

# Chapter 11 – Stormwater Management

## TABLE OF CONTENTS

<b>CHAPTER 11 - STORMWATER MANAGEMENT</b> .....	<b>11-1</b>
<b>11.1 Introduction</b> .....	<b>11-1</b>
<b>11.2 Design Policy</b> .....	<b>11-2</b>
<b>11.3 General Design Criteria</b> .....	<b>11-3</b>
11.3.1 Introduction .....	11-3
11.3.2 Pre-Development Conditions .....	11-3
11.3.3 Hydrology .....	11-3
11.3.4 Multi-Use Facilities .....	11-4
11.3.4.1 Quality versus Quantity .....	11-4
11.3.4.2 Temporary versus Permanent .....	11-4
11.3.5 Impounding Facilities .....	11-4
11.3.6 Regional Facilities .....	11-5
11.3.7 Right of Way/Permanent Easements .....	11-6
11.3.8 Fencing .....	11-6
11.3.9 Plan Details .....	11-7
11.3.9.1 Stormwater Management Profiles and Cross Sections .....	11-7
11.3.9.2 Stormwater Management Details – R&B Standard SWM-DR.....	11-7
11.3.9.3 Stormwater Management Summary .....	11-8
11.3.9.4 Method of Measurement – Basis of Payment.....	11-8
11.3.10 Maintenance .....	11-8
<b>11.4 Part II B Design Criteria</b> .....	<b>11-10</b>
11.4.1 Water Quality .....	11-10
11.4.2 Water Quantity .....	11-12
11.4.2.1 Channel Protection .....	11-12
11.4.2.2 Flood Protection.....	11-14
11.4.2.3 Sheet Flow .....	11-15
<b>11.5 Part II B Design Concepts</b> .....	<b>11-16</b>
11.5.1 Water Quality .....	11-16
11.5.1.1 Offsite Water Quality Compliance .....	11-17
11.5.1.2 Compliance Spreadsheets .....	11-17
11.5.1.3 Land Cover and Soil Groups .....	11-18
11.5.2 Water Quantity .....	11-19
11.5.2.1 Channel Protection .....	11-19
11.5.2.1.1 Manmade Stormwater Conveyance System .....	11-20
11.5.2.1.2 Restored Conveyance System .....	11-20
11.5.2.1.3 Natural Conveyance System .....	11-20
11.5.2.1.4 Energy Balance .....	11-20
11.5.2.1.5 Improvement Factor (I.F.) .....	11-21
11.5.2.1.6 Forested Conditions .....	11-22
11.5.2.1.7 Limits of Analysis .....	11-23
11.5.2.1.8 Runoff Reduction .....	11-23
11.5.2.1.9 Increases in Peak Rate of Runoff .....	11-25
11.5.2.2 Flood Protection .....	11-25
11.5.2.2.1 Conveyance System Definition .....	11-26
11.5.2.2.2 Localized Flooding not Currently Experienced.....	11-26
11.5.2.2.3 Localized Flooding Currently Experienced .....	11-26

	11.5.2.2.4	Compliance with the Flood Protection Criteria .....	11-27
	11.5.2.2.5	Limits of Analysis .....	11-27
	11.5.2.3	Sheet Flow .....	11-28
<b>11.5.3</b>		<b>Part II B Design Procedures and Sample Problems .....</b>	<b>11-29</b>
	11.5.3.1	SWM Plan Requirements .....	11-29
	11.5.3.2	Water Quality – Runoff Reduction Method (RRM) Procedure .....	11-29
	11.5.3.2.1	BMPs In-Series/Treatment Trains .....	11-34
	11.5.3.2.2	Offsite Water Quality Compliance Options .....	11-34
	11.5.3.2.3	Water Quality – Sample Problem – New Development .....	11-35
	11.5.3.2.4	Water Quality – Sample Problem – Redevelopment .....	11-44
	11.5.3.3	Water Quantity – Channel Protection Procedure .....	11-53
	11.5.3.3.1	Channel Protection - Limits of Analysis Sample Problems .....	11-56
	11.5.3.3.2	Channel Protection - Adjusted CN Sample Problem .....	11-57
	11.5.3.3.3	Channel Protection - Manmade System Sample Problem .....	11-57
	11.5.3.3.4	Channel Protection - Restored System Sample Problem .....	11-58
	11.5.3.3.5	Channel Protection - Natural System Sample Problem .....	11-58
	11.5.3.4	Water Quantity – Flood Protection Procedure .....	11-60
	11.5.3.4.1	Flood Protection - Limits of Analysis .....	11-64
	11.5.3.4.2	Flood Protection - Limits of Analysis Sample Problems .....	11-64
	11.5.3.4.3	Flood Protection Sample Problem – Localized Flooding Not Currently Experienced .....	11-65
	11.5.3.4.4	Flood Protection Sample Problem – Localized Flooding Currently Experienced .....	11-68
<b>11.5.4</b>		Pretreatment .....	11-69
<b>11.5.5</b>		Treatment Volume Computation .....	11-70
<b>11.5.6</b>		Detention Time Computation and Orifice Sizing .....	11-70
	11.5.6.1.1	Average Hydraulic Head Method (Method #2 from VSWMH) - VDOT Preferred Method .....	11-71
	11.5.6.1.2	Maximum Hydraulic Head Method (Method #1 from VSWMH) ..	11-73
	11.5.6.1.3	Tv Hydrograph .....	11-73
	11.5.6.1.4	Alternative Method of Routing WQV to Find Drawdown Time ...	11-74
<b>11.5.7</b>		Preliminary Detention Volume Computation .....	11-74
	11.5.7.1.1	Modified Rational Method, Simplified Triangular Hydrograph ....	11-74
		Routing .....	11-74
	11.5.7.1.2	Critical Storm Duration Method .....	11-76
	11.5.7.1.3	Pagan Volume Estimation Method .....	11-77
	11.5.7.1.4	Sample Problems – Using 3 Methods to Estimate Volume of Storage for Quantity Control .....	11-77
<b>11.5.8</b>		Preliminary Basin Sizing .....	11-80
<b>11.5.9</b>		Final Basin Sizing – Reservoir Routing .....	11-82
	11.5.9.1	Storage – Indication Method Routing Procedure .....	11-82
	11.5.9.2	Storage – Indication Method Routing Sample Problem #1 .....	11-85
<b>11.6</b>		<b>Part II C Design Criteria .....</b>	<b>11-100</b>
	11.6.1	Water Quality .....	11-100
	11.6.2	Water Quantity .....	11-102
	11.6.3	Compensatory Treatment .....	11-103
	11.6.4	Embankment (Dam) .....	11-104
	11.6.5	Basin Grading .....	11-105
	11.6.6	Sediment Forebay .....	11-107
<b>11.7</b>		<b>Part II C Design Concepts .....</b>	<b>11-109</b>
	11.7.1	Water Quality .....	11-109
	11.7.2	Water Quantity .....	11-110
	11.7.3	Extended Detention vs. Retention .....	11-110
	11.7.4	Detention Time .....	11-111
	11.7.5	Release Rates .....	11-111

11.7.5.1	Channel Erosion Control – Q <sub>1</sub> Control .....	11-112
11.7.5.2	Flooding .....	11-114
11.7.5.3	Water Quality Control .....	11-115
<b>11.7.6</b>	<b>Outlet Hydraulics .....</b>	<b>11-120</b>
11.7.6.1	Orifice .....	11-120
11.7.6.2	Weirs .....	11-120
11.7.6.3	Types of Outlet Structures .....	11-121
<b>11.7.6.3.1</b>	<b>General .....</b>	<b>11-121</b>
<b>11.7.6.3.2</b>	<b>SWM-1 (VDOT Standard) .....</b>	<b>11-121</b>
<b>11.7.6.3.3</b>	<b>Weir Wall (Stormwater Management Dam) .....</b>	<b>11-122</b>
11.7.6.4	Routing .....	11-122
<b>11.8</b>	<b>Part II C Design Procedures and Sample Problems .....</b>	<b>11-123</b>
<b>11.8.1</b>	<b>Documentation Requirements .....</b>	<b>11-123</b>
<b>11.8.2</b>	<b>Water Quality Volume Computation and BMP Selection Procedure .....</b>	<b>11-123</b>
<b>11.8.3</b>	<b>Detention Time Computation and Orifice Sizing .....</b>	<b>11-125</b>
11.8.3.1	Average Hydraulic Head Method (Method #2) - VDOT Preferred Method .....	11-125
11.8.3.2	Maximum Hydraulic Head Method (Method #1) .....	11-126
11.8.3.3	WQV Hydrograph (HYG) .....	11-127
11.8.3.4	Alternative Method of Routing WQV to Find Drawdown Time .....	11-127
<b>11.8.4</b>	<b>Channel Erosion Control Volume – Q<sub>1</sub> Control .....</b>	<b>11-127</b>
<b>11.8.5</b>	<b>Preliminary Detention Volume Computation .....</b>	<b>11-129</b>
11.8.5.1	Modified Rational Method, Simplified Triangular Hydrograph Routing .....	11-129
11.8.5.2	Critical Storm Duration Method .....	11-131
11.8.5.3	Pagan Volume Estimation Method .....	11-132
11.8.5.4	Sample Problems – Using 3 Methods to Estimate Volume of Storage for Quantity Control .....	11-133
<b>11.8.6</b>	<b>Determine Preliminary Basin Size .....</b>	<b>11-135</b>
<b>11.8.7</b>	<b>Final Basin Sizing-Reservoir Routing .....</b>	<b>11-136</b>
11.8.7.1	Storage – Indication Method Routing Procedure .....	11-136
11.8.7.2	Storage – Indication Method Routing Sample Problem .....	11-139
11.8.7.3	SWM Basin Design: Sample Problem .....	11-145
<b>11.9</b>	<b>References .....</b>	<b>11-151</b>

### List of Tables

Table 11-1.	WQV Orifice Sizes .....	11-71
Table 11-2.	Storage Characteristics .....	11-84
Table 11-3.	Runoff Hydrographs.....	11-86
Table 11-4.	Stage-Discharge-Storage Data .....	11-88
Table 11-5.	Storage Routing for the 2-yr Storm.....	11-90
Table 11-6.	Storage Routing for the 10-yr Storm.....	11-90
Table 11-7.	BMP Selection Table .....	11-102
Table 11-8.	Summary of Design Criteria for Dry and Wet Basins .....	11-107
Table 11-9.	Functional Goals of Stormwater BMPs.....	11-110
Table 11-10.	WQV Orifice Sizes .....	11-125
Table 11-11.	Storage Characteristics .....	11-137
Table 11-12.	Runoff Hydrographs.....	11-140

Table 11-13. Stage-Discharge-Storage Data .....	11-141
Table 11-14. Storage Routing for the 2-yr Storm.....	11-143
Table 11-15. Storage Routing for the 10-yr Storm.....	11-143

### **List of Figures**

Figure 11-1. Simplified Triangular Hydrograph Method.....	11-75
Figure 11-2. Pagan Method Curve.....	11-77
Figure 11-3. Stage-Storage Curve.....	11-83
Figure 11-4. Stage-Discharge Curve .....	11-83
Figure 11-5. Storage Characteristics Curve.....	11-84
Figure 11-6. Runoff Hydrographs .....	11-91
Figure 11-7. Typical SWM Basin Dam.....	11-105
Figure 11-8. Typical Sediment Forebay Plan and Section .....	11-108
Figure 11-9. Removal Rates vs. Detention Time.....	11-111
Figure 11-10. Simplified Triangular Hydrograph Method.....	11-131
Figure 11-11. Pagan Method Curve.....	11-133
Figure 11-12. Stage-Storage Curve.....	11-136
Figure 11-13. Stage-Discharge Curve .....	11-137
Figure 11-14. Storage Characteristics Curve .....	11-138
Figure 11-15. Runoff Hydrographs .....	11-144

### **List of Appendices**

Appendix 11B-1 SWM and TSB Summary Table
Appendix 11B-2 Redevelopment/Surplus Credit Tracking Form (LD-458)
Appendix 11C-1 SWM Facility Tabulation Sheet

# ***Chapter 11 - Stormwater Management***

## ***11.1 Introduction***

Development of watersheds has the potential for generally causing an increase in the peak flow rate of stormwater runoff. This increase is often associated with flood damage, erosion, and siltation control problems, and increased pollutant loads. Urban development has been identified as having a direct impact on the hydrologic cycle by reducing, or even eliminating, the natural storage capacity of the land. These natural storage areas are then replaced with impervious and managed pervious surfaces. Impervious cover prevents the infiltration of rainfall into the soil and increases the speed and quantity of rainfall runoff to the outfall. Increased stormwater runoff impacts water quality, stream channel erosion, and localized flooding. For a watershed with no defined, or inadequate, outfall, the total volume of runoff is critical and SWM facilities (SWMFs) can be used to store the increases in volume and control peak discharge rates.

The goal of stormwater management (SWM) is to inhibit the deterioration of the aquatic environment by instituting a program that maintains both water quantity and quality post-development runoff characteristics, as nearly as practicable, equal to or better than pre-development runoff characteristics, and to limit the post-development peak discharge flow rates to match the predevelopment peak discharge flow rates.

### **Stormwater Quality Control**

Stormwater quality control pertains to reducing the amount of pollutants discharged by land development projects. This goal is achieved by reducing the volume of runoff to reduce pollutant loads, treating runoff to reduce pollutant concentration and loads, or a combination of both.

### **Stormwater Quantity Control**

Stormwater quantity control (or flooding and erosion control) pertains to replicating the water quantity post-development runoff characteristics and site hydrology, as nearly as practicable, equal to or better than the existing pre-development runoff characteristics, including runoff volume and peak rate of runoff.

## **11.2 Design Policy**

Acts of the General Assembly have resulted in the issuance of the Stormwater Management Regulation and Erosion and Sediment Control Regulations. The Virginia Stormwater Management Program Regulation can be obtained at the Virginia Legislative Information System (LIS) web site at

<https://law.lis.virginia.gov/admincode/title9/agency25/chapter870/>. The Erosion and Sediment Control Regulations can also be obtained at the LIS web site at <https://law.lis.virginia.gov/admincode/title9/agency25/chapter840/>.

The general application to highway drainage design associated with these regulations is addressed here and also in the latest version of VDOT Location & Design Instructional & Informational Memorandum IIM-LD-195. Effective July 1, 2014, Water Quality and Water Quantity are now governed by two (2) distinct methods, depending on whether the project follows Part II B or Part II C technical criteria. Please refer to IIM-LD-195 which provides guidance in determining which technical criteria governs for a given project.

## **11.3 General Design Criteria**

### **11.3.1 Introduction**

Depending on which technical criteria (Parts II B & II C) governs a given project, the design elements for Stormwater Management Facilities (SWMFs) or Best Management Practices (BMPs) will be different. For example, Part II B includes 9VAC25-870-62 through 9VAC25-870-92, and Part II C includes 9VAC25-870-93 through 9VAC25-870-99.

For those projects following Part II C, BMPs will closely follow design criteria as still recognized by the Virginia Stormwater Management Handbook (Blue Book) at <http://www.deq.virginia.gov/Programs/Water/StormwaterManagement/Publications.aspx> and also pursuant to the information in this chapter. VDOT published a Part II C BMP Design Manual of Practice for designers and plan reviewers to use and reference at [http://www.virginiadot.org/business/resources/LocDes/Part II C BMP Design Manual.pdf](http://www.virginiadot.org/business/resources/LocDes/Part_II_C_BMP_Design_Manual.pdf)

For those projects following Part II B, the design of BMPs will follow design criteria as identified in the VDOT Part II B BMP Design Manual of Practice located at [http://www.virginiadot.org/business/resources/LocDes/Part II B BMP MOP Combined 6-24-19.pdf](http://www.virginiadot.org/business/resources/LocDes/Part_II_B_BMP_MOP_Combined_6-24-19.pdf); as well as DEQ-approved water quality BMP standards and special provisions. A designer can use the standards and specifications from the Virginia Stormwater BMP Clearinghouse; however, the designer is encouraged to follow the design practices in this Chapter, as these are specific to VDOT from a construction and maintenance standpoint.

### **11.3.2 Pre-Development Conditions**

For purposes of computing pre-development runoff, all pervious lands on the site shall be assumed to be in “good” hydrologic condition in accordance with the U.S. Department of Agriculture's Natural Resources Conservation Service (NRCS) standards, regardless of conditions existing at the time of computation. Pre-development runoff calculations utilizing other hydrologic conditions may be utilized provided that it is demonstrated to and approved by the DEQ that actual site conditions warrant such considerations.

### **11.3.3 Hydrology**

Hydrology should be performed using the appropriate hydrograph procedures presented in Chapter 6, Hydrology.

Unless otherwise specified, the prescribed design storms are the 1-year (yr), 2-yr, and 10-yr 24-hour storms using the site-specific rainfall precipitation frequency data recommended by the U.S. National Oceanic and Atmospheric Administration (NOAA) Atlas 14. Partial duration time series shall be used for the precipitation data.

Pre-development and post-development runoff characteristics and site hydrology shall be verified by site inspections, topographic surveys, available soil mapping or studies, and calculations consistent with good engineering practices.

Unless otherwise specified, all hydrologic analyses shall be based on the existing watershed characteristics and how the ultimate development condition of the subject project will be addressed.

### **11.3.4 Multi-Use Facilities**

#### **11.3.4.1 Quality versus Quantity**

SWMFs may function as both quantity control and quality control facilities, and are also known as a Best Management Practices or BMPs. Some facilities may only be needed for either quality or quantity control.

#### **11.3.4.2 Temporary versus Permanent**

Permanent SWMF may be utilized as temporary sediment basins during the construction phase of the project, and if so, the design of the SWMF will need to address this dual function. The design that is needed for a permanent SWMF may need to be altered to provide additional temporary sediment storage volume that is in excess of the applicable design volume. For design purposes, the two volumes (temporary sediment storage volume and post-construction volume) should not be added together, but rather the larger of the two should govern the facility's design.

The additional volume needed for temporary sediment storage may be provided by excavating the bottom of the basin lower than that required for the WQV. The basin's permanent outlet control structure can be temporarily altered to serve as the control structure for the temporary sediment basin (see Standard SWM-DR of VDOT's R&B Standards and the Virginia ESC Handbook). When the project is nearing completion, and the basin is no longer needed for temporary sediment control, the basin can be converted to satisfy the permanent SWMF requirements by regrading (excavating and/or filling) and removing any temporary control structure appurtenances.

### **11.3.5 Impounding Facilities**

SWM wet ponds and extended detention ponds that are not covered by the Impounding Structure Regulations (4VAC50-20 et seq.) shall, at a minimum, be engineered for structural integrity for the 100-yr storm event.

Construction of SWM impoundment structures or facilities may occur in karst areas only after a study of the geology and hydrology of the area has been conducted to determine the presence or absence of karst features that may be impacted by stormwater runoff and facility placement.



Permanent SWM impoundment structures or facilities shall only be constructed in karst features after completion of a geotechnical investigation that identifies any necessary modifications to the facility to ensure its structural integrity and maintain its water quality and quantity efficiencies. Any Class V Underground Injection Control Well registration statements for stormwater discharges to improved sinkholes shall be included in the SWPPP.

### **11.3.6 Regional Facilities**

There are many cases where it is more feasible to develop one major SWMF to control a large watershed area rather than a number of small individual facilities controlling small drainage areas within the large watershed. The concept of regional SWMFs is endorsed by VDOT provided that certain requirements are met.

When applicable, the regional facility shall comply with the impounding structure regulations (4VAC50-20 et seq.).

The regional facility is allowed to address water quality requirements and where allowed, water quantity requirements, in accordance with sections 9VAC25-870-69 and 9VAC25-870-92 of the VSMP regulations.

Development and/or use of regional SWMFs must be a joint undertaking by VDOT and the local governing body. VDOT shall not be owner of any such facility. The site must be part of a Master SWM Plan developed and/or approved by the local VSMP Authority and/or DEQ and any agreements related to the VDOT use of these facilities must be consummated between VDOT and the local governing body. VDOT may enter into an agreement with a private individual or corporation provided the local governing body has a DEQ approved SWM program that complies with the VSMP Regulations and the proper agreements for maintenance and liability of the regional facility have been executed between the local governing body and the private individual or corporation and any such agreements are referenced in the agreement between VDOT and the private individual or corporation.

When VDOT agrees to the use of an existing or future VDOT roadway embankment as an impounding structure for a regional facility, the roadway embankment must be designed or retrofitted appropriately for such use. The VDOT R/W line will normally be set at the inlet face of the main drainage structure.

The design of regional SWMFs must address any mitigation needed to meet the water quality and quantity requirements of any known future VDOT projects within the contributing watershed. Regional SWMFs located upstream of a proposed VDOT roadway shall provide sufficient mitigation for any water quality and quantity impacts of runoff from the proposed roadway project which may not pass through the proposed facility.

Any questions or concerns related to the use of an offsite regional SWMF to satisfy the VDOT post-development SWM requirements should be discussed between the SWM Plan Designer and the appropriate DEQ regional office prior to entering into any agreements with either private or public entities.

### **11.3.7 Right of Way/Permanent Easements**

Permanent SWMFs may be placed in fee R/W or in permanent easements.

