CHAPTER 3

STORMWATER HYDROLOGY

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3.1 METHODS FOR ESTIMATING STORMWATER RUNOFF

3.1.1 Introduction to Hydrologic Methods

Hydrology deals with estimating peak flows, volumes, and time distributions of stormwater runoff. The analysis of these parameters is fundamental to the design of stormwater management facilities, such as storm drainage systems and best management practices. In the hydrologic analysis of a development site, there are a number of variable factors that affect the nature of stormwater runoff from the site. Some of the factors that need to be considered include:

- Rainfall amount and storm distribution
- Drainage area size, shape and orientation
- Ground cover and soil type
- Slopes of terrain and stream channel(s)
- Antecedent moisture condition
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Characteristics of the local drainage system

There are a number of empirical hydrologic methods that can be used to estimate runoff characteristics for a site or drainage subbasin; however, the following methods presented in this section have been selected to support hydrologic site analysis for the design methods and procedures included in the Manual:

- Rational Method
- SCSNRCS TR-55 Unit Hydrograph Method
- U.S. Geological Survey (USGS) Regression Equations
- Water Quality Treatment Volume Calculation
- Water Balance Calculations

These methods were selected based upon a verification of their accuracy in duplicating local hydrologic estimates for a range of design storms throughout the state and the availability of equations, nomographs, and computer programs to support the methods.

Table 3.1.1-1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications. Table 3.1.1-2 provides some limitations on the use of several methods.

In general:

- The Rational Method is recommended for small highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters.
- The USGS regression equations are recommended for drainage areas with characteristics within the ranges given for the equations. The USGS equations

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should be used with caution when there are significant storage areas within the drainage basin or where other drainage characteristics indicate that general regression equations might not be appropriate.

Method	Manual Section	Rational Method	SCS <u>NRCS</u> TR-55 Method	USGS Equations	Water Quality Volume
Water Quality Volume (WQ _v)	2.2				✓
Channel Protection Volume (Cp _v)	2.2		✓		
Overbank Flood Protection (Q _{p25})	2.2		✓	✓	
Extreme Flood Protection (Q _f)	2.2		✓	✓	
Storage Facilities	3.3		✓	✓	
Outlet Structures	3.4		✓	✓	
Gutter Flow and Inlets	5.2	✓			
Storm Drain Pipes	5.2	✓	✓	✓	
Culverts	5.3	✓	✓	✓	
Small Ditches	5.4	√	✓	✓	
Open Channels	5.4		✓	✓	
Energy Dissipation	5.5		✓	✓	

able 3.1.1-2 Constraints on Using Recommended Hydrologic Methods			
Method	Size Limitations ¹	Comments	
Rational	0 – 25 <mark>200</mark> acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems. Shall Net not to be used for storage design.	
SCSNRCS	TR-55 ² 0 – 2000 acre	s* Method can be used for estimating peak flows and hydrographs for all design applications.	
USGS	25 acres to 25 mi ²	Method can be used for estimating peak flows for all design applications.	
USGS	128 acres to 25 mi ²	Method can be used for estimating hydrographs for all design applications.	
Water Quality	Limits set for each BMP	Method used for calculating the Water Quality Volume (WQ $_{v}$)	
² There are ma		he stormwater management facility (e.g., culvert, inlet). ich as HEC-1) that utilize this methodology asin simplified peak flow only.	

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If other hydrologic methods are to be considered and used by a local review authority or design engineer, the method should first be calibrated to local conditions and tested for accuracy and reliability. If local stream gage data are available, these data can be used to develop peak discharges and hydrographs. The user is referred to standard hydrology textbooks for statistical procedures that can be used to estimate design flood events from stream gage data.

Note: 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

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runoff from the basin is complex and too little data are available on the factors influencing the rainfall-runoff relationship to expect exact solutions.

3.1.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 3.1.2-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

-					
<u>Symbol</u>	Definition	<u>Units</u>			
А	Drainage area	acres			
B _f	Baseflow	acre-feet			
С	Runoff coefficient	-			
Cf	Frequency factor	-			
CN	SCSNRCS TR-55-runoff curve number	· -			
CPv	Channel Protection Volume	acre-feetft3			
d	Time interval	hours			
E	Evaporation	ft			
Et	Evapotranspiration	ft			
Fp	Pond and swamp adjustment factor	-			
Gh	Hydraulic gradient				
lori	Runoff intensity	in/hr			
I	Percent of impervious cover	%			
I	Infiltration	ft			
la	Initial abstraction from total rainfall	in			
k _h	Infiltration rate	ft/day			
L	Flow length	ft			
n	Manning roughness coefficient	-			
NRCS	Natural Resources Conservation Servi	<u>ce</u>			
Of	Overflow	acre-feetft3			
Р	Accumulated rainfall	in			
P ₂	2-year, 24-hour rainfall	in			
Pw	Wetted perimeter	ft			
PF	Peaking factor				
Q	Rate of runoff	cfs (or inches)			
Q_d	Developed runoff for the design storm	in			
Q _f	Extreme Flood Protection Volume	<u>ft³acre-feet</u>			
Qi	Peak inflow discharge	cfs			
Q_{o}	Peak outflow discharge	cfs			
Qp	Peak rate of discharge	cfs			
Q _{p25}	Overbank Flood Protection Volume	ft ³ acre-feet			
Q _{wq}	Water Quality peak rate of discharge	cfs			
q	Storm runoff during a time interval	in			
q _u	Unit peak discharge	cfs (or cfs/mi ² /inch)			
Ŕ	Hydraulic radius	ft			
R₀	Runoff	ft ³ acre-feet			
R _v	Runoff Coefficient	_			
S	Ground slope	ft/ft or %			
S	Potential maximum retention	in			
S	Slope of hydraulic grade line	ft/ft			
SCS	Soil Conservation Service	_			

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T	Channel top width	ft
T _L	Lag time	hours
T _p	Time to peak	hr
T _t	Travel time	hours
t t _c TIA V V Vr Vs WQv	Time Time of concentration Total impervious area Velocity Pond volume Runoff volume Storage volume Water Quality Volume	min min % ft/s <u>ft³acre-feet</u> <u>ft³acre-feet</u> ft ³ acre-feet

3.1.3 Rainfall Estimation

The first step in any hydrologic analysis is an estimation of the rainfall that will fall on the site for a given time period. The amount of rainfall can be quantified with the following characteristics:

Duration (hours) – Length of time over which rainfall (storm event) occurs *Depth (inches)* – Total amount of rainfall occurring during the storm duration *Intensity (inches per hour)* – Depth divided by the duration

The Frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of *exceedence probability* or *return period*.

Exceedence Probability – Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically 1 year *Return Period* – Average length of time between events that have the same duration and volume

Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedence probability of 0.01 and a return period of 100 years.

Rainfall intensities for any location across Georgia can be obtained through the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 publication, or online using the *Precipitation Frequency Data Server* database (<u>http://hdsc.nws.noaa.gov/hdsc/pfds/</u>). NOAA precipitation data should be used for all hydrologic analysis at the given locations. Additional information regarding how the values in this database were derived can be accessed using the link above.

The tabular precipitation data provided within the database are applicable for storm durations from 5 minutes to 60 days. In addition to the tabular data, the NOAA precipitation database also has a graphical display that shows the intensity duration frequency curve of any given precipitation data. Figure 3.1.3-2-1 shows an example Intensity-Duration-Frequency (IDF) Curve for Athens, Georgia, for up to 10 storms (1-year – 1,000-year). These curves are plots of the tabular values. No values are given for

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times less than 5 minutes.

Figure 3.1.3-3-2 (included as the 10-year 24-hour values from TP40) shows that the rainfall values vary south to north with generally constant values in a "V" pattern from east to west in central and south Georgia.

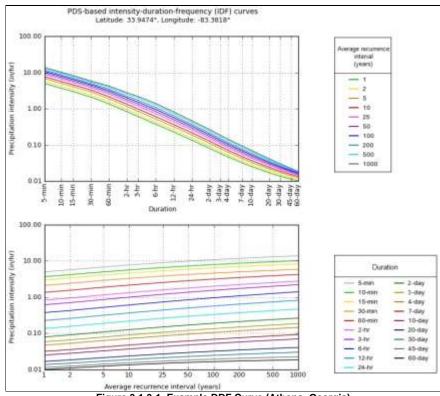


Figure 3.1.3-1 Example DDF Curve (Athens, Georgia) (Source: NOAA Atlas 14, Volume 9, Version 2, 2013)

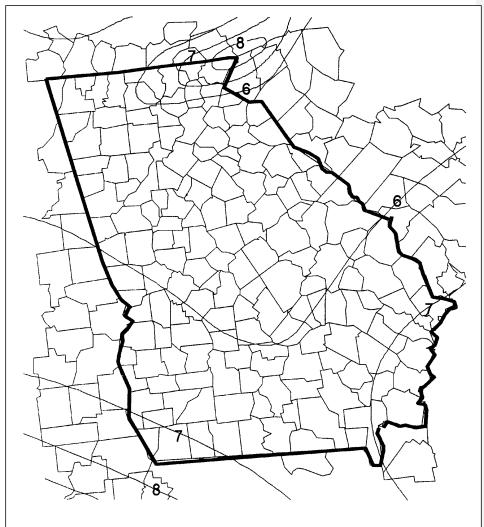


Figure 3.1.3-2 Rainfall Isohyetal Lines (10-year, 24-hour values)

3.1.4 Rational Method

3.1.4.1 Introduction

An important formula for determining the peak runoff rate is the Rational Formula. It is characterized by:

- · Consideration of the entire drainage area as a single unit
- · Estimation of flow at the most downstream point only
- The assumption that rainfall is uniformly distributed over the drainage area and is constant over time

The Rational Formula follows the assumption that:

- The predicted peak discharge has the same probability of occurrence (return period) as the used rainfall intensity (I)
- The runoff coefficient (C) is constant during the storm event

When using the Rational Method some precautions should be considered:

- □ In determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility.
- Since the Rational Method uses a composite C and a single t_c value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), then basin should be divided into sub-drainage basins.
- □ The charts, graphs, and tables included in this section are given to assist the engineer in applying the Rational Method. The engineer should use sound engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate that these adjustments are appropriate.

3.1.4.2 Application

The Rational Method can be used to estimate stormwater runoff peak flows for the design of gutter flows, drainage inlets, storm drain pipe, culverts, and small ditches. It is most applicable to small, highly impervious areas. The recommended maximum drainage area that should be used with the Rational Method is $\frac{25-200}{200}$ acres.

The Rational Method should not be used for storage design or any other application where a more detailed routing procedure is required. However, due to the popularity of the Modified Rational method among Georgia practitioners for design of small detention facilities, a method has been included in Section 3.3. The normal use of the Modified Rational method significantly under predicts detention volumes, but the improved method in Section 3.3 corrects this deficiency in the method and can be used for detention design for drainage areas up to 5 acres.

The Rational Method should also not be used for calculating peak flows downstream of bridges, culverts or storm sewers that may act as restrictions and impact the peak rate of discharge.

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3.1.4.3 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, t_c (the time required for water to flow from the most remote point of the basin to the location being analyzed).

The Rational Formula is expressed as follows:

(3.1.1)

Where: Q = maximum rate of runoff (cfs)

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a duration equal to the t_c (in/hr)

A = drainage area contributing to the design location (acres)

The coefficients given in Table 3.1.4-2 are applicable for storms of 5-year to 10-year frequencies. Less frequent, higher intensity storms may require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin Engineers, 1969). The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor C_f . The Rational Formula now becomes:

$Q = C_f CIA$

(3.1.2)

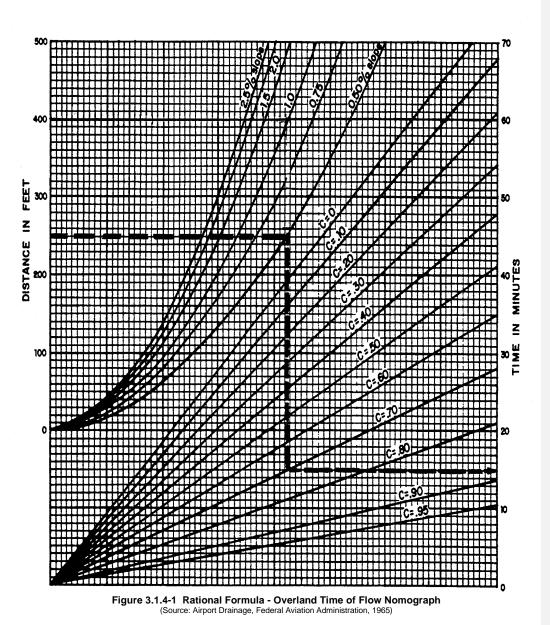
The C_{f} values that can be used are listed in Table 3.1.4-1. The product of C_{f} times C shall not exceed 1.0.

Table 3.1.4-1 Frequency Factors for Rat	ional Formula	
Recurrence Interval (years)	<u>C</u> f	
10 or less	1.0	
25	1.1	
50	1.2	
100	1.25	

3.1.4.4 Time of Concentration

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). The time of concentration consists of an overland flow time to the point where the runoff is concentrated or enters a defined drainage feature (e.g., open channel) plus the time of flow in a closed conduit or open channel to the design point.

Figure 3.1.4-1 can be used to estimate overland flow time. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter.



To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the nomograph shown in Figure 3.1.4-2. Note: time of concentration cannot be

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less than 5 minutes.

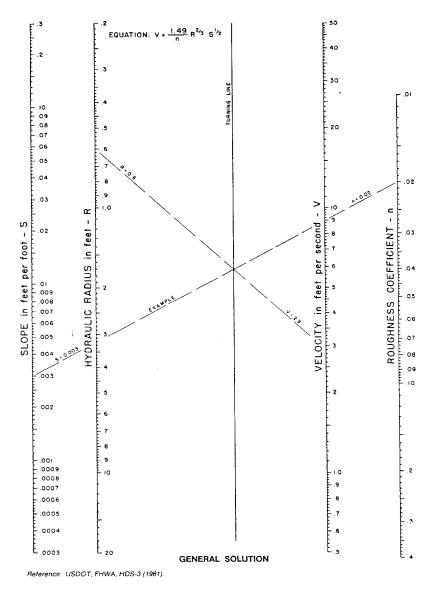


Figure 3.1.4-2 Manning's Equation Nomograph (Source: USDOT, FHWA, HDS-3 (1961))

Another method that can be used to determine the overland flow portion of the time of concentration is the "Kinematic Wave Nomograph" (Figure 3.1.4-3). The kinematic wave method incorporates several variables including rainfall intensity and Manning's "n". In using the nomograph, the engineer has two unknowns starting the computations: the time of concentration and the rainfall intensity. A value for the rainfall intensity "I" must be assumed. The travel time is determined iteratively.

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If one has determined the length, slope and roughness coefficient, and selected a rainfall intensity table, the steps to use Figure 3.1.4-3 are as follows:

(Step 1) Assume a rainfall intensity.

- (Step 2) Use Figure 3.1.4-3 (or the equation given in the figure) to obtain the first estimate of time of concentration.
- (Step 3) Using the time of concentration obtained from Step 2, use the appropriate rainfall intensity from NOAA Atlas 14 and find the rainfall intensity corresponding to the computed time of concentration. If this rainfall intensity corresponds with the assumed intensity, the problem is solved. If not, proceed to Step 4.
- (Step 4) Assume a new rainfall intensity that is between that assumed in Step 1 and that determined in Step 3.
- (Step 5) Repeat Steps 1 through 3 until there is good agreement between the assumed rainfall intensity and that obtained from the rainfall intensity tables.

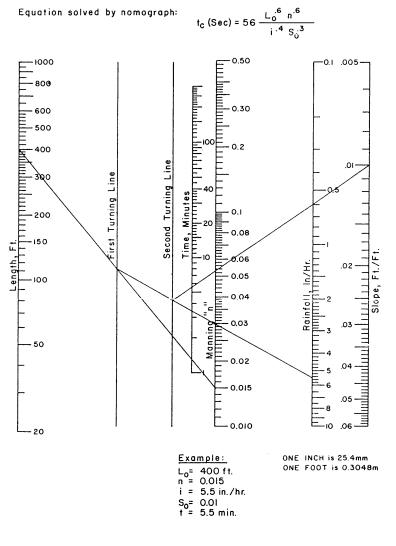


Figure 3.1.4-3 Kinematic Wave Nomograph (Source: Manual For Erosion And Sediment Control In Georgia, 1996)

Generally, the time of concentration for overland flow is only a part of the overall design problem. Often one encounters swale flow, confined channel flow, and closed conduit flow-times that must be added as part of the overall time of concentration. When this situation is encountered, it is best to compute the confined flow-times as the first step in the overall determination of the time of concentration. This will give the designer a rough estimate of the time involved for the overland flow, which will give a better first start on the rainfall intensity assumption. For example, if the flow time in a channel is 15 minutes and the overland flow time from the ridge line to the channel is 10 minutes, then the total time of concentration is 25 minutes.

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Other methods and charts may be used to calculate overland flow time if approved by the local review authority.

Two common errors should be avoided when calculating time of concentration. First, in some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. Second, when designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 50 feet for impervious areas should be done only after careful consideration.

3.1.4.5 Rainfall Intensity (I)

The rainfall intensity (I) is the average rainfall rate in in/hr for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from data given from NOAA Atlas 14.

3.1.4.6 Runoff Coefficient (C)

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 3.1.4-2 gives the recommended runoff coefficients for the Rational Method.

Description of Area	Runoff Coefficients (C)
Lawns:	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat, 2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
Unimproved areas (forest)	0.15
Business:	
Downtown areas	0.95
Neighborhood areas	0.70
Residential:	
Single-family areas	0.50
Multi-units, detached	0.60
Multi-units, attached	0.70
Suburban	0.40
Apartment dwelling areas	0.70

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Table 3.1.4-2 Recommended Runoff Coefficient Values	(continued)
Industrial:	
Light areas	0.70
Heavy areas	0.80
Parks, cemeteries	0.25
Playgrounds	0.35
Railroad yard areas	0.40
Streets:	
Asphalt and Concrete	0.95
Brick	0.85
Drives, walks, and roofs	0.95
Gravel areas	0.50
Graded or no plant cover	
Sandy soil, flat, 0 - 5%	0.30
Sandy soil, flat, 5 - 10%	0.40
Clayey soil, flat, 0 - 5%	0.50
Clayey soil, average, 5 - 10%	0.60

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 3.1.4-2 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as rooftops, asphalt, and concrete streets and sidewalks. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

It should be remembered that the Rational Method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

It may be that using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) will yield a higher peak runoff value than by using the whole site. This should be checked particularly in areas where the overland portion is grassy (yielding a long t_c) to avoid underestimating peak runoff.

3.1.4.7 Example Problem

Following is an example problem that illustrates the application of the Rational Method to estimate peak discharges.

Estimates of the maximum rate of runoff are needed at the inlet to a proposed culvert for a 25-year return period.

Site Data

From a topographic map of the City of Roswell and a field survey, the area of the drainage

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basin upstream from the point in question is found to be 23 acres. In addition the following data were measured:

Average overland slope = 2.0% Length of overland flow = 50 ft Length of main basin channel = 2,250 ft Slope of channel - .018 ft/ft = 1.8% Roughness coefficient (n) of channel was estimated to be 0.090 From existing land use maps, land use for the drainage basin was estimated to be: Residential (single family) - 80% Graded - sandy soil, 3% slope - 20%

From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be: Lawn - sandy soil, 2% slope

Overland Flow

A runoff coefficient (C) for the overland flow area is determined from Table 3.1.4-2 to be 0.10.