It is recommended that all permanent SWM features (dams, risers, storage area etc.) be placed within fee R/W initially. Outfall ditches and similar features may initially be placed in permanent easements.

The final decision on R/W versus permanent easement should be made prior to the R/W (or similar) phase of the project development process based on information obtained at the Field Inspection, Design Public Hearing and/or other such plan review milestones.

VDOT will generally be amenable to the desires of the affected landowners regarding the fee R/W or permanent easement issue.

The multiple use of property for SWMFs and other features, such as utilities, is permissible. The decision on such use must be made on a case-by-case basis.

Permanent easements and/or other properties acquired through the R/W acquisition process, and which are considered a part of the “site” in determining the post-development SWM requirements for the project, are to remain under the ownership/control of VDOT for the life of the project and such property is to be identified/designated on the plans and legally encumbered for the purpose of SWM.

### **11.3.8 Fencing**

Fencing of SWMFs is normally not required and should not be considered for most practices due to:

- Insignificant Hazard – For detention basins (no permanent water pool), significant ponding of water in the basin should only occur with very heavy rainfall events and the maximum ponded depth should typically be no more than about 3’. Ponds and lakes are almost never fenced, even though they may be located in subdivisions and have deep, permanent water pools.
- Limits Maintenance Operations – Fencing could hinder the performance of both routine and long term maintenance operations. Fencing could become damaged during major maintenance operations and have to be repaired or replaced.

Fencing of SWMFs may occasionally be needed and should be used when:

- The basin is deep with a ponded depth greater than about 3' and/or has steep side slopes with two or more side slopes steeper than 3:1, or
- The basin is in close proximity to schools, playgrounds or similar areas where children may be expected to frequent, or
- It is recommended by the VDOT Field Inspection Review Team (or other such plan reviewing group), the VDOT Residency Administrator or the City/County (where City/County will take over maintenance responsibility), or
- A chain or gate will be needed on some basins to prohibit vehicular access for dumping or other undesirable access. The designer should seek input as appropriate from the District Hydraulic Engineer or the District Roadside Manager to determine any prohibition requirements.

Where fencing is proposed, access gate(s) of sufficient size to accommodate maintenance equipment are to be provided. Appropriate security mechanisms for the gates are to be provided to prevent/deter unauthorized entry.

For non-fenced basins, a chain barricade (see Standard CR-1 of VDOT's R&B Standards) or gate may be needed across the vehicular entrance to prohibit non-authorized access if there is a concern with illegal dumping or other undesirable activities at the site.

"No Trespassing" signs shall be considered for use on all basins, whether fenced or unfenced, and should be recommended, as needed, by the VDOT Field Inspection Review Team or other such plan reviewing group.

### **11.3.9 Plan Details**

#### **11.3.9.1 Stormwater Management Profiles and Cross Sections**

- To be provided for all SWMFs.

#### **11.3.9.2 Stormwater Management Details – R&B Standard SWM-DR**

- Includes details for debris rack, trash rack, concrete cradle, water quality orifice and modifications for use of SWMF as a temporary sediment basin.
- Specify at each SWMF location requiring any of the noted items.
- The location and the size of the water quality orifice or any other required openings in the control structure shall be specified in the description/details for the control structure for each SWMF.

### **11.3.9.3 Stormwater Management Summary**

- All drainage items related to the construction of SWMFs shall be summarized, by location, in the Drainage Summary for the project.
- All water quality requirements related to redevelopment projects shall be summarized on the Water Quality Redevelopment Tabulation Sheet in Appendix 11B-2.
- All incidental items related to the construction of SWMFs shall be summarized, by location, in the Incidental Summary for the project.
- Stormwater Management Excavation and Borrow or Embankment fill, if needed, are to be included in the totals on the Grading Diagram and/or Summary.

### **11.3.9.4 Method of Measurement – Basis of Payment**

- Stormwater Management Drainage Structure (SWM-1): Basis of payment to be linear feet (LF) measured from invert of structure to top of concrete. Price bid includes cost of trash rack, debris rack and holder, temporary dewatering device and temporary metal plates.
- Stormwater Management Dam (weir wall): Basis of payment to be cubic yards (CY) of Concrete Class A3 Miscellaneous and pounds (LBS) of Reinforcing Steel.
- Concrete Cradle: Basis of payment to be cubic yards (CY) of Concrete Class A3 Miscellaneous
- Grading:
  - Excavation for SWM basins will be measured and paid for as cubic yards (CY) of SWM Basin Excavation.
  - Fill material needed for dams or berms will be measured and paid for as cubic yards (CY) of Regular Excavation, Borrow Excavation or Embankment.
  - The Grading Diagram is to reflect how the cubic yards (CY) of SWM Management Basin Excavation and cubic yards (CY) of Embankment or Borrow is to be distributed.

### **11.3.10 Maintenance**

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed SWMFs. To assure acceptable performance and function, the designer should review the latest version of the VDOT Post-Construction BMP Inspection and Maintenance Manual as part of the design consideration.

The Manuals will identify the requirements for maintenance of SWMFs, the schedule for inspection and maintenance operations, and the identification of persons responsible for the maintenance. Proper design should focus on minimizing maintenance requirements by addressing the following potential problems:

- Both weed growth and grass maintenance should be addressed in the plan and design. When practical given R/W constraints, concerns may be addressed by

constructing side slopes no steeper than 3:1 so that they can be maintained using available power-driven equipment, such as tractor mowers.

- Sedimentation may be controlled by constructing forebays to contain sediment for easy removal.
- Bank deterioration can be controlled with protective lining, vegetation, or by limiting bank slopes.
- Standing water or soggy surfaces may be eliminated by sloping basin bottoms toward the outlet, or by constructing underdrain facilities to lower water tables. These measures also assist in mosquito control.
- Outlet structures should be selected to minimize the possibility of blockage.
- Locate the facility for easy access so that maintenance associated with litter and damage to fences and perimeter plantings can be conducted on a regular basis.
- A 10' wide access for inspection and maintenance personnel should be provided at each SWMF. The Standard PE-1 details shown in VDOT's Road and Bridge Standards should be used for vehicular entrances. A turnaround should be provided on vehicular entrances when needed based upon accessibility. Appropriate surface material should be provided for each vehicular entrance. The designer should seek input as appropriate from the District Hydraulic Engineer or the District Roadside Manager to determine the vehicular access requirements.
- VDOT maintenance procedures include inspecting each SWMF on an annual basis, and inspecting each SWMF after any storm that causes the capacity of the principal spillway to be exceeded. Basins should also have accumulated sediment removed about every 5 to 10 years or as necessary based upon a SWMF inspection.

## **11.4 Part II B Design Criteria**

### **11.4.1 Water Quality**

The following minimum design criteria and statewide standards for SWM shall be applied to the site:

- 1) New development. The total phosphorus load of new development projects shall not exceed 0.41 lb/ac/yr, as calculated pursuant to 9VAC25-870-65.
- 2) For prior developed lands on linear projects, a designer can reduce 20% below the pre-development total phosphorus load. However, the total phosphorus load shall not be required to be reduced to below the applicable standard for new development unless a more stringent standard has been established by a locality. As such, the following approach is recommended for VDOT linear projects utilizing the VRRM spreadsheets:
  - a. The calculations shall be limited to the disturbed area of the project, plus any new right of way or new permanent easements secured specifically for the project. Areas that qualify as “routine maintenance” (see the Virginia Stormwater Management Act § 62.1-44.15:34), such as mill and pave areas, should not be included as they are not required to meet the water quality requirements of Part II B.
  - b. The designer can utilize the VRRM redevelopment spreadsheet provided by DEQ (ensuring to utilize the most current, corrected version and checking the “linear development” box) to calculate compliance. The DEQ VRRM redevelopment spreadsheet will also address new development by comparing changes in land cover between the existing and proposed conditions. It is no longer necessary to use two spreadsheets: one for new development and one for redevelopment (development on prior developed land).
- 3) Compliance with the water quality design criteria shall be determined utilizing the Virginia Runoff Reduction Method or another equivalent methodology that is approved by the State Water Control Board.

- 4) The following BMPs are accepted by VDOT to effectively reduce pollutant loads and/or runoff volume:
  - Soil Amendments
  - Permeable Pavement
  - Grass Channel
  - Bioretention (and Bioretention Filter)
  - Infiltration
  - Dry Swale
  - Wet Swale
  - Sheet Flow to Vegetated Filter/Open Space
  - Extended Detention Pond
  - Filtering Practice
  - Constructed Wetland
  - Wet Pond
- 5) Manufactured Treatment Devices (MTDs) or proprietary BMPs accepted by DEQ may be utilized, when accepted by VDOT, in accordance with the design guidance and efficiencies approved by DEQ. See the VDOT Approved Products Lists or Special Products Evaluation List for manufactured or proprietary BMPs acceptable to VDOT: <http://www.virginiadot.org/business/bu-materials-New-Products.asp>.
- 6) Where a site drains to more than one HUC (as defined in 9VAC25-870-10), the pollutant load reduction requirements shall be applied independently within each HUC unless reductions are achieved in accordance with a comprehensive watershed stormwater management plan in accordance with 9VAC25-870-92.\*
- 7) Offsite compliance options may be used to meet required pollutant reductions, including the following:
  - a. Offsite controls utilized in accordance with a comprehensive SWM plan adopted pursuant to the VSMP Regulation,
  - b. Pollutant loading pro rata share programs established pursuant to § 15.2-2243 of the Code of Virginia,
  - c. The nonpoint nutrient offset program established pursuant to § 62.1-44.15:35 of the Code of Virginia,
  - d. Other offsite options approved by\* the DEQ, and
  - e. When VDOT has additional properties available within the same HUC or upstream HUC that the project directly discharges to, or within the same

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\* Rev. 10/23

watershed, offsite SWMFs on those properties may be utilized to meet the required pollutant reductions from the land-disturbing activity.

## **11.4.2 Water Quantity**

Compliance with the minimum standards set out in this section is deemed to satisfy the requirements of subdivision 19 of 9VAC25-840-40 (Minimum Standard 19 or MS-19) for ESC Plans.

The U.S. Department of Agriculture's Natural Resources Conservation Service (NRCS) synthetic 24-hour rainfall distribution and models, including, but not limited to NRCS Methods (including TR-55, TR-20, or EFH-2); hydrologic and hydraulic methods developed by the U.S. Army Corps of Engineers; or other standard hydrologic and hydraulic methods, shall be used to conduct the analyses of SWM compliance.

### **11.4.2.1 Channel Protection**

Concentrated stormwater flow shall be released into a stormwater conveyance system and meet the criteria in subdivision 1, 2, or 3 below, from the point of discharge to the limits of analysis as defined in subdivision 4 below.

1. Manmade stormwater conveyance systems. When stormwater from a development is discharged to a manmade stormwater conveyance system, following the land-disturbing activity, either:
  - a. The manmade stormwater conveyance system shall convey the post-development peak flow rate from the 2-yr 24-hour storm event without causing erosion of the system. Detention of stormwater or downstream improvements may be incorporated into the approved land-disturbing activity to meet this criterion, at the discretion of VDOT; or
  - b. The peak discharge requirements for concentrated stormwater flow to natural stormwater conveyance systems in subdivision 3 of this subsection shall be met.
2. Restored stormwater conveyance systems. When stormwater from a development is discharged to a restored stormwater conveyance system that has been restored using natural design concepts, following the land-disturbing activity, either:
  - a. The development shall be consistent, in combination with other stormwater runoff, with the design parameters of the restored stormwater conveyance system that is functioning in accordance with the design objectives; or
  - b. The peak discharge requirements for concentrated stormwater flow to natural stormwater conveyance systems in subdivision 3 of this subsection shall be met.
3. Natural stormwater conveyance systems. When stormwater from a development is discharged to a natural stormwater conveyance system, the maximum peak flow rate from the 1-yr 24-hour storm following the land-disturbing activity shall be calculated either:



a. In accordance with the following methodology (referred to as the Energy Balance):

$$Q_{\text{Developed}} = \text{I.F.} \times \left( \frac{Q_{\text{Pre-Developed}} \times RV_{\text{Pre-Developed}}}{RV_{\text{Developed}}} \right)$$

Under no condition shall  $Q_{\text{Developed}}$  be higher than  $Q_{\text{Pre-Developed}}$  nor shall  $Q_{\text{Developed}}$  be required to be less than that calculated in the following equation:

$$Q_{\text{Developed}} = \left( \frac{Q_{\text{Forest}} \times RV_{\text{Forest}}}{RV_{\text{Developed}}} \right)$$

Where:

I.F. (Improvement Factor) = 0.8 for sites > 1 acre LDA or 0.9 for sites ≤ 1 acre LDA

$Q_{\text{Developed}}$  = the allowable peak flow rate of runoff from the developed site for the 1-yr 24-hour storm.

$RV_{\text{Developed}}$  = the volume of runoff from the site in the developed condition for the 1-yr 24-hour storm.

$Q_{\text{Pre-Developed}}$  = the peak flow rate of runoff from the site in the pre-developed condition for the 1-yr 24-hour storm.

$RV_{\text{Pre-Developed}}$  = the volume of runoff from the site in pre-developed condition for the 1-yr 24-hour storm.

$Q_{\text{Forest}}$  = the peak flow rate of runoff from the site in a forested condition for the 1-yr 24-hour storm.

$RV_{\text{Forest}}$  = the volume of runoff from the site in a forested condition for the 1-yr 24-hour storm.

b. In accordance with another methodology that is demonstrated to achieve equivalent results and is approved by the Board.

4. Limits of analysis. Unless subdivision 3 of this subsection is utilized to show compliance with the channel protection criteria, stormwater conveyance systems shall be analyzed for compliance with channel protection to a point where either:<sup>\*</sup>

a. Based on land area, the site's contributing drainage area is less than or equal to 1.0% of the total watershed area; or

b. Based on peak flow rate, the site's peak flow rate from the 1-yr 24-hour storm is less than or equal to 1.0% of the existing peak flow rate from the 1-yr 24-hour storm, prior to the implementation of any stormwater quantity control measures.

The point where the limit of analysis occurs does not need to be included in the evaluation of adequacy, but the system from the outfall or point of discharge up to<sup>†</sup>

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\* Rev. 10/23

† Rev. 10/23

the limits of analysis must be analyzed. If there is a natural channel anywhere in the system from the outfall up to the limit of analysis, then the energy balance will apply. Note that the limits of analysis are applied separately for each outfall or point of discharge from a regulated land-disturbing activity.

#### **11.4.2.2 Flood Protection**

For the purposes of this section, flooding and all flow rates are to be analyzed by the use of a 10-yr 24-hour storm event. Concentrated stormwater flow shall be released into a stormwater conveyance system and shall meet one of the following criteria as demonstrated by use of acceptable hydrologic and hydraulic methodologies:

1. If the stormwater conveyance system currently does not experience localized flooding: The point of discharge releases stormwater into a stormwater conveyance system that, following the land-disturbing activity, confines the post-development peak flow rate within the stormwater conveyance system, this provision is satisfied. Detention or downstream improvements may be incorporated into the land-disturbing activity to satisfy this criterion.
2. If the stormwater conveyance system currently experiences localized flooding the point of discharge either:
  - a. Confine the post-development peak flow rate within the stormwater conveyance system to avoid the localized flooding (detention or downstream improvements may accomplish this), or;
  - b. Release a post-development peak flow rate that is less than the pre-development peak flow rate (no downstream analysis is required if this option is employed).
3. Limits of analysis. Unless otherwise stated in 2.b above, stormwater conveyance systems shall be analyzed for compliance with flood protection criteria to a point where:
  - a. The site's contributing drainage area is less than or equal to 1.0% of the total watershed area draining to a point of analysis in the downstream stormwater conveyance system;
  - b. Based on peak flow rate, the site's peak flow rate is less than or equal to 1.0% of the existing peak flow rate prior to the implementation of any stormwater quantity control measures, or;
  - c. The stormwater conveyance system enters a mapped floodplain or other flood-prone area where development is prohibited. Flood-prone areas may include, but are not limited to, the floodplain, the floodway, the flood fringe, wetlands, riparian buffers, or other areas adjacent to the main channel.

The point where the limit of analysis occurs does not need to be included in the evaluation of adequacy, but the system from the outfall or point of discharge up to the limits of analysis must be analyzed. If there is a natural channel anywhere in the system from the outfall up to the limit of analysis, then the energy balance will apply. Note that the limits of analysis are applied separately for each outfall or point of discharge from a regulated land-disturbing activity.

### **11.4.2.3 Sheet Flow**

Increased volumes of sheet flow resulting from pervious or disconnected impervious areas, or from physical spreading of concentrated flow through level spreaders, must be identified and evaluated for potential impacts on down-gradient properties or resources. Increased volumes of sheet flow that will cause or contribute to erosion, sedimentation, or flooding of down gradient properties or resources shall be diverted to a SWMF or a stormwater conveyance system that conveys the runoff without causing down-gradient erosion, sedimentation, or flooding. If all runoff from the site is at the point of analysis sheet flow and the conditions of this subsection are met, no further water quantity controls are required. The designer is required to document that increases in sheet flows meet these conditions.

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\* Rev. 10/23

## **11.5 Part II B Design Concepts**

### **11.5.1 Water Quality**

Stormwater runoff can have a significant impact on the environment. Various pollutants are found in stormwater runoff, and studies show that the common sources of these pollutants are atmospheric deposition, urban and agricultural land uses, and natural spaces. Impervious surfaces, such as parking lots, rooftops, and roads associated with land development serve to accumulate and transport these pollutants to receiving streams. Also, the conversion of pervious surfaces such as undisturbed forest, meadow, and other open spaces to managed turf can increase runoff and the amount of pollution in the runoff.

Control of stormwater quality offers the following potential benefits:

- Improved surface water quality through runoff reduction;
- Recharge of groundwater resources;
- Maintenance of historic base flow rates and stream hydrology; and
- Protection of surface water quality through treatment of runoff.

Under the Virginia Runoff Reduction Method (VRRM), water quality design relies on three mechanisms to control pollution on stormwater runoff: reduce pollutant sources by minimizing land disturbance through environmental site design; reducing runoff volume by retaining and infiltrating runoff; and treating the remaining runoff through the application of stormwater best management practices (BMPs). The selection and sizing of BMPs, which use a number of physical, chemical, and biological mechanisms to control pollutants, is generally based upon the contributing drainage area, storage and treatment volume required, and flow rate through the system.

One of the first considerations in selecting a stormwater BMP is the functional goal of the BMP. The main components of stormwater management (SWM) are: water quality, channel protection, and flood protection. Any one or a combination of these components will dictate the functional goal of the BMP design. In general, SWMFs can be categorized into water quality BMPs and water quantity (channel and flood protection) facilities.

The use of some BMPs is limited by site or watershed feasibility factors, such as environmental impacts, drainage area or watershed size, topographic constraints, underlying soils and geology, seasonal high groundwater table, and other constraints specific to a project.

The BMPs designed for water quality control provide varying levels of runoff reduction and pollutant removal. Phosphorus is the keystone pollutant targeted for removal in Virginia, although sediment and nitrogen remain pollutants of concern and are addressed in water quality BMP design.

### 11.5.1.1 Offsite Water Quality Compliance

Offsite alternatives as describe in 9VAC25-870-69 may be utilized to meet the design criteria for water quality under certain situations. Offsite options shall not be allowed:

- Unless the selected offsite option achieves the necessary pollutant load reductions prior to the commencement of the construction of the proposed project. Where the offsite option will be constructed as a part of the proposed VDOT project, the offsite option must be completed and functional prior to the completion of the VDOT project, or
- In violation of local water quality-based limitations at the point of discharge that are consistent with the determinations made pursuant to a TMDL Implementation Plan, contained in a MS4 Program Plan approved by DEQ, or as otherwise may be established or approved by DEQ.

A common offsite compliance option used for VDOT projects is the purchase of nutrient credits as discussed in IIM-LD-251, which reflects the requirements in 9VAC25-900-91.\*

### 11.5.1.2 Compliance Spreadsheets

Compliance with the water quality design criteria set out in subdivisions A 1 and A 2 of 9VAC25-870-63 shall typically be determined by utilizing the Virginia Runoff Reduction Method (Virginia Runoff Reduction Method: Instructions & Documentation, March 28, 2011) or another equivalent methodology that is approved by the State Water Control Board. VDOT may utilize the DEQ VRRM Compliance Spreadsheets or any proprietary or non-proprietary spreadsheet or software which properly incorporates the VRRM for assessing compliance with pollutant removal requirements. Designers are responsible for ensuring that their use of the software (inputs and outputs) is consistent with the VRRM and applicable technical criteria. Load reductions for new development and redevelopment will typically be calculated using these tools, unless separate calculations are required due to limitations in the tool (e.g. in some instances, the DEQ VRRM Redevelopment Spreadsheet overestimates required load reductions for prior developed lands with high existing turf loadings. In these instances, VDOT may elect to utilize an alternate spreadsheet which calculates required load reductions in accordance with the method). Additional calculations will be prepared in general accordance with the Virginia Runoff Reduction Method.

The BMPs addressed in the most recent VDOT BMP Design Manual of Practice and contained in the most recent versions of VDOT's approved special provisions and standard insertable sheets for Runoff Reduction Practices are preferred to satisfy these criteria, and will be adapted by the Engineer as appropriate. Other approved BMPs found on the Virginia Stormwater BMP Clearinghouse Website may also be utilized, if deemed acceptable by VDOT, including manufactured BMPs approved by the DEQ.