Time of Concentration

From Figure 3.1.4-1 with an overland flow length of 50 ft, slope of 2% and a C of 0.10, the overland flow time is 10 min. Channel flow velocity is determined from Figure 3.1.4-2 to be 3.1 ft/s (n = 0.090, R = 1.62 (from channel dimensions) and S = .018). Therefore,

Flow Time = $\frac{2,250 \text{ feet}}{(3.1 \text{ ft/s})/(60 \text{ s/min})}$ = 12.1 minutes

and $t_c = 10 + 12.1 = 22.1 \text{ min}$ (use 22 min)

Rainfall Intensity

From NOAA Atlas 14, and using a duration equal to 22 minutes,

I₂₅ (25-yr return period) = 4.88 in/hr

Runoff Coefficient

A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the values from Table 3.1.4-2.

Land Use	Percent of Total Land Area	Runoff Coefficient	Weighted Runoff Coefficient*
Residential (single family)	.80	.50	.40
Graded area	.20	.30	.06

Total Weighted Runoff Coefficient = <u>.46</u>

*Column 3 equals column 1 multiplied by column 2.

Peak Runoff

The estimate of peak runoff for a 25-yr design storm for the given basin is:

 $Q_{25} = C_f CIA = (1.10)(.46)(4.88)(23) = 57 cfs$

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3.1.5 SCSNRCS TR-55 Hydrologic Method

3.1.5.1 Introduction

The Soil Conservation Service¹ (SCSNRCS TR-55) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCSNRCS TR-55 approach, however, is more sophisticated in that it 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. Details of the methodology can be found in the SCSNRCS -National Engineering Handbook, Part 630, Section 4, Hydrology.

A typical application of the SCSNRCS TR-55 method includes the following basic steps:

- (1) Determination of curve numbers that represent different land uses within the drainage area.
- (2) Calculation of time of concentration to the study point.
- (3) Using the Type II or Type III rainfall distribution, total and excess rainfall amounts are determined. Note: See Figure 3.1.5-1 for the geographic boundaries for the different <u>SCSNRCS TR-55</u> rainfall distributions.
- (4) Using the unit hydrograph approach, the hydrograph of direct runoff from the drainage basin can be developed.

3.1.5.2 Application

The <u>SCSNRCS TR-55</u> method can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows. The simplified method of

subsection 3.1.5.7 can be used for drainage areas up to 2,000 acres. Thus, the <u>SCSNRCS TR-55</u> method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches and open channels, and energy dissipators.

3.1.5.3 Equations and Concepts

The hydrograph of outflow from a drainage basin is the sum of the elemental hydrographs from all the sub-areas of the basin, modified by the effects of transit time through the basin and storage in the stream channels. Since the physical characteristics of the basin including shape, size and slope are constant, the unit hydrograph approach assumes that there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. Thus, the unit hydrograph is a typical hydrograph for the basin with a runoff volume under the hydrograph equal to one (1.0) inch from a storm of specified duration. For a storm of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the runoff volume. Therefore, a storm that produces 2 inches of runoff would have a hydrograph with a flow equal to twice the flow of the unit hydrograph. With 0.5 inches of runoff, the flow of the hydrograph would be one-half of the flow of the unit hydrograph.

The following discussion outlines the equations and basin concepts used in the <u>SCSNRCS TR-55</u> method.

<u>Drainage Area</u> - 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, combine hydrographs from different sub-basins as applicable, and/or route flows to

¹ The Soil Conservation Service is now known as the Natural Resources Conservation Service (NRCS)

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points of interest.

<u>Rainfall</u> - The <u>SCSNRCS TR-55</u> method applicable to the State of Georgia is based on a storm event that has a Type II or Type III time distribution. These distributions are used to distribute the 24-hour volume of rainfall for the different storm frequencies (Figure 3.1.5-1).

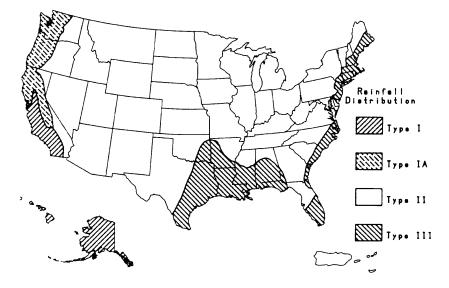


Figure 3.1.5-1 Approximate Geographic Boundaries for <u>SCSNRCS TR-55</u> Rainfall Distributions

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by <u>SCSNRCS TR-55</u> from experimental plots for numerous soils and vegetative cover conditions. The following <u>SCSNRCS TR-55</u> runoff equation is used to estimate direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$\mathbf{Q} = \frac{(\mathbf{P} - \mathbf{I}_{\alpha})^2}{(\mathbf{P} - \mathbf{I}_{\alpha}) + \mathbf{S}}$$
(3.1.3)

Where: Q = accumulated direct runoff (in)

P = accumulated rainfall (potential maximum runoff) (in)

 $I_a\,$ = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)

S = potential maximum soil retention (in)

An empirical relationship used in the $\frac{\text{SCS}_{NRCS} \text{ TR-55}}{\text{method for estimating } I_a \text{ is:}}$

$I_a = 0.2S$

(3.1.4)

This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment.

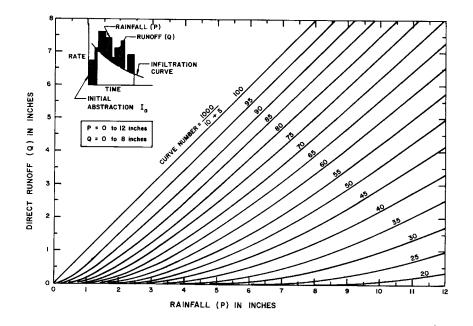
Substituting 0.2S for I_a in equation 3.1.3, the equation becomes:

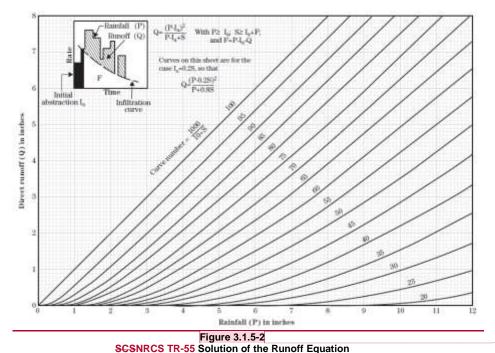
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$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$
(3.1.5)

Where: S = 1000/CN - 10 and $CN = \frac{SCS_{NRCS} TR-55}{CS_{NRCS} TR-55}$ curve number

Figure 3.1.5-2 shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurs on a watershed with a curve number of 85.





(Source: SCSNRCS TR-55, NEH630, 2004TR-55, Second Edition, June 1986)

Equation 3.1.5 can be rearranged so that the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

CN = 1000/[10 + 5P + 10Q – 10(Q ² + 1.25QP) ^{1/2}]

3.1.5.4 Runoff Factor

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The <u>SCSNRCS TR-55</u> method uses a combination of soil conditions and land uses (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. The higher the CN, the higher the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the <u>SCSNRCS TR-55</u> has divided soils into four hydrologic soil groups.

- Group A Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.
- Group B Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.
- Group C Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.
- Group D Soils having a high runoff potential due to very slow infiltration rates. These soils

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consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

A list of soils throughout the State of Georgia and their hydrologic classification can be found in the publication *Urban Hydrology for Small Watersheds*, 2nd *Edition, Technical Release Number 55, 1986.* Soil Survey maps can be obtained online at the United States Department of Agriculture (USDA) Natural Resource Conservation Commission's (NRCS) web soil survey online tool to classify the soil type. (http://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm)

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. Average antecedent soil moisture conditions (AMC II) are recommended for most hydrologic analysis, except in the design of state-regulated Category I dams where AMC III may be required. Areas with high water table conditions. This should be considered a calibration parameter for modeling against real calibration data. Table 3.1.5-1 gives recommended curve number values for a range of different land uses.

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but sees the drainage area as a uniform land use represented by the composite curve number.

Land Use	Percent of Total Land Area	Curve Number	Weighted Curve Number (% area x CN)
Residential 1/8 acre Soil group B	80%	85	68
Meadow Good condition Soil group C	20%	71	14

Composite curve numbers for a drainage area can be calculated by using the weighted method as presented below.

The different land uses within the basin should reflect a uniform hydrologic group represented by a single curve number. Any number of land uses can be included, but if their spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

3.1.5.5 Urban Modifications of the SCSNRCS TR-55 Method

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for developed areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 3.1.5-1 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system. It is possible that curve number values from urban areas could be reduced by not directly connecting impervious surfaces to the drainage system, but allowing runoff to flow as sheet flow over significant pervious areas.

The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

Connected Impervious Areas

The CNs provided in Table 3.1.5-1 for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

- (a) Pervious urban areas are equivalent to pasture in good hydrologic condition, and
- (b) Impervious areas have a CN of 98 and are directly connected to the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 3.1.5-1 are not applicable, use

Figure 3.1.5-3 to compute a composite CN. For example, Table 3.1.5-1 gives a CN of 70 for a 1/2-acre lot in hydrologic soil group B, with an assumed impervious area of 25%. However, if the lot has 20% impervious area and a pervious area CN of 61, the composite CN obtained from Figure 3.1.5-3 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

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Cover description		Curve numbers for hydrologic soil groups				
Cover type and hydrologic condition		erage percent pervious area ²	А	в	С	D
Cultivated land:	without conservation with conservation to		72 62	81 71	88 78	91 81
Pasture or range lan	d: poor condition good condition		68 39	79 61	86 74	89 80
Meadow: good condit	ion		30	58	71	78
Wood or forest land	thin stand, poor cov good cover	/er	45 25	66 55	77 70	83 77
Poor conditior Fair condition	parks, golf courses, a (grass cover <50%) (grass cover 50% to 7 n (grass cover > 75%)	'5%)	³ 68 49 39	79 69 61	86 79 74	89 84 80
Impervious areas: Paved parking (excluding righ	lots, roofs, driveways nt-of-way)	, etc.	98	98	98	98
right-of-way) Paved; open o	and storm drains (excl ditches (including right ing right-of-way) right-of-way)	U U	98 83 76 72	98 89 85 82	98 92 89 87	98 93 91 89
Urban districts: Commercial and bus Industrial	siness	85% 72%	89 81	92 88	94 91	95 93
Residential districts 1/8 acre or less (town 1/4 acre 1/3 acre 1/2 acre 1 acre 2 acres		65% 38% 30% 25% 20% 12%	77 61 57 54 51 46	85 75 72 70 68 65	90 83 81 80 79 77	92 87 86 85 84 82
Developing urban a	reas and (pervious areas		77	86	91	94

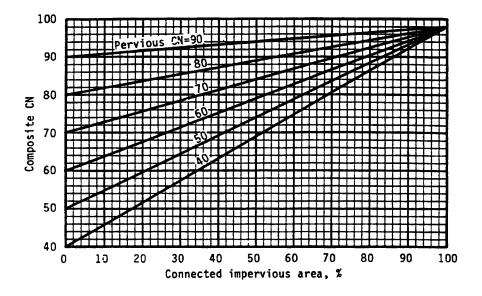
³ CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

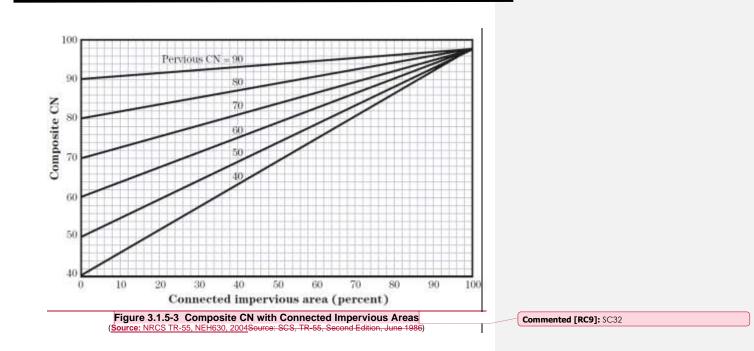
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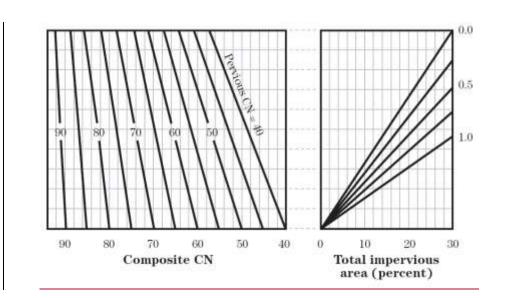
Unconnected Impervious Areas

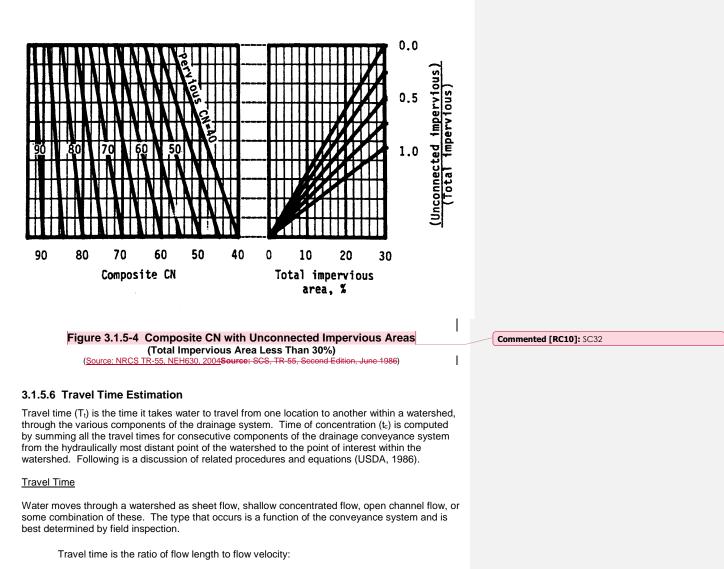
Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use Figure 3.1.5-4 if total impervious area is less than 30% or (2) use Figure 3.1.5-3 if the total impervious area is equal to or greater than 30%, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When impervious area is less than 30%, obtain the composite CN by entering the right half of Figure 3.1.5-4 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1/2-acre lot with 20% total impervious area (75% of which is unconnected) and pervious CN of 61, the composite CN from Figure 3.1.5-4 is 66. If all of the impervious area is connected, the resulting CN (from Figure 3.1.5-3) would be 68.









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

(3.1.7)

 $\begin{array}{ll} \mbox{Where:} & T_t = travel time (hr) \\ \mbox{L} &= flow length (ft) \\ \mbox{V} &= average velocity (ft/s) \\ \mbox{3600} = conversion factor from seconds to hours } \end{array}$

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Sheet Flow

Sheet flow can be calculated using the following formula:

$$T_{t} = \frac{0.42 (nL)^{0.8}}{60 (P_{2})^{0.5} (S)^{0.4}}$$

(3.1.8)

 $\begin{array}{ll} \mbox{Where:} & T_t\mbox{=} travel time (hr) \\ & n = \mbox{Manning roughness coefficient (see Table 3.1.5-2)} \\ & L = \mbox{flow length (ft),} \\ & P_2 = 2\mbox{-year, 24-hour rainfall} \\ & S = \mbox{land slope (ft/ft)} \end{array}$

Surface Description	<u>n</u>
Smooth surfaces (concrete, asphalt,	
gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses ²	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods ³	
Light underbrush	0.40
Dense underbrush	0.80
n values are a composite of information by Engman (1986).	
udes species such as weeping lovegrass, bluegrass, buffalo gr	es blue grama grass and native grass mixt
	s the only part of the plant cover that will

Source: SCSNRCS TR-55, TR-55, Second Edition, June 1986.

Shallow Concentrated Flow

After a maximum of 50 to 100 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure 3.1.5-5, in which average velocity is a function of watercourse slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be computed from using Figure 3.1.5-5, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

Unpaved	V = 16.13(S) ^{0.5}	(3.1.9)
Paved	V = 20.33(S) ^{0.5}	(3.1.10)

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Where: V = average velocity (ft/s) S = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity using Figure 3.1.5-5 or equations 3.1.9 or 3.1.10, use equation 3.1.7 to estimate travel time for the shallow concentrated flow segment.

Open Channels

Velocity in channels should be calculated from the Manning equation. Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where channels have been identified by the local municipality, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity for travel time calculations is usually determined for bank-full elevation assuming low vegetation winter conditions.

Manning's equation is
$$V = \frac{1.49 (R)^{2/3} (S)^{1/2}}{n}$$

Where: V = average velocity (ft/s)

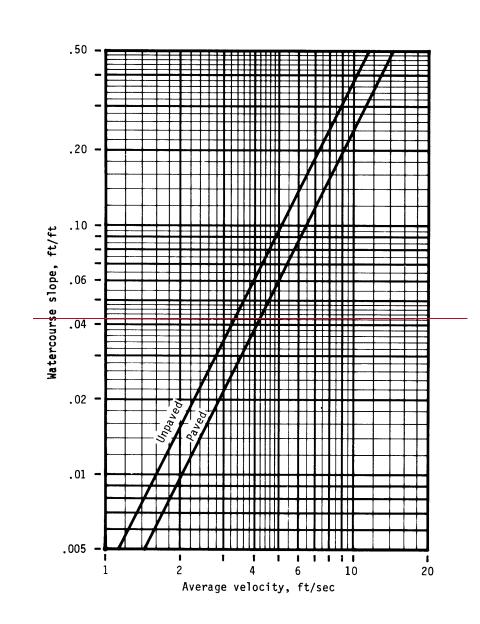
- R = hydraulic radius (ft) and is equal to A/P_w
- A = cross sectional flow area (ft^2)
- P_w = wetted perimeter (ft)
- S = slope of the hydraulic grade line (ft/ft)
- n = Manning's roughness coefficient for open channel flow

After average velocity is computed using equation 3.1.11, T_t for the channel segment can be estimated using equation 3.1.7.

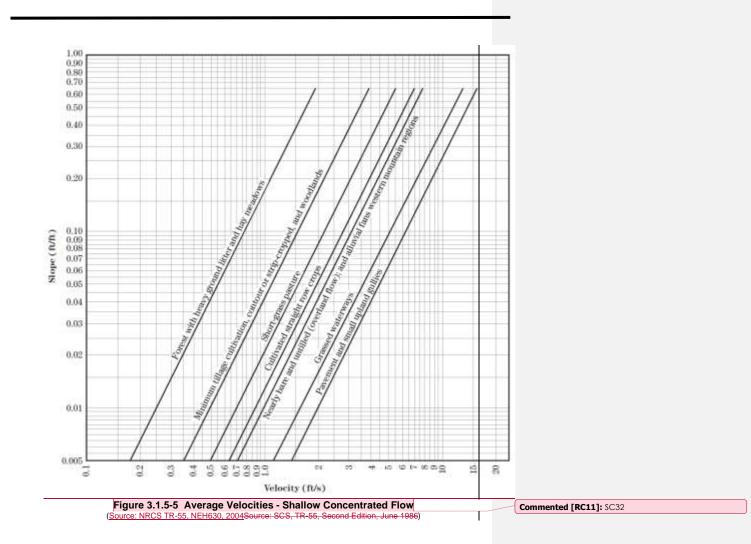
Limitations

- Equations in this section should not be used for sheet flow longer than 50 feet for impervious land uses.
- $\hfill\square$ In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate $t_c.$
- A culvert or bridge can act as detention structure if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert or bridge.

(3.1.11)



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3.1.5.7 Simplified SCSNRCS TR-55 Peak Runoff Rate Estimation

The following <u>SCSNRCS TR-55</u> procedures were taken from the <u>SCSNRCS TR-55</u> Technical Release 55 (USDA, 1986) which presents simplified procedures to calculate storm runoff volume and peak rate of discharges. These procedures are applicable to small drainage areas (typically less than 2,000 acres) with homogeneous land uses that can be described by a single CN value. The peak discharge equation is:

 $Q_p = q_u AQF_p$

(3.1.12)

Where: Q_p = peak discharge (cfs)

- q_u = unit peak discharge (cfs/mi²/in)
- A = drainage area (mi²)
- Q = runoff (in)
- F_p = pond and swamp adjustment factor

The input requirements for this method are as follows:

- t_c hours
- Drainage area mi²
- Type II or type III rainfall distribution
- 24-hour design rainfall
- CN value
- Pond and Swamp adjustment factor (If pond and swamp areas are spread throughout the watershed and are not considered in the t_c computation, an adjustment is needed.)