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\* Rev. 10/23

Deviations in design from the VDOT BMP Design Manual of Practice, VDOT approved special provisions and standard insertable sheets, or the Virginia BMP Clearinghouse must be approved.

Compliance with the water quality criteria are evaluated generally using the entire site. However, where a site drains to more than one 6<sup>th</sup> Order HUC, the pollutant load reduction requirements shall be applied independently within each HUC unless reductions are achieved in accordance with the options described in Section 11.4.1 (7).

### **11.5.1.3 Land Cover and Soil Groups**

There are three categories of land cover identified by the VRRM: Forest/Open Space, Managed Turf, and Impervious Cover. Definitions of the three categories of land cover are provided in the definitions chapter of the Drainage Manual and DEQ guidance on use of the VRRM.

There are limited circumstances in which forest/open space will be allowed within the development site, and subsequently allowed to be identified as such using the VRRM. These include the following:

1. Surface area of stormwater BMPs that are NOT wet ponds, have some type of vegetative cover, and that do not replace an otherwise impervious surface. (BMPs in this category include bioretention, dry swale, grass channel, ED pond that is not mowed routinely, stormwater wetland, soil amended areas that are vegetated, and infiltration practices that have a vegetated cover.)
2. Utility rights-of-way that will be left in a natural vegetated state (can include areas that will be bush-hogged no more than four times per year)

In general, areas to be disturbed during construction in the R/W that do not meet the criteria above will be considered managed turf as the area will be assumed to be compacted unless soil restoration practices are implemented following disturbance.

If a disturbed area is expected to remain forest/open space because compaction will not occur, this area must be identified on the plan including the rationale for it not being compacted. The plan must also include maintenance requirements (i.e. mowing and fertilizer requirements) for these areas to maintain forest/open space conditions after construction is complete. These areas must be identified early in the process to allow the District Hydraulics Engineer an opportunity to review and approve.

If an area is to remain undisturbed, the plan must include provisions to exclude equipment from entering the area during construction and must include maintenance requirements (i.e. mowing and fertilizer requirements) for these areas to remain forest/open space conditions after construction is complete. These areas must be identified early in the process to allow the District Hydraulics Engineer an opportunity to review and approve.

Hydrologic soil group determinations can be made using the National Resources Conservation Service (NRCS) web soil surveys.

## 11.5.2 Water Quantity

Controlling the quantity of stormwater runoff can provide the following potential benefits:

- Prevention or reduction of increases in peak runoff rate, volume, and other characteristics caused by land disturbance and urban development;
- Decrease downstream channel erosion;
- Mitigation of downstream drainage capacity problems (localized flooding);
- Recharge of groundwater resources;
- Reduction or elimination of the need for downstream outfall improvements; and,
- Maintenance of historic base flow rates and stream hydrology.

Note that the new Water Quantity criteria for channel and flood protection are applied at each outfall from a land-disturbing project. Compliance must be shown at each outfall, not as an aggregate for the entire land-disturbing activity. The application of Water Quantity criteria by outfall may result in the use of different criteria applied to demonstrate compliance at each outfall.

### 11.5.2.1 Channel Protection

Channel protection criteria require that concentrated stormwater runoff is released to a manmade, restored, or natural stormwater conveyance system. For manmade and restored channels, the designer must demonstrate adequacy of the channel from the point of discharge to the limits of analysis. If inadequate for the post-development conditions, the designer must provide for detention, runoff reduction, or channel improvements. For discharges to natural channels, the Energy Balance criteria must be applied at each point of discharge.\*

Whenever the Energy Balance is used to achieve compliance for channel protection, a channel adequacy analysis is not necessarily required. However, a channel analysis is recommended for both manmade and restored channels to determine if a SWMF is required for compliance with the water quantity criteria, and to optimize the size of the necessary SWMFs, as the control requirements for the manmade and restored channels could be less than those required to satisfy the Energy Balance.

Defaulting to a design based upon the Energy Balance when the conveyance system is manmade or restored, without first conducting a channel adequacy analysis, could result in higher costs for R/W acquisition, design, construction, and maintenance. The designer should consider the additional cost for the SWMF versus the cost for the channel adequacy analysis. While the channel adequacy analysis may require additional field

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\* Rev. 10/23

survey, hydrologic analysis, and hydraulic modeling, the cost for the analysis may be less than the cost of implementing a SWMF designed for the Energy Balance that is larger than needed to meet the criteria for a manmade or restored channel.

#### **11.5.2.1.1 Manmade Stormwater Conveyance System**

A manmade stormwater conveyance system, including ditches, swales, curb, gutter, storm sewer, culverts, or other components must convey the post-development peak flow rate from the 2-yr 24-hour\* storm event without causing erosion of the system. The system must be shown adequate from the point of discharge to the limits of analysis (see 11.5.2.1.7). Where a manmade channel is inadequate for the post-development condition, site design, stormwater detention, or channel improvements can be used to obtain compliance in the 2-yr 24-hour storm event. Alternatively, the Energy Balance criteria can be applied to achieve compliance, regardless of the adequacy of the manmade stormwater conveyance system. Engineering methods for evaluating the adequacy of manmade stormwater conveyance systems are discussed in more detail in Chapters 4, 6, 7, and 9 of the Drainage Manual.

#### **11.5.2.1.2 Restored Conveyance System**

A restored conveyance system, such as a restored or relocated stream based upon natural channel design concepts, must meet the hydrologic and hydraulic parameters used for the restoration design. The system must be shown adequate from the point of discharge to the limits of analysis (see 11.5.2.1.7). Where a restored channel is inadequate for the post-development condition, site design, stormwater detention, or channel improvements can be used to obtain compliance. The Energy Balance criteria can also be applied to achieve compliance, regardless of the adequacy of the restored conveyance system. Design methods for evaluating and designing restored conveyance systems can be found in Chapters 4, 6, and 7 of the Drainage Manual.

#### **11.5.2.1.3 Natural Conveyance System**

For a natural conveyance system, the Energy Balance must be applied for compliance with the channel protection criteria. The methods for applying the Energy Balance are discussed below. A natural channel need not be analyzed for channel adequacy, as application of the Energy Balance is considered adequate for channel protection.

#### **11.5.2.1.4 Energy Balance**

In order to move beyond the traditional control of just the peak rate of runoff and address other runoff characteristics such as volume, frequency, and duration, a Technical Advisory Panel developed the Energy Balance method. While the term “energy” is used to describe the method, the computation is actually more loosely related to a “power”

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\* Rev. 10/23



balance, but the difference is not important to the development and application of the method to protect channels.

The fundamental concept of the Energy Balance is to further reduce the post-developed peak rate of runoff to below the pre-developed peak rate of runoff in inverse proportion to the increase in runoff volume for the post-developed condition. While application of the Energy Balance may not directly reduce the runoff volume resulting from development, it allows more time for infiltration and reduces the frequency of channel-forming storm events, providing channel hydrology similar to existing low flow or base flow conditions.

The Energy Balance is based upon the 1-yr return period storm instead of the 2-yr storm previously required for natural channels in Minimum Standard 19 (and still required for manmade channels). The Energy Balance equation used to calculate the allowable peak rate of runoff was summarized in the Design Criteria section:

$$Q_{\text{Developed}} = \text{I.F.} \times \left( \frac{Q_{\text{Pre-Developed}} \times \text{RV}_{\text{Pre-Developed}}}{\text{RV}_{\text{Developed}}} \right)$$

Where:

I.F. (Improvement Factor) = 0.8 for sites > 1 acre LDA or 0.9 for sites ≤ 1 acre LDA

$Q_{\text{Developed}}$  = the allowable peak flow rate of runoff from the developed site for the 1-yr 24-hour storm.

$\text{RV}_{\text{Developed}}$  = the volume of runoff from the site in the developed condition for the 1-yr 24-hour storm.

$Q_{\text{Pre-Developed}}$  = the peak flow rate of runoff from the site in the pre-developed condition for the 1-yr 24-hour storm.

$\text{RV}_{\text{Pre-Developed}}$  = the volume of runoff from the site in pre-developed condition for the 1-yr 24-hour storm.

#### **11.5.2.1.5 Improvement Factor (I.F.)**

As noted previously, the amended Stormwater Management Act requires that a land-disturbing activity improve upon the existing runoff characteristics when the existing receiving channel is already eroding. To address this requirement, an Improvement Factor (I.F.) was added to the Energy Balance method to require a further reduction in the peak rate of runoff for the post-development condition. For small projects ≤ 1-acre in area, the I.F. is 0.9, resulting in a minimum 10% reduction in the peak rate of runoff. Projects > 1-acre must apply an I.F. of 0.8 or a minimum 20% reduction in the allowable peak runoff rate.

In some cases, where it can be demonstrated that the existing channel is not experiencing erosion in the pre-development condition, the I.F. may be waived for calculating the allowable peak rate of runoff for the post-development condition. However, use of the waiver will require a channel adequacy analysis to demonstrate that the existing channel is adequate for the existing conditions and that an improvement is not necessary. Waivers may be granted by DEQ provided that VDOT coordinates with DEQ and submits a written request to DEQ requesting the exception. Waiver/exception requests must be

submitted in writing to the VDOT Central Office L&D MS4/SWM Program for review and coordination with the DEQ Central Office. The request shall include documentation of the need for the waiver/exception. The documentation shall describe all means and methods evaluated for meeting the water quality/quantity requirements and the reasons why specific means or methods were determined not feasible. The documentation shall also state that the exception being requested is the minimum necessary to afford relief.

#### **11.5.2.1.6 Forested Conditions**

When applying the Energy Balance method to a land-disturbing activity, the allowable peak rate of runoff could be less than the runoff from the site if it were undeveloped in a good forested condition. This can occur from the application of the I.F. on a site where the runoff peak rate or volume does not increase appreciably from the pre-developed condition. In this case, the allowable peak rate of runoff should not be less than the forested condition calculated as follows:

$$Q_{\text{Developed}} = \left( \frac{Q_{\text{Forest}} \times RV_{\text{Forest}}}{RV_{\text{Developed}}} \right)$$

Where:

$Q_{\text{Developed}}$  = the allowable peak flow rate of runoff from the developed site for the 1-yr 24-hour storm.

$RV_{\text{Developed}}$  = the volume of runoff from the site in the developed condition for the 1-yr 24-hour storm.

$Q_{\text{Forest}}$  = the peak flow rate of runoff from the site in a forested condition for the 1-yr 24-hour storm.

$RV_{\text{Forest}}$  = the volume of runoff from the site in a forested condition for the 1-yr 24-hour storm.

### **11.5.2.1.7 Limits of Analysis**

Unlike Minimum Standard 19, where the receiving channel was analyzed for adequacy immediately below the land development site outfall (or at the outlet of a storm sewer when the project outfalls to a storm sewer), the channel protection criterion requires that manmade and restored channels be analyzed and demonstrated to be adequate from the point of discharge to the “limits of analysis”. This is the point in the drainage system where the site’s contributing drainage area is  $\leq 1\%$  of the total watershed area, or the point in the system where the peak rate of runoff from the 1-yr 24-hour storm for the post-developed condition prior to the implementation of any stormwater quantity controls is less than 1% of the total watershed peak rate of runoff.\*

The point where the limit of analysis occurs does not need to be included in the evaluation of adequacy, but the system from the point of discharge up to the limits of analysis must be analyzed. If there is a natural channel anywhere in the system from the point of discharge to the limit of analysis, then the energy balance will apply. Note that the limits of analysis are applied separately for each outfall or point of discharge from a regulated land-disturbing activity.

### **11.5.2.1.8 Runoff Reduction**

A consequence of the new Runoff Reduction Method (RRM) discussed previously under Water Quality is the reduction in runoff volume associated with BMPs that provide RRM credit. By reducing the runoff volume with RRM facilities, the allowable discharge under the Energy Balance method may be higher because the overall increase in runoff volume is lower for the post-development condition.

To reflect the reduced runoff volume when RRM facilities are included in the SWM design, an adjustment in the post-development Runoff Curve Number (CN) can be calculated using the VRRM spreadsheets or other engineering methods acceptable to VDOT and DEQ (see section on modeling).

An adjusted CN must be calculated individually for each storm event (1-, 2-, and 10-yr 24-hour), as the CN reduction provided will vary based upon the storm. In general terms, the CN reduction is calculated assuming that the RRM Treatment Volume provided is treated as additional initial abstraction ( $I_a$ ) in the CN Method from the NRCS Technical Release 55 (TR-55) *Urban Hydrology for Small Watersheds*. As the RRM retention volume is constant and based upon the SWMF design, but the total runoff volume varies for each storm event, a reduced CN would be calculated and used in hydrologic calculations downstream of the RRM facilities.

Here is the method used to calculate adjusted CNs using modified equations 2-3 and 2-4 of TR-55:

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\* Rev. 10/23

$$Q - R = \frac{(P - 0.2 \times S_{adj})^2}{(P + 0.8 \times S_{adj})}$$

Where:

Q = runoff (watershed inches) for the 1-, 2-, or 10-yr 24-hour storm event before RRM practices are applied

R = volume of retention storage provided by runoff reduction practices (watershed inches) = runoff reduction volume (ft<sup>3</sup>)/watershed drainage area (acres) x 12 (inches/foot)/43,560 (ft<sup>2</sup>/acre)

P = rainfall (inches) for the 1-, 2-, or 10-yr 24-hour storm event

S<sub>adj</sub> = potential maximum retention after runoff begins (inches) adjusted for runoff reduction volume

The value of Q prior to runoff reduction is calculated for the watershed using standard TR-55 graphical peak discharge methodology; the retention storage volume R provided by runoff reduction practices is typically calculated in the VRRM spreadsheet in ft<sup>3</sup>, converted to watershed inches, and subtracted from Q; and the equation solved for an adjusted S value (S<sub>adj</sub>). The S<sub>adj</sub> value is then used to determine the adjusted CN using the following relationship from TR-55:

$$CN_{adj} = \frac{1000}{S_{adj} + 10}$$

Where:

CN<sub>adj</sub> = adjusted CN calculated individually for the 1-, 2-, and 10-yr 24-hour storm events

The solution for S<sub>adj</sub> involves a quadratic equation, and multiple techniques are available to solve. However, **the Virginia RRM Spreadsheet solves the equation for S<sub>adj</sub> and provides CN<sub>adj</sub> for the 1-, 2-, and 10-yr storms to simplify the process for the designer.**

Note that the RRM Spreadsheet adjusts the CN using units of watershed inches instead of acre-feet or cubic feet. As long as the pre- and post-development drainage areas remain the same at the outfall, the use of watershed inches is valid. However, when the drainage area at an outfall changes from the pre- to post-developed condition, the CN adjustments used for the Energy Balance should be made using units of volume, such as acre-feet or cubic feet.

While the RRM Spreadsheet is ideal for adjusting CNs for simple drainage areas, there are times when changes in drainage area due to development or more complex drainage networks exceed the capabilities of the RRM spreadsheet. In this case, the designer will have to use other means to apply the CN methodology of TR-55 and compute adjusted CN values for a project.

The adjusted CN for each storm event (1-, 2-, and 10-yr 24-hour) is then used in the revised hydrologic calculations to determine the adjusted peak rate of runoff and runoff volume after RRM practices in each storm. The adjusted peak rate of runoff and runoff volume are used to determine compliance for water quantity as already discussed, including the Energy Balance method.

Note that the runoff retention volume provided via RRM practices should not be used to adjust CNs and also as storage for storm routing for water quantity control. The volume reduction provided should be used for one or the other, but not both. However, should a SWMF include non-retention storage (detention) for water quantity above or outside of the RRM practice retention volume, then the additional storage could be used in storm routing to demonstrate additional reductions in peak rate of runoff due to detention.

Typically, it is advantageous for storage-based practices (such as bioretention facilities) to disregard the potential CN adjustment value, and be routed including the pore storage (in the media, and the choker/reservoir stone), the surface storage, and the outlet characteristics. It is also commonplace to neglect underdrain flows (this assumption was used in the derivation of the Effective CN method) due to their relatively insignificant effect on the overall discharge, in effect modeling the practice as an empty plugged bathtub.

An example would be taking credit for the retention volume present in the engineered soil media, sump stone, and ponding in a bioretention facility. If the total retention volume is used to adjust the CN, then the same volume should not be used as storage to route a storm through the bioretention facility to take credit for peak runoff reduction. However, the retention volume in the engineered media and sump stone could be used to calculate an adjusted CN, and the ponding volume used for detention volume routing through the facility. In this case, the runoff reduction volume used to adjust the CN would need to be calculated separately from the RRM Spreadsheet, which assumes that all runoff retention storage is used for CN adjustment.

#### **11.5.2.1.9      *Increases in Peak Rate of Runoff***

In no case shall the Energy Balance method be used to justify an increase in post-developed peak rate of runoff from a regulated land-disturbing project. This can occur when the site design results in a post-development runoff volume that is less than the pre-development volume, such as when runoff reduction controls are used. For all regulated land-disturbing activities, the post-developed peak rate of runoff shall not exceed the pre-developed peak rate of runoff.

#### **11.5.2.2    Flood Protection**

Local flood protection in Virginia for storm events smaller than the 100-yr event was addressed under Minimum Standard 19 of the Virginia Erosion and Sediment Control Regulations (9VAC25-840-40) and in the flooding criteria in Part II C of the Virginia Stormwater Management Program (VSMP) Regulation (9VAC-25-870-98).

In the case of linear transportation projects, the criteria in the VSMP Regulation did not require the control of the post-developed stormwater runoff for flooding, except in

accordance with a watershed or regional SWM plan. With the amended VSMP Regulation, linear transportation projects are no longer exempt from the Flood Protection criteria. The application of the new Flood Protection criteria to VDOT projects is discussed below.

#### **11.5.2.2.1 Conveyance System Definition**

The new VSMP Regulation defines a “stormwater conveyance system” as a combination of drainage components that are used to convey stormwater discharge, either within or downstream of the land-disturbing activity. This includes:

"Manmade stormwater conveyance system" means a pipe, ditch, vegetated swale, or other stormwater conveyance system constructed by man except for restored stormwater conveyance systems;

"Natural stormwater conveyance system" means the main channel of a natural stream and the flood-prone area adjacent to the main channel; or

"Restored stormwater conveyance system" means a stormwater conveyance system that has been designed and constructed using natural channel design concepts. Restored stormwater conveyance systems include the main channel and the flood-prone area adjacent to the main channel.

Note that both the natural and restored systems include the main channel and the flood-prone area adjacent to the main channel when considering localized flooding and Flood Protection.

An analysis of the system in the existing conditions is necessary to establish if localized flooding occurs in the 10-yr 24-hour storm event. Historic flood records and anecdotal evidence may also be useful in documenting existing flood conditions, although both should be supported by hydrologic and hydraulic modeling to support the existence of flooding.

#### **11.5.2.2.2 Localized Flooding not Currently Experienced**

When localized flooding does not occur under pre-developed conditions (i.e., the stormwater conveyance system contains the 10-yr 24-hour storm event), the post-development discharge from project must be confined within the stormwater conveyance system. If this does not occur, detention of stormwater, system improvements, or a combination of both may be used to make the system adequate.

#### **11.5.2.2.3 Localized Flooding Currently Experienced**

When localized flooding does occur under pre-developed conditions (i.e., the stormwater conveyance system does not contain the 10-yr 24-hour storm event), either:

- a. The post-development peak discharge from the project must be confined within the stormwater conveyance system to avoid localized flooding, and detention of

stormwater, system improvements, or a combination of both may be used to make the system adequate; or,

- b. The post-development peak flow rate must be less than the pre-developed peak flow rate for the 10-yr 24-hour storm event.

#### **11.5.2.2.4 Compliance with the Flood Protection Criteria**

Compliance with the flood protection criteria can be achieved by detaining the post-development 10-yr 24-hour peak runoff at each project outfall to below the pre-development rate; however, there are cases where detention to the pre-developed peak runoff rate may not be necessary, and a system adequacy analysis is required to demonstrate the need.

If it is demonstrated via an adequacy analysis that the stormwater conveyance system does not contain the post-developed peak rate of runoff, then the designer must consider the options for detention and system improvements.

In general, detention may be preferred to system improvements due to the cost of acquiring additional R/W and long-term maintenance of the system improvements. However, the system improvements required to mitigate minor increases in peak runoff may be less intrusive and expensive than providing detention and meet other project goals for SWM and environmental protection. The designer must consider the total cost of options before making a final decision to provide detention, make system improvements, or a combination of both.

#### **11.5.2.2.5 Limits of Analysis**

Stormwater conveyance systems must be analyzed and demonstrated to be adequate downstream up to the point in the drainage system where the site contributing drainage area is  $\leq 1\%$  of the total watershed area.

The limits of analysis can also be the point in the downstream system where the peak rate of runoff from the 10-yr, 24-hour storm for the post-developed condition is less than 1% of the existing peak rate of runoff for the watershed.

The point where the limit of analysis occurs does not need to be included in the evaluation of adequacy, but the system from the outfall or point of discharge up to the limits of analysis must be analyzed. If there is a natural channel anywhere in the system from the outfall up to the limit of analysis, then the energy balance will apply. Note that the limits of analysis are applied separately for each outfall or point of discharge from a regulated land-disturbing activity.