Computations for the peak discharge method proceed as follows:

- The 24-hour rainfall depth is determined from the precipitation data in the NOAA Atlas 14 publication, or online using the *Precipitation Frequency Data Server* database (<u>http://hdsc.nws.noaa.gov/hdsc/pfds/</u>).
- (2) The runoff curve number, CN, is estimated from Table 3.1.5-1 and direct runoff, Q_p, is calculated using equation 3.1.12.
- (3) The CN value is used to determine the initial abstraction, I_a, from Table 3.1.5-3, and the ratio I_a/P is then computed (P = accumulated 24-hour rainfall).
- (4) The watershed time of concentration is computed using the procedures in subsection 3.1.5.6 and is used with the ratio l_a/P to obtain the unit peak discharge, q_{up}, from Figure 3.1.5-6 for the Type II rainfall distribution and Figure 3.1.5-7 for the Type III rainfall distribution. If the ratio l_a/P lies outside the range shown in the figures, either the limiting values or another peak discharge method should be used. Note: Figures 3.1.5-6 and 3.1.5-7 are based on a peaking factor of 484. If a peaking factor of 300 is needed, these figures are not applicable and the simplified <u>SCSNRCS TR-55</u> method should not be used.
- (5) The pond and swamp adjustment factor, F_{p} , is estimated from below:

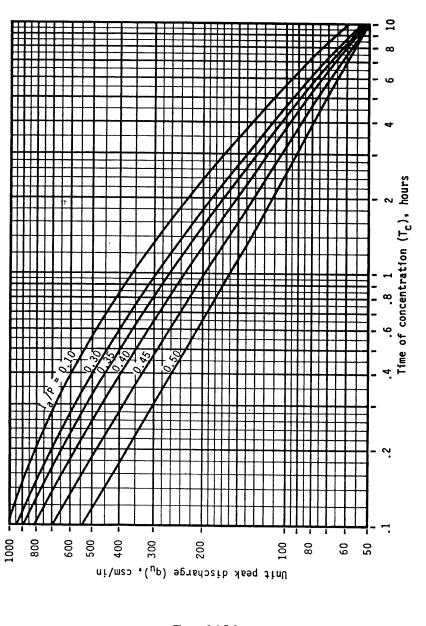
Pond and Swamp Areas (%*)	<u>F</u> ₀
0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

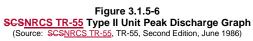
*Percent of entire drainage basin

(6) The peak runoff rate is computed using equation 3.1.12.

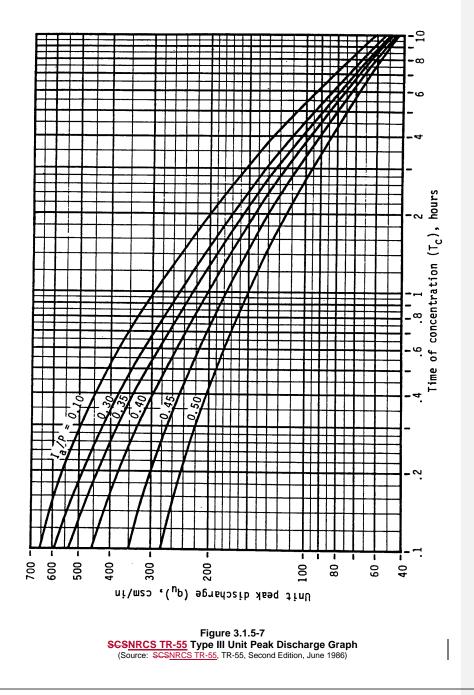
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Table 3.1.5-3 Ia Values	for Runoff C	urve Numbers	
Curve Number	<u>l₄ (in)</u>	Curve Number	<u>l_a (in)</u>
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.774	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		
Source: SCS<u>NRCS TR-55</u>, TR-5	55, Second Editi	on, June 1986	





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3.1.5.8 Example Problem 1

Compute the 100-year peak discharge for a 50-acre wooded watershed located in Peachtree City, which will be developed as follows:

- Forest land good cover (hydrologic soil group B) = 10 ac
- Forest land good cover (hydrologic soil group C) = 10 ac
- 1/3 acre residential (hydrologic soil group B) = 20 ac
- Industrial development (hydrological soil group C) = 10 ac

Other data include the following: Total impervious area = 18 acres, % of pond / swamp area = 0

Computations

(1) Calculate rainfall excess:

- The 100-year, 24-hour rainfall is 8.22 inches (From NOAA Atlas 14).
- The 1-year, 24-hour rainfall is 3.37 inches (From NOAA Atlas 14).
- Composite weighted runoff coefficient is:

Dev. #	Area	<u>% Total</u>	CN	Composite CN
1	10 ac.	0.20	55	11.0
2	10 ac.	0.20	70	14.0
3	20 ac.	0.40	72	28.8
4	10 ac.	0.20	91	18.2
Total	50 ac.	1.00		72

* from Equation 3.1.5, Q (100-year) = 4.89 inches Q_d (1-year developed) = 1.0 inches

(2) Calculate time of concentration

The hydrologic flow path for this watershed = 1,890 ft

Segment	Type of Flow	Length (ft) Slope (%)
1	Overland n = 0.24	40	2.0
2	Shallow channel	750	1.7
3	Main channel*	1100	0.50

* For the main channel, n = .06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

Segment 1 - Travel time from equation 3.1.8 with $P_2 = 3.84$ inches (From NOAA Atlas 14)

 $T_t = [0.42(0.24 \ X \ 40)^{0.8}] \ / \ [(3.84)^{0.5} \ (.020)^{0.4}] = 6.26 \ minutes$

 $\begin{array}{l} \mbox{Segment 2 - Travel time from Figure 3.1.5-5 or equation 3.1.9} \\ \mbox{V = 2.1 ft/sec (from equation 3.1.9)} \\ \mbox{T}_t = 750 \ / \ 60 \ (2.1) = 5.95 \ \mbox{minutes} \end{array}$

Segment 3 - Using equation 3.1.11 $V = (1.49/.06) (1.43)^{0.67} (.005)^{0.5} = 2.23 \text{ ft/sec}$ $T_t = 1100 \ / \ 60 \ (2.23) = 8.22 \ \text{minutes}$

 $t_c = 6.26 + 5.95 + 8.22 = 20.43$ minutes (.34 hours)

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 I_{e}/P = (.778 / 8.23) = .095 (Note: Use I_{e}/P = .10 to facilitate use of Figure 3.1.5-6. Straight line interpolation could also be used.)

- (4) Unit discharge q_u (100-year) from Figure 3.1.5-6 = 650 csm/in, q_u (1-year) = 580 csm/in
- (5) Calculate peak discharge with F_p = 1 using equation 3.1.12

Q₁₀₀ = 650 (50/640)(4.89)(1) = 248 cfs

3.1.5.9 Hydrograph Generation

In addition to estimating the peak discharge, the <u>SCSINRCS TR-55</u> method can be used to estimate the entire hydrograph from a drainage area. The <u>SCSINRCS TR-55</u> has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for small drainage areas (less than 2,000 acres). The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, <u>SCSINRCS TR-55</u> has developed hydrograph procedures to be used to generate composite flood hydrographs. For the developed hydrograph from a homogeneous developed drainage area and drainage areas that are not homogeneous, where hydrographs need to be generated from sub-areas and then routed and combined at a point downstream, the engineer is referred to the procedures outlined by the <u>SCSINRCS TR-55</u> in the 1986 version of TR-55 available from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

The unit hydrograph equations used in the SCSNRCS TR-55 method for generating hydrographs includes a constant to account for the general land slope in the drainage area. This constant, called a peaking factor, can be adjusted when using the method. A default value of 484 for the peaking factor represents rolling hills – a medium level of relief. SCSNRCS TR-55 indicates that for mountainous terrain the peaking factor can go as high as 600, and as low as 300 for flat (coastal) areas. Referring to Figure 3.1.6-1, which shows the different hydrologic regions developed by the USGS for the state of Georgia, Region 3 represents the primary region of the state where modification of the peaking factor from 484 to 300 is most often warranted if the individual watershed possesses flat terrain.

As a result of hydrologic/hydraulic studies completed in the development of this Manual, the following are recommendations related to the use of different peaking factors:

- The <u>SCSNRCS TR-55</u> method can be used without modification (peaking factor left at 484) in Regions 1, 2 and 4 generally when performing modeling analysis.
- The <u>SCSNRCS TR-55</u> method can be modified in that a peaking factor of 300 can be used for modeling generally in Region 3 when watersheds are flat and have significant storage in the overbanks. These watersheds would be characterized by:
 - □ Mild Slopes (less than 2% slope)
 - Significant surface storage throughout the watershed in the form of standing water during storm events or inefficient drainage systems

The <u>SCSNRCS TR-55</u> method can be similarly adjusted for any watershed that has flow and storage characteristics similar to a typical Region 3 stream

The development of a runoff hydrograph from a watershed is a laborious process not normally done by hand. For that reason only an overview of the process is given here to assist the designer in reviewing and understanding the input and output from a typical computer program. There are choices of computational interval, storm length (if the 24-hour storm is not going to be used), and other "administrative" parameters that are peculiar to each computer program.

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The development of a runoff hydrograph for a watershed or one of many sub-basins within a more complex model involves the following steps:

- Development or selection of a design storm hyetograph. Often the <u>SCSNRCS TR-55</u> 24hour storm described in subsection 3.1.5.3 is used. This storm is recommended for use in Georgia.
- (2) Development of curve numbers and lag times for the watershed using the methods described in subsections 3.1.5.4, 2.1.5.5, and 3.1.5.6.
- (3) Development of a unit hydrograph from either the standard (peaking factor of 484) or coastal area (peaking factor of 300) dimensionless unit hydrographs. See discussion below.
- (4) Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the <u>SCSNRCS TR-55</u> rainfall-runoff equation (Equation 3.1.5).
- (5) Application of each increment of excess rainfall to the unit hydrograph to develop a series of runoff hydrographs, one for each increment of rainfall (this is called "convolution").
- (6) Summation of the flows from each of the small incremental hydrographs (keeping proper track of time steps) to form a runoff hydrograph for that watershed or sub-basin.

To assist the designer in using the <u>SCSNRCS TR-55</u> unit hydrograph approach with a peaking factor of 300, Figure 3.1.5-8 and Table 3.1.5-4 have been developed. The unit hydrograph is used in the same way as the unit hydrograph with a peaking factor of 484.

The procedure to develop a unit hydrograph from the dimensionless unit hydrographs in the table below is to multiply each time ratio value by the time-to-peak (Tp) and each value of q/q_u by q_u calculated as:

 $q_{U} = (PF A) / (T_{p})$

(3.1.13)

Where: qu= unit hydrograph peak rate of discharge (cfs)

PF = peaking factor (either 484 or 300)

 $A = area (mi^2)$

d = rainfall time increment (hr) T_p = time to peak = d/2 + 0.6 T_C (hr)

For ease of spreadsheet calculations, the dimensionless unit hydrographs for 484 and 300 can be approximated by the equation:

 $\frac{q}{q_u} = \left[\frac{t}{T_p} e^{\left(1-\frac{t}{T_p}\right)}\right]^X$ (3.1.14)

Where X is 3.79 for the PF=484 unit hydrograph and 1.50 for the PF=300 unit hydrograph.

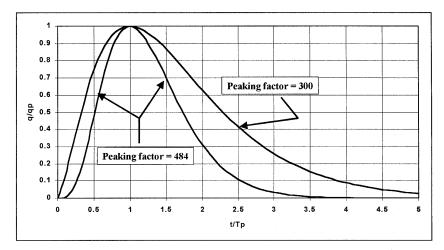


Figure 3.1.5-8 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

Table 3.1.5-4
 Dimensionless
 Unit
 Hydrographs

	48	84	3	00
t/Tt	q/qu	Q/Qp	q/qu	Q/Qp
0.0	0.0	0.0	0.0	0.0
0.1	0.005	0.000	0.122	0.006
0.2	0.046	0.004	0.296	0.019
0.3	0.148	0.015	0.469	0.041
0.4	0.301	0.038	0.622	0.070
0.5	0.481	0.075	0.748	0.105
0.6	0.657	0.125	0.847	0.144
0.7	0.807	0.186	0.918	0.186
0.8	0.916	0.255	0.966	0.231
0.9	0.980	0.330	0.992	0.277
1.0	1.000	0.406	1.000	0.324
1.1	0.982	0.481	0.993	0.370
1.2	0.935	0.552	0.974	0.415
1.3	0.867	0.618	0.945	0.459
1.4	0.786	0.677	0.909	0.501
1.5	0.699	0.730	0.868	0.541
1.6	0.611	0.777	0.823	0.579
1.7	0.526	0.817	0.775	0.615
1.8	0.447	0.851	0.727	0.649
1.9	0.376	0.879	0.678	0.680
2.0	0.312	0.903	0.631	0.710
2.1	0.257	0.923	0.584	0.737
2.2	0.210	0.939	0.539	0.762
2.3	0.170	0.951	0.496	0.785
2.4	0.137	0.962	0.455	0.806

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	484		300	
t/Tt	q/qu	Q/Qp	q/qu	Q/Qp
2.5	0.109	0.970	0.416	0.825
2.6	0.087	0.977	0.380	0.843
2.7	0.069	0.982	0.346	0.859
2.8	0.054	0.986	0.314	0.873
2.9	0.042	0.989	0.285	0.886
3.0	0.033	0.992	0.258	0.898
3.1	0.025	0.994	0.233	0.909
3.2	0.020	0.995	0.211	0.919
3.3	0.015	0.996	0.190	0.928
3.4	0.012	0.997	0.171	0.936
3.5	0.009	0.998	0.153	0.943
3.6	0.007	0.998	0.138	0.949
3.7	0.005	0.999	0.124	0.955
3.8	0.004	0.999	0.111	0.960
3.9	0.003	0.999	0.099	0.965
4.0	0.002	1.000	0.089	0.969
4.1			0.079	0.972
4.2			0.071	0.976
4.3			0.063	0.979
4.4			0.056	0.981
4.5			0.050	0.984
4.6			0.044	0.986
4.7			0.039	0.987
4.8			0.035	0.989
4.9			0.031	0.990
5.0			0.028	0.992
5.1			0.024	0.993
5.2			0.022	0.994
5.3			0.019	0.995
5.4			0.017	0.996
5.5			0.015	0.996
5.6			0.013	0.997
5.7			0.012	0.997
5.8			0.010	0.998
5.9			0.009	0.998
6.0			0.008	0.999
6.1			0.007	0.999
6.2			0.006	0.999
6.3			0.006	1.000

 Table 3.1.5-4
 Dimensionless Unit Hydrographs (continued)

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3.1.5.10 Example Problem 2

Compute the unit hydrograph for the 50-acre wooded watershed in example 3.1.5.8.

Computations

(1) Calculate T_p and time increment

The time of concentration (T_c) is calculated to be 20.24 minutes for this watershed. If we assume a computer calculation time increment (d) of 3 minutes then:

 $T_p = d/2 + 0.6T_c = 3/2 + 0.6 * 20.24 = 13.64$ minutes (0.227 hrs)

(2) Calculate q_{pu}

 $q_{pu} = PF A/T_p = (484 * 50/640)/(0.227) = 166 cfs$

For a PF of 300 q_{pu} would be:

 $q_{pu} = PF A/T_p = (300 * 50/640)/(0.227) = 103 cfs$

(3) Calculate unit hydrograph for both 484 and 300.

Based on spreadsheet calculations using equations 3.1.13 and 3.1.14, the table below has been derived.

Tii	me	48	34	30	00
t/Tp	time (min)	q/qpu	Q	q/qpu	q
0	0	0	0.00	0	0.00
0.22	3.0	0.06	10.26	0.33	34.18
0.44	6.0	0.37	61.74	0.68	69.60
0.66	9.0	0.75	124.79	0.89	91.99
0.88	12.0	0.97	161.37	0.99	101.85
1	13.64	1	166	1	103
1.10	15.0	0.98	163.39	0.99	102.35
1.32	18.0	0.85	141.70	0.94	96.74
1.54	21.0	0.66	110.45	0.85	87.64
1.76	24.0	0.48	79.61	0.75	76.98
1.98	27.0	0.33	54.06	0.64	66.03
2.20	30.0	0.21	35.02	0.54	55.59
2.42	33.0	0.13	21.84	0.45	46.10
2.64	36.0	0.08	13.19	0.37	37.76
2.86	39.0	0.05	7.77	0.30	30.60
3.08	42.0	0.03	4.47	0.24	24.58
3.30	45.0	0.02	2.52	0.19	19.60
3.52	48.0	0.01	1.40	0.15	15.52
3.74	51.0	0.00	0.76	0.12	12.21
3.96	54.0	0.00	0.41	0.09	9.57
4.18	57.0	0.00	0.22	0.07	7.46
4.40	60.0	0.00	0.12	0.06	5.79
4.62	63.0	0.00	0.06	0.04	4.48
4.84	66.0	0.00	0.03	0.03	3.45
5.06	69.0	0.00	0.02	0.03	2.65
5.28	72.0	0.00	0.01	0.02	2.03

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Ti	me	48	34	30	00
t/Tp	time (min)	q/qpu	Q	q/qpu	q
5.50	75.0	0.00	0.00	0.02	1.55
5.72	78.0			0.01	1.18
5.94	81.0			0.01	0.90
6.16	84.0			0.01	0.68
6.38	87.0			0.01	0.52
6.60	90.0			0.00	0.39
6.82	93.0			0.00	0.30
7.04	96.0			0.00	0.22
7.26	99.0			0.00	0.17
7.48	102.0			0.00	0.13
7.70	105.0			0.00	0.09
7.92	108.0			0.00	0.07
8.14	111.0			0.00	0.05
8.36	114.0			0.00	0.04
8.58	117.0			0.00	0.03
8.80	120.0			0.00	0.02
9.01	123.0			0.00	0.02
9.23	126.0			0.00	0.01
9.45	129.0			0.00	0.01
9.67	132.0			0.00	0.01
9.89	135.0			0.00	0.01
10.11	138.0			0.00	0.00

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3.1.6 U.S. Geological Survey Peak Flow and Hydrograph Method

3.1.6.1 Introduction

For the past 20 years the USGS has been collecting rain and streamflow data at various sites within the Atlanta metropolitan area and throughout the state of Georgia. The data from these efforts have been used to calibrate a USGS rainfall-runoff model. The U.S. Geological Survey Model was then used to develop peak discharge regression equations for the 2-, 5-, 10-, 25-, 50- and 100-year floods. In addition, the USGS used the statewide database to develop a dimensionless hydrograph that can be used to simulate flood hydrographs from rural and urban streams in Georgia. This USGS information is specific to geographical regions of Georgia. Figure 3.1.6-1 shows the locations of these different regions.

3.1.6.2 Application

The USGS regression method is used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows for larger drainage areas:

- 25 acres and larger for peak flow estimation
- 128 acres and larger for hydrograph generation

The USGS method can be used for most design applications, including the design of storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches and open channels, and energy dissipators.