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\* Rev. 10/23

Unlike the channel protection analysis, the flood protection analysis can also end when the stormwater conveyance system enters a mapped floodplain or other flood-prone area adopted by local ordinance.

### **11.5.2.3 Sheet Flow**

When the post-developed condition results in increases in sheet flow runoff volume from a project, including the dispersal of concentrated flow using level spreaders and other energy dissipating techniques, the designer must identify the discharges and evaluate the runoff increase for impacts to waterways and properties. If the evaluation demonstrates the potential to cause or contribute to erosion, sedimentation, or flooding below the project, then the increased sheet flows must be directed to a SWMF or stormwater conveyance system that is adequate for the increased runoff. When all discharges from a project are sheet flow and the criteria are met, no further controls are required for channel or flood protection.

It should be noted that the conversion of concentrated flow to sheet flow via level spreaders and energy dissipaters becomes more difficult as the volume and peak rate of runoff increases. This is especially true for concentrated discharges from a culvert or storm sewer system, or when the area below the outfall is sloped away from the discharge. If the designer chooses to convert concentrated flow to sheet flow, the evaluation and design of control structures must be carefully documented and supported via engineering computations. The area required to convert concentrated flow to sheet flow may exceed available R/W and easement for larger outfalls, so flow may need to remain concentrated to make the best use of existing land available for drainage and SWM.

The potential for erosion and sedimentation from increases in sheet flows from a project shall be evaluated. Flooding must be considered for increases in sheet flow volume as well, using good engineering practice and acceptable hydrologic and hydraulic evaluation. Designers shall describe how the potential for flooding, erosion and sedimentation from increased sheet flow was evaluated and how no adverse impact on downstream waterways and properties were determined.



## 11.5.3 Part II B Design Procedures and Sample Problems

### 11.5.3.1 SWM Plan Requirements

The following documentation will be required for SWMF/BMP design:

- Documentation requirements presented in Chapter 6, Hydrology.
- Computations for determination of the pre- and post-development peak runoff rates and runoff volumes for the design storms.
- Water Quantity (Channel and Flood Protection) computations for the relevant design storms, including determination of the limits of analysis.
- Water Quality computations based upon the Runoff Reduction Method (RRM).
- SWMF Tabulation Sheet when submitting final plans.
- Provide all documentation from storm routing. This would generally include inflow and outflow hydrographs, stage-storage curves, discharge rating curves for the spillway(s), and routing summaries. This information would be generated by various computer modeling software.
- Basin grading and spillway(s) details and specifications provided.
- Complete (C) and Minimum (M) plan projects shall show SWM measures in the plan assembly and may include insert sheets specific to each SWM measure for clarity.
- No-plan (N) and other types of projects (including maintenance activities) that have an abbreviated plan assembly must conform to the requirements of the VSMP Regulations and VPDES Construction General Permit where the land disturbance value exceeds the applicable land disturbance thresholds for such.

The plan design details for BMPs shall be appropriately sealed and signed by a qualified person licensed or registered in the Commonwealth of Virginia as an architect, professional engineer, land surveyor, landscape architect, or certified professional soil scientist, as regulated by the Virginia Department of Professional Regulation.

### 11.5.3.2 Water Quality – Runoff Reduction Method (RRM) Procedure

The Virginia Runoff Reduction Method is described and detailed in documents published by the Virginia DEQ, including Guidance Memorandums, supporting white papers from the method developer, training materials, and the VRRM Spreadsheets. Please see the DEQ documents for more in-depth procedures on implementation of the RRM.

For this process, it is assumed that the VRRM Spreadsheets are being used to demonstrate compliance with the water quality criteria. If another acceptable model or method is being used, follow the instruction and directions for that model or method. Some of the same steps may apply.

**Step 1. Select Project Type and Open VRRM Spreadsheet**

*Determine if the project qualifies as New Development or Redevelopment (see the definitions for “New Development” and “Redevelopment” in Chapter 1 Appendix A-1.)*

*Based on the type of development, start a new VRRM Spreadsheet for the project and enter information to identify the project.*

*Note that water quality compliance must be demonstrated by Hydrologic Unit Code (HUC). If a project drains to more than one HUC, a VRRM Spreadsheet may be required for each HUC to demonstrate compliance in each HUC individually. Overtreatment in one HUC is not allowed to compensate for compliance in another HUC using onsite BMPs, unless the overtreatment is in an adjacent upstream HUC.*

**Step 2. Enter Project Information on Site Tab**

*Collect overall project site parameters and enter into the Site tab in the VRRM Spreadsheet, including:*

- *Land Cover type (Forest/Open Space, Managed Turf, or Impervious, not the land cover types from TR-55)*
- *Hydrologic Soil Group or HSG for each land cover type (A, B, C, or D as defined by the USDA for TR-55).*
- *Sub-area for each combination of Land Cover type and HSG.*

*Check that the total area for each Land Cover type and overall project area are correct in the summary table for the sub-areas entered.*

*Note that the New Development spreadsheet only needs the Post-Development project parameters.*

*The Redevelopment spreadsheet requires site information for the pre-development (or existing) conditions, as well as the post-development (or proposed) conditions.*

*The Redevelopment spreadsheet has the option to calculate alternate water quality reduction requirements for linear development projects, such as roadways. Make sure the cell is marked “Yes” for a linear redevelopment project.*

*Please note that the latest redevelopment spreadsheet from DEQ can compute the removal required for both new development and redevelopment on a project, so it is not necessary to break out the land cover by new development or prior development or use both the new development and redevelopment spreadsheets for a project.*

**Step 3. Review Project Site Summary and Pollutant Removal Requirements**

*The spreadsheet should update the VRRM Site calculations and report the following information for the overall site on the tab:*

- *Runoff coefficients (Rv)*
- *Total Phosphorous (TP) Load Reduction Required (lb/yr)*
- *Land Cover Summary*
- *Treatment Volume (cubic feet or ft<sup>3</sup>)*
- *Nutrient Loads (lb/yr)*

**Step 4. Identify Outfalls and Provide Drainage Area Information**

*Based upon the topography, proposed drainage network, and site design, identify the location(s) of project outfall(s).*

*Determine the contributing drainage area to each outfall, as well as the sub-areas for Land Cover type by HSG. The VRRM Spreadsheet can handle up to 5 outfalls, one each on the worksheet tabs labeled D.A. A to D.A. E.*

*For each outfall, enter the contributing sub-area to the outfall based upon the distribution of Land Cover type and HSG, as was done for the overall project on the Site tab. The D.A. tab will calculate the total drainage area and Runoff coefficient (Rv) for the outfall.*

**Step 5. Selection and Application of Stormwater BMPs**

*Based upon the topography, drainage design, R/W available, site constraints, and other SWM requirements, select stormwater best management practices (BMPs) for use in the outfall.*

*Note that the entire contributing drainage area to an outfall may not be treated in one BMP. The contributing drainage area to the outfall can be broken into smaller areas for the selection and application of distributed BMPs. BMPs can also be placed in series to provide a treatment train with higher pollutant removal efficiencies (see Section 11.5.3.1.1).*

**Step 5a. Apply Runoff Reduction (RR) Practices**

*Begin with Runoff Reduction (RR) practices that are suitable for the contributing drainage area, as they generally provide a greater pollutant removal rate than conventional BMPs that only provide treatment. These include:*

- *Vegetated Roof (not likely to be used on transportation projects)*
- *Rooftop Disconnection (not likely to be used on transportation projects)*
- *Permeable Pavement*
- *Grass Channel*
- *Dry Swale*

- *Bioretention*
- *Infiltration*
- *Extended Detention*
- *Sheetflow to Filter/Open Space*

*See the VDOT BMP Design Manual of Practice or the Virginia BMP Clearinghouse for detailed information on the selection, application, and design of RR practices.*

*Once RR practices have been selected, enter the acreage of managed turf and impervious area draining to each practice in the spreadsheet. The spreadsheet will automatically calculate the runoff reduction, remaining runoff volume, total treatment volume, pollutant load, pollutant removed, and remaining pollutant load for each practice.*

*Note that RR practices can be placed downstream of other BMPs to create a treatment train of BMPs in series. See Section 11.5.3.1.1 for a discussion.*

*The purchase of nutrient credits to address post-construction water quality reduction requirements for construction activities shall be considered the preferred alternative when available and economically feasible.*

*If the project site area is less than 5 acres, up to 100% of the Phosphorous reduction can be achieved via offsite options; or, if the removal rate is less than 10 lb/yr, up to 100% of the Phosphorous reduction can be achieved via offsite options.*

*If the project site area is greater than 5 acres and the Phosphorous removal rate is greater than 10 lb/yr, up to 25% of the Phosphorous reduction can be achieved via offsite options. In some cases, more than 25% can be purchased if it can be shown that achieving 75% removal onsite is not practicable. This requires submitting a waiver to DEQ to secure approval for purchasing more than 25% of the credits.*

*See Section 11.5.3.2.2 below for a discussion of offsite water quality compliance options.*

#### *Step 5b. Review Water Quality Compliance*

*Once RR practices have been applied to each outfall, go to the Water Quality Compliance tab to determine if the selected BMPs meet the water quality requirements for the overall site.*

*If the TP load reduction target has been met for the project site, then proceed to detailed design for the BMPs selected, Step 7.*

*If the TP load reduction target has been exceeded for the project site, consider optimization of the RR practices selected to reduce the TP reduction to the target load. Return to Step 5a.*

*If the TP load reduction target has not been met, consider the application of additional RR practices, the use of RR practices in series, or move on to Step 5c to select conventional treatment options for compliance.*

#### **Step 5c. Apply Conventional BMPs**

*If the RR practices selected do not satisfy the Phosphorous Load reduction requirements for the project, then the use of conventional BMPs (no runoff reduction provided) should be considered. The conventional BMPs included in the VRRM Spreadsheet are:*

- *Wet Swales*
- *Filtering Practices*
- *Constructed Wetlands*
- *Wet Ponds*
- *Manufactured Treatment Devices (MTDs)*

*See the VDOT BMP Design Manual of Practice or the Virginia BMP Clearinghouse for detailed information on the selection, application, and design of conventional BMPs.*

*Note that conventional BMPs can be placed downstream of other BMPs to create a treatment train of BMPs in series. See Section 11.5.3.2.1 for a discussion.*

#### **Step 5d. Review Water Quality Compliance**

*Once conventional BMPs have been applied, go to the Water Quality Compliance tab to determine if the selected BMPs meet the water quality requirements for the overall site.*

*If the TP load reduction target has been met for the project, then proceed to detailed design for the BMPs selected, Step 7.*

*If the TP load reduction target has been exceeded for the project site, consider optimization of the BMPs selected to reduce the TP reduction to the target load. Return to Steps 5a and 5c.*

*If the TP load reduction target has not been met, apply additional BMPs, the use of BMPs in-series, or move on to Step 6 to investigate offsite compliance options.*

## Step 7. *BMP Design*

*Once compliance with the pollutant load reduction requirements is achieved by application of BMPs and offsite compliance options, the designer should proceed with detailed design of BMPs. The results of the VRRM Spreadsheet should be retained to verify the Treatment Volume required for each RR practice and conventional BMP.*

*For detailed information on the design of BMPs, see the VDOT BMP Design Manual of Practice or the Virginia BMP Clearinghouse.*

### **11.5.3.2.1 *BMPs In-Series/Treatment Trains***

SWM BMPs (RR and conventional) can be placed downstream of other BMPs to create a “treatment train” of practices in-series. The use of BMPs in series provides greater pollutant removal rates and allows smaller controls to be distributed throughout a project, closer to the pollutant sources.

This is accomplished by going to the spreadsheet entries for each upstream BMP and selecting the Downstream Practice to be employed via a dropdown list. The spreadsheet will automatically enter the remaining runoff volume and pollutant load from the upstream BMP to the calculations for the downstream BMP.

Note that if other areas drain to the downstream BMP (but not the upstream BMP in series), then those areas must be entered separately into the spreadsheet in the section for the downstream BMP. Note that the water quality sizing for BMPs with a runoff reduction practice upstream is reduced by the volume of runoff reduction upstream. However, all downstream facilities should be evaluated for proper conveyance and freeboard, as appropriate, using the full contributing drainage area.

### **11.5.3.2.2 *Offsite Water Quality Compliance Options***

When a project meets the requirements that allow offsite water quality compliance, the designer should consider the option, especially when the R/W or easements available for SWM onsite are limited and site constraints make onsite BMPs difficult or costly to implement. A cost comparison of the onsite versus offsite options should include capital costs (R/W, easements, and construction) as well as the long-term maintenance costs. Often, the cost for purchasing offsite nutrient credits to meet water quality load reductions is less costly, especially when land costs and long term maintenance are included in the analysis.

Also, the designer must consider the need for water quantity control for the project. While conventional BMPs provide no Runoff Reduction, RR practices reduce the runoff volume and provide water quantity control. The Runoff Reduction provided by the RR practice and the water quantity control provided should be considered in a comparison of the onsite versus offsite design options, especially if a SWMF will be required for water quantity control.

The most common form of offsite compliance is the purchase of Nutrient Credits from a Nutrient Credit Bank trading under the nonpoint nutrient offset program established pursuant to § 62.1-44.15:35 of the Code of Virginia. VDOT maintains contracts with approved Nutrient Credit Banks across the Commonwealth serving most of the major tributaries in Virginia. The cost per credit (lb/yr) has been fixed under the competitive, negotiated contracts. Note that the contracts have a fixed term and are re-advertised and negotiated periodically. See IIM-LD-251 for details on the VDOT Nutrient Credit purchase program.

Note that there are other ways to achieve water quality compliance offsite for a project. These include the payment into a *pro rata* system used to construct nutrient reduction BMPs in the same watershed; development under a comprehensive SWM plan adopted pursuant to the VSMP Regulation; on other VDOT owned properties in the same HUC or upstream HUC as the land disturbance activity; and other options approved by VDOT and the DEQ. These options are all unique and will not be developed further in this document. Contact the District Hydraulics Engineer, State Hydraulics & Utilities Engineer, or State MS4 Program Manager to coordinate and develop offsite options other than Nutrient Credit purchases.

#### **11.5.3.2.3 Water Quality – Sample Problem – New Development**

Assume a VDOT project with a 1.7-acre site draining to an outfall. The soils are all classified as HSG C by the USDA Soil Survey. In the pre-development condition, the entire site is undisturbed forest. In the post-developed condition, the site will include 0.8 acres of impervious roadway and shoulder, with the balance managed turf. *What are the  $R_v$ , total TP load, TP Load Reduction Required, and total Treatment Volume for the site?*

- Step 1. Use the VRRM Spreadsheet for New Development for the project, as the pre-development condition is described as “undisturbed forest”.
- Step 2. On the Site tab, enter the project name and date of the calculations. Complete the Land Cover table for the post-development conditions:

## Post-Development Project (Treatment Volume and Loads)

### Land Cover (acres)

	A Soils	B Soils	C Soils	D Soils	Totals
<b>Forest/Open Space (acres)</b> -- undisturbed, protected forest/open space or reforested land					0.00
<b>Managed Turf (acres)</b> -- disturbed, graded for yards or other turf to be mowed/managed			0.90		0.90
<b>Impervious Cover (acres)</b>			0.80		0.80
					1.70

Note that the pre-development condition land cover is not required for the New Development tab, as the allowable TP load based upon the water quality criteria is 0.41 lb/acre/yr for New Development, regardless of the type of land cover in the pre-development condition.

- Step 3. The spreadsheet automatically calculates the post-development Requirements for the Site Area:

Post-Development Requirement for Site Area	
TP Load Reduction Required (lb/yr)	1.49

This is the difference between the allowable TP load of 0.41 lb/acre/yr for new development and the total TP load in the post-development condition, which is provided in the Land Cover Summary.



A Land Cover Summary for the post-development condition is provided:

<b>LAND COVER SUMMARY -- POST DEVELOPMENT</b>	
<b>Land Cover Summary</b>	
Forest/Open Space Cover (acres)	0.00
Weighted Rv (forest)	0.00
% Forest	0%
Managed Turf Cover (acres)	0.90
Weighted Rv (turf)	0.22
% Managed Turf	53%
Impervious Cover (acres)	0.80
Rv (impervious)	0.95
% Impervious	47%
Site Area (acres)	<b>1.70</b>
Site Rv	<b>0.56</b>
<b>Treatment Volume and Nutrient Loads</b>	
<b>Treatment Volume (acre-ft)</b>	0.0798
<b>Treatment Volume (cubic feet)</b>	<b>3,478</b>
<b>TP Load (lb/yr)</b>	<b>2.18</b>
<b>TN Load (lb/yr) (Informational Purposes Only)</b>	15.63

Based upon the results reported by the VRRM Spreadsheet, Site Tab, the Site Rv is 0.56, the total TP Load for the site is 2.18 lb/yr, the TP Load reduction required is 1.49 lb/yr (68% reduction from the total TP Load), and the total Treatment Volume required to achieve complete runoff reduction (no increase in runoff volume) is 3,478 ft<sup>3</sup>.

- Step 4. Assume the entire site drains to one outfall, so move to the second tab labeled "D.A. A" and enter the Land Cover information for the project. For this example, the Land Cover information for Drainage Area A is the same as for the project site:

**Drainage Area A Land Cover (acres)**

	A Soils	B Soils	C Soils	D Soils	Totals	Land Cover Rv
Forest/Open Space (acres)					0.00	0.00
Managed Turf (acres)			0.90		0.90	0.22
Impervious Cover (acres)			0.80		0.80	0.95
				<b>Total</b>	1.70	

Step 5a. The first step in providing onsite treatment is to consider the use of Runoff Reduction practices. For the example, assume the roadway is open section with roadside drainage via swales and ditches. Also, assume the soils have low permeability and are unsuitable for infiltration (based upon a site specific soil investigation), but the depth to seasonal high groundwater and bedrock is > 6'.

Based upon this information and the information provided in the VDOT BMP Design Manual of Practice and Virginia BMP Clearinghouse, select a Dry Swale #1 as our Runoff Reduction practice.

As the entire site drains to the outfall via proposed Dry Swales, enter the entire site area into the spreadsheet under the columns "Managed Turf Credit Area" and "Impervious Cover Credit Area":

**Stormwater Best Management Practices (RR = Runoff Reduction)**

Practice	Runoff Reduction Credit (%)	Managed Turf Credit Area (acres)	Impervious Cover Credit Area (acres)	Volume From Upstream Practice (ft <sup>3</sup> )
<b>5. Dry Swale (RR)</b>				
5.a. Dry Swale #1 (Spec #10)	40	0.90	0.80	0

Once the information for the contributing drainage area to the Dry Swales is entered, the spreadsheet will calculate the Runoff Reduction volume provided by the practice, the remaining volume of runoff not removed via the Runoff Reduction practice, the total Treatment Volume for the practice, the untreated TP load to the practice, the TP removed by the practice, and any remaining TP load after runoff reduction and treatment:

**Stormwater Best Management Practices (RR = Runoff Reduction)**

Practice	Runoff Reduction (ft <sup>3</sup> )	Remaining Runoff Volume (ft <sup>3</sup> )	Total BMP Treatment Volume(ft <sup>3</sup> )	Untreated Phosphorus Load to Practice (lb)	Phosphorus Removed By Practice (lb)	Remaining Phosphorus Load (lb)
<b>5. Dry Swale (RR)</b>						
5.a. Dry Swale #1 (Spec #10)	1,391	2,087	3,478	2.18	1.13	1.05

Step 5b. Based upon the results above, has 100% Runoff Reduction been achieved by the application of Dry Swales to the project? Has the TP Load reduction required been satisfied to achieve compliance?

According to the Site tab, 3,478 ft<sup>3</sup> of Runoff Reduction is required to reduce the runoff volume to pre-development condition, but only 1,391 ft<sup>3</sup> is achieved in the Dry Swales, so 100% Runoff Reduction is not met. Also, 1.49 lb/yr of TP reduction is required for site compliance, but the Dry Swales only provide 1.13 lb/yr of reduction.

Compliance with the Water Quality requirements for the site can also be verified on the Water Quality Compliance tab:

## Site Results (Water Quality Compliance)

### Runoff Reduction Volume and TP By Drainage Area

	D.A. A
RUNOFF REDUCTION VOLUME ACHIEVED (ft <sup>3</sup> )	1,391
TP LOAD AVAILABLE FOR REMOVAL (lb/yr)	2.18
TP LOAD REDUCTION ACHIEVED (lb/yr)	1.13
TP LOAD REMAINING (lb/yr)	1.05
NITROGEN LOAD REDUCTION ACHIEVED (lb/yr)	8.59

### Total Phosphorus

FINAL POST-DEVELOPMENT TP LOAD (lb/yr)	2.18
TP LOAD REDUCTION REQUIRED (lb/yr)	1.49
TP LOAD REDUCTION ACHIEVED (lb/yr)	1.13
TP LOAD REMAINING (lb/yr):	1.05
REMAINING TP LOAD REDUCTION REQUIRED (lb/yr):	0.35

Note that there is a remaining TP Load reduction required of 0.35 lb/yr, as shown in the tab in red font. Had the TP Load reduction been satisfied, the number would be green.

Step 5c. As the TP Load reduction was not satisfied with the first application of RR practices, further treatment is necessary. Assuming no additional RR practices can be used, look at adding a conventional BMP downstream of the Dry Swales for treatment in-series.

On Tab D.A. A, return to the Dry Swale #1 and go to the column labeled "Downstream Practice to be Employed". Using the pull down menu, select a Filtering Practice #1 design for the conventional BMP.

**Stormwater Best Management Practices (RR = Runoff Reduction) --Select from dropdown lists--**

Practice	Untreated Phosphorus Load to Practice (lb)	Phosphorus Removed By Practice (lb)	Remaining Phosphorus Load (lb)	...	Downstream Practice to be Employed
<b>5. Dry Swale (RR)</b>					
5.a. Dry Swale #1 (Spec #10)	2.18	1.13	1.05	...	11.a. Filtering Practice #1

When this is done, note that the spreadsheet automatically populates information for the downstream BMP. Scroll down to the row labeled “11.a. Filtering Practice #1 (Spec #15)” and note that the remaining runoff from the upstream Dry Swales is now entering the proposed BMP:

**Stormwater Best Management Practices (RR = Runoff Reduction)**

Practice	Volume from Upstream Practice (ft <sup>3</sup> )	Runoff Reduction (ft <sup>3</sup> )	Remaining Runoff Volume (ft <sup>3</sup> )	Total BMP Treatment Volume (ft <sup>3</sup> )	Phosphorus Load from Upstream Practices (lb)
<b>11. Filtering Practices (no RR)</b>					
11.a. Filtering Practice #1 (Spec #12)	2,087	0	2,087	2,087	1.05

Note that the remaining volume of runoff and TP load from the Dry Swales is now in the calculation for the downstream BMP. Compare the value for “Volume from Upstream Practice” from the Filtering Practice #1 with the “Remaining Runoff Volume” for the Dry Swales. Also, compare the “Phosphorus Load from Upstream Practices” for the Filtering Practice #1 with the value reported as “Remaining Phosphorus Load” for the Dry Swales.

(Note that no Land Cover values were entered for the Filtering Practice #1, as it receives all of the remaining runoff from the upstream Dry Swales. If additional areas discharged to the Filtering Practice, but not through the Dry Swales, then the additional areas would be entered as Land Cover in acres for the Filtering Practice #1.)

Step 5d. Check water quality compliance for the overall project on the “Water Quality Compliance” tab:

**Site Results (Water Quality Compliance)**

**Runoff Reduction Volume and TP By Drainage Area**

	D.A. A
<b>RUNOFF REDUCTION VOLUME ACHIEVED (ft<sup>3</sup>)</b>	1,391
<b>TP LOAD AVAILABLE FOR REMOVAL (lb/yr)</b>	2.18
<b>TP LOAD REDUCTION ACHIEVED (lb/yr)</b>	1.76
<b>TP LOAD REMAINING (lb/yr)</b>	0.42

**Total Phosphorus**

<b>FINAL POST-DEVELOPMENT TP LOAD (lb/yr)</b>	2.18
<b>TP LOAD REDUCTION REQUIRED (lb/yr)</b>	1.49
<b>TP LOAD REDUCTION ACHIEVED (lb/yr)</b>	1.76
<b>TP LOAD REMAINING (lb/yr):</b>	0.42
<b>REMAINING TP LOAD REDUCTION REQUIRED (lb/yr):</b>	<b>0.00      **</b>

**\*\* TARGET TP REDUCTION EXCEEDED BY 0.28 LB/YEAR \*\***

*Based upon the results above, has 100% Runoff Reduction been achieved for the project by the addition of a Filtering Device #1 in series with Dry Swales? Has the TP Load reduction required been satisfied to achieve compliance?*

According to the Site tab, 3,478 ft<sup>3</sup> of Runoff Reduction is required to reduce the runoff volume to pre-development condition, but only 1,391 ft<sup>3</sup> is achieved in the Filtering Device and Dry Swales, so 100% Runoff Reduction is not met.

However, 1.76 lb/yr of TP reduction is provided for the proposed treatment train of Dry Swales in-series with a Filtering Practice #1. This exceeds the 1.49 lb/yr of TP reduction required for compliance by 0.28 lb/yr as shown for “TP Load Remaining” and a note in the tab.

As the TP load reduction exceeds the water quality criterion, the design should be optimized to reduce the excess, unless the additional 0.28 lb/yr of TP reduction can be used for compliance on another project in the same HUC, or for compliance with VDOT’s Chesapeake Bay TMDL Watershed Action Plan.

- Step 6. Instead of adding a Filtering Practice #1 in-series with the Dry Swales, consider the option to provide compliance offsite.

*Does the project qualify for the use of offsite options for compliance? If so, what % of the total TP Load Reduction Required can be met using offsite options?*

Recall that the sample project area is 1.7-acres (< 5-acres) and the TP Load Reduction Required is 1.49 lb/yr (< 10 lb/yr). For the sample project, 100% of the TP Load Reduction Required could be purchased according to the water quality offsite compliance criteria.

Another option would be to construct the Dry Swales as a RR practice and drainage system, but purchase the remaining credits, instead of putting a Filtering Practice #1 downstream of the Dry Swales. In this case, the Dry Swales alone provide a TP Load Reduction of 1.13 lb/yr, so the designer could choose to purchase the 0.35 lb/yr difference to bring the project into compliance with the water quality criterion.

Also, the designer must consider the need for water quantity control for the project. While the Filtering Device #1 provides no Runoff Reduction, the proposed Dry Swales reduce the runoff volume and provide water quantity control. The Runoff Reduction provided by the RR practice should be considered in a comparison of the design options, especially if a SWMF will be required for water quantity control.

In order to select the best option, the designer should consider the cost of the Dry Swales and Filtering Device #1 versus the cost of obtaining offsite compliance. The need for SWMFs for water quantity control should also be considered. The cost comparison should include both the capital costs (R/W, easements, and construction) as well as the long-term maintenance costs.

- Step 7. Using the information from the VRRM Spreadsheet and the selected water quality treatment design option (including the Treatment Volume required), design each BMP (RR practice or conventional) using the design standards in the VDOT BMP Design Manual of Practice, supplemented by the information provided in the Virginia BMP Clearinghouse.

**11.5.3.2.4 Water Quality – Sample Problem – Redevelopment**

Assume for the previous sample that the pre-development condition included 0.2 acres of impervious area and 0.3 acres of managed turf, with the balance undisturbed forest. The soils remain classified as HSG C by the USDA Soil Survey. In the post-developed condition, the site will include 0.8 acres of total impervious area (roadway and shoulder) and 0.9 acres of managed turf.

Step 1. As the project site was previously developed with impervious area and managed turf, the project constitutes “development on prior developed lands”, also referred to as Redevelopment. Use the VRRM Spreadsheet for Redevelopment for this sample project.

Step 2. On the Redevelopment Site tab, enter the project name and date of the calculations. As the project includes impervious roadway and shoulder, assume it is a linear development and mark the appropriate box “Yes”.

Complete the Land Cover table for both the pre- and post-development conditions:

**Pre-ReDevelopment Land Cover (acres)**

	A Soils	B Soils	C Soils	D Soils	Totals
<b>Forest/Open Space (acres)</b> -- undisturbed, protected forest/open space or reforested land			1.20		1.20
<b>Managed Turf (acres)</b> -- disturbed, graded for yards or other turf to be mowed/managed			0.30		0.30
<b>Impervious Cover (acres)</b>			0.20		0.20
					1.70

**Post-Development Land Cover (acres)**

	A Soils	B Soils	C Soils	D Soils	Totals
<b>Forest/Open Space (acres)</b> -- undisturbed, protected forest/open space or reforested land					0.00
<b>Managed Turf (acres)</b> -- disturbed, graded for yards or other turf to be mowed/managed			0.90		0.90
<b>Impervious Cover (acres)</b>			0.80		0.80
<b>Area Check</b>	OK.	OK.	OK.	OK.	1.70



Note that the pre-development condition land cover is required for the Redevelopment tab, as the TP load reduction requirement for development on prior developed lands where the net impervious cover area remains the same is a 20% reduction below existing conditions (for projects 1-acre or larger). Projects less than 1-acre with no net increase in impervious cover only require a 10% reduction below the existing pollutant load.

Also note that the spreadsheet compares the pre- and post-development impervious cover to determine what portion of a project is considered redevelopment (requiring a 20%/10% reduction from existing) versus additional net impervious area. When the post-development net impervious cover area increases above the pre-development area, the additional impervious area added must meet the new development criteria of 0.41 lb/acre/yr.

There is compliance criterion for linear development projects on prior developed lands (redevelopment). When the box in the Site tab is marked "Yes", noting the project is a linear redevelopment, the spreadsheet automatically calculates the alternate criterion by applying a 20% reduction to the pre-development TP load. The spreadsheet applies the allowable load for linear development project as the compliance goal for the sample project, as the project is a linear development.

- Step 3. The spreadsheet automatically calculates the post-development pollutant reduction requirements for the Site:

Post-Development Requirement for Site Area	
TP Load Reduction Required (lb/yr)	1.43
Linear Project TP Load Reduction Required (lb/yr): 1.49	

A detailed look at the results on the Site tab includes a Land Cover Summary and pollutant loading computation results for the pre- and post-development conditions. Note that two columns are included in the Pre-Redevelopment Land Cover Summary: the first is for the overall pre-development site conditions (1.70 acres), while the second is for the pre-development conditions adjusted to remove the additional area converted to new impervious cover.

<b>LAND COVER SUMMARY -- PRE-REDEVELOPMENT</b>		
<i>Land Cover Summary-Pre</i>		
<b>Pre-ReDevelopment</b>	<b>Listed</b>	<b>Adjusted<sup>1</sup></b>
Forest/Open Space Cover (acres)	1.20	0.60
Weighted Rv(forest)	0.04	0.04
% Forest	71%	55%
Managed Turf Cover (acres)	0.30	0.30
Weighted Rv(turf)	0.22	0.22
% Managed Turf	18%	27%
Impervious Cover (acres)	0.20	0.20
Rv(impervious)	0.95	0.95
% Impervious	12%	18%
<b>Total Site Area (acres)</b>	<b>1.70</b>	<b>1.10</b>
<b>Site Rv</b>	<b>0.18</b>	<b>0.25</b>

<sup>1</sup>Adjusted Land Cover Summary:

Pre-ReDevelopment land cover minus pervious land cover (forest/open space or managed turf) acreage proposed for new impervious cover.

Adjusted total acreage is consistent with Post-ReDevelopment acreage (minus acreage of new impervious cover).

Column I shows load reduction requirement for new impervious cover (based on new development load limit, 0.41 lbs/acre/year).

The redevelopment post-development (Post-ReDevelopment) summary is more complicated than the New Development spreadsheet results, because of the application of the redevelopment criteria to the pre-development load (including the pre-development impervious cover, but not the new impervious cover) and application of the new development criteria for the additional impervious cover. Also, the spreadsheet calculates the alternate criterion required for linear development.

## LAND COVER SUMMARY -- POST DEVELOPMENT

<i>Land Cover Summary-Post (Final)</i>		<i>Land Cover Summary-Post</i>		<i>Land Cover Summary-Post</i>	
<b>Post ReDev. &amp; New Impervious</b>		<b>Post-ReDevelopment</b>		<b>Post-Development New Impervious</b>	
Forest/Open Space Cover (acres)	0.00	Forest/Open Space Cover (acres)	0.00		
Weighted Rv(forest)	0.00	Weighted Rv(forest)	0.00		
% Forest	0%	% Forest	0%		
Managed Turf Cover (acres)	0.90	Managed Turf Cover (acres)	0.90		
Weighted Rv (turf)	0.22	Weighted Rv (turf)	0.22		
% Managed Turf	53%	% Managed Turf	82%		
Impervious Cover (acres)	0.80	ReDev. Impervious Cover (acres)	0.20		
Rv(impervious)	0.95	Rv(impervious)	0.95	Rv(impervious)	0.95
% Impervious	47%	% Impervious	18%		
<b>Final Site Area (acres)</b>	<b>1.70</b>	<b>Total ReDev. Site Area (acres)</b>	<b>1.10</b>		
<b>Final Post Dev Site Rv</b>	<b>0.56</b>	<b>ReDev Site Rv</b>	<b>0.35</b>		

Treatment Volume and Nutrient Load calculation results are reported for the pre- and post-redevelopment conditions as well:

Treatment Volume and Nutrient Load		
<b>Pre-ReDevelopment Treatment Volume (acre-ft)</b>	0.0253	0.0233
<b>Pre-ReDevelopment Treatment Volume (cubic feet)</b>	1,104	1,016
<b>Pre-ReDevelopment TP Load (lb/yr)</b>	<b>0.69</b>	<b>0.64</b>
Pre-ReDevelopment TP Load per acre (lb/acre/yr)	0.41	0.58
Baseline TP Load (lb/yr) (0.41 lbs/acre/yr applied to pre-redevelopment area excluding pervious land proposed for new impervious cover)	0.45	

Treatment Volume and Nutrient Load					
Final Post-Development Treatment Volume (acre-ft)	0.0798	Post-ReDevelopment Treatment Volume (acre-ft)	0.0323	Post-Development Treatment Volume (acre-ft)	0.0475
Final Post-Development Treatment Volume (cubic feet)	3,478	Post-ReDevelopment Treatment Volume (cubic feet)	1,408	Post-Development Treatment Volume (cubic feet)	2,069
Final Post-Development TP Load (lb/yr)	2.18	Post-ReDevelopment Load (TP) (lb/yr)*	0.88	Post-Development TP Load (lb/yr)	1.30
Final Post-Development TP Load per acre (lb/acre/yr)	1.29	Post-ReDevelopment TP Load per acre (lb/acre/yr)	0.80		
		Max. Reduction Required (Below Pre-ReDevelopment Load)	20%		

The load reduction required for the standard redevelopment and the net increase in impervious area are calculated and reported separately:

TP Load Reduction Required for Redeveloped Area (lb/yr)	0.37	TP Load Reduction Required for New Impervious Area (lb/yr)	1.05
---------------------------------------------------------	------	------------------------------------------------------------	------

Note that the TP Load reduction required for the standard redevelopment criterion, including a redevelopment reduction of 20% from the pre-development load (minus the pervious cover area converted to impervious cover in post-development) and an allowable loading of 0.41 lb/acre/yr for the net increase in impervious area. Based upon the results of the spreadsheet, the standard redevelopment reduction required is (0.37 lb/yr + 1.05 lb/yr) = 1.42 lb/yr rounded to 1.43 lb/yr in the spreadsheet. Standard redevelopment on

VDOT projects would not apply to linear roadways, but could apply to design for facilities and rest areas.

The alternate TP Load required for linear redevelopment projects is 20% less than the pre-development TP load. The pre-development TP load is reported as 0.69 lb/yr, so the allowable TP Load for linear development would be  $(1.0 - 0.2) \times 0.69 \text{ lb/yr} = 0.55 \text{ lb/yr}$ . The TP Load required reduction from the post-development load of 2.18 lb/yr is  $(2.18 \text{ lb/yr} - 0.55 \text{ lb/yr}) = 1.63 \text{ lb/yr}$ .

Lastly, the water quality criteria include a minimum allowable load based upon conditions of forest/open space, calculated using 0.41 lb/acre/yr as the allowable load. For the 1.70 acre sample project, the minimum allowable TP Load is  $(0.41 \text{ lb/acre/yr} \times 1.70 \text{ acres}) = 0.70 \text{ lb/yr}$ . The TP Load required reduction in this case is the post-development load minus the minimum allowable, or  $(2.18 \text{ lb/yr} - 0.70 \text{ lb/yr}) = 1.48 \text{ lb/yr}$  rounded to 1.49 lb/yr in the spreadsheet.

A comparison of the TP Load reduction required by the three criteria shows that the least stringent reduction requirement would be the standard redevelopment reduction of 1.43 lb/yr, followed by the minimum allowable load reduction of 1.49 lb/yr, while the most stringent TP load reduction is based upon the linear development criterion at 1.63 lb/yr.

*Based on the spreadsheet results reported above, which TP Load Reduction Requirement (lb/yr) must the sample project satisfy? What is the Treatment Volume required for 100% Runoff Reduction in ft<sup>3</sup>?*

In the end, the spreadsheet requires the minimum allowable load criterion and reports a TP Load reduction requirement of 1.49 lb/yr as the “Linear Project TP Load Reduction Required” on the Site spreadsheet. The Water Quality Compliance tab confirms the value applied. Note that this is more stringent than the standard redevelopment load reduction of 1.43 lb/yr, but if the project is a linear development, then the standard redevelopment criterion does not apply.

The Treatment Volume required for the post-redevelopment condition, in order to provide for 100% Runoff Reduction to address the increase in runoff volume, is reported as 3,478 ft<sup>3</sup>.

- Step 4. Unlike the first example for New Development, assume that only the post-development impervious cover area of 0.8 acres is being treated before discharging to the outfall, with the balance of the project site (managed turf) not being treated.

Move to the second tab labeled “D.A. A” and enter the post-development Land Cover information for the project. For this example, the Land Cover information for Drainage Area A is the same as for the project site:

**Drainage Area A Land Cover (acres)**

	A Soils	B Soils	C Soils	D Soils	Totals	Land Cover Rv
Forest/Open Space (acres)					0.00	0.00
Managed Turf (acres)			0.90		0.90	0.22
Impervious Cover (acres)			0.80		0.80	0.95
<b>Total</b>					<b>1.70</b>	

Step 5a. The first step in providing onsite treatment is to consider the use of Runoff Reduction practices. For the redevelopment example, assume only the post-development impervious cover is being treated to achieve water quality compliance for the site. Also, assume the soils have low permeability and are unsuitable for infiltration (based upon a site specific soil investigation), but the depth to seasonal high groundwater and bedrock is > 6’.

Based upon this information and the information provided in the VDOT BMP Design Manual of Practice and Virginia BMP Clearinghouse, start by selecting a Bioretention #2 as our RR practice.

Unlike the New Development example where the entire site was treated by BMPs, only the post-development impervious cover is being treated in this example. Enter the post-development impervious cover area into the spreadsheet under the column “Impervious Cover Credit Area” for practice “6.b. Bioretention #2 or Micro-Bioretention #2 (Spec #9)”:

**Stormwater Best Management Practices (RR = Runoff Reduction)**

Practice	Runoff Reduction Credit (%)	Managed Turf Credit Area (acres)	Impervious Cover Credit Area (acres)	Volume From Upstream Practice (ft <sup>3</sup> )
<b>6. Bioretention (RR)</b>				
6.b. Bioretention #2 or Micro-Bioretention #2 (Spec #9)	80		0.80	0

Once the information for the drainage to the BMP is entered, the spreadsheet will calculate the Runoff Reduction volume provided by the practice, the remaining volume of runoff not removed via the Runoff Reduction practice, the total Treatment Volume for the practice, the untreated TP load to the practice, the TP removed by the practice, and any remaining TP load after runoff reduction and treatment:

**Stormwater Best Management Practices (RR = Runoff Reduction)**

Practice	Runoff Reduction (ft <sup>3</sup> )	Remaining Runoff Volume (ft <sup>3</sup> )	Total BMP Treatment Volume(ft <sup>3</sup> )	Untreated Phosphorus Load to Practice (lb)	Phosphorus Removed By Practice (lb)	Remaining Phosphorus Load (lb)
<b>6. Bioretention (RR)</b>						
6.b. Bioretention #2 or Micro-Bioretention #2 (Spec #9)	2,207	552	2,759	1.73	1.56	0.17

Step 5b. *Based upon the results above, has 100% Runoff Reduction been achieved by the application of RR practice to the project? Has the TP Load reduction required been satisfied to achieve compliance?*

According to the Site tab, 3,478 ft<sup>3</sup> of Runoff Reduction is required to reduce the runoff volume to pre-development condition, but only 2,207 ft<sup>3</sup> is achieved in the Bioretention #2, so 100% Runoff Reduction is not met.

Also, 1.49 lb/yr of TP reduction is required for site compliance, and the Bioretention #2 practice provides 1.56 lb/yr of reduction, so the water quality criteria should be satisfied.

Compliance with the water quality requirements for the site can also be verified on the Water Quality Compliance tab:

### Site Results (Water Quality Compliance)

#### Runoff Reduction Volume and TP By Drainage Area

	D.A. A
<b>RUNOFF REDUCTION VOLUME ACHIEVED (ft<sup>3</sup>)</b>	2,207
<b>TP LOAD AVAILABLE FOR REMOVAL (lb/yr)</b>	2.18
<b>TP LOAD REDUCTION ACHIEVED (lb/yr)</b>	1.56
<b>TP LOAD REMAINING (lb/yr)</b>	0.63

#### Total Phosphorus    **LINEAR PROJECT:**

<b>FINAL POST-DEVELOPMENT TP LOAD (lb/yr)</b>	2.18
<b>TP LOAD REDUCTION REQUIRED (lb/yr)</b>	1.49
<b>TP LOAD REDUCTION ACHIEVED (lb/yr)</b>	1.56
<b>TP LOAD REMAINING (lb/yr):</b>	0.63

**REMAINING TP LOAD REDUCTION REQUIRED (lb/yr):                    0.00**

**\*\* TARGET TP REDUCTION EXCEEDED BY 0.07 LB/YEAR \*\***

Note that there is excess TP Load reduction of 0.07 lb/yr, as shown in the tab and noted by the text. Had the TP Load reduction not been satisfied, the number would be red.