3.1.6.3 Peak Discharge Equations

For a complete description of the USGS regression equations presented below, consult the latest USGS publications regarding both rural and urban flood frequencies. Based on the current USGS publications, a watershed is determined to be urban if 10% or more of the watershed basin is impervious. USGS regression equations have been removed from this Manual due to their periodic update. Check the USGS publications website for the most recent publications and regression equations. At the time of this Manual update, the following publications were available:

- <u>Urban Method</u>: Methods for Estimating the Magnitude and Frequency of Floods for Urban and Small, Rural Streams in Georgia, South Carolina, and North Carolina, 2011 (http://pubs.usgs.gov/sir/2014/5030/)
- <u>Rural Method</u>: Magnitude and Frequency of Rural Floods in the Southeastern United States, 2006: Volume 1, Georgia (<u>http://pubs.usgs.gov/sir/2009/5043/</u>)

In addition to the publications, USGS has also developed a spreadsheet tool to assist the designer in computing flood-frequency characteristics, for both the urban and rural methods. The spreadsheets are downloadable, using the links provided above, as Microsoft Excel documents.

3.1.6.4 Peak Discharge Limitations for Urban and Rural Basins

Each USGS regression equation uses variables that represent the following:

- Drainage area (DRNAREA, DA, or A)
- Percent impervious cover; and
- Percent developed land

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The most recent version of the USGS publications should also be used to verify the limitations of these variables within the peak discharge equations. These equations should not be used on any variables which have physical characteristics outside of their appropriate range.

Figure 3.1.6-1 USGS Hydrologic Regions in Georgia (Source: USGS, 2011)

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3.1.6.5 Hydrographs

The USGS has developed a dimensionless hydrograph for Georgia streams having drainage areas of less than 500 mi². This dimensionless hydrograph can be used to simulate flood hydrographs for rural and urban streams throughout the State of Georgia. For a complete description of the USGS dimensionless hydrograph, consult the USGS publication *Simulation of Flood Hydrographs for Georgia Streams, Water-Resources Investigation Report 86-4004.* Table 3.1.6-2 lists the time and discharge ratios for the dimensionless hydrograph.

Time Ratio	Discharge Ratio	Time Ratio	Discharge Ratio
<u>(t/T_L)</u>	(Q/Q_p)	<u>(t/T_L)</u>	<u>(Q/Q</u> _p)
0.25	0.12	1.35	0.62
0.30	0.16	1.40	0.56
0.35	0.21	1.45	0.51
0.40	0.26	1.50	0.47
0.45	0.33	1.55	0.43
0.50	0.40	1.60	0.39
0.55	0.49	1.65	0.36
0.60	0.58	1.70	0.33
0.65	0.67	1.75	0.30
0.70	0.76	1.80	0.28
0.75	0.84	1.85	0.26
0.80	0.90	1.90	0.24
0.85	0.95	1.95	0.22
0.90	0.98	2.00	0.20
0.95	1.00	2.05	0.19
1.00	0.99	2.10	0.17
1.05	0.96	2.15	0.16
1.10	0.92	2.20	0.15
1.15	0.86	2.25	0.14
1.20	0.80	2.30	0.13
1.25	0.74	2.35	0.12
1.30	0.68	2.40	0.11

The lag time equations for calculating the dimensionless hydrograph are:

North of the Fall Line (rural):	
$T_{L} = 4.64 A^{0.49} S^{-0.21}$	(3.1.15)
South of the Fall Line (rural):	
T _L = 13.6A ^{0.43} S ^{-0.31}	(3.1.16)
Regions 1, 2 and 3 (urban):	
T _L = 7.86A ^{0.35} TIA ^{-0.22} S ^{-0.31}	(3.1.17)

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Region 4 (urban):

TL = 6.10A^{0.35}TIA^{-0.22}S^{-0.31}

(3.1.18)

 $\begin{array}{ll} \mbox{Where:} & T_L = lag time (hours) \\ & A = drainage area (mi^2) \\ & S = main channel slope (ft/mi) \\ & TIA = total impervious area (percent) \\ \end{array}$

Using these lag time equations and the dimensionless hydrograph, a runoff hydrograph can be determined after the peak discharge is calculated.

3.1.6.6 Hydrograph Limitations

Following are the limitations of the variables within the lag time equations. The lag time equation should not be used for drainage areas that have physical characteristics outside the limits listed below:

Physical Characteristics	Minimum	Maximum	<u>Units</u>
North of the Fall Line (rural) A - Drainage Area S - Main Channel Slope	0.3 5.0	500 200	mi ² feet per mile
South of the Fall Line (rural) A - Drainage Area S - Main Channel Slope	0.2 1.3	500 60	mi ² feet per mile
Regions 1, 2 & 3 (urban) A - Drainage Area S - Main Channel Slope TIA - Total Impervious Area	0.04 9.4 1.0	19.1 772.0 61.6	mi ² feet per mile percent
Region 4 (urban) A - Drainage Area S - Main Channel Slope TIA - Total Impervious Area	0.12 19.4 6.1	2.9 110.0 42.4	mi ² feet per mile percent

3.1.6.7 Example Problem

For the 100-year flood, calculate the peak discharge for rural and developed conditions for the following drainage area located in Region 1 in the Atlanta metro area. For the developed conditions, develop the flood hydrograph for this drainage area.

Drainage Area = 175 acres = 0.273 mi²

- Main Channel Slope = 117 ft/mi
- Percent Impervious Area = 32%

Peak Discharge Calculations

100-year Rural Peak Discharge:

 $Q_{100} = 776(DA)^{0.594}$ (Taken from the most recent publication) $Q_{100} = 776(0.273)^{0.594} = 359$ cfs 100-year Developed (Urban) Peak Flow:

 $Q_{100} = 753(DRNAREA)^{0.8038}10^{(0.0024*IMPNLCD06)}$ (Taken from the most recent publication) $Q_{100} = 753(0.273)^{0.8038}10^{(0.0024*32)} = 317$ cfs

Flood Hydrograph Calculations

Lag Time Calculations

 $T_L = 7.86A^{0.35}TIA^{-0.22}S^{-0.31} = 7.86 \ (0.273)^{0.35} \ (32)^{-0.22} \ (117)^{-0.31} = 0.53 \ hours$

Hydrograph Calculations

Using the dimensionless USGS hydrograph given in Table 3.1.6-2, the following calculations are done to determine the ordinates of the flood hydrograph.

Time (t) = $t/T_L \times 0.53$ t/T_L from Table 3.1.6-2

Discharge (Q) = $Q/Q_p \times 561$ Q/Q_p from Table 3.1.6-2

Coordinates for the flood hydrograph are given in Table 3.1.6-4 on the next page.

ime Ratio	Time (t)	Discharge Ratio	Discharge
<u>(t/T_L)</u>	Hours	<u>(Q/Q</u> _p)	(cfs)
0.25	0.13	0.12	67
0.30	0.16	0.16	90
0.35	0.19	0.21	118
0.40	0.21	0.26	146
0.45	0.24	0.33	185
0.50	0.27	0.40	224
0.55	0.29	0.49	275
0.60	0.32	0.58	325
0.65	0.34	0.67	376
0.70	0.37	0.76	426
0.75	0.40	0.84	471
0.80	0.42	0.90	505
0.85	0.45	0.95	533
0.90	0.48	0.98	550
0.95	0.50	1.00	561
1.00	0.53	0.99	555
1.05	0.56	0.95	539
1.10	0.58	0.92	516
1.15	0.61	0.86	482
1.20	0.64	0.80	402
1.25	0.66	0.74	449
1.30	0.69	0.68	381
	0.89	0.62	348
1.35			
1.40	0.74	0.56	314
1.40 1.50	0.77 0.80	0.51 0.47	286 264
1.50	0.80	0.43	264
1.60	0.85	0.39	219
1.65	0.87	0.36	202
1.70	0.90	0.33	185
1.75	0.93	0.30 0.28	168
1.80	0.95		157
1.85	0.98	0.26	146
1.90	1.01	0.24	135
1.95	1.03	0.22	123
2.00	1.06	0.20	112
2.05	1.09	0.19	107
2.10	1.11	0.17	95
2.15	1.14	0.16	90
2.20	1.17	0.15	84
2.25	1.19	0.14	79
2.30	1.22	0.13	73
2.35 2.40	1.25 1.27	0.12 0.11	67 62

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3.1.7 Water Quality Volume and Peak Flow

3.1.7.1 Water Quality Volume Calculation

The Water Quality Volume (WQ_v) is the retention or treatment volume required to remove a significant percentage of the stormwater pollution load, defined in this Manual as an 80% removal of the average annual post-development total suspended solids (TSS) load. This is achieved by intercepting and retaining or treating a portion of the runoff from all storms and all the runoff from 85% of the storms that occur on average during the course of a year.

The water quality treatment volume is calculated by multiplying the 85^{th} percentile annual rainfall event by the volumetric runoff coefficient (R_v) and the site area. R_v is defined as:

$$R_v = 0.05 + 0.009(I)$$

Where: I = percent of impervious cover (%)

For the state of Georgia, the average 85^{th} percentile annual rainfall event is 1.2 inches. Therefore, WQ_v is calculated using the following formula:

$$WQ_{v} = \frac{1.2 R_{v} A}{12}$$
(3.1.20)

 $\begin{array}{l} \mbox{Where: } WQ_v = \mbox{water quality volume (acre-feet)} \\ R_v = \mbox{volumetric runoff coefficient} \\ A = \mbox{total drainage area (acres)} \end{array}$

 WQ_v can be expressed in inches simply as $1.2(R_v) = Q_{wv}$

3.1.7.2 Water Quality Volume Peak Flow Calculation

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as for sand filters and infiltration trenches. An arbitrary storm would need to be chosen using the Rational Method, and conventional <u>SCSNRCS TR-55</u> methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2 inches. This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the treatment practice due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

The following procedure can be used to estimate peak discharges for small storm events. It relies on the Water Quality Volume and the simplified peak flow estimating method above. A brief description of the calculation procedure is presented below.

(Step 1) Using WQ_v, a corresponding Curve Number (CN) is computed utilizing the following equation:

$CN = 1000/[10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25 Q_{wv}P)^{1/2}]$

Where, P = rainfall, in inches (use 1.2 inches for the Water Quality Storm in Georgia) Q_{wv} = Water Quality Volume, in inches (1.2R_v)

(Step 2) Once a CN is computed, the time of concentration (t_c) is computed (based on the methods described in this section).

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- (Step 3) Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_{wq}) for the water quality storm event is computed using a slight modification of the Simplified <u>SCSNRCS TR-55</u> Peak Runoff Rate Estimation technique of subsection 3.1.5.7. Use appropriate rainfall distribution type (either Type II or Type III in Georgia).
 - Read initial abstraction (Ia), compute I_a/P
 - Read the unit peak discharge (q_u) for appropriate tc
 - Using WQ_v , compute the peak discharge (Q_{wq})

 $\mathbf{Q}_{wq} = \mathbf{q}_u * \mathbf{A} * \mathbf{Q}_{wv}$

where Q_{wq} = the water quality peak discharge (cfs) q_u = the unit peak discharge (cfs/mi²/inch) A = drainage area (mi²) Q_{wv} = Water Quality Volume, in inches (1.2R_v)

3.1.7.3 Example Problem

•

Using the data and information from the example problem in subsection 3.1.5.8 calculate the water quality volume and the water quality peak flow.

Calculate water quality volume (WQv)

Compute volumetric runoff coefficient, Rv

 $R_V = 0.05 + (0.009) = 0.05 + (0.009)(18/50 \times 100\%) = 0.37$

Compute water quality volume, WQ_v $WQ_v = 1.2(R_v)(A)/12 = 1.2(.37)(50)/12 = 1.85$ acre-feet

Calculate water quality peak flow

Compute runoff volume in inches, $\mathsf{Q}_{wv}\!\!:$

 $Q_{wv} = 1.2 R_v = 1.2 * 0.37 = 0.44$ inches

Computer curve number:

 $\begin{array}{l} \text{CN} = 1000/[10 + 5\text{P} + 10\text{Q} - 10(\text{Q}_{\text{WV}}^2 + 1.25 \text{ Q}_{\text{WV}}\text{P})\%] \\ \text{CN} = 1000/[10 + 5^*1.2 + 10^*0.252 - 10(0.252^2 + 1.25^*0.252^*1.2)\%] \\ = 84 \end{array}$

 $t_c = 0.34$ (computed previously)

$$\begin{split} S &= 1000/CN - 10 = 1000/84 - 10 = 1.90 \text{ inches} \\ 0.2S &= I_a = 0.38 \text{ inches} \\ I_a/P &= 0.38/1.2 = 0.317 \end{split}$$

Find qu:

From Figure 3.1.5-6 for $I_a/P = 0.317$ $q_u = 535 \text{ cfs/mi}^2/\text{in}$

Compute water quality peak flow:

 $Qwq = q_u * A * Q_{wv} = 535 * 50/640 * 0.44 = 18.4 cfs$

3.1.8 Water Balance Calculations

3.1.8.1 Introduction

Water balance calculations help determine if a drainage area is large enough, or has the right characteristics, to support a permanent pool of water during average or extreme conditions. When in doubt, a water balance calculation may be advisable for any best management practice that maintains a permanent volume of stormwater.

The details of a rigorous water balance are beyond the scope of this manual. However, a simplified procedure is described herein that will provide an estimate of pool viability and point to the need for more rigorous analysis. Water balance can also be used to help establish planting zones in a wetland design.

3.1.8.2 Basic Equations

Water balance is defined as the change in volume of the permanent pool resulting from the total inflow minus the total outflow (actual or potential):

$$\Delta \quad \mathbf{V} = \Sigma \mathbf{I} - \Sigma \mathbf{O}$$

Where:

Where:

The inflows consist of rainfall, runoff and baseflow into the pond. The outflows consist of infiltration, evaporation, evapotranspiration, and surface overflow out of the best management practice. Equation 3.1.21 can be changed to reflect these factors.

$\Delta V = P + Ro + Bf - I - E - Et - Of \qquad (3.1.22)$

 $\begin{array}{l} \mathsf{P} = \mathsf{precipitation} \ (\mathsf{ft}) \\ \mathsf{Ro} = \mathsf{runoff} \ (\mathsf{ac-ft}) \\ \mathsf{Bf} = \mathsf{baseflow} \ (\mathsf{ac-ft}) \\ \mathsf{I} = \mathsf{infiltration} \ (\mathsf{ft}) \\ \mathsf{E} = \mathsf{evaporation} \ (\mathsf{ft}) \\ \mathsf{Et} = \mathsf{evapotranspiration} \ (\mathsf{ft}) \\ \mathsf{Of} = \mathsf{overflow} \ (\mathsf{ac-ft}) \end{array}$

Rainfall (P) - Rainfall values can be obtained from NOAA Atlas 14 at:

http://hdsc.nws.noaa.gov/hdsc/pfds/

Monthly values are commonly used for calculations of values over a season. Rainfall is then the direct amount that falls on the permanent pool surface for the period in question. When multiplied by the permanent pool surface area (in acres) it becomes acre-feet of volume.

<u>Runoff (R_o)</u> – Runoff is equivalent to the rainfall for the period times the "efficiency" of the watershed, which is equal to the ratio of runoff to rainfall. In lieu of gage information, Q/P can be estimated one of several ways. The best method would be to perform long-term simulation modeling using rainfall records and a watershed model. Two other methods have been proposed.

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(3.1.21)

Equation 3.1.19 gives a ratio of runoff to rainfall volume for a particular storm. If it can be assumed that the average storm that produces runoff has a similar ratio, then the R_v value can serve as the ratio of rainfall to runoff. Not all storms produce runoff in an urban setting. Typical initial losses (often called "initial abstractions") are normally taken between 0.1 and 0.2 inches. When compared to the rainfall records in Georgia, this is equivalent of about a 10% runoff volume loss. Thus a factor of 0.9 should be applied to the calculated R_v value to account for storms that produce no runoff. Equation 3.1.23 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

$$Q = 0.9 PR_v$$

(3.1.23)

Where: P = precipitation (in) Q = runoff volume (in) R_v = volumetric runoff coefficient [see equation 3.1.19]

Ferguson (1996) has performed simulation modeling in an attempt to quantify an average ratio on a monthly basis. For the Atlanta area he has developed the following equation:

Q = 0.235P/S ^{0.64} – 0.161	(3.1.24)
--------------------------------------	----------

Where:

Where:

P = precipitation (in) Q = runoff volume (in) S = potential maximum retention (in) [see equation 3.1.5]

<u>Baseflow (Bf)</u> – Most stormwater ponds and wetlands have little, if any, baseflow, as they are rarely placed across perennial streams. If so placed, baseflow must be estimated from observation or through theoretical estimates. Methods of estimation and baseflow separation can be found in most hydrology textbooks.

<u>Infiltration (I)</u> – Infiltration is a very complex subject and cannot be covered in detail here. The amount of infiltration depends on soils, water table depth, rock layers, surface disturbance, the presence or absence of a liner in the pond, and other factors. The infiltration rate is governed by the Darcy equation as:

I = infiltration (ac-ft/day)A = cross sectional area through which the water infiltrates (ac) $K_h = saturated hydraulic conductivity or infiltration rate (ft/day)$ $G_h = hydraulic gradient = pressure head/distance$

 $G_{\rm h}$ can be set equal to 1.0 for pond bottoms and 0.5 for pond sides steeper than about 4:1. Infiltration rate can be established through testing, though not always accurately. As a first cut estimate Table 3.1.8-2 can be used.

Material	Hydraulic Conductivity					
Material	in/hr	ft/day				
ASTM Crushed Stone No. 3	50,000	100,000				
ASTM Crushed Stone No. 4	40,000	80,000				
ASTM Crushed Stone No. 5	25,000	50,000				
ASTM Crushed Stone No. 6	15,000	30,000				
Sand	8.27	16.54				
Loamy sand	2.41	4.82				
Sandy loam	1.02	2.04				
Loam	0.52	1.04				
Silt loam	0.27	0.54				
Sandy clay loam	0.17	0.34				
Clay loam	0.09	0.18				
Silty clay loam	0.06	0.12				
Sandy clay	0.05	0.10				
Silty clay	0.04	0.08				
Clay	0.02	0.04				

<u>Evaporation (E)</u> – Evaporation is from an open water surface. Evaporation rates are dependent on differences in vapor pressure, which, in turn, depend on temperature, wind, atmospheric pressure, water purity, and shape and depth of the pond. It is estimated or measured in a number of ways, which can be found in most hydrology textbooks. Pan evaporation methods are also used though there are only two pan evaporation sites active in Georgia (Lake Allatoona and Griffin). A pan coefficient of 0.7 is commonly used to convert the higher pan value to the lower lake values.

Table 3.1.8-3 gives pan evaporation rate distributions for a typical 12-month period based on pan evaporation information from five stations in and around Georgia. Figure 3.1.8-1 depicts a map of annual free water surface (FWS) evaporation averages for Georgia based on a National Oceanic and Atmospheric Administration (NOAA) assessment done in 1982. FWS evaporation differs from lake evaporation for larger and deeper lakes, but can be used as an estimate of it for the type of structural stormwater ponds and wetlands being designed in Georgia. Total annual values can be estimated from this map and distributed according to Table 3.1.8-3.

Table	Table 3.1.8-2 Evaporation Monthly Distribution										
J	F	м	Α	м	J	J	Α	S	0	N	D
3.2%	4.4%	7.4%	10.3%	12.3%	12.9%	13.4%	11.8%	9.3%	7.0%	4.7%	3.2%

Evapotranspiration (Et). Evapotranspiration consists of the combination of evaporation and transpiration by plants. The estimation of Et for crops in Georgia is well documented and has become standard practice. However, for wetlands the estimating methods are not documented, nor are there consistent studies to assist the designer in estimating the demand wetland plants would put on water volumes. Values for turf are given in Table 3.1.8-1 based on the Blaney-Criddle method. Literature values for various places in the United States vary around the free water surface lake evaporation values. Estimating Et only becomes important when wetlands are being designed and emergent vegetation covers a significant portion of the pond surface. In these cases conservative estimates of lake evaporation should be compared to crop-based Et estimates can be obtained from typical hydrology textbooks or from the web sites mentioned above.