Step 5c. The TP Load reduction required for the entire project was satisfied with the first application of RR practices to just the post-development impervious cover; therefore, further treatment is not necessary.

As the TP load reduction exceeds the water quality requirement, the design could be optimized to reduce the excess. The designer should check with the District Hydraulics Engineer to determine if the excess reduction should remain for use as offsite credit or for TMDL compliance.

According to the Site tab, 3,478 ft<sup>3</sup> of RR retention storage is required to reduce the runoff volume to pre-development conditions. Only 2,207 ft<sup>3</sup> is achieved in the RR practice, so 100% Runoff Reduction is not met, but the Bioretention #2 provides 63% of the RR retention storage, which will decrease the post-development runoff considerably and help achieve significant water quantity control.



- Step 6. Instead of using the RR practice to achieve water quality compliance onsite, consider the option to provide compliance offsite.

*Does the project qualify for the use of offsite nutrient credits for compliance? If so, what % of the total TP Load Reduction Required can be met using offsite options?*

Based upon the offsite criterion, a project with a site area less than 5-acres or a TP Load Reduction Requirement less than 10 lb/yr can use offsite options for water quality compliance, up to 100% of the TP Load Reduction Required. Recall that the sample project area is 1.7-acres (< 5-acres) and the TP Load Reduction Required is 1.49 lb/yr (< 10 lb/yr). Therefore, 100% of the TP Load Reduction Required could be purchased.

In order to select the best option (onsite versus offsite compliance), the designer should consider the cost of the Bioretention #2 versus the cost of obtaining offsite compliance through a value engineering analysis. The need for SWMFs for water quantity control should also be considered, since the Bioretention #2 provides a significant decrease in post-development runoff to meet water quantity goals for channel and flood protection. The value engineering cost comparison should include both the capital costs (R/W, easements, and construction) as well as the long-term maintenance costs over the operating life of the project and BMP.

The designer should consider the need for water quantity control for the project. The Runoff Reduction provided by the RR practice should be considered in a comparison of the design options, especially if a SWMF will be required for water quantity control.

- Step 7. Using the information from the VRRM Spreadsheet and the selected water quality treatment option, including the Treatment Volume required, design the BMP using the design standards in the VDOT BMP Design Manual of Practice, supplemented by the design standards in the Virginia BMP Clearinghouse.

### **11.5.3.3 Water Quantity – Channel Protection Procedure**

- Step 1. Identify project outfalls and receiving stormwater conveyance systems.*

*Determine project site (the project site, for the purposes of determining water quality and quantity compliance includes only the regulated land-disturbing activity) drainage area for each project outfall and the total drainage area to each stormwater conveyance system (system).*

- Step 2. Determine the limits of analysis in the system based upon the drainage area at each outfall and the total drainage area to the system:*

$$DA_{\text{outfall}} \leq \frac{DA_{\text{system}}}{100}$$

*Where:*

$DA_{\text{outfall}}$  = site drainage area at the outfall (acres or square miles)

$DA_{\text{system}}$  = total drainage area for the system at the limits of analysis (use units consistent with  $DA_{\text{outfall}}$ )

- Step 3. Determine the type of system below each outfall to the limits of analysis in order to identify the applicable channel protection criteria.
- Step 4. Determine the pre- and post-development peak rate and volume of runoff for each outfall and the system (see Chapter 4 Hydrology)
- Step 5. Using runoff information developed in Step 5, determine the limits of analysis at the point in the system where the total pre-development peak rate of runoff is 100 times greater than the outfall peak rate of runoff for each outfall during the 1-yr 24-hour storm:

$$Q_{\text{outfall}} \leq \frac{Q_{\text{system}}}{100}$$

*Where:*

$Q_{\text{outfall}}$  = post-development peak rate of runoff from the site at the outfall in the absence of SWM/BMP for the 1-yr 24-hour storm (cubic feet or second or cfs)

$Q_{\text{system}}$  = pre-development peak rate of runoff for the system at the limits of analysis (use units consistent with  $Q_{\text{outfall}}$ )

Use the limits of analysis for the point closest to the outfall based upon the two methods in Steps 3 and 5.

- Step 6. Determine the adequacy of the stormwater conveyance system from the outfall or point of discharge up to the limits of analysis using hydraulic calculation methods discussed in Chapters 6 to 9 of the Drainage Manual.

Include CN adjustment (see Section 11.5.2.1.8) where runoff reduction practices with retention storage are provided to adjust the allowable peak rate of runoff.

If the system is **manmade**, go to Step 6a.

If the system is **restored**, go to Step 6b.

If the system is **natural**, go to Step 6c.

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\* Rev. 10/23

Step 6a. For a **manmade** stormwater conveyance system (system), assess the system's ability to resist erosion for the post-development 2-yr 24-hour storm.

*If the system is unable to resist erosion in the post-development condition, use the methods outlined in Chapter 11, Sections 11.5.6 to 11.5.9 to design and incorporate SWMFs/BMPs into the plan to meet the channel protection criterion.*

*Alternatively, consider the option to make downstream system improvements to meet the criterion; a combination of SWM and system improvements; or apply the Energy Balance method at the outfall. (Go to Step 7c)*

Step 6b. For **restored** systems designed using natural channel design, evaluate the hydrologic and hydraulic design parameters used to restore the system to determine if the restored channel will function as designed in the post-development runoff conditions.

*If the system is unable to function as originally designed in the post-development condition, use the methods outlined in Chapter 11, Sections 11.5.6 to 11.5.9 to design and incorporate SWMFs/BMPs into the plan to meet the criterion for the system.*

*Alternatively, consider the option to make downstream system improvements to meet the criterion; a combination of SWM and system improvements; or apply the Energy Balance method at the outfall. (Go to Step 7c)*

Step 6c. For **natural** systems (or when selected as the design option under Steps 7a or 7b), apply the Energy Balance (see Section 11.5.2.1.4) using the pre- and post-development peak rate and volume of runoff for the contributing site area to each outfall. Where the post-development runoff from the site increases or the improvement factor (I.F.) applies, provide SWMFs or BMPs to reduce runoff and detain to allowable peak rates.

*Check to confirm that the allowable developed peak discharge resulting from the Energy Balance is not less than that required if the project site was forested. If it is less, than the allowable peak discharge should be the same as the project site in the forested condition.*

Step 7. Confirm that the **limits of analysis** identified and used for applying the channel protection criterion is the closest to the outfall based upon the two methods used in Steps 2 and 5.

### 11.5.3.3.1 Channel Protection - Limits of Analysis Sample Problems

- Step 1. Given an outfall with a drainage area of 0.5 acres, what would the minimum total drainage area in the downstream system be at the limit of analysis for the channel protection criterion?
- Step 2. Determine the limits of analysis in the system based upon the post-development drainage area at the outfall and the pre-development total drainage area in the system:

$$DA_{\text{outfall}} \leq \frac{DA_{\text{system}}}{100}$$

$$DA_{\text{system}} \geq DA_{\text{outfall}} \times 100$$

$$DA_{\text{system}} \geq 0.5 \text{ acres} \times 100 \geq 50 \text{ acres}$$

The adequacy analysis can end just before the point in the manmade system where the total pre-development system drainage area is 50 acres.

- Step 3. Assume the system below the outfall to the limits of analysis based upon drainage area comparison is a manmade system.
- Step 4. By applying standard hydrologic methods, it is determined that the post-development peak discharge from the site at the project outfall is 0.8 cfs, while the pre-development peak discharge in the system at the limits of analysis based upon the drainage areas is 105 cfs.
- Step 5. *Could the limits of analysis change based upon the peak discharges?* Determine the total pre-development peak rate of runoff at the point in the system is 100 times the outfall post-development peak rate of runoff:

$$Q_{\text{outfall}} \leq \frac{Q_{\text{system}}}{100}$$

$$Q_{\text{system}} \geq Q_{\text{outfall}} \times 100$$

$$Q_{\text{system}} \geq 0.8 \text{ cfs} \times 100 = 80 \text{ cfs}$$

Compare the peak rate of runoff at the limits of analysis identified in Step 2 based upon drainage area comparison (105 cfs) against the calculated peak rate of runoff that is 100 times greater than the outfall peak rate (80 cfs)

$$105 \text{ cfs} \geq 80 \text{ cfs}$$

As 105 cfs is greater than 80 cfs, the limits of analysis could be moved upstream from the point where the system drainage area is 100 times the project outfall drainage area to the point where the peak rate of runoff in pre-

development conditions is 80 cfs, which is 100 times the project outfall peak rate of runoff of 0.8 cfs.

### 11.5.3.3.2 Channel Protection - Adjusted CN Sample Problem

Step 6. A 0.5 acre site drains to a project outfall both in the pre- and post-development conditions. The site area has a post-development CN of 88. In order to meet water quality criteria, the RRM was applied and a BMP selected that provides a runoff reduction volume of 478 cubic feet. The 1-yr 24-hour storm precipitation depth is 2.75 inches and the runoff for the drainage area before applying runoff reduction practices is 1.60 inches. What is the adjusted CN for the 1-yr 24-hour storm after application of the runoff reduction BMP?

$$Q - R = \frac{(P - 0.2 \times S_{adj})^2}{(P + 0.8 \times S_{adj})}$$

Where:

Q = 1.60 inches

R = 478 cubic feet / (0.5 acre x 43560 square feet/acre) x 12 inches/foot = 0.26 inches

Q - R = 1.60 - 0.26 = 1.34 inches

P = 2.75 inches

$$1.34 = \frac{(2.75 - 0.2 \times S_{adj})^2}{(2.75 + 0.8 \times S_{adj})}$$

Compute  $S_{adj}$  using by solving the quadratic equation:  $S_{adj} = 1.85$  inches

Solve for  $CN_{adj}$ :

$$CN_{adj} = \frac{1000}{S_{adj} + 10} = \frac{1000}{1.85 + 10} = \frac{1000}{11.85} = 84$$

Alternatively, the adjusted CN for the 1-yr 24-hour storm can be obtained from the VRRM Spreadsheet.

### 11.5.3.3.3 Channel Protection - Manmade System Sample Problem

Step 6a. The process for conducting an adequacy analysis of a manmade stormwater conveyance system is discussed in detail in Chapters 7 (Ditches and Channels), 8 (Culverts), and 9 (Storm Drains). The analysis must demonstrate that the manmade system will resist the erosive forces in a 2-yr 24-hour storm for the post-development condition, including the total runoff to the system from offsite.

If the system is inadequate at any location from the outfall down to the limits of analysis, use detention design methods to provide attenuation in a SWMF such

that the system is adequate; design system improvements using the methods in Chapters 7, 8, and 9 to make the system adequate; or apply the Energy Balance method to comply with the Channel Protection criterion.

#### **11.5.3.3.4 Channel Protection - Restored System Sample Problem**

Step 6b. Assume the receiving system at the limits of analysis is a restored channel designed by the application of natural channel design techniques. The process for conducting an adequacy analysis of a restored stormwater conveyance system designed and constructed using natural channel design techniques is more complicated than for a manmade system. Some of the methods discussed in detail in Chapters 6 for open systems may apply, but more importantly the post-development condition must be reviewed to determine if the original restoration design is adequate for the post-development hydrologic and hydraulic conditions. The analysis must demonstrate that the restored system was designed and constructed to resist the flow conditions used for the natural channel design event(s) used in the system restoration, including the total runoff to the system.

If the restored system is inadequate at any location from the outfall down to the limits of analysis, use the detention design methods to provide attenuation in a SWMF such that the restored system is adequate for the post-developed condition; design system improvements using natural channel design methods to address the post-development runoff conditions; or apply the Energy Balance method to comply with the Channel Protection criterion.

#### **11.5.3.3.5 Channel Protection - Natural System Sample Problem**

Step 6c. Assume a VDOT project outfall with a site area of 1.7 acres (pre- and post-development conditions) discharging to a natural system. The USDA classifies the soils in the drainage area as HSG C, with pre-development land cover of 1.7 acres of open space in grass in good condition (CN = 74), and post-development land cover of 0.9 acres grass in good condition and 0.8 acres of impervious roadway (weighted CN = 85). The pre-development time of concentration is 20 minutes and the post-development time of concentration is 10 minutes.

Pre- and post-development hydrology has been developed using the TR-55 methodology presented in Chapter 4 and a VRRM Spreadsheet prepared for water quality computations. The 1-yr 24-hour precipitation for the project (obtained from Atlas 14) is 2.73 inches. The pre-development peak flow rate of runoff and runoff volume for the 1-yr 24-hour storm are 1.28 cfs and 0.603 in (0.0854 ac-ft), respectively.

For the post-development condition, the peak flow rate of runoff for the 1-year 24-hour storm is 3.22 cfs and the runoff volume is 1.233 in (0.1747 ac-ft), before the application of any runoff reduction practices.

*What is the allowable discharge rate of runoff from the site at the outfall?*

As the receiving system is natural below the outfall to the limits of analysis, the channel protection criteria require the application of the Energy Balance equation:

$$Q_{\text{Allowable}} = Q_{\text{Developed}} = \text{I.F.} \times \left( \frac{Q_{\text{Pre-Developed}} \times \text{RV}_{\text{Pre-Developed}}}{\text{RV}_{\text{Developed}}} \right)$$

Where:

I.F. = 0.80 (site > 1-acre)

$Q_{\text{Pre-Developed}} = 1.28$  cfs

$\text{RV}_{\text{Pre-Developed}} = 0.603$  in (0.0854 ac-ft)

$\text{RV}_{\text{Developed}} = 1.233$  in (0.1747 ac-ft)

$$Q_{\text{Allowable}} = Q_{\text{Developed}} = 0.80 \times \left( \frac{1.28 \times 0.114}{1.233} \right) = 0.10 \text{ cfs}$$

To meet the Energy Balance, the allowable Q must be reduced from 3.22 cfs to 0.10 cfs, which is a 3.12 cfs or 97% reduction in post-development peak flow for the 1-yr 24-hour storm.

Check the peak discharge from the 1.7 acre site assuming the land cover is forest (woods in good condition), using TR-55 Methodology. For woods in good condition (CN = 70 for HSG C), the peak rate of runoff for the 1-yr 24-hour is computed to be 1.21 cfs, which is higher than the computed allowable peak rate using the Energy Balance (0.10 cfs). Therefore, the allowable peak rate of runoff for the sample problem is the value for woods in good condition or 1.21 cfs. This requires a reduction of 2.01 cfs or 62% from the post-development peak rate of runoff.

In situations where the point of discharge includes drainage from the site (regulated land-disturbing activity) and additional undisturbed area from outside the project site, the improvement factor applies only to the regulated land-disturbing activity. In these instances it is necessary to separate the total drainage to the outfall into that coming from the project site and that coming from undisturbed off-site areas. Designers should determine the total peak discharge at the point of discharge (pre and post-development) including the project site and any undisturbed areas that may drain to the point of discharge. BMPs or SWMFs should be applied such that the post-development peak flow rate at the point of discharge is equal to or less than the total pre-development discharge from undisturbed areas plus the allowable Q from the project site.

*If RRM practices are provided for the 1.7 acres drainage area to address water quality in the form of a dry swale #2 design providing 2,087 ft<sup>3</sup> of runoff reduction, what would the allowable peak discharge at the outfall be for the developed condition?*

First, determine the adjusted CN for the 1-yr 24-hour storm at the project outfall:

$$Q - R = \frac{(P - 0.2 \times S_{adj})^2}{(P + 0.8 \times S_{adj})}$$

Where:

Q = 1.233 inches

R = 2,087 cubic feet/ (1.7 acres x 43560 square feet/acre) x 12 inches/foot = 0.338 inches (*from the RRM Spreadsheet*)

Q - R = 1.233 - 0.338 = 0.895 inches

P = 2.73 inches

$$0.895 = \frac{(2.73 - 0.2 \times S_{adj})^2}{(2.73 + 0.8 \times S_{adj})}$$

Solving the quadratic equation for  $S_{adj} = 2.97$ .

Solve for  $CN_{adj}$ :

$$CN_{adj} = \frac{1000}{S_{adj} + 10} = \frac{1000}{2.97 + 10} = \frac{1000}{12.97} = 77$$

The RRM Spreadsheet “Runoff Volume and CN” tab for the project calculates the adjusted CN for the 1-yr 24-hour storm as 79, based upon the RV numbers computed in the spreadsheet, as opposed to the Q values (watershed inches) calculated via TR-55.

Using  $CN_{adj} = 77$  in the TR-55 hydrologic calculations, determine the adjusted peak runoff ( $Q_{Developed}$ ) and runoff volume ( $RV_{Developed}$ ) for the post-development condition after the application of the RRM practices: 2.05 cfs and 0.756 inches, respectively.

With the CN adjusted post-development peak rate of runoff, the required reduction would be  $(2.05 \text{ cfs} - 1.21 \text{ cfs}) = 0.84 \text{ cfs}$  or  $(0.84 \text{ cfs}/3.22 \text{ cfs} \times 100\%) = 26\%$  of the unadjusted post-development peak rate of runoff.

By applying dry swales in the project drainage area to address water quality via the RRM, the designer also reduced the runoff volume through curve number (CN) adjustment. The CN adjustment decreased the reduction in peak rate of runoff from the developed site from 2.01 cfs (62% reduction) to 0.84 cfs (26%). This will reduce the storage volume required for detention to meet the natural channel protection criteria.

#### 11.5.3.4 Water Quantity – Flood Protection Procedure

*Step 1. Identify project outfalls, receiving stormwater conveyance systems, and mapped floodplain(s) or flood prone area(s) identified in a study (such as a FEMA floodplain identified in a FIRM or a FIS, or a local floodplain map supported by a study)*



Step 2. *If the stormwater conveyance system (system) immediately below an outfall is mapped as a floodplain or flood prone area, then the flood protection criteria is satisfied and no further analysis is required for that outfall.*

*Otherwise, proceed to Step 3.*

Step 3. *Determine project site drainage area for each project outfall.*

Step 4. *Determine the limits of analysis in the system based upon the drainage area at each outfall and the total drainage area to the system:*

$$DA_{\text{outfall}} \leq \frac{DA_{\text{system}}}{100}$$

*Where:*

$DA_{\text{outfall}}$  = *project drainage area at the outfall (acres or square miles)*

$DA_{\text{system}}$  = *total drainage area for the system at the limits of analysis (use units consistent with  $DA_{\text{outfall}}$ )*

*If a floodplain or flood prone area is mapped in the system upstream of the limits of analysis determined in Step 4, then the flood protection analysis can stop at the mapped floodplain or flood prone area. Go to Step 8 to determine the adequacy of the system to the limits of analysis.*

*Otherwise, continue to Step 5.*

Step 5. *Determine the pre- and post-development peak rate and volume of runoff for each outfall and the system for the 10-yr 24-hour storm (see Chapter 4 Hydrology)*

Step 6. *Using runoff information developed in Step 5, determine the limits of analysis at the point in the system where the total pre-development peak rate of runoff is 100 times greater than the outfall peak rate of runoff for each outfall during the 10-yr 24-hour storm:*

$$Q_{\text{outfall}} \leq \frac{Q_{\text{system}}}{100}$$

*Where:*

$Q_{\text{outfall}}$  = *post-development peak rate of runoff at the outfall in the absence of SWM/BMP for the 10-yr 24-hour storm (cubic feet or second or cfs)*

$Q_{\text{system}}$  = *pre-development peak rate of runoff for the system at the limits of analysis (use units consistent with  $Q_{\text{outfall}}$ )*

*If a floodplain or flood prone area is mapped in the system upstream of the limits of analysis determined in Step 6, then the flood protection analysis can stop at the mapped floodplain or flood prone area. Go to Step 8 to determine the adequacy of the system to the limits of analysis.*

Otherwise, continue to Step 7.

- Step 7. *If a floodplain or flood prone area is mapped in the system upstream of the limits of analysis determined in Steps 4 and 6, then the flood protection analysis can stop at the mapped floodplain or flood prone area. Go to Step 8 to determine the adequacy of the system to the limits of analysis.*

*Use the limits of analysis for the point closest to the outfall based upon the two methods in Steps 4 and 6.*

- Step 8. *Is there documentation available to demonstrate the performance of the system below the outfall in the pre-development conditions during the 10-yr 24-hour the system is currently flooding or not flooding?*

*If so, go to Step 8a.*

*If not, go to Step 8b.*

- Step 8a. *If the available documentation demonstrates that the system below an outfall does not currently experience localized flooding during a 10-yr 24-hour storm event, then go to Step 9.*

*If there is documentation that the system below an outfall currently experiences localized flooding during the 10-yr 24-hour event, go to Step 10.*

- Step 8b. *If there is no documentation that a system below an outfall does or does not currently experiencing flooding during the 10-yr 24-hour storm, hydraulic modeling of the system in the pre-development condition should be completed to determine if flooding is reasonably expected. See Chapters 6, 7, 8, and 9 for details on hydraulic modeling of systems.*

*If modeling of the pre-development condition demonstrates that no flooding is reasonably expected in the system down to the limits of analysis, proceed to Step 9. If the modeling documents potential flooding in the system to the limits of analysis below the outfall under the pre-development condition, then proceed to Step 10.*

*Note that this step may be necessary just to define the stormwater conveyance system for flood protection, which includes both the main channel and adjacent flood prone areas.*

- Step 9. *For a system that does not currently experiencing localized flooding:*

*Using the hydrologic modeling results from Step 5, prepare a hydraulic analysis for system using the post-development 10-yr 24-hour storm to determine if the post-development peak flow rate is confined within the system to the limits of analysis.*

*If the post-development peak flow rate is contained within the system to the limits of analysis, then the system is adequate below the outfall and no further SWM is required for flood protection.*

*If the post-development peak flow rate is not contained within the system to the limits of analysis, provide stormwater detention or system improvements to make the system adequate. Go to Step 11.*

**Step 10.** *For a system that currently experiences localized flooding:*

*Using the hydrologic modeling results from Step 5, prepare a hydraulic analysis of the system using the post-development 10-yr 24-hour storm to determine if the post-development peak flow rate is confined within the system to the limits of analysis.*

*If the post-development peak flow rate is contained within the system to the limits of analysis, then the system is adequate below the outfall and no further SWM is required for flood protection.*

*If the post-development peak flow rate is not contained within the system to the limits of analysis:*

- *Provide stormwater detention or system improvements to make the system adequate for the post-development condition, or*
- *Provide a design that releases the post-development peak flow rate to less than the pre-development peak flow rate for the 10-yr 24-hour storm.*

*Go to Step 11.*

**Step 11.** *See the methods in Chapter 11, Sections 11.5.6 to 11.5.9 for design of detention facilities to make the system adequate for the post-development condition, or to provide a post-development peak runoff rate that is less than pre-development when the system below the outfall is currently experiencing flooding.*

*See the methods in Chapters 6, 7, 8, and 9 for making the system below the outfall adequate for the post-development peak rate of runoff.*

#### **11.5.3.4.1 Flood Protection - Limits of Analysis**

Determining the limits of analysis is critical in demonstrating compliance for Flood Protection. The limits of analysis for flood protection must extend downstream to\* the point in the system below the outfall where the total system drainage area is 100 times greater than the project outfall drainage area; to the point where the total system peak rate of runoff is 100 times greater than the project outfall peak rate of runoff, both based upon the 10-yr 24-hour storm; or to the point in the system where a mapped floodplain exists.

Note that the system must be analyzed from the outfall to the limits of analysis, not just below the outfall or just at the limits of analysis. This may require extensive hydrologic and hydraulic modeling to demonstrate that the system is adequate.

Before just choosing to provide detention for a project outfall, the designer should assess the adequacy of the existing system to determine the extent of the system (main channel and adjacent flood prone area), whether flooding exists pre-development, and what peak flows the system can adequately convey without causing flooding of properties and waterways. The adequacy analysis could result in higher allowable peak runoff for the post-development condition, especially if there is existing flooding occurring.

Note that systems that are not currently experiencing flooding require detention to the peak rate that is adequate, not just detention to the pre-development peak rate of runoff from the project at that outfall. Depending upon the system, this could mean a peak runoff less than or greater than the pre-development condition. A system adequacy determination to the limits of analysis will allow the designer to optimize the SWM plan for flood protection.

#### **11.5.3.4.2 Flood Protection - Limits of Analysis Sample Problems**

Step 4. Given an outfall from a project with a contributing site area of 10.3 acres, what would the minimum total drainage area in the downstream system be at the limit of analysis for the flood protection criterion?

$$DA_{\text{outfall}} \leq \frac{DA_{\text{system}}}{100}$$

$$DA_{\text{system}} \geq DA_{\text{outfall}} \times 100$$

$$DA_{\text{system}} \geq 10.3 \text{ acres} \times 100 \geq 1,030 \text{ acres}$$

The adequacy analysis can end at the point in the manmade system where the total system drainage area is 1,030 acres.

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\* Rev. 10/23

- Step 6. The post-development peak discharge for the 10-yr 24-hour storm from the site is computed to be 187 cfs, while the pre-development peak discharge in the system at the limits of analysis based upon the drainage area comparison above is 15,895 cfs. Could the limits of analysis change based upon the peak discharges?

$$Q_{\text{outfall}} \leq \frac{Q_{\text{system}}}{100}$$

$$Q_{\text{system}} \geq Q_{\text{outfall}} \times 100$$

$$Q_{\text{system}} \geq 187 \text{ cfs} \times 100 \geq 18,700 \text{ cfs}$$

$$15,895 \text{ cfs} < 18,700 \text{ cfs}$$

As  $Q_{\text{system}}$  (15,895 cfs) is less than  $Q_{\text{outfall}} \times 100$  (18,700 cfs), the limits of analysis cannot be moved upstream based upon flow comparison.

- Step 7. If the system below the outfall is mapped in a Flood Insurance Rate Map (FIRM) adopted by FEMA as a Zone AE Special Flood Hazard Area (SFHA). The floodplain mapping begins at a point in the system below the outfall where the total drainage area is 1 square mile. *Can the limits of analysis for the flood protection criterion be moved?*

The system drainage area below the outfall at the limits of analysis based upon comparison of drainage areas is 1,030 acres. The floodplain mapping generally begins in the system at a point with a total drainage area of 1 square mile, or 640 acres. As the system drainage area where floodplain mapping begins (640 acres) is smaller than the system drainage area for  $DA_{\text{outfall}} \times 100$ , the flood prone area is located upstream. Therefore, the limits of analysis for flood protection can be moved upstream to the point in the system where flooding is mapped. This can be confirmed by using the FIRM to determine if the floodplain mapping begins before the point in the system where the drainage area is equal to  $DA_{\text{outfall}} \times 100$ .

#### **11.5.3.4.3 Flood Protection Sample Problem – Localized Flooding Not Currently Experienced**

- Step 1. A project with a single outfall discharges to a natural stormwater conveyance system. No mapped floodplain or flood prone areas (as identified in studies) are found immediately below the outfall.
- Step 2. As the system at the outfall is not mapped as a floodplain or flood prone area, a system adequacy analysis is required to satisfy the flood protection criteria.
- Step 3. The post-development drainage area at the project outfall is 2.3 acres.

- Step 4. Based upon drainage area comparisons, the limits of analysis are located in the receiving system where the total drainage area is 230 acres.
- Step 5. The post-development 10-yr 24-hour peak runoff rate at the project outfall is 12.6 cfs. At the limits of analysis identified by drainage area comparison, the pre-development 10-yr 24-hour peak rate of runoff is 990 cfs.
- Step 6. The pre-development peak rate of runoff for the 10-yr 24-hour storm at the limits of analysis based upon drainage areas (990 cfs) is less than 100 times the post-development project outfall peak rate of runoff for the 10-yr 24-hour storm ( $100 \times 12.6 \text{ cfs} = 1,260 \text{ cfs}$ ). This means that the limits of analysis based upon a comparison of the peak rate of runoff is located downstream of the limits based upon the drainage area comparison.
- Step 7. As no floodplain or flood prone areas are mapped downstream of the project outfall, and the limits of analysis based upon the drainage area comparison is closer to the project outfall than the limits based upon comparison of the peak rates of runoff, the adequacy analysis can end at the limits of analysis based upon the drainage area comparison.
- Step 8. Historic flood data and anecdotal information for the area downstream of the project provide no evidence of flooding currently experienced in the 10-yr 24-hour storm.
- Step 8a. As there exists documentation demonstrating that the system below an outfall does not currently experience localized flooding during a 10-yr 24-hour storm event, the flood adequacy analysis can move to Step 9.
- Step 9. Given the hydrologic information developed in Step 5 for the 10-yr 24-hour storm event, a hydraulic analysis of the receiving system capacity from the project outfall to the limits of analysis is conducted using good hydrologic engineering methods and practices presented in Chapters 6 and 12 of the Drainage Manual.

For the sample problem, the receiving system (main channel and adjacent flood prone areas) is well defined, so the pre-development peak rate of runoff does not need to be modeled hydraulically to determine the pre-development flood prone areas. However, the system is modeled to determine that the capacity at the limits of analysis is 1,050 cfs. *Is the system adequate for the post-development flows in the 10-yr 24-hour storm?*

In Step 6 it was determined that the post-development peak rate of runoff for the system at the limits of analysis is 1,260 cfs. As 1,260 cfs is greater than the system capacity of 1,050 cfs, the system does not confine the post-development peak rate of runoff and the flood protection criteria is not met. To confirm this conclusion, the system can be modeled hydraulically with the post-development peak rate of runoff to determine if the water surface elevations increase and flooding goes outside of the flood prone areas adjacent to the

main channel. Move on to Step 11 to make changes to the land-disturbing activity or receiving system such that the criterion is met.

Note that the system must adequately convey the post-development peak rate of runoff from the outfall to the limits of analysis, not just at the limits of analysis. While a system may prove adequate at the limits of analysis, there may be segments between the outfall and limits of analysis that are not adequate to convey the post-development peak rate of runoff for the 10-yr, 24-hr storm. The best way to demonstrate the overall adequacy of the system for the post-developed conditions is to generate modeling for the system along its length downstream to the limits of analysis.

- Step 11. As the existing system below the outfall does not convey the post-development peak rate of runoff in the sample problem, either detention or stormwater system improvements may be incorporated into the land-disturbing activity to meet the flood protection capacity.

Note that as the pre-development system is deemed not to currently experience flooding based on historic record or anecdotal evidence, the design goal for detention is to meet the system capacity, not to discharge less than the pre-development peak rate of runoff. If system improvements are chosen to address flood protection, then the improvements should be designed for the post-development runoff conditions. Also, a combination of detention and system improvements is an alternative.

Because the system was deemed adequate for the pre-development conditions based upon existing evidence that flooding was not occurring, it is possible that the peak rate of runoff for the system capacity may actually be higher than the pre-development peak rate of runoff. When that is the case, detention should be designed to achieve the system capacity, when the existing system is not currently experiencing flooding.

Given that system improvements usually require work outside of the project area, necessitating additional R/W or easements, and requiring water quality permits for work in waters of the U.S., detention of runoff within the project limits is likely to be the most efficient and effective option. A cost comparison of detention onsite versus offsite system improvements should be made to determine the best option to address flood protection.

However, if system improvements are pursued due to cost effectiveness or other project benefits or site constraints, which require additional R/W or easements, work outside of the project area, and water quality permits for work in waters of the U.S., then it may make sense to use just system improvements.

*What if the flood prone area adjacent to the main channel is modeled for the pre-development peak rate of runoff and it is determined that the 10-yr 24-hour storm event is not confined within the system?*

If this occurs, then there is now information available that counters the historic record and anecdotal evidence that supported a conclusion of no localized flooding. In this case, it may be necessary to re-evaluate the flood protection criteria assuming that the system does currently experience localized flooding. An example of applying the flood protection criteria for systems that currently experience flooding is presented in the next section.

#### **11.5.3.4.4 Flood Protection Sample Problem – Localized Flooding Currently Experienced**

*What if the system in the previous sample problem was currently experiencing flooding in pre-development flow conditions, based upon historic flood record, anecdotal evidence, a system specific floodplain study, or by the designer when routing the pre-development peak rate of runoff through the system in a hydraulic model?*

Step 8a. As there exists documentation demonstrating that the system below the outfall does currently experience localized flooding during a 10-yr 24-hour storm event, the flood adequacy analysis can move to Step 10.

Step 10. Given the hydrologic information developed in Step 5 for the 10-yr 24-hour storm event, a hydraulic analysis of the receiving system capacity from the project outfall to the limits of analysis is conducted using good hydrologic engineering methods and practices presented in Chapters 6 and 12 of the Drainage Manual.

For the sample problem, the receiving system (main channel and adjacent flood prone areas) is well defined, so the pre-development peak rate of runoff does not need to be modeled hydraulically to determine the pre-development flood prone areas. However, the system is modeled to determine that the capacity at the limits of analysis is 1,050 cfs. *Is the system adequate for the post-development flows in the 10-yr 24-hour storm?*

In Step 6 it was determined that the post-development peak rate of runoff for the system at the limits of analysis is 1,260 cfs. As 1,260 cfs is greater than the system capacity of 1,050 cfs, the system does not confine the post-development peak rate of runoff and the flood protection criteria is not met. Move on to Step 11 to make changes to the land-disturbing activity or receiving system such that the criterion is met.

Note that the system must adequately convey the post-development peak rate of runoff from the outfall to the limits of analysis, not just at the limits of analysis. While a system may prove adequate at the limits of analysis, there may be segments between the outfall and limits of analysis that are not adequate to convey the post-development peak rate of runoff for the 10-yr, 24-hr storm. The best way to demonstrate the overall adequacy of the system for the post-developed conditions is to generate modeling for the system along its length downstream to the limits of analysis.



Step 11. As the existing system below the outfall does not convey the post-development peak rate of runoff in the sample problem, either detention or stormwater system improvements may be incorporated into the land-disturbing activity to meet the flood protection capacity.

Note that as the pre-development system is deemed to currently experience flooding based on historic record or anecdotal evidence, the design goal for detention is to either meet the system capacity or to discharge less than the pre-development peak rate of runoff from the project at the outfall in the post-development conditions. This is different than the requirement for a system that is not currently experiencing flooding, as it allows the option to provide onsite detention to pre-development peak rates of runoff instead of providing detention to the system capacity. In this case, the designer should evaluate both detention criteria to determine which requires the least onsite detention and design for that criterion.

For example, if the detention required to meet the system adequacy is a 1-acre basin area storing 4 acre-feet of runoff, and the detention required to detain to less than the pre-development peak rate of runoff only requires a 0.7-acre basin area storing 2.5 acre-feet, then the smaller detention basin is likely to be more cost effective to implement. The designer should do a cost comparison, including the cost for R/W and easements as well as construction and maintenance, to demonstrate which basin is most cost effective for the final design.

If system improvements are chosen to address flood protection, then the improvements should be designed for the post-development runoff conditions. Also, a combination of detention and system improvements is an alternative.

As noted in the previous example, system improvements usually require work outside of the project area, necessitating additional R/W or easements, and requiring water quality permits for work in waters of the U.S. For this reason, detention of runoff is likely to be the most efficient and effective option. A cost comparison of detention versus offsite system improvements should be made to determine the best option to address flood protection.

If system improvements are pursued due to cost effectiveness or other project benefits or site constraints, which require additional R/W or easements, work outside of the project area, and water quality permits for work in waters of the U.S., then it may make sense to use just system improvements.

#### **11.5.4 Pretreatment**

Pretreatment is an important component of most water quality BMPs. Its purpose is to remove gross pollutants (sand, grit, gravel, trash, and debris) from stormwater runoff in an area that is easier to access and maintain, protecting the primary BMP downstream from contamination and extending the maintenance life of the overall BMP. Pretreatment features may require more frequent maintenance themselves, but their intent is to lengthen the time

between maintenance activities in the primary BMP. Examples of pretreatment practices include vegetated filter strips, sediment forebays, energy dissipaters, and manufactured treatment devices.

Some of the new RR practices require more than one form of pretreatment in order to achieve runoff reduction and higher pollutant removal efficiencies. Details on pretreatment selection and design are found in the VDOT Part II B BMP Design Manual of Practice and the Virginia BMP Clearinghouse Standards and Specifications.

### 11.5.5 Treatment Volume Computation

Treatment Volume (Tv) for water quality replaces the old concept of “Water Quality Volume (WQV)” in the Virginia Runoff Reduction Method (VRRM). The value of Tv is the product of the “Target Rainfall Event” (1.00 inch), the site area (acres), and the site Rv from the VRRM, with a unit conversion:

$$T_v = 1.00 \text{ inches} \times \text{Site Area acres} \times R_v \times \left( \frac{1 \text{ foot}}{12 \text{ inches}} \right)$$

*What is the Tv for a 1.7 acre site with a computed post-development Rv of 0.56?*

$$T_v = 1.00 \text{ inches} \times 1.7 \text{ acres} \times 0.56 \times \left( \frac{1 \text{ foot}}{12 \text{ inches}} \right) = 0.0793 \text{ acre – feet}$$

The VRRM Spreadsheet automatically calculates the water quality Tv requirements for the entire site on the Site tab after the land cover information is entered.

Once BMPs are applied to a drainage area, the Tv for the contributing drainage area to each BMP must be calculated for proper sizing of the BMP. The VRRM Spreadsheet also calculates the Tv to each BMP in the drainage area tabs, using the contributing drainage area and the Rv calculated for the contributing drainage area to each BMP. For BMPs in-series, the Tv for a downstream BMP is based upon the Tv for the upstream BMP minus the RR retention storage volume provided by the upstream RR practice.

Some BMP designs require a storage or treatment volume that is more than one Tv to achieve the published pollutant reduction rates. This is presented in the VDOT BMP Design Manual of Practice and the Virginia BMP Clearinghouse Standards. For example, the Bioretention Level 2 and the Extended Detention Level 2 standards require a design using 1.25 times Tv to achieve higher removal rates. A Wet Pond #2 design is based upon 1.5 times the Tv, but has a higher pollutant removal rate than a Wet Pond #1.

### 11.5.6 Detention Time Computation and Orifice Sizing

A water quality extended-detention basin treats runoff by detaining it and releasing it over a specified amount of time. In theory, extended-detention of the required Treatment Volume will allow the particulate pollutants to settle out of runoff, functioning similarly to a permanent pool in a Wet Pond. The Virginia BMP Clearinghouse Standard for Extended Detention Level 1 specifies 24-hours or less draw down for the average Tv time, while the Extended Detention Level 2 design requires an average Tv draw down time of 36 hours.

This is a brim draw down time, beginning at the time of peak storage of the water quality volume. Brim drawdown time means the time required for the entire calculated volume to drain 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. The extended detention orifice can be sized using either of the following methods, taken from the Virginia Stormwater Management Handbook (VSWMH), 1999 edition:

- Using the **average hydraulic** head associated with the required  $T_v$  and draw down time. This is the VDOT preferred option.
- Using the **maximum hydraulic** head associated with the  $T_v$ , calculate the orifice size needed to achieve the required draw down time and route the  $T_v$  through the basin to verify the actual storage volume used and the drawdown time.

After calculating the needed orifice size the designer should select the nearest nominal size opening from Table 11-10.

**Table 11-1. WQV Orifice Sizes**

Diameter	
Inches	Square Feet
1/2	0.001
3/4	0.003
1	0.005
1 1/2	0.012
2	0.022
2 1/2	0.034
3	0.049
3 1/2	0.067
4	0.087
4 1/2	0.110
5	0.136
5 1/2	0.165
6	0.196

**11.5.6.1.1 Average Hydraulic Head Method (Method #2 from VSWMH) - VDOT Preferred Method**

The average hydraulic head method is the preferred method for determining the required orifice size. It is quicker and easier than the maximum hydraulic head method, which requires a routing to verify the drawdown time. It is also noted that the difference in orifice size produced by the two different methods is insignificant, (i.e. 2 inches versus 2 1/2 inches).

Average Hydraulic Head Sample Problem:

Find the orifice size for the required treatment volume for an Extended Detention Level 1 design using the average hydraulic head method, where:

$$h_{\max} = 1.1 \text{ ft.}$$
$$Tv = 8,720 \text{ ft}^3$$

Step 1. Calculate the average head:

$$h_{\text{avg}} = \frac{1.1 \text{ ft}}{2} = 0.55 \text{ ft}$$

Note: Actual h for an orifice is measure from the center of the orifice. Since the size of this orifice is unknown and assumed small, use  $h_{\max} = 1.1 \text{ ft}$ .

Step 2. Determine the draw down time for an Extended Detention Level 1 design from the VDOT BMP Manual of Practice: average 24-hr detention time.

Step 3. Calculate the discharge through the orifice based for the required treatment volume based upon the required detention time:

$$Q_{\text{avg}} = \frac{Tv}{\text{Extended Detention Time}} = \frac{8,720 \text{ ft}^3}{24 \text{ hr} \times 3,600 \frac{\text{sec}}{\text{hr}}} = 0.101 \text{ cfs}$$

Step 4. Calculate the orifice area by rearranging the orifice equation:

$$A = \frac{Q_{\text{avg}}}{C\sqrt{2 \times g \times h_{\text{avg}}}} = \frac{0.101 \text{ cfs}}{0.6\sqrt{2 \times 32.2 \times 0.55}} = 0.0283 \text{ ft}^2$$

Step 5. From Table 11-10, select a 2-inch orifice with  $A = 0.022 \text{ ft}^2$ .

Step 6. The Tv hydrograph should then be routed through the basin to determine if the residence time is approximately 24 hours.

### 11.5.6.1.2 Maximum Hydraulic Head Method (Method #1 from VSWMH)

The maximum hydraulic head method uses the maximum discharge and results in a slightly larger orifice than the same procedure using the average hydraulic head method. The routing allows the designer to verify the performance of the calculated orifice size. However, as a result of the routing effect, the actual basin storage volume used to achieve the drawdown time will be less than the computed brim drawdown volume.