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<u>Overflow (Of)</u> – Overflow is considered as excess runoff, and in water balance design is either not considered, since the concern is for average values of precipitation, or is considered lost for all volumes above the maximum pond storage. Obviously, for long-term simulations of rainfall-runoff, large storms would play an important part in pond design.

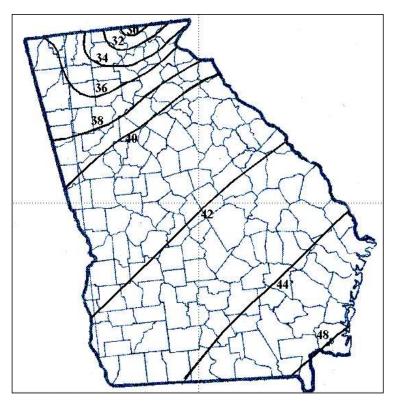


Figure 3.1.8-1 Average Annual Free Water Surface Evaporation (in inches) (Source: NOAA, 1982)

3.1.8.3 Example Problem

Austin Acres, a 26-acre site in Augusta, is being developed along with an estimated 0.5-acre surface area pond. There is no baseflow. The desired pond volume to the overflow point is 2 acre-feet. Will the site be able to support the pond volume? From the basic site data we find that the site is 75% impervious with sandy clay loam soil.

- From equation 3.1.19, $R_v = 0.05 + 0.009$ (75) = 0.73. With the correction factor of 0.9 the watershed efficiency is 0.65.
- The annual lake evaporation from Figure 3.1.8-1 is about 42 inches.
- For a sandy clay loam the infiltration rate is I = 0.34 ft/day (Table 3.1.8-2).
- From a grading plan it is known that about 10% of the total pond area is sloped greater than 1:4.
- Monthly rainfall for Augusta was found from the Web site provided above.

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Table 3.1.8-4 shows summary calculations for this site for each month of the year.

Information f	for Austin Acres
	Information f

Shower and	. d	. ŧ.,		A	M	J		Α.	s	0	N	D
Daysimo	31	28	31	30	. 31	30	31.	31	30	. 31	30	. 3
Precipitation (in)	4.05	A.27	4.65	5.31	3.77	4.13	8.24	45	3.02	2.84	2.48	3.4
Ewap Dist	3.2%	4.4%	7.4%	10.3%	12.3%	12.9%	13.4%	11.8%	9.3%	7.0%	4.7%	3.29
Ro (ac-f0	5.70	6.01	6.65	4.66	5.31	5.82	5.97	6.34	4.25	4.00	3.49	4.7
P (ac.fl)	0.12	0.18	0.19	0.14	0.18	0.17	0.18	0.10	0.13	0.12	0.10	0.1
E (ac-R)	0.06	D D8	0.13	0.18	0.22	0.23	0.23	0.21	0.1B	0.12	0.08	0.0
1 (ic-ft)	5.01	4.62	6.01	4.85	5.01	4.05	5.01	5.01	4.85	6.01	4.05	-5.0
Balance (ac-N)	0.8t	1.59	161	0.29	0.24	0.92	19.91	13f	-0.63	+1.01	-1:38	-8.13
Running Balance (ac.4)	0.81	2.00	2.00	1.77	2.00	2.00	2.00	2.00	1.37	0.36	0.00	0.00

Explanation of Table:

- 1. Months of year
- 2. Days per month
- 3. Monthly precipitation from web site is shown in Figure 3.1.8-2.
- 4. Distribution of evaporation by month from Table 3.1.8-3.
- 5. Watershed efficiency of 0.65 times the rainfall and converted to acre-feet.
- 6. Precipitation volume directly into pond equals precipitation depth times pond surface area divided by 12 to convert to acre-feet
- 7. Evaporation equals monthly percent of 42 inches from line 4 converted to acre-feet
- Infiltration equals infiltration rate times 90% of the surface area plus infiltration rate times 0.5 (banks greater than 1:4) times 10% of the pond area converted to acre-feet
- 9. Lines 5 and 6 minus lines 7 and 8
- 10. Accumulated total from line 10 keeping in mind that all volume above 2 acre-feet overflows and is lost in the trial design

It can be seen that for this example the pond has potential to go dry in winter months. This can be remedied in a number of ways including compacting the pond bottom, placing a liner of clay or geosynthetics, and changing the pond geometry to decrease surface area.

			Tota	1 Predi	pital	tion			Sno	W.	#Day:	Prec:	ip.
	Nean						Day Max	Nean	High-	-Yr	=>.10	=>.50	*>1.
Ja	4.05	8,91	87	0.75	81	2.78	25/1978	D.3	2.3	88	7	3	1
Fe	4.27	7.67	61	0.69	68	3,50	5/1985	1.0	14.0	73	77576865446	3	1 1 1 1 1 1 1 1 1 1 1 1
Ha.	4.65	11.92	80	0.88	68	5.31	10/1967	0.0	1.1	80	7	3 2 2	1
Ap.	3.31	8.43	61	0.60	70	2,71	27/1961	0.0	0.0	0	5	2	1
Ma	3,77	9,61	79	1.57	87	4.44	31/1981	0.0	0.0	0	7		1
Jn	4.13	8.84	89	0.60	64	2,95	12/1964	0.0	0.0	0	6	3	1
31	4.24	11.43	67	1.02	87	3.67	21/1979	0.0	0.0	0	8	2	1
<u>àu</u>	4.50	11.34	86	0.65	80	5.95	29/1964	0.0	0.0	0	6	3	1
Se.	3.02	9.51	75	0.31	84	4.55	20/1975	0.0	0.0	0	5	2	1
óe.	2.84	14,82	90	0.01	63	5,32	12/1990	0.0	0.0	0	4	2	1
No	2.48	7.76	85	0.57	73	3.43	22/1985	0.0	0.0	0	4	2	1
De	3.40	8,65	81	0.96	80	2,89	16/1970	0.0	0.4	71	6	J Z 3 2 2 3 3	1
<u>kn</u>	44.66	66.04	64	32.96	78	5.95	29/08/64	1.4	14.4	73	71	30	13
Wi	11.71	20.26	87	5.62	86	3.50	5/02/85	1.3	14.4	73	20	8	3
Sp	11.73	19,93	84	4.00	85	5.31	10/03/67	0.0	1.1	80	18	в	3
su	12.87	24.89	64	7.08	80	5.95	29/08/64	0.0	0.0	0	20	8 8 9 5	3344
Fa	8.35	18.50	90	1.96	84	5.32	12/10/90	0.0	0.0	0	12	5	2

Figure 3.1.8-2 Augusta Precipitation Information

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3.1.9 Downstream Hydrologic Assessment

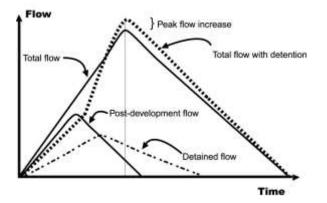
The purpose of the overbank flood protection and extreme flood protection criteria is to protect downstream properties from flood increases due to upstream development. These criteria require the designer to control peak flow at the outlet of a site such that post-development peak discharge equals pre-development peak discharge. It has been shown that in certain cases this does not always provide effective water quantity control downstream from the site and may actually exacerbate flooding problems downstream. The reasons for this have to do with (1) the timing of the flow peaks, and (2) the total increase in volume of runoff. Further, due to a site's location within a watershed, there may be very little reason for requiring overbank flood control from a particular site. This section outlines a suggested procedure for determining the impacts of post-development stormwater peak flows and volumes on downstream flows that a community may require as part of a development to over-detain to protect downstream properties or may even warrant a reduction/elimination of detention because of the timing of peak discharges within the watershed.

3.1.9.1 Reasons for Downstream Problems

Flow Timing

If water quantity control (detention) structures are indiscriminately placed in a watershed and changes to the flow timing are not considered, the structural control may actually increase the peak discharge downstream. The reason for this may be seen in Figure 3.1.9-1. The peak flow from the site is reduced appropriately, but the timing of the flow is such that the combined detained peak flow (the larger dashed triangle) is actually higher than if no detention were required. In this case, the shifting of flows to a later time brought about by the detention pond actually makes the downstream flooding worse than if the post-development flows were not detained.





Increased Volume

An important impact of new development is an increase in the total runoff volume of flow. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with downstream tributaries to increase the downstream peak flows.

Figure 3.1.9-2 illustrates this concept. The figure shows the pre- and post-development hydrographs from a development site (Tributary 1). The post-development runoff hydrograph meets the flood protection criteria (i.e., the post-development peak flow is equal to the pre-

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development peak flow at the outlet from the site). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than pre-development combined flow. This is because the increased volume and timing of runoff from the developed site increases the combined flow and flooding downstream. In this case, the detention volume would have to have been increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

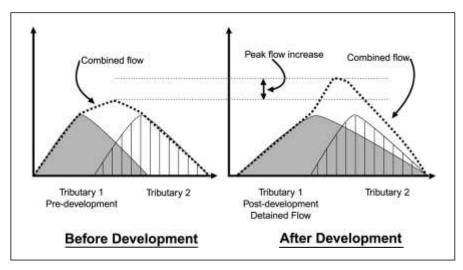


Figure 3.1.9-2 Effect of Increased Post-Development Runoff Volume with Detention on a Downstream Hydrograph

3.1.9.2 The Ten-Percent Rule

In this Manual the "ten percent" criterion has been adopted as the most flexible and effective approach for ensuring that stormwater quantity detention ponds actually attempt to maintain predevelopment peak flows throughout the system downstream.

The ten-percent rule recognizes the fact that a structural control providing detention has a "zone of influence" downstream where its effectiveness can be felt. Beyond this zone of influence the structural control becomes relatively small and insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, that zone of influence is considered to be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. For example, if the structural control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater.

Typical steps in the application of the ten-percent rule are:

- (1) Determine the target peak flow for the site for predevelopment conditions.
- (2) Using a topographic map determine the lower limit of the zone of influence (10% point).
- (3) Using a hydrologic model determine the pre-development peak flows and timing of those peaks at each tributary junction beginning at the pond outlet and ending at the next tributary junction beyond the 10% point.
- (4) Change the land use on the site to post-development and rerun the model.

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- (5) Design the structural control facility such that the overbank flood protection (25-year) postdevelopment flow does not increase the peak flows at the outlet and the determined tributary junctions.
- (6) If it does increase the peak flow, the structural control facility must be redesigned or one of the following options considered:
 - Control of the overbank flood volume (Q_{p25}) may be waived by the local authority saving the developer the cost of sizing a detention basin for overbank flood control. In this case the ten-percent rule saved the construction of an unnecessary structural control facility that would have been detrimental to the watershed flooding problems. In some communities this situation may result in a fee being paid to the local government in lieu of detention. That fee would go toward alleviating downstream flooding or making channel or other conveyance improvements.
 - Work with the local government to reduce the flow elevation through channel or flow conveyance structure improvements downstream.
 - Obtain a flow easement from downstream property owners to the 10% point.

Even if the overbank flood protection requirement is eliminated, the water quality treatment (WQ_v), channel protection (CP_v), and extreme flood protection (Q_t) criteria will still need to be addressed.

3.1.9.3 Example Problem

Figure 3.1.9-3 illustrates the concept of the ten-percent rule for two sites in a watershed.

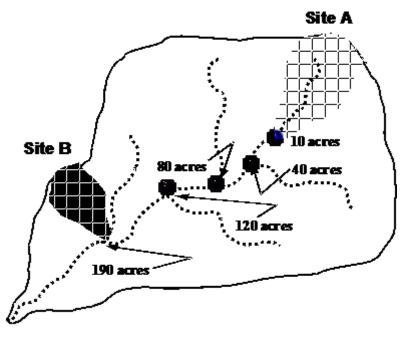


Figure 3.1.9-3 Example of the Ten-Percent Rule

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Discussion

Site A is a development of 10 acres, all draining to a wet ED stormwater pond. The overbank flooding and extreme flood portions of the design are going to incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked "80 acres." The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The assumption is that if there is no peak flow increase at the 80-acre point then there will be no increase through the next stream reach downstream through the 10% point (100 acres) to the 120-acre point. The designer constructs a simple HEC-1 model of the 80-acre areas using single existing condition sub-watersheds for each tributary. Key detention structures existing in other tributaries must be modeled. An approximate curve number is used since the *actual* peak flow is not key for initial analysis; only the increase or decrease is important. The accuracy in curve number determination is not as significant as an accurate estimate of the time of concentration. Since flooding is an issue downstream, the pond is designed (through several iterations) until the peak flow does not increase at junction points downstream to the 80-acre point.

Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin with care taken to estimate the time of concentration for input into the TR-20 model of the watershed. The model shows that a detention facility, in this case, will actually <u>increase</u> the peak flow in the stream.

3.2 METHODS FOR ESTIMATING STORMWATER VOLUME REDUCTION – DESIGN WORKSHEET

3.2.1 Volume Reduction Design

3.2.1.1 IntroductionIntroduction

The runoff reduction approach to addressing Water Quality requirements is discussed in detail in Volume 1 and sections 2.2.2 and 2.2.3 of Volume 2. Best management practices that incorporate stormwater runoff reduction are provided in Chapter 4. Within each BMP section of the manual, design steps have been provided for each unique application and practice.

As a Georgia Stormwater Management Manual design aid, a site development review tool has been created to aid in the design and documentation of stormwater management requirements. This tool can be accessed and downloaded at the following location:

http://www.northgeorgiawater.org/stormwater/tools-for-local-governments

Appendix E provides a more detailed discussion of the tool itself and also includes design examples for different development scenarios. Within the tool, an "Instructions" and "Tool Flowchart" tab have been created to provide specific input requirements and responsibilities of the end user. These same instructions and workflow process are particularly beneficial for local communities who will ultimately review the information as part of their stormwater management review process.

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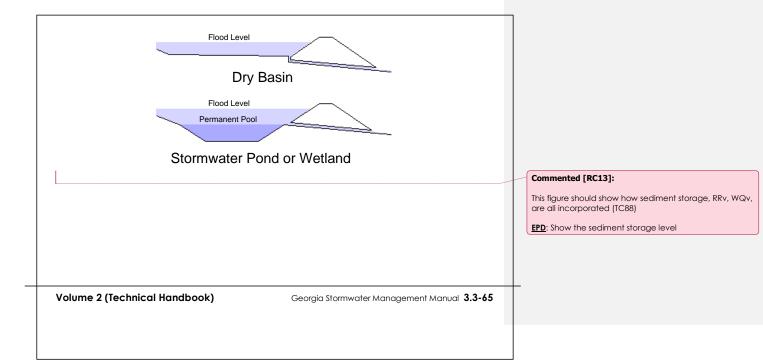
3.3 STORAGE DESIGN

3.3.1 General Storage Concepts

3.3.1.1 Introduction

This section provides general guidance on stormwater runoff storage for meeting stormwater management control requirements (i.e., water quality treatment, downstream channel protection, overbank flood protection, and extreme flood protection).

Storage of stormwater runoff within a stormwater management system is essential to providing the extended detention of flows for water quality treatment and downstream channel protection, as well as for peak flow attenuation of larger flows for overbank and extreme flood protection. Runoff storage can be provided within an on-site system through the use of best management practices and/or nonstructural features and landscaped areas. Figure 3.3.1-1 illustrates various storage facilities that can be considered for a development site.



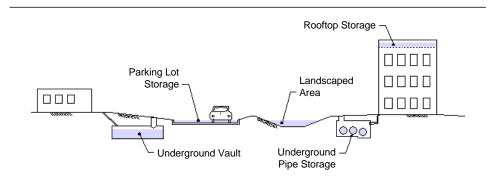


Figure 3.3.1-1 Examples of Typical Stormwater Storage Facilities

Stormwater *detention* is used to reduce the peak discharge and detain runoff for a specified short period of time. Detention volumes are designed to completely drain after the design storm has passed. Detention is used to meet overbank flood protection criteria, and extreme flood criteria where required.

Extended detention (ED) is used to drain a runoff volume over a specified period of time, typically 24 hours, and is used to meet channel protection criteria. Some structural control designs (wet ED pond, micropool ED pond, and shallow ED marsh) also include extended detention storage of a portion of the water quality volume.

Retention facilities are designed to contain a permanent pool of water, such as stormwater ponds and wetlands, which is used for water quality treatment.

Storage facilities are often classified on the basis of their location and size. *On-site* storage is constructed on individual development sites. *Regional* storage facilities are constructed at the lower end of a subwatershed and are designed to manage stormwater runoff from multiple projects and/or properties. A discussion of regional stormwater controls is found in Section 4.1.

Storage can also be categorized as *on-line* or *off-line*. On-line storage uses a structural control facility that intercepts flows directly within a conveyance system or stream. Off-line storage is a separate storage facility to which flow is diverted from the conveyance system. Figure 3.3.1-2 illustrates on-line versus off-line storage.

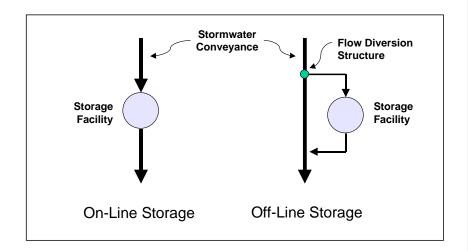


Figure 3.3.1-2 On-Line versus Off-Line Storage

3.3.1.2 Storage Classification

Stormwater storage(s) can be classified as detention, extended detention or retention. Some facilities include one or more types of storage.

3.3.1.3 Stage-Storage Relationship

A stage-storage curve defines the relationship between the depth of water and storage volume in a storage facility (see Figure 3.3.1-3). The volume of storage can be calculated by using simple geometric formulas expressed as a function of depth.

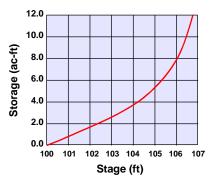


Figure 3.3.1-3 Stage-Storage Curve

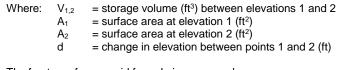
The storage volume for natural basins may be developed using a topographic map and the double-end area, frustum of a pyramid, prismoidal or circular conic section formulas.

The double-end area formula (see Figure 3.3.1-4) is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d$$
(3.3.1)

Figure 3.3.1-4 Double-End Area Method

ZERO AREA AT 100 ft



The frustum of a pyramid formula is expressed as:

$$V = d/3 [A_1 + (A_1 \times A_2)^{0.5} + A_2]/3$$

(3.3.2)

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Where: V = volume of frustum of a pyramid (ft³)

- d = change in elevation between points 1 and 2 (ft)
- A_1 = surface area at elevation 1 (ft²)
- A_2 = surface area at elevation 2 (ft²)

The prismoidal formula for trapezoidal basins is expressed as:

$$V = LWD + (L + W) ZD^{2} + 4/3 Z^{2} D^{3}$$
(3.3.3)

Where: V = volume of trapezoidal basin (ft^3)

L

Ζ

= length of basin at base (ft)

W = width of basin at base (ft)

D = depth of basin (ft) Z = side slope factor, r

= side slope factor, ratio of horizontal to vertical

The circular conic section formula is:

$V = 1.047 D (R_1^2 + R_2^2 + R_1 R_2)$	(3.3.4)
$V = 1.047 D (3 R_1^2 + 3ZDR_1 + Z_2D^2)$	(3.3.5)

Where: R1, R2 = bottom and surface radii of the conic section (ft)

D = depth of basin (ft)

= side slope factor, ratio of horizontal to vertical

3.3.1.4 Stage-Discharge Relationship

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility (see Figure 3.3.1-5). A typical storage facility has two outlets or spillways: a principal outlet and a secondary (or emergency) outlet. The principal outlet is usually designed with a capacity sufficient to convey the design flows without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. This spillway should be designed taking into account the potential threat to downstream areas if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway. For more details, see Section 3.4, *Outlet Structures*.

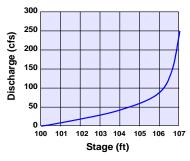


Figure 3.3.1-5 Stage-Discharge Curve

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3.3.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 3.3.2-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3.3.2-1	Symbols and Definitions	
<u>Symbol</u>	Definition	<u>Units</u>
A	Cross sectional or surface area	ft ²
Am	Drainage area	mi ²
С	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
t	Routing time period	sec
g	Acceleration due to gravity	ft/s ²
Ĥ	Head on structure	ft
Hc	Height of weir crest above channel bottom	ft
К	Coefficient	-
I	Inflow rate	cfs
L	Length	ft
Q,q	Peak inflow or outflow rate	cfs, in
R	Surface Radii	ft
S, Vs	Storage volume	ft ³
t _b	Time base on hydrograph	hrs
TI	Duration of basin inflow	hrs
t _P	Time to peak	hrs
Vs, S	Storage volume	ft3, in, acre-f
Vr	Volume of runoff	ft3, in, acre-f
W	Width of basin	ft
Z	Side slope factor	-

3.3.3 General Storage Design Procedures

3.3.3.1 Introduction

This section discusses the general design procedures for designing storage to provide standard detention of stormwater runoff for overbank and extreme flood protection (Q_{p25} and Q_f).

The design procedures for all structural control storage facilities are the same whether or not they include a permanent pool of water. In the latter case, the permanent pool elevation is taken as the "bottom" of storage and is treated as if it were a solid basin bottom for routing purposes.

It should be noted that the location of structural stormwater controls is very important as it relates to the effectiveness of these facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in

different downstream locations. Therefore, a downstream peak flow analysis should be performed as part of the storage facility design process (see subsection 3.1.9).

In multi-purpose multi-stage facilities such as stormwater ponds, the design of storage must be integrated with the overall design for water quality treatment objectives. See Chapter 4 for further guidance and criteria for the design of structural stormwater controls.

An important consideration in these designs is the sediment volume that the system is capable of storing before performance and/or capacity are reduced. For larger watersheds, or regional detention facilities, **T**the sediment volume is estimated from the sediment produced per year, times the years between dredging or similar maintenance. Provisions should be made in the layout to facilitate access for dredging equipment to the storage area and -the maximum sediment depth should be defined. Maintenance plans should discuss dredging and set a time interval for evaluation such as once per year.

For smaller watersheds, and all other stormwater detention facilities, sedimentation can be addressed by a local community requirement for an as-built pond survey and/or certification process. During the construction phase of a development or redevelopment, sedimentation occurs and can reduce the storage capacity of the post-construction stormwater detention basin drastically. Once final stabilization of a site has occurred, the accumulated sediment should be removed and the detention basin surveyed to comply with the originally approved design volume. It is understood that sedimentation of a fully stabilized site over time is minimal for smaller watersheds, when compared to a larger watershed during active construction activities.

3.3.3.2 Data Needs

The following data are needed for storage design and routing calculations:

- Inflow hydrograph for all selected design storms
- Stage-storage curve for proposed storage facility
- Stage-discharge curve for all outlet control structures
- Estimate of sediment deposited per year and the number of years desired before dredging.

3.3.3.3 Design Procedure

A general procedure for using the above data in the design of storage facilities is presented below.

- (Step 1) Compute inflow hydrograph for runoff from the 25- (Q_{p25}), and 100-year (Q_f) design storms using the hydrologic methods outlined in Section 3.1. Both existing- and post-development hydrographs are required for 25-year design storm.
- (Step 2) Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see subsection 3.2.4).
- (Step 3) Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. <u>Include the estimated volume of sediment</u> <u>storage (if appropriate)</u>. The maximum storage requirement calculated from Step 2 should be used. From the selected shape determine the maximum

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depth in the pond.

- (Step 4) Select the type of outlet and size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.
- (Step 5) Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model. If the routed postdevelopment peak discharges from the 25-year design storm exceed the existing-development peak discharges, then revise the available storage volume, outlet device, etc., and return to Step 3.
- (Step 6) Perform routing calculations using the 100-year hydrograph to determine if any increases in downstream flows from this hydrograph will cause damages and/or drainage and flooding problems. If problems will be created (e.g., flooding of habitable dwellings, property damage, or public access and/or utility interruption) then the storage facility must be designed to control the increased flows from the 100-year storm. If not then consider emergency overflow from runoff due to the 100-year (or larger) design storm and established freeboard requirements.
- (Step 7) Evaluate the downstream effects of detention outflows for the 25- and 100-year storms to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system until a confluence point is reached where the drainage area being analyzed represents 10% of the total drainage area (see subsection 3.1.9).
- (Step 8) Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

Routing of hydrographs through storage facilities is critical to the proper design of these facilities. Although storage design procedures using inflow/outflow analysis without routing have been developed, their use in designing detention facilities has not produced acceptable results in many areas of the country, including Georgia.

Although hand calculation procedures are available for routing hydrographs through storage facilities, they are very time consuming, especially when several different designs and iterations are evaluated. Many standard hydrology and hydraulics textbooks give examples of hand-routing techniques. For this Manual, it assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given here.

3.3.4 Preliminary Detention Calculations

3.3.4.1 Introduction

Procedures for preliminary detention calculations are included here to provide a simple method that can be used to estimate storage needs and also provide a quick check on the results of using different computer programs. Standard routing should be used for actual (final) storage facility calculations and design.

3.3.4.2 Storage Volume

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For small drainage areas, a preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 3.3.4-1.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_{s} = 0.5T_{I}(Q_{i} - Q_{0})$$

(3.3.6)

Where: Vs

V_s = storage volume estimate (ft³) Q_i = peak inflow rate (cfs)

- Q_0 = peak outflow rate (cfs)
- T_i = duration of basin inflow (s)

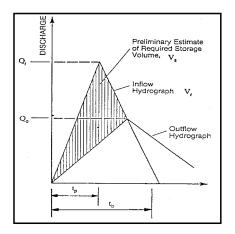


Figure 3.3.4-1 Triangular-Shaped Hydrographs (For Preliminary Estimate of Required Storage Volume)

3.3.4.3 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff and Singh, 1976).

- (Step 1) Determine input data, including the allowable peak outflow rate, Q₀, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.
- (Step 2) Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and the following equation:

$$\frac{V_s}{V_R} = \frac{1.291 \times \left(1 - \frac{Q_o}{Q_I}\right)^{0.753}}{\left(\frac{t_p}{t_b}\right)^{0.411}}$$

(3.3.7)

- Where: V_S = volume of storage (in)
 - V_r = volume of runoff (in)
 - Q₀ = outflow peak flow (cfs)
 - Q_i = inflow peak flow (cfs)
 - t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak]
 - t_p = time to peak of the inflow hydrograph (hr)
- (Step 3) Multiply the volume of runoff, Vr, times the ratio V_S/Vr, calculated in Step 2 to obtain the estimated storage volume V_S.

3.3.4.4 Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

- (Step 1) Determine volume of runoff, V_r, peak flow rate of the inflow hydrograph, Q_i, time base of the inflow hydrograph, t_b, time to peak of the inflow hydrograph, t_p, and storage volume V_S.
- (Step 2) Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Wycoff and Singh, 1976):

$$Q_0/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546}$$

Where: Q₀= outflow peak flow (cfs)

- Q_i= inflow peak flow (cfs)
- V_S= volume of storage (in)
- V_r= volume of runoff (in)
- t_b= time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak]
- t_p= time to peak of the inflow hydrograph (hr)
- (Step 3) Multiply the peak flow rate of the inflow hydrograph, Q_i , times the potential peak flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, Q_0 , for the selected storage volume.

3.3.5 Channel Protection Volume Estimation

3.3.5.1 Introduction

The Simplified SCSNRCS TR-55 Peak Runoff Rate Estimation approach (see subsection 3.1.5.7) can be used for estimation of the Channel Protection Volume (CP_v) for storage facility design.

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(3.3.8)

This method should not be used for standard detention design calculations. See either subsection 3.3.4 or the modified rational method in subsection 3.3.6 for preliminary detention calculations without formal routing.

3.3.5.2 Basic Approach

For CP_v estimation, using Figures 3.1.5-6 and 3.1.5-7 in Section 3.1, the unit peak discharge (q_u) can be determined based on I_a/P and time of concentration (t_c). Knowing q_u and T (extended detention time, typically 24 hours), the q_o/q_l ratio (peak outflow discharge/peak inflow discharge) can be estimated from Figure 3.3.5-1.

Using the following equation from TR-55 for a Type II or Type III rainfall distribution, $V_{\text{S}}/V_{\text{r}}$ can be calculated.

Note: Figure 3.3.4-1 can also be used to estimate V_S/V_r.

$V_{s}/V_{r} = 0.682 - 1.43 (q_{0}/q_{i}) + 1.64 (q_{0}/q_{i})^{2} - 0.804 (q_{0}/q_{i})^{3}$

The required storage volume can then be calculated by:

$$V_{s} = \frac{(V_{s}/V_{r})(Q_{d})(A)}{12}$$
(3.3.10)

 $\begin{array}{ll} \mbox{Where:} & V_S \mbox{ and } V_r \mbox{ are defined above} \\ & Q_d = \mbox{the developed runoff for the design storm (inches)} \end{array}$

A = total drainage area (acres)

While the TR-55 short-cut method reports to incorporate multiple stage structures, experience has shown that an additional 10-15% storage is required when multiple levels of extended detention are provided inclusive with the 25-year storm.

3.3.5.3 Example Problem

Compute the 100-year peak discharge for a 50-acre wooded watershed located in Peachtree City, which will be developed as follows:

- Forest land good cover (hydrologic soil group B) = 10 ac
- Forest land good cover (hydrologic soil group C) = 10 ac
- 1/3 Acre residential (hydrologic soil group B) = 20 ac
- Industrial development (hydrological soil group C) = 10 ac

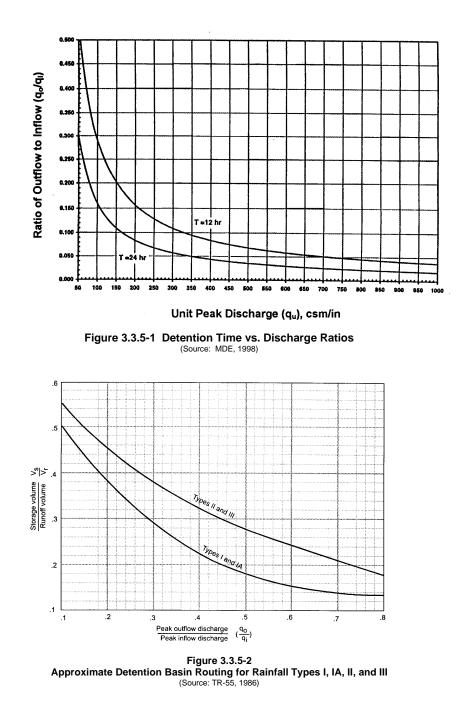
Other data include the following:

- Total impervious area = 18 acres
- % of pond and swamp area = 0

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(3.3.9)



Computations

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(1) Calculate rainfall excess:

- The 100-year, 24-hour rainfall is 8.22 inches (From NOAA Atlas 14).
- The 1-year, 24 hour rainfall is 3.37 inches (From NOAA Atlas 14).
- Composite weighted runoff coefficient is:

Dev. #	Area (ac)	% Total	CN	Composite CN
1	10	0.20	55	11.0
2	10	0.20	70	14.0
3	20	0.40	72	28.8
4	10	0.20	91	18.2
Total	50	1.00		72

* From equation 3.1.6, Q (100-year) = 4.89 inches Q_d (1-year developed) = 1.0 inches

(2) Calculate time of concentration

The hydrologic flow path for this watershed = 1,890 ft

Segment	Type of Flow	Length (ft)	Slope (%)
1	Overland n = 0.24	40	2.0 %
2	Shallow channel	750	1.7 %
3	Main channel*	1100	0.50 %

* For the main channel, n = .06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

Segment 1 - Travel time from equation 2.1.9 with $P_2 = 3.84$ in (From NOAA Atlas 14)

 $T_t = [0.42(0.24 \times 40)^{0.8}] / [(3.84)^{0.5} (.020)^{0.4}] = 6.26 \text{ minutes}$

```
 \begin{array}{l} \mbox{Segment 2 - Travel time from Figure 3.1.5-5 or equation 3.1.9} \\ \mbox{V = 2.1 ft/sec (from equation 3.1.9)} \\ \mbox{T}_t = 750 \ / \ 60 \ (2.1) = 5.95 \ \mbox{minutes} \end{array}
```

 $\begin{array}{l} \mbox{Segment 3 - Using equation 3.1.11} \\ V = (1.49 / .06) \; (1.43)^{0.67} \; (.005)^{0.5} = 2.23 \; \mbox{ft/sec} \\ T_t = 1100 \; / \; 60 \; (2.23) = 8.22 \; \mbox{minutes} \end{array}$

 $t_c = 6.26 + 5.95 + 8.22 = 20.43$ minutes (.34 hours)

(3) Calculate I_a/P for C_n = 72 (Table 3.1.5-1), I_a = .778 (Table 3.1.5-3)

 I_a/P = (.778 / 8.23 = .095 (Note: Use I_a/P = .10 to facilitate use of Figure 3.1.5-6. Straight line interpolation could also be used.)

(4) Unit discharge q_u (100-year) from Figure 3.1.5-6 = 650 csm/in, q_u (1-year) = 580 csm/in

(5) Calculate peak discharge with $F_p = 1$ using equation 3.1.12

Q₁₀₀ = 650 (50/640)(4.89)(1) = 248 cfs

(6) Calculate water quality volume (WQ_v)

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Compute runoff coefficient, R_v $R_v = 0.50 + (IA)(0.009) = 0.50 + (18)(0.009) = 0.21$

Compute water quality volume, WQ_{ν} WQ_{ν} = 1.2(R_{\nu})(A)/12 = 1.2(.21)(50)/12 = 1.05 acre-feet

(7) Calculate channel protection volume ($CP_v = V_S$)

Knowing q_u (1-year) = 580 csm/in from Step 3 and T (extended detention time of 24 hours), find q_0/q_i from Figure 3.3.5-1. $q_0/q_i = 0.03$

For a Type II rainfall distribution, $V_S/V_r = 0.682 - 1.43 (q_0/q_l) + 1.64 (q_0/q_l)^2 - 0.804 (q_0/q_l)^3$ $V_S/V_r = 0.682 - 1.43 (0.03) + 1.64 (0.03) - 0.804 (0.03) = 0.64$

Therefore, stream channel protection volume with Q_d (1-year developed) = 1.0 inches, from Step 1, is $CP_v = V_S = (V_S/V_f)(Q_d)(A)/12 = (0.64)(1.0)(50)/12 = 2.67$ acre-feet

3.3.6 The Modified Rational Method

3.3.6.1 Introduction

For drainage areas of <u>less than 5 acres</u>, a modification of the Rational Method can be used for the estimation of storage volumes for detention calculations.

The Modified Rational Method uses the peak flow calculating capability of the Rational Method paired with assumptions about the inflow and outflow hydrographs to compute an approximation of storage volumes for simple detention calculations. There are many variations on the approach. Figure 3.3.6-1 illustrates one application. The rising and falling limbs of the inflow hydrograph have a duration equal to the time of concentration (t_c). An allowable target outflow is set (Q_a) based on pre-development conditions. The storm duration is t_d , and is varied until the storage volume (shaded gray area) is maximized. It is normally an iterative process done by hand or on a spreadsheet. Downstream analysis is not possible with this method as only approximate graphical routing takes place.

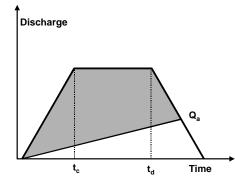


Figure 3.3.6-1 Modified Rational Definitions

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Commented [RC14]: TC179 – will remain as is. Modified Rational method discussed here, not rational method.

3.3.6.2 Design Equations

The design of detention using the Modified Rational Method is presented as a noniterative approach suitable for spreadsheet calculation (Debo & Reese, 1995).

The allowable release rate can be determined from:

$$Q_a = C_a i A$$
 (3.3.11)

Where: Qa = allowable release rate (cfs)

- Ca = predevelopment Rational Method runoff coefficient
- = rainfall intensity for the corresponding time of concentration (in/hr) i.
- A = area (acres)

The critical duration of storm, the time value to determine rainfall intensity, at which the storage volume is maximized is:

$$T_{d} = \sqrt{\frac{2CAab}{Q_{a}}} - b \tag{3.3.12}$$

Where: T_d = critical storm duration (min)

Q_a = allowable release rate (cfs) С

= developed condition Rational Method runoff coefficient

A = area (acres)

a, b = rainfall factors dependent on location and return period taken from Table 3.3.6-1

The required storage volume, in cubic feet can be obtained from equation 3.3.13.

$$V_{\text{preliminary}} = 60 \left[\text{CAa} - (2\text{CabAQ}_a)^{1/2} + (Q_a/2) \left(\text{b-t_c} \right) \right]$$
(3.3.13a)

Where: V_{preliminary} = preliminary required storage (ft³)

 V_{max} = required storage (ft³)

= time of concentration for the developed condition (min) to

 $P_{180} = 3$ -hour (180-minute) storm depth (in)

P_{td} = storm depth for the critical duration (in)

all other variables are as defined above

The equations above include the use of an adjustment factor to the calculated storage volume to account for undersizing. The factor (P_{180}/P_{td}) is the ratio of the 3-hour storm depth for the return frequency divided by the rainfall depth for the critical duration calculated in equation 3.3.12.

The Modified Rational Method also often undersizes storage facilities in flat and more sandy areas where the target discharge may be set too large, resulting in an oversized orifice. In these locations a C factor of 0.05 to 0.1 should be used.

3.3.6.3 Example Problem

A 5-acre site is to be developed in Atlanta. Based on site and local information, it is determined that channel protection is not required and that limiting the 25-year and 100-year storm is also not required. The local government has determined that the development must detain the 2-year and 10-year storms. Rainfall values are taken from NOAA Atlas 14. The following key information is obtained:

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(3.3.13b)

		minutes and C fa
Steps	2 - year	10 - year
t _c (min)	21	21
i (in/hr)	3.34	4.51
Q _a (equation 3.3.11) (cfs)	3.67	4.96
a (from Table 3.3.6-1)	123.19	184.23
b (from Table 3.3.6-1)	15.91	19.96
V _{max} (equation 3.3.13) (ft ³)	16,017	23,199
P ₁₈₀ (from NOAA Atlas 14) (in)	2.43	3.42
T _d (equation 3.3.12) (min)	49	57
P _{td} (from NOAA Atlas 14) (in)	1.62	2.70
V _{max} (equation 3.3.13) (ft ³)	24,025	29,385

 Table 3.3.6-1
 Rainfall Factors "a" and "b" for the Modified Rational Method (1-year through 100-year return periods)

City				Re	eturn Inte	erval		
City		1	2	<u>5</u>	<u>10</u>	<u>25</u>	<u>50</u>	<u>100</u>
Albony	bany		159.17	198.14	230.00	271.84	305.29	341.98
Albany	b	16.02	19.72	22.52	24.49	26.00	26.97	28.23
Atlanta	а	97.05	123.19	157.99	184.23	219.21	249.86	278.71
Allania	b	12.88	15.91	18.44	19.96	21.13	22.28	23.01
Athens	а	106.01	126.29	162.23	187.80	224.41	253.05	281.69
Amens	b	15.41	16.95	19.57	20.87	22.19	22.99	23.68
Augusta	а	119.32	142.78	171.04	192.10	221.48	247.98	271.24
Augusta	b	17.05	19.12	20.34	20.96	21.40	22.10	22.32
Bainbridge	а	128.79	171.90	215.02	245.38	291.64	329.59	367.38
Dambiluge	b	16.39	21.13	24.33	25.87	27.73	29.12	30.26
Brunswick	а	177.81	191.06	233.75	266.24	314.79	352.59	367.38
DIUIISWICK	b	26.30	24.13	27.51	29.49	31.77	33.16	34.22
Columbus	а	113.09	142.00	177.92	205.63	246.52	273.92	306.45
	b	15.67	17.87	20.34	21.88	23.63	24.11	25.13
Macon	а	111.40	139.06	176.78	203.43	242.56	272.93	306.45
Wacon	b	15.48	17.68	20.55	21.94	23.47	24.38	25.59
Metro	а	93.15	116.20	148.58	171.22	201.95	227.07	254.06
Chattanooga	b	14.25	15.97	18.00	18.91	19.60	20.12	20.84
Peachtree City	а	101.63	125.43	160.73	185.58	219.86	250.95	277.86
Feachtree City	b	13.72	15.94	18.64	19.91	21.02	22.25	22.81
Rome	а	88.91	120.41	159.75	188.99	229.97	264.15	292.64
Kome	b	12.10	16.05	19.06	20.82	22.51	23.81	24.21
Roswell	а	93.33	126.28	159.12	182.23	219.74	246.68	273.06
I/O2MCII	b	12.28	16.92	19.00	19.96	21.54	22.17	22.67
Savannah	а	135.97	178.06	230.29	266.68	325.90	373.89	418.97
Gavannan	b	19.41	23.22	28.28	30.80	34.41	36.82	38.60
Тоссоа	а	114.77	124.54	164.15	192.50	234.48	266.57	299.01
10000	b	19.58	17.40	20.33	21.85	23.67	24.65	25.51

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Valdosta	а	132.93	165.35	203.32	229.47	269.41	301.00	333.57
	b	16.72	19.94	22.63	23.79	25.20	26.10	26.98
Vidalia	а	120.40	161.23	201.42	230.71	272.84	310.23	343.58
viualia	b	15.00	20.17	23.69	25.24	26.80	28.32	29.15

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3.4 OUTLET STRUCTURES

3.4.1 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 3.4.1-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3.4.1-1	Symbols and Definitions	
<u>Symbol</u>	Definition	<u>Units</u>
A,a	Cross sectional or surface area	ft ²
Am	Drainage area	mi ²
В	Breadth of weir	ft
С	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/s ²
Н	Head on structure	ft
Hc	Height of weir crest above channel bottom	ft
K,k	Coefficient	-
L	Length	ft
n	Manning's n	-
Q,q	Peak inflow or outflow rate	cfs, in
Vu	Approach velocity	ft/s
WQv	Water quality volume	ac ft
W	Maximum cross sectional bar width	
	facing the flow	in
х	Minimum clear spacing between bars	in
θ	Angle of v-notch	degrees
θg	Angle of the grate with respect to	-
2	the horizontal	degrees

3.4.2 Primary Outlets

3.4.2.1 Introduction

Primary outlets provide the critical function of the regulation of flow for structural stormwater controls. There are several different types of outlets that may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.

For a single stage system, the stormwater facility can be designed as a simple pipe or culvert. For multistage control structures, the inlet is designed considering a range of design flows.

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A stage-discharge curve is developed for the full range of flows that the structure would experience. The outlets are housed in a riser structure connected to a single outlet conduit. An alternative approach would be to provide several pipe or culvert outlets at different levels in the basin that are either discharged separately or are combined to discharge at a single location.

This section provides an overview of outlet structure hydraulics and design for stormwater storage facilities. The design engineer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

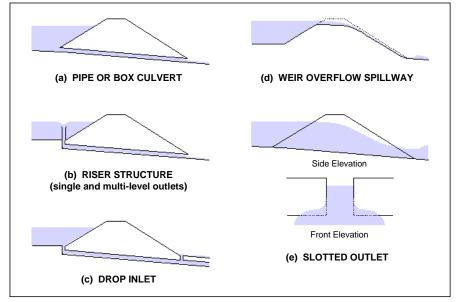


Figure 3.4.2-1 Typical Primary Outlets

3.4.2.2 Outlet Structure Types

There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in stormwater facility design:

- Orifices
- Perforated risers
- Pipes / Culverts
- Sharp-crested weirs

- Broad-crested weirsV-notch weirs
- Proportional weirs
- Combination outlets

Each of these outlet types has a different design purpose and application:

- Water quality and channel protection flows are normally handled with smaller, more
 protected outlet structures such as reverse slope pipes, hooded orifices, orifices
 located within screened pipes or risers, perforated plates or risers, and V-notch
 weirs.
- Larger flows, such as overbank protection and extreme flood flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway through

the embankment. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.

3.4.2.3 Orifices

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice.

For a single orifice, as illustrated in Figure 3.4.2-2(a), the orifice discharge can be determined using the standard orifice equation below.

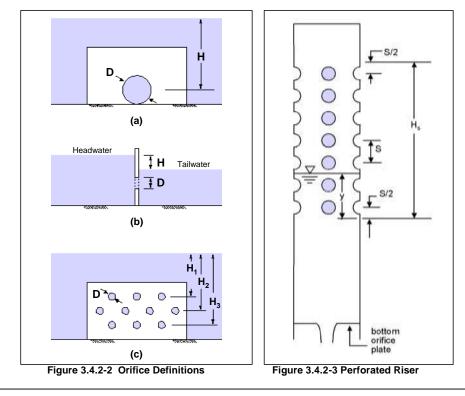
 $Q = CA (2gH)^{0.5}$

(3.4.1)

Where: = the orifice flow discharge (cfs) Q

- = discharge coefficient С А
 - = cross-sectional area of orifice or pipe (ft²)
 - = acceleration due to gravity (32.2 ft/s^2)
- g D = diameter of orifice or pipe (ft)
- н = effective head on the orifice, from orifice center to the water surface

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces as shown in Figure 3.4.2-2(b).



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When the material is thinner than the orifice diameter, with sharp edges, a coefficient of 0.6 should be used. For square-edged entrance conditions the generic orifice equation can be simplified:

$Q = 0.6A (2gH)^{0.5} = 3.78D^2H^{0.5}$

(3.4.2)

When the material is thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges are rounded, a coefficient of 0.92 can be used.

Flow through multiple orifices, such as the perforated plate shown in Figure 3.4.2-2(c), can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

Perforated orifice plates for the control of discharge can be of any size and configuration. However, the Denver Urban Drainage and Flood Control District has developed standardized dimensions that have worked well. Table 3.4.2-1 gives appropriate dimensions. The vertical spacing between hole centerlines is always 4 inches.

	Minimum	Flow Area per Row (in ²)					
Hole Diameter (in)	Column Hole Centerline Spacing (in)	1 column	2 columns	3 columns			
1/4	1	0.05	0.1	0.15			
5/16	2	0.08	0.15	0.23			
3/8	2	0.11	0.22	0.33			
7/16	2	0.15	0.3	0.45			
1/2	2	0.2	0.4	0.6			
9/16	3	0.25	0.5	0.75			
5/8	3	0.31	0.62	0.93			
11/16	3	0.37	0.74	1.11			
3/4	3	0.44	0.88	1.32			
13/16	3	0.52	1.04	1.56			
7/8	3	0.6	1.2	1.8			
15/16	3	0.69	1.38	2.07			
1	4	0.79	1.58	2.37			
1 1/16	4	0.89	1.78	2.67			
1 1/8	4	0.99	1.98	2.97			
1 3/16	4	1.11	2.22	3.33			
1 1/4	4	1.23	2.46	3.69			
1 5/16	4	1.35	2.7	4.05			
1 3/8	4	1.48	2.96	4.44			
1 7/16	4	1.62	3.24	4.86			
1 1/2	4	1.77	3.54	5.31			
1 9/16	4	1.92	3.84	5.76			
1 5/8	4	2.07	4.14	6.21			
1 11/16	4	2.24	4.48	6.72			
1 3/4	4	2.41	4.82	7.23			
1 13/16	4	2.58	5.16	7.74			
1 7/8	4	2.76	5.52	8.28			
1 15/16	4	2.95	5.9	8.85			
2	4	3.14	6.28	9.42			
	ber of columns re	fers to parallel o	olumns of holes				
inimum steel plate thickness	1/4"	5/16"		3/8"			

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For rectangular slots the height is normally 2 inches with variable width. Only one column of rectangular slots is allowed.

Figure 3.4.2-4 provides a schematic of an orifice plate outlet structure for a wet ED pond showing the design pool elevations and the flow control mechanisms.

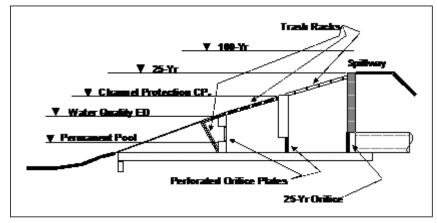


Figure 3.4.2-4 Schematic of Orifice Plate Outlet Structure

3.4.2.4 Perforated Risers

A special kind of orifice flow is a perforated riser as illustrated in Figure 3.4.2-3. In the perforated riser, an orifice plate at the bottom of the riser, or in the outlet pipe just downstream from the elbow at the bottom of the riser, controls the flow. It is important that the perforations in the riser convey more flow than the orifice plate so as not to become the control.

Referring to Figure 3.4.2-3, a shortcut formula has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988):

$$Q = C_{p} \frac{2A_{p}}{3H_{s}} \sqrt{2g} H^{3/2}$$
(3.4.3)

Where:

= discharge (cfs) = discharge coefficient for perforations (normally 0.61) Cp

= cross-sectional area of all the holes (ft²)

A_p H_s = distance from S/2 below the lowest row of holes to S/2 above the

3.4.2.5 Pipes and Culverts

C

Discharge pipes are often used as outlet structures for stormwater control facilities. The design of these pipes can be for either single or multi-stage discharges. A reverse-slope underwater pipe is often used for water quality or channel protection outlets.

top row (ft)

Pipes smaller than 12 inches in diameter may be analyzed as a submerged orifice as long as H/D is greater than 1.5. Note: For low flow conditions when the flow reaches and begins to overflow the pipe, weir flow controls (see subsection 3..4.2.6). As the stage increases the flow will transition to orifice flow.

Pipes greater than 12 inches in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet control culvert nomographs and procedures given in Section 5.3, Culvert Design, or by using equation 3.4.4 (NRCS, 1984).

The following equation is a general pipe flow equation that is derived through the use of the Bernoulli and continuity principles.

$Q = a[(2gH) / (1 + k_m + k_pL)]0.5$

(3.4.4)

Where:	Q g H k _p L	 discharge (cfs) pipe cross sectional area (ft²) acceleration of gravity (ft/s²) elevation head differential (ft) coefficient of minor losses (use 1.0) pipe friction coefficient = 5087n²/D^{4/3} pipe length (ft)
--------	------------------------------------	---

2.3.2.6 Sharp-Crested Weirs

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear as it overflows, the overflow is termed a sharp-crested weir. If the sides of the weir also cause the through flow to contract, it is termed an end-contracted sharpcrested weir. Sharp-crested weirs have stable stage-discharge relations and are often used as a measurement device. A sharp-crested weir with no end contractions is illustrated in Figure 3.4.2-5(a). The discharge equation for this configuration is (Chow, 1959):

$Q = [(3.27 + 0.4(H/H_c)] LH^{1.5}]$

(3.4.5)

Where: Q = discharge (cfs)

- Н = head above weir crest excluding velocity head (ft)
- = height of weir crest above channel bottom (ft) Hc
- L = horizontal weir length (ft)

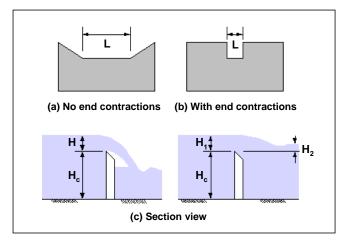


Figure 3.4.2-5 Sharp-Crested Weir

A sharp-crested weir with two end contractions is illustrated in Figure 3.4.2-5(b). The discharge equation for this configuration is (Chow, 1959):

Where: Q = discharge (cfs)

L

- H = head above weir crest excluding velocity head (ft)
- H_c = height of weir crest above channel bottom (ft)
 - = horizontal weir length (ft)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Q_{s} = Q_{f} (1 - (H_{2}/H_{1})^{1.5})^{0.385}$$

(3.4.7)

(3.4.6)

Where: Q_S = submergence flow (cfs)

- Q_f = free flow (cfs)
- H₁ = upstream head above crest (ft)
- H_2 = downstream head above crest (ft)

3.4.2.7 Broad-Crested Weirs

A weir in the form of a relatively long raised channel control crest section is a *broad-crested* weir. The flow control section can have different shapes, such as triangular or circular. True broadcrested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. For example, a thick wall or a flat stop log can act like a sharp-crested weir when the approach head is large enough that the flow springs from the upstream corner. If upstream head is small enough relative to the top profile length, the stop log can act like a broad-crested weir (USBR, 1997).

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The equation for the broad-crested weir is (Brater and King, 1976):

$Q = CLH^{1.5}$

(3.4.8)

Where: Q = discharge (cfs)

- C = broad-crested weir coefficient
- L = broad-crested weir length perpendicular to flow (ft)
- H = head above weir crest (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 3.4.2-2.

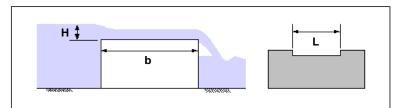


Figure 3.4.2-6 Broad-Crested Weir

Measured Head (H)*	Weir Crest Breadth (b) in feet										
In feet	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.27	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

3.4.2.8 V-Notch Weirs

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The discharge through a V-notch weir (Figure 3.4.2-7) can be calculated from the following equation (Brater and King, 1976).

Q = 2.5 tan (θ /2) H^{2.5}

(3.4.9)

Where: Q

- = discharge (cfs)
- θ = angle of V-notch (degrees) H = head on apex of notch (ft)

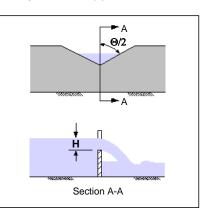


Figure 3.4.2-7 V-Notch Weir

3.4.2.9 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head. A typical proportional weir is shown in Figure 3.4.2-8. Design equations for proportional weirs are (Sandvik, 1985):

Q = 4.97 a^{0.5} b (H - a/3) (3.4.10) x/b = 1 - (1/3.17) (arctan (y/a)^{0.5}) (3.4.11)

Where: Q = discharge (cfs) Dimensions a, b, H, x, and y are shown in Figure 3.4.2-8

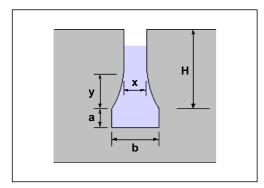


Figure 3.4.2-8 Proportional Weir Dimensions

3.4.2.10 Combination Outlets

Combinations of orifices, weirs and pipes can be used to provide multi-stage outlet control for different control volumes within a storage facility (i.e., water quality volume, channel protection volume, overbank flood protection volume, and/or extreme flood protection volume).

They are generally of two types of combination outlets: shared outlet control structures and separate outlet controls. Shared outlet control is typically a number of individual outlet openings (orifices), weirs or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Figure 3.4.2-9 shows an example of a riser designed for a wet ED pond.

The orifice plate outlet structure in Figure 3.4.2-4 is another example of a combination outlet.

Separate outlet controls are less common and may consist of several pipe or culvert outlets at different levels in the storage facility that are either discharged separately or are combined to discharge at a single location.

The use of a combination outlet requires the construction of a composite stagedischarge curve (as shown in Figure 3.4.2-10) suitable for control of multiple storm flows. The design of multi-stage combination outlets is discussed later in this section.

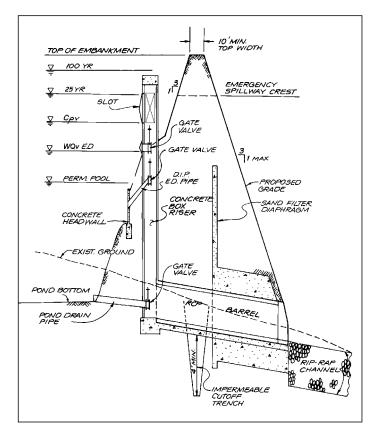


Figure 3.4.2-9 Schematic of Combination Outlet Structure

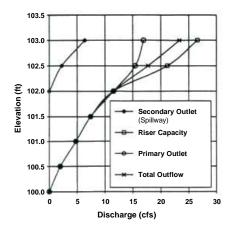


Figure 3.4.2-10 Composite Stage-Discharge Curve

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3.4.3 Extended Detention (Water Quality and Channel Protection) Outlet Design

3.4.3.1 Introduction

Extended detention orifice sizing is required in design applications that provide extended detention for downstream channel protection or the ED portion of the water quality volume. In both cases an extended detention orifice or reverse slope pipe can be used for the outlet. For a structural control facility providing both WQ_v extended detention and CP_v control (wet ED pond, micropool ED pond, and shallow ED wetland), there will be a need to design two outlet orifices – one for the water quality control outlet and one for the channel protection drawdown.

(This following procedures are based on the water quality outlet design procedures included in the Virginia Stormwater Management Handbook, 1999)

The outlet hydraulics for peak control design (overbank flood protection and extreme flood protection) is usually straightforward in that an outlet is selected that will limit the peak flow to some predetermined maximum. Since volume and the time required for water to exit the storage facility are not usually considered, the outlet design can easily be calculated and routing procedures used to determine if quantity design criteria are met.

In an extended detention facility for water quality treatment or downstream channel protection, however, the storage volume is detained and released over a specified amount of time (e.g., 24-hours). The release period is a brim drawdown time, beginning at the time of peak storage of the water quality volume until the entire calculated volume drains out of the basin. This assumes that the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention outlet can be sized using either of the following methods:

- (1) Use the maximum hydraulic head associated with the storage volume and maximum flow, and calculate the orifice size needed to achieve the required drawdown time, and route the volume through the basin to verify the actual storage volume used and the drawdown time.
- (2) Approximate the orifice size using the average hydraulic head associated with the storage volume and the required drawdown time.

These two procedures are outlined in the examples below and can be used to size an extended detention orifice for water quality and/or channel protection.

3.4.3.2 Method 1: Maximum Hydraulic Head with Routing

A wet ED pond sized for the required water quality volume will be used here to illustrate the sizing procedure for an extended-detention orifice.

Given the following information, calculate the required orifice size for water quality design.

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Given: Water Quality Volume (WQ_v) = 0.76 ac ft = 33,106 ft³ Maximum Hydraulic Head (H_{max}) = 5.0 ft (from stage vs. storage data)

(Step 1) Determine the <u>maximum</u> discharge resulting from the 24-hour drawdown requirement. It is calculated by dividing the Water Quality Volume (or Channel Protection Volume) by the required time to find the average discharge, and then multiplying by two to obtain the maximum discharge.

 Q_{avg} = 33,106 ft^3 / (24 hr)(3,600 s/hr) = 0.38 cfs Q_{max} = 2 * Q_{avg} = 2 * 0.38 = 0.76 cfs

(Step 2) Determine the required orifice diameter by using the orifice equation (3.4.8) and

 Q_{max} and H_{max} :

 $Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$

A = 0.76 / 0.6[(2)(32.2)(5.0)]^{0.5} = 0.071 ft³

Determine pipe diameter from A = $3.14d^2/4$, then d = $(4A/3.14)^{0.5}$

 $D = [4(0.071)/3.14]^{0.5} = 0.30 \text{ ft} = 3.61 \text{ in}$

Use a 3.6-inch diameter water quality orifice.

Routing the water quality volume of 0.76 ac ft through the 3.6-inch water quality orifice will allow the designer to verify the drawdown time, as well as the maximum hydraulic head elevation. The routing effect will result in the actual drawdown time being less than the calculated 24 hours. Judgment should be used to determine whether the orifice size should be reduced to achieve the required 24 hours or if the actual time achieved will provide adequate pollutant removal.

3.4.3.3 Method 2: Average Hydraulic Head and Average Discharge

Using the data from the previous example (3.4.3.2) use Method 2 to calculate the size of the outlet orifice.

Given: Water Quality Volume (WQ_v) = 0.76 ac ft = 33,106 ft³ Average Hydraulic Head (h_{avg}) = 2.5 ft (from stage vs storage data)

(Step 1) Determine the average release rate to release the water quality volume over a 24-hour time period.

 $Q = 33,106 \text{ ft}^3 / (24 \text{ hr})(3,600 \text{ s/hr}) = 0.38 \text{ cfs}$

(Step 2) Determine the required orifice diameter by using the orifice equation (3.4.8) and the average head on the orifice:

$$Q = CA(2gH)^{0.5}$$
, or $A = Q / C(2gH)^{0.5}$

 $A = 0.38 / 0.6[(2)(32.2)(2.5)]^{0.5} = 0.05 \text{ ft}^3$

Determine pipe diameter from A = $3.14r^2 = 3.14d^2/4$, then d = $(4A/3.14)^{0.5}$

 $D = [4(0.05)/3.14]^{0.5} = 0.252 \text{ ft} = 3.03 \text{ in}$ Use a 3-inch diameter water quality orifice.

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Use of Method 1, utilizing the maximum hydraulic head and discharge and routing, results in a 3.6-inch diameter orifice (though actual routing may result in a changed orifice size) and Method 2, utilizing average hydraulic head and average discharge, results in a 3.0-inch diameter orifice.

3.4.4 Multi-Stage Outlet Design

3.4.4.1 Introduction

A combination outlet such as a multiple orifice plate system or multi-stage riser is often used to provide adequate hydraulic outlet controls for the different design requirements (e.g., water quality, channel protection, overbank flood protection, and/or extreme flood protection) for stormwater ponds, stormwater wetlands and detention-only facilities. Separate openings or devices at different elevations are used to control the rate of discharge from a facility during multiple design storms. Figures 3.4.2-4 and 3.4.2-9 are examples of multi-stage combination outlet systems.

A design engineer may be creative to provide the most economical and hydraulically efficient outlet design possible in designing a multi-stage outlet. Many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. The stage-discharge table or rating curve is a composite of the different outlets that are used for different elevations within the multi-stage riser (see Figure 3.4.2-10)

3.4.4.2 Multi-Stage Outlet Design Procedure

Below are the steps for designing a multi-stage outlet. Note that if a structural control facility will not control one or more of the required storage volumes (WQ_v, CP_v, Q_{p25}, and Q_t), then that step in the procedure is skipped.

- (Step 1) <u>Determine Stormwater Control Volumes</u>. Using the procedures from Sections 3.1 and 3.3, estimate the required storage volumes for water quality treatment (WQ_v), channel protection (CP_v), and overbank flood control (Q_{p25})and extreme flood control (Q_f).
- (Step 2) <u>Develop Stage-Storage Curve</u>. Using the site geometry and topography, develop the stage-storage curve for the facility in order to provide sufficient storage for the control volumes involved in the design.
- (Step 3) <u>Design Water Quality Outlet</u>. Design the water quality extended detention (WQ_v-ED) orifice using either Method 1 or Method 2 outlined in subsection 3.4.3. If a permanent pool is incorporated into the design of the facility, a portion of the storage volume for water quality will be above the elevation of the permanent pool. The outlet can be protected using either a reverse slope pipe, a hooded protection device, or another acceptable method (see subsection 3.4.5).
- (Step 4) <u>Design Channel Protection Outlet</u>. Design the stream channel protection extended detention outlet (CP_v-ED) using either method from subsection 3.4.3. For this design, the storage needed for channel protection will be "stacked" on top of the water quality volume storage elevation determined in Step 3. The total stage-discharge rating curve at this point will include water quality control orifice and the outlet used for stream channel protection. The

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outlet should be protected in a manner similar to that for the water quality orifice.

(Step 5) <u>Design Overbank Flood Protection Outlet</u>. The overbank protection volume is added above the water quality and channel protection storage. Establish the Q_{p25} maximum water surface elevation using the stage-storage curve and subtract the CP_v elevation to find the 25-year maximum head. Select an outlet type and calculate the initial size and geometry based upon maintaining the predevelopment 25-year peak discharge rate. Develop a stage-discharge curve for the combined set of outlets (WQ_v, CP_v and Q_{p25}).

This procedure is repeated for control (peak flow attenuation) of the 100-year storm (Q_f) , if required.

(Step 6) <u>Check Performance of the Outlet Structure</u>. Perform a hydraulic analysis of the multi-stage outlet structure using reservoir routing to ensure that all outlets will function as designed. Several iterations may be required to calibrate and optimize the hydraulics and outlets that are used. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This usually requires that the structure have a larger cross-sectional area than the outlet conduit.

The hydraulic analysis of the design must take into account the hydraulic changes that will occur as depth of storage changes for the different design storms. As shown in Figure 3.4.4-1, as the water passes over the rim of a riser, the riser acts as a weir. However, when the water surface reaches a certain height over the rim of a riser, the riser will begin to act as a submerged orifice. The designer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place for an outlet where this change in hydraulic conditions will change. Also note in Figure 3.4.4-1 that as the elevation of the water increases further, the control can change from barrel inlet flow control to barrel pipe flow control. Figure 3.4.4-2 shows another condition where weir flow can change to orifice flow, which must be taken into account in the hydraulics of the rating curve as different design conditions results in changing water surface elevations.

- (Step 7) <u>Size the Emergency Spillway</u>. It is recommended that all stormwater impoundment structures have a vegetated emergency spillway (see subsection 3.4.6). An emergency spillway provides a degree of safety to prevent overtopping of an embankment if the primary outlet or principal spillway should become clogged, or otherwise inoperative. The 100-year storm should be routed through the outlet devices and emergency spillway to ensure the hydraulics of the system will operate as designed.
- (Step 8) <u>Design Outlet Protection</u>. Design necessary outlet protection and energy dissipation facilities to avoid erosion problems downstream from outlet devices and emergency spillway(s). See Section 5.5, *Energy Dissipation Design*, for more information.
- (Step 9) <u>Perform Buoyancy Calculations</u>. Perform buoyancy calculations for the outlet structure and footing. Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water.
- (Step 10) Provide Seepage Control. Seepage control should be provided for the outflow

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pipe or culvert through an embankment. The two most common devices for controlling seepage are (1) filter and drainage diaphragms and (2) anti-seep collars.

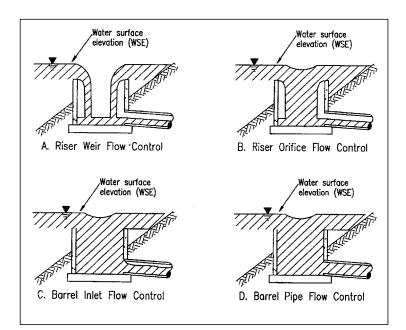


Figure 3.4.4-1 Riser Flow Diagrams (Source: VDCR, 1999)

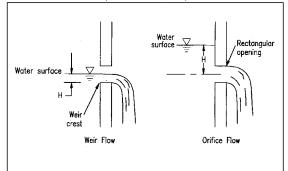


Figure 3.4.4-2 Weir and Orifice Flow (Source: VDCR, 1999)

3.4.5 Extended Detention Outlet Protection

Small low flow orifices such as those used for extended detention applications can easily clog, preventing the structural control from meeting its design purpose(s) and potentially causing adverse impacts. Therefore, extended detention orifices need to be adequately protected from clogging. There are a number of different anti-clogging designs, including:

- The use of a reverse slope pipe attached to a riser for a stormwater pond or wetland with a permanent pool (see Figure 3.4.5-1). The inlet is submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond.
- The use of a hooded outlet for a stormwater pond or wetland with a permanent pool (see Figures 3.4.5-2 and 3.4.5-3).
- Internal orifice protection through the use of an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket (see Figure 3.4.5-4).
- Internal orifice protection through the use of an adjustable gate valves can to achieve an equivalent orifice diameter.

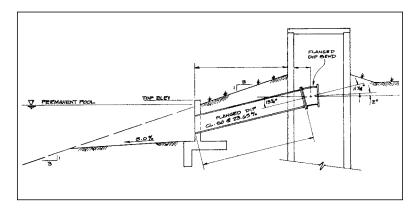


Figure 3.4.5-1 Reverse Slope Pipe Outlet

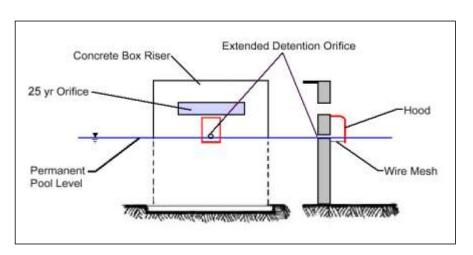


Figure 3.4.5-2 Hooded Outlet

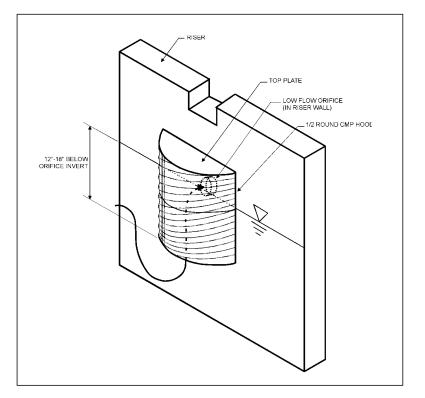


Figure 3.4.5-3 Half-Round CMP Orifice Hood

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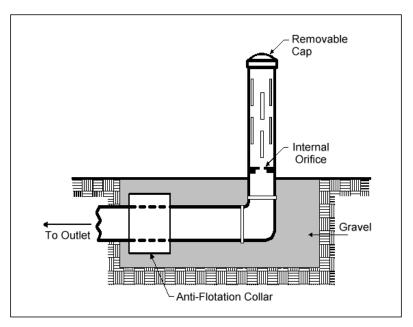


Figure 3.4.5-4 Internal Control for Orifice Protection

3.4.6 Trash Racks and Safety Grates

3.4.6.1 Introduction

The susceptibility of larger inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions:

- Keeping debris away from the entrance to the outlet works where they will not clog the critical portions of the structure
- Capturing debris in such a way that relatively easy removal is possible
- Ensuring that people and large animals are kept out of confined conveyance and outlet areas
 Providing a safety system that prevents anyone from being drawn into the outlet and allows
- them to climb to safety

When designed properly, trash racks serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985; Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors, including head losses through the rack, structural convenience, safety and size of outlet. Well-designed trash racks can also have an aesthetically pleasing appearance.

An example of trash racks used on a riser outlet structure is shown in Figure 3.4.6-1. Additional track-<u>trash</u> rack design can be found in Appendix C. The inclined vertical bar

rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

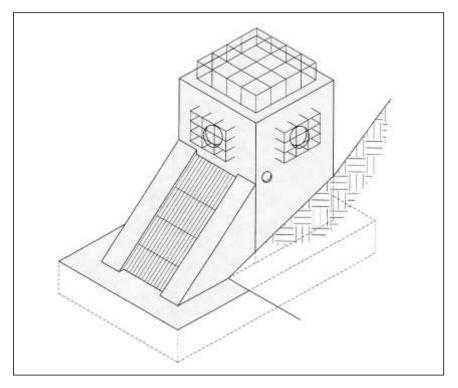


Figure 3.4.6-1 Example of Various Trash Racks Used on a Riser Outlet Structure (Source: VDCR, 1999)

3.4.6.2 Trash Rack Design

Trash racks must be large enough such that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice.

The surface area of all trash racks should be maximized and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level—the slower the approach flow, the flatter the angle. Rack opening rules-of-thumb are found in literature. Figure 3.4.6-2 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack.

Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978; UDFCD, 1992). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore head losses through the grate should be calculated. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Metcalf & Eddy (1972) give the following equation (based on German experiments) for losses. Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50% is chosen as a working assumption.

$$H_g = K_{g1} (w/x)^{4/3} (V_u^2/2g) \sin \theta_g$$

(3.4.12)

Where: H_g = head loss through grate (ft)

- K_{g1} = bar shape factor:
 - 2.42 sharp edged rectangular
 - 1.83 rectangular bars with semicircular upstream faces
 - 1.79 circular bars
- 1.67 rectangular bars with semicircular up- and downstream faces
- w = maximum cross-sectional bar width facing the flow (in)
- x = minimum clear spacing between bars (in)
- V_u = approach velocity (ft/s)
- θ_g = angle of the grate with respect to the horizontal (degrees)

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

$$H_g = \frac{K_g^2 V_u^2}{(3.4.13)}$$

Where K_{g2} is defined from a series of fit curves as:

- sharp edged rectangular (length/thickness = 10) K_{g2} = 0.00158 - 0.03217 A_r + 7.1786 $A_r{}^2$
- sharp edged rectangular (length/thickness = 5) K_{g2} = -0.00731 + 0.69453 A_r + 7.0856 A_r^2
- round edged rectangular (length/thickness = 10.9) K_{g2} = -0.00101 + 0.02520 A_r + 6.0000 A_r^2
- circular cross section $K_{g2} = 0.00866 + 0.13589 \; A_r + 6.0357 \; A_r^2$

and A_r is the ratio of the area of the bars to the area of the grate section.

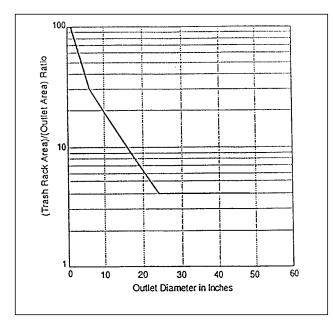


Figure 3.4.6-2 Minimum Rack Size vs. Outlet Diameter (Source: UDCFD, 1992)

3.4.7 Secondary Outlets

3.4.7.1 Introduction

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure 3.4.7-1 shows an example of an emergency spillway.

In many cases, on-site stormwater storage facilities do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir.

By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor.

3.4.7.2 Emergency Spillway Design

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see Figure 3.4.7-1). The emergency spillway is proportioned to pass flows in excess of the design flood (typically the 100-year flood or greater) without allowing excessive velocities and without overtopping of the embankment. Flow in the emergency spillway is open channel flow (see Section 54, *Open Channel Design*, for more information). Normally, it is assumed that critical depth occurs at the control section.

NRCS (SCSNRCS TR-55) manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both the inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper the 3:1 horizontal to vertical.

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated below.

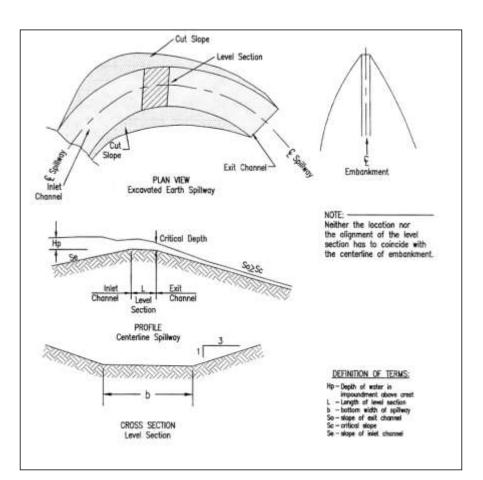


Figure 3.4.7-1 Emergency Spillway (Source: VDCR, 1999)

References

Ferguson, B. and Debo, T., 1990. On-Site Stormwater Management.

Ferguson, B., 1996. <u>Estimation of Direct Runoff in the Thornthwaite Water Balance</u>. Prof. Geographer, October 1996, pp. 263-271.

Hershfield, D. M., 1961. Rainfall Frequency Atlas of the United States, Technical Paper 40.

NOAA, 1977. Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States, NOAA Technical Memo. NWS HYDRO-35.

NOAA, 1982. <u>Evaporation Atlas for the Contiguous 48 United States</u>, NOAA Technical Report NWS 33.

Pitt, Robert, 1994. <u>Small Storm Hydrology</u>. Unpublished report. Department of Civil Engineering, University of Alabama at Birmingham.

Schueler, Thomas R., 1987. <u>Controlling Urban Runoff</u>. Washington Metropolitan Water Resources Planning Board.

Maryland Department of the Environment, 2000. <u>Maryland Stormwater Design Manual</u>, <u>Volumes I and II</u>. Prepared by Center for Watershed Protection (CWP).

U. S. Department of Agriculture, Soil Conservation Service, Engineering Division, 1985. SCSNRCS TR-55 National Engineering Handbook.

U. S. Department of Agriculture, Soil Conservation Service, Engineering Division, 1986. <u>Urban</u> <u>Hydrology for Small Watersheds</u>. Technical Release 55 (TR-55).

U. S. Department of Transportation, Federal Highway Administration, 1984. <u>Hydrology</u>. Hydraulic Engineering Circular No. 19.

U. S. Geological Survey, 2000. <u>Lagtime Relations for Urban Streams In Georgia</u>. Water-Resources Investigation Report 00-4049.

U. S. Geological Survey, 1994. <u>Flood-frequency Relations for Urban Streams in Georgia</u>. Water-Resources Investigation Report 95-4017.

U. S. Geological Survey, 1986. <u>Simulation of Flood Hydrographs for Georgia Streams</u>. Water-Resources Investigation Report 86-4004.

U. S. Geological Survey, 1993. <u>Techniques for Estimating Magnitude and Frequency of Floods in</u> <u>Rural Basins of Georgia</u>. Water Resources Investigation Report 93-4016.

Wright-McLaughlin Engineers, 1969. <u>Urban Storm Drainage Criteria Manual</u>. Volumes I and II. Prepared for the Denver Regional Council of Governments, Denver, Colorado.

Debo, Thomas N., and Andrew J. Reese, 1995. <u>Municipal Storm Water Management</u>. Lewis Publishers: CRC Press, Inc., Boca Raton, Florida.

Metropolitan Government of Nashville and Davidson County, 1988. <u>Stormwater</u> <u>Management Manual - Volume 2 Procedures</u>. Prepared by AMEC, Inc. (formerly The Edge Group) and CH2M Hill.

Wycoff, R. L. and U. P. Singh, 1976. Preliminary Hydrologic Design of Small Flood

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1

<u>Detention Reservoirs</u>. Water Resources Bulletin. Vol. 12, No. 2, pp 337-49. Brater, E. F. and H. W. King, 1976. <u>Handbook of Hydraulics</u>. 6th ed. New York: McGraw Hill Book Company.

Chow, C. N., 1959. Open Channel Hydraulics. New York: McGraw Hill Book Company.

Debo, Thomas N and Andrew J. Reese, 1995. <u>Municipal Storm Water Management</u>. Lewis Publishers: CRC Press, Inc., Boca Raton, Florida.

McEnroe, B.M., J.M. Steichen and R. M. Schweiger, 1988. <u>Hydraulics of Perforated</u> <u>Riser Inlets for Underground Outlet Terraces</u>, Trans ASAE, Vol. 31, No. 4, 1988.

NRCS, 1984. <u>Engineering Field Manual for Conservation Practices</u>, Soil Conservation Service, Engineering Division, Washington, D.C.

Virginia Department of Conservation and Recreation, 1999. <u>Virginia Stormwater</u> <u>Management Handbook</u>.

Sandvik, A., 1985. <u>Proportional Weirs for Stormwater Pond Outlets</u>. Civil Engineering, March 1985, ASCE pp. 54-56.

Metropolitan Government of Nashville and Davidson County, 1988. <u>Stormwater</u> <u>Management Manual - Volume 2 Procedures</u>. Prepared by AMEC, Inc. (formerly The Edge Group) and CH2M Hill.

United States Bureau of Reclamation, <u>Water Measurement Manual</u>, http://www.usbr.gov/wrrl/fmt/wmm/

Urban Drainage and Flood Control District, 1999. Criteria Manual, Denver, CO.

Wycuff, R. L. and U. P. Singh, 1976. <u>Preliminary Hydrologic Design of Small Flood</u> <u>Detention Reservoirs</u>. Water Resources Bulletin. Vol. 12, No. 2, pp 337-49.