Maximum Hydraulic Head Sample Problem:

*Find the orifice size for the required treatment volume for an Extended Detention Level 1 design using the maximum hydraulic head method, where:*

$$h_{\max} = 1.1 \text{ ft.}$$

$$Q_{\text{avg}} = 0.101 \text{ cfs}$$

Step 1. Calculate the maximum  $Q_{\max}$ :

$$Q_{\max} = 2 \times Q_{\text{avg}} = 2 \times 0.101 \text{ cfs} = 0.202 \text{ cfs}$$

Step 2. Using  $h_{\max}$  and  $Q_{\max}$ , compute the orifice size for the required treatment volume using the maximum hydraulic head method.

Calculate the orifice area by rearranging the orifice equation:

$$A = \frac{Q_{\max}}{C\sqrt{2 \times g \times h_{\max}}} = \frac{0.202 \text{ cfs}}{0.6\sqrt{2 \times 32.2 \times 1.1}} = 0.040 \text{ ft}^2$$

Step 3. From Table 11-10, select a 2½-inch orifice with  $A = 0.034 \text{ ft}^2$

Step 4. Route the  $T_v$  hydrograph through the basin using the 2½-inch orifice.

NOTE: The routing of the  $T_v$  hydrograph thru a basin may not be possible with some routing software where hydrographs lasting longer than 24 hours are not accommodated. The problem is due to detention times greater than 24-hours to achieve hydrograph draw down to 0 cfs, possibly greater than 30 or more hours at very low flows.

### 11.5.6.1.3 $T_v$ Hydrograph

To develop a runoff hydrograph for the  $T_v$ , the designer should use the “Target Rainfall Event” of 1 inch, HSG and land cover for post-development conditions, and the hydrograph development techniques presented in Chapter 6 of the Drainage Manual.

The NRCS unit hydrographs will probably be the easiest hydrographs to provide the required treatment volume for an extended detention basin or other SWMF. The land cover conditions should be based upon the NRCS CN method. Based upon the VRRM,

“Forest/Open Space” uses the CNs for “Woods, Good”; “Managed Turf” uses the CNs for “Open Space, Good”; and “Impervious Cover” has a CN = 98 for all HSG.

#### **11.5.6.1.4      *Alternative Method of Routing WQV to Find Drawdown Time***

The Stormwater Management Handbook, Vol. II, 1999 edition, defines “brim drawdown time” as the time the treatment volume elevation is reached until the basin is emptied. This is based upon a storm producing only the amount of runoff required for the Tv, based upon a “Target Rainfall Event” of 1 inch.

The normally required routing of a storm larger than the “Target Rainfall Event” of 1 inch for quantity control can also be used for drawdown time with some adjustment providing that the routing software will accommodate a duration greater than 24-hours. The receding limb of the inflow hydrograph will need to be showing either 0.0 or 0.01 cfs inflow up to a time of 30 hours for an Extended Detention Level 1 design, and up to 48 hours for an Extended Detention Level 2 design.

By this method the drawdown time for Tv is actually from the time that the ponded depth recedes to the treatment volume elevation with no more inflow (remember that this method is for storm events > 1-inch) until the basin is “empty” (receding limb of the inflow hydrograph will need to be showing either 0.0 or 0.01 cfs).

### **11.5.7    Preliminary Detention Volume Computation**

Three methods are presented for estimating the volume of storage needed for peak flow attenuation (quantity control). The estimated storage volumes are approximate and the designer will need to select the most appropriate volume in order to determine the preliminary basin size.

#### **11.5.7.1.1      *Modified Rational Method, Simplified Triangular Hydrograph Routing***

Information needed includes the hydrology and hydrographs for the watershed or drainage area to be controlled, calculated by using one of the methods as outlined in Chapter 6, and the allowable release rates for the facility, as established by the water quantity criteria for channel and flood protection.

##### ***Step 1.    Determine BMP requirements***

Calculate the water quality Treatment Volume (Tv) using the steps presented previously.

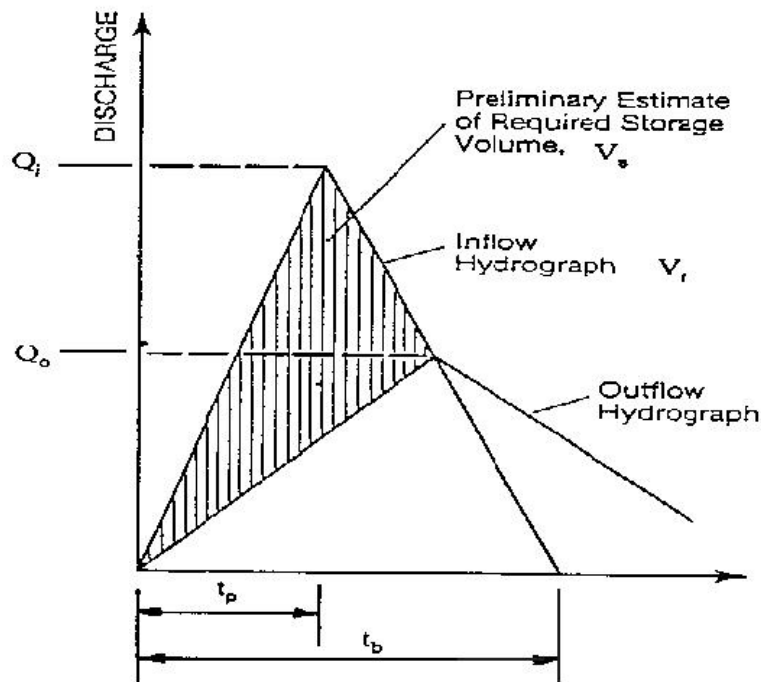
##### ***Step 2.    Compute allowable release rates***

Compute the pre- and post-developed hydrology for the site outfall using the methods presented previously. In either case, the post-developed hydrology will provide the peak discharge into the basin as a peak discharge (cfs), a runoff

volume (watershed inches, acre-feet, or ft<sup>3</sup>), or a runoff hydrograph (cfs over storm and runoff duration). Refer to Chapter 6, Hydrology, on developing peak discharge, runoff volume, and runoff hydrographs.

**Step 3. Estimate the required storage volume**

The information required includes the developed condition peak rate of runoff, or runoff hydrograph, and the allowable release rates for the appropriate design storms. These methods provide a preliminary estimate of the storage volume required for peak flow attenuation.



**Figure 11-1. Simplified Triangular Hydrograph Method**

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

$$V_s = \frac{1}{2} T_b (Q_i - Q_o)$$

Where:

- V<sub>s</sub> = Storage volume estimate, ft<sup>3</sup>
- Q<sub>i</sub> = Peak inflow rate, cfs
- Q<sub>o</sub> = Peak outflow rate, cfs
- T<sub>b</sub> = Duration of basin inflow, sec.

### 11.5.7.1.2 Critical Storm Duration Method

The critical storm duration method is used to calculate the maximum storage volume for a detention facility. This critical storm duration is the storm duration that generates the greatest volume of runoff and, therefore, requires the most storage. The required storage volume is represented by the area between the inflow hydrograph and the outflow hydrograph. The area can be approximated using the following equation:

$$V = \left[ Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3q_o t_c}{4} \right] 60$$

Where:

$V$  = Required storage volume, ft<sup>3</sup>

$Q_i$  = Inflow peak discharge, cfs, for the critical storm duration,  $T_d$

$T_c$  = Time of concentration, min.

$q_o$  = Allowable peak outflow, cfs

$T_d$  = Critical storm duration, min.

The first derivative of the critical storage volume equation with respect to time is an equation that represents the slope of the storage volume curve plotted versus time. When the equation above is set to equal zero, and solved for  $T_d$ , it represents the time at which the slope of the storage volume curve is zero, or at a maximum.

The equation for the critical storm duration is:

$$T_d = \sqrt{\frac{2CAa \left( b - \frac{t_c}{4} \right)}{q_o}} - b$$

Where:

$T_d$  = Critical storm duration, min.

$C$  = Runoff coefficient

$A$  = Drainage area, ac.

$a$  &  $b$  = Rainfall constants developed for storms of various recurrence intervals and various geographic locations

$t_c$  = Time of concentration, min.

$q_o$  = Allowable peak outflow, cfs

\*The  $a$  &  $b$  rainfall constants are not to be used for any other purpose.

The Department has developed a computer program entitled "CRITSTRM" for performing these computations. Access is available upon request at the following web address:

<http://www.virginiadot.org/business/locdes/notification.asp>

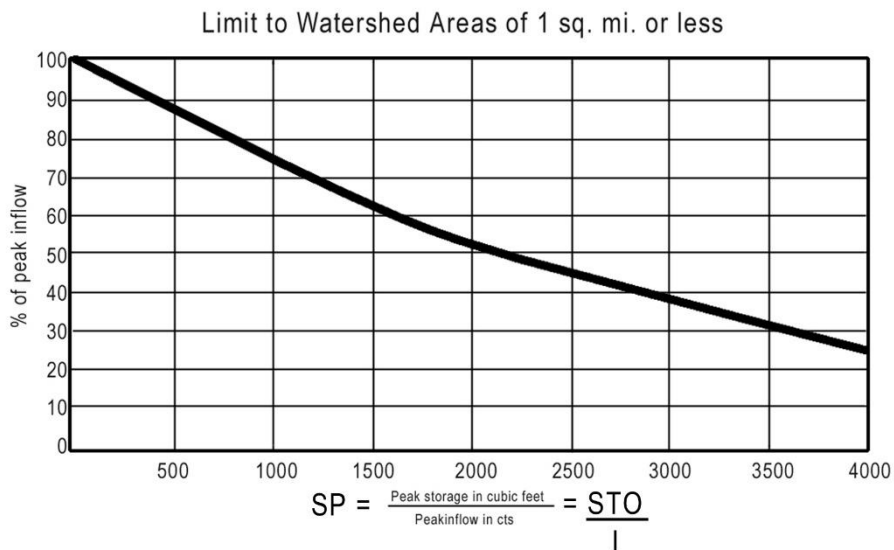


### 11.5.7.1.3 Pagan Volume Estimation Method

This method is appropriate for use with small basins serving watersheds of 200 acres or less. For this method, data from many small basins was compiled and the curve in Figure 11-11 was developed. This curve is used to determine the storage volume for a given drainage area by dividing the pre-development peak inflow by the post-development peak inflow.

Knowing the percentage of peak inflow, the storage parameter (peak storage in cubic feet over peak inflow in cubic feet per second) can be found by moving horizontally over the y-axis to the curve and down to the x-axis.

By multiplying the storage parameter by the peak inflow, the approximate peak storage can be found. This method should be used only as a first trial. Experience has shown that this method is conservative.



**Figure 11-2. Pagan Method Curve**

*Step 1. Determine pre- and post-development peak discharges.*

*Step 2. Determine the Storage Parameter (SP).*

SP is determined from Figure 11-11 drawing a line from the percentage of peak inflow ( $Q_o/Q_i$ ) to the line and reading the factor along the base of the figure.

*Step 3. Compute the Maximum Storage Volume (STO):*

$$STO = SP(I)$$

### 11.5.7.1.4 Sample Problems – Using 3 Methods to Estimate Volume of Storage for Quantity Control

Given the following information, estimate the volume of storage required for water quantity control:

Condition	Rational Method			Q
	DA	C	T <sub>c</sub>	
Pre-developed	25 ac	0.38	52 min	24 cfs
Post-developed	25 ac	0.59	21 min	65 cfs

*Method 1: Modified Triangular Hydrograph Method*

Based on the triangular hydrograph method, solve for V<sub>s</sub> as follows:

$$V_s = \frac{1}{2} T_b(Q_i - Q_o)$$

Where:

V<sub>s</sub> = Storage volume estimate, ft<sup>3</sup>

Q<sub>i</sub> = 65 cfs

Q<sub>o</sub> = 24 cfs

T<sub>b</sub> = 2 x T<sub>c</sub> (post-development) = 2 x 21 min = 42 min = 2,520 sec

$$V_s = \frac{1}{2} (2,520)(65 - 24) = 51,660 \text{ ft}^3$$

*Method 2: Critical Storm Duration Method*

Based on the critical storm duration method, determine the critical storm duration T<sub>d</sub> as follows:

a = 189.2

b = 22.1

C = 0.59 (post-development)

A = 25 acres

t<sub>c</sub> = 21 min (post-development)

q<sub>o</sub> = 24 cfs (Allowable outflow based on pre-development)

$$T_d = \sqrt{\frac{2CAa \left(b - \frac{t_c}{4}\right)}{q_o}} - b$$

$$T_d = \sqrt{\frac{2(0.59)(25)(189.2) \left(22.1 - \frac{21}{4}\right)}{24}} - 22.1$$

$$T_d = 40.5 \text{ min}$$

Solve for the critical storm duration intensity (I):

$$I = \frac{a}{b + 40.5} = \frac{189.2}{22.1 + 40.5} = 3.02 \text{ in/hr}$$

Determine the peak inflow (Q) using the Rational Method equation and the critical storm duration intensity (I):

$$Q = C_f C_i A = 1.0(0.59)(3.02)(25) = 44.5 \text{ cfs}$$

Determine the required storage volume (V) for the critical storm duration ( $T_d$ ):

$$V = \left[ Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3q_o t_c}{4} \right] 60$$
$$V = \left[ 44.5(40.5) + \frac{44.5(21)}{4} - \frac{24(40.5)}{2} - \frac{3(24)(21)}{4} \right] 60$$
$$V = 70,313 \text{ ft}^3$$

*Method 3: Pagan Method*

Based on the Pagan Method, solve for the estimated storage volume as follows:

$$\frac{Q_o}{Q_i} = \frac{24}{65} = 0.37 \times 100\% = 37\%$$

Determine the value for SP from Figure 11-5:

$$SP = 3,100 \text{ sec}$$

Use the relationship between STO, SP, and I to calculate STO:

$$STO = SP(I) = 3,100 (65) = 201,500 \text{ ft}^3$$

A comparison of the results for the 3 methods applied to the sample problem:

Method	Estimated Storage Volume, V (ft <sup>3</sup> )
Triangular Hydrograph	51,660
Critical Storm Duration	70,313
Pagan Method	201,500

Note that the Pagan Method estimates a much higher detention volume than either the Triangular Hydrograph or Critical Storm Duration methods.

### 11.5.8 Preliminary Basin Sizing

Based upon the estimated storage volume requirements calculated by the three methods above, determine the preliminary size of the basin. Assume the basin will have a rectangular shaped base, 2:1 length to width ratio, and an optimum depth of 4 feet. The basin will have 3:1 side slopes, but for the first size estimate, size the basin assuming vertical sides for a first estimate.

*Method 1: Simplified Triangular Hydrograph Method*

Calculate the footprint assuming a 4-ft depth:

$$\frac{51,660}{4} = 12,915 \text{ ft}^2$$

Assuming a rectangular shape with 2:1 length to width ratio:

$$(L) \times (W) = 12,915 \text{ ft}^2$$

$$L = 2(W)$$

$$2(W) \times 1(W) = 2(W^2) = 12,915 \text{ ft}^2$$

$$W = \sqrt{\frac{12,915 \text{ ft}^2}{2}} = 80 \text{ ft}$$

$$L = 2(W) = 2(80) = 160 \text{ ft}$$

Check the volume using the dimensions calculated:

$$V = L \times W \times D = 160 \times 80 \times 4 = 51,200 \text{ ft}^3 > 51,660 \text{ ft}^3 \checkmark$$

*Method 2: Critical Storm Duration Method*

Calculate the footprint assuming a 4-ft depth:

$$\frac{70,313}{4} = 17,578 \text{ ft}^2$$

Assuming a rectangular shape with 2:1 length to width ratio:

$$(L) \times (W) = 17,578 \text{ ft}^2$$

$$LL = 2(WW)$$

$$2(W) \times 1(W) = 2(W^2) = 17,578 \text{ ft}^2$$

$$W = \sqrt{\frac{17,578 \text{ ft}^2}{2}} = 94 \text{ ft}$$

$$L = 2(W) = 2(94) = 188 \text{ ft}$$

Check the volume using the dimensions calculated:

$$V = L \times W \times D = 188 \times 94 \times 4 = 70,688 \text{ ft}^3 > 70,313 \text{ ft}^3 \checkmark$$

*Method 3: Pagan Method*

Calculate the footprint assuming a 4-ft depth:

$$\frac{201,500}{4} = 50,375 \text{ ft}^2 \text{ft}^2$$

Assuming a rectangular shape with 2:1 length to width ratio:

$$(L) \times (W) = 50,375 \text{ ft}^2 (L) \times (W) = 50,375 \text{ ft}^2$$

$$L = 2(W)$$

$$2(W)(W) \times 1(W)(W) = 2(W^2W^2) = 50,375 \text{ ft}^2 \text{ft}^2$$

$$W = \sqrt{\frac{50,375 \text{ ft}^2}{2}} W = \sqrt{\frac{50,375 \text{ ft}^2}{2}} = 159 \text{ ftft}$$

$$L = 2(W) = 2(159) = 318 \text{ ft}$$

Check the volume using the dimensions calculated:

$$V = L \times W \times DV = L \times W \times D = 318 \times 159 \times 4 = 202,248 \text{ ft}^3 \text{ft}^3 > 201,500 \text{ ft}^3 \text{ft}^3 \checkmark$$

A comparison of the results for the 3 methods applied to the sample problem:

Method	Estimated Dimensions (ft)		
	Length	Width	Depth
Triangular Hydrograph	160	80	4
Critical Storm Duration	188	94	4
Pagan Method	318	159	4

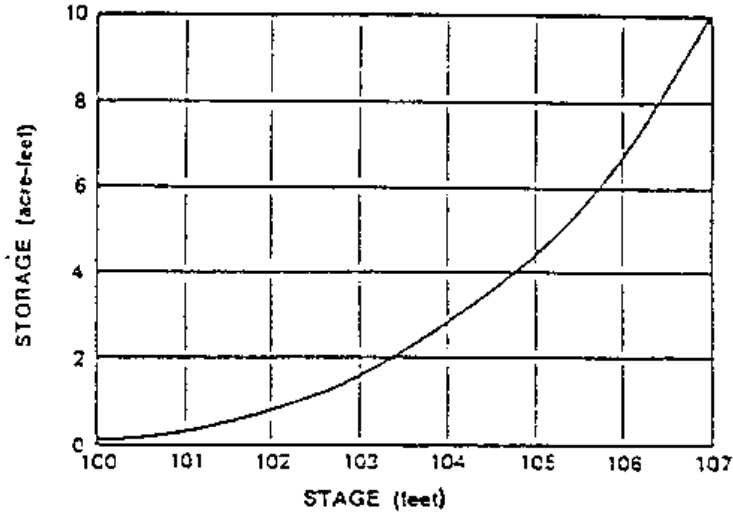
Note the differences in the results for the 3 methods. The only way to confirm the actual storage required to detain to allowable peak flow rates is to design the basin using the estimated storage and route the storms to confirm that detention is achieved. The design for the basin should be optimized to reflect the dimensions that provide sufficient storage without oversizing and driving up the cost to construct and maintain the facility.

## 11.5.9 Final Basin Sizing – Reservoir Routing

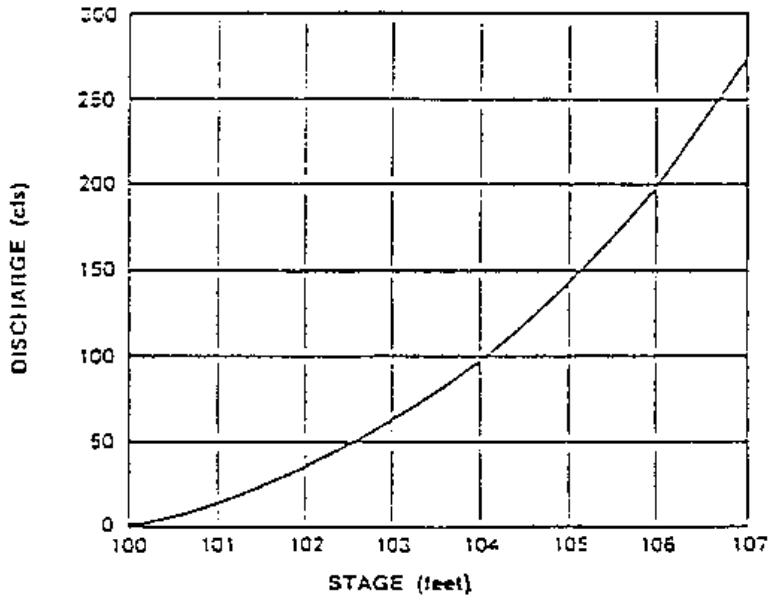
### 11.5.9.1 Storage – Indication Method Routing Procedure

The following procedure presents the basic principles of performing routing through a reservoir or storage facility (Puls Method of storage routing). Routing is most often completed with computer software, which develops the stage-discharge and stage-storage curves within the program.

*Step 1. Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility. Example stage-storage and stage-discharge curves are shown in Figure 11-12 and Figure 11-13 respectively.*



**Figure 11-3. Stage-Storage Curve**



**Figure 11-4. Stage-Discharge Curve**

- Step 2. Select a routing time period ( $\Delta t$ ) to provide at least five points on the rising limb of the inflow hydrograph. Use  $t_p$  divided by 5 to 10 for  $\Delta t$ .*
- Step 3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of  $S \pm \frac{0}{2} \Delta T$  versus stage. An example tabulation of storage characteristics curve data is shown in Table 11-2.*

**Table 11-2. Storage Characteristics**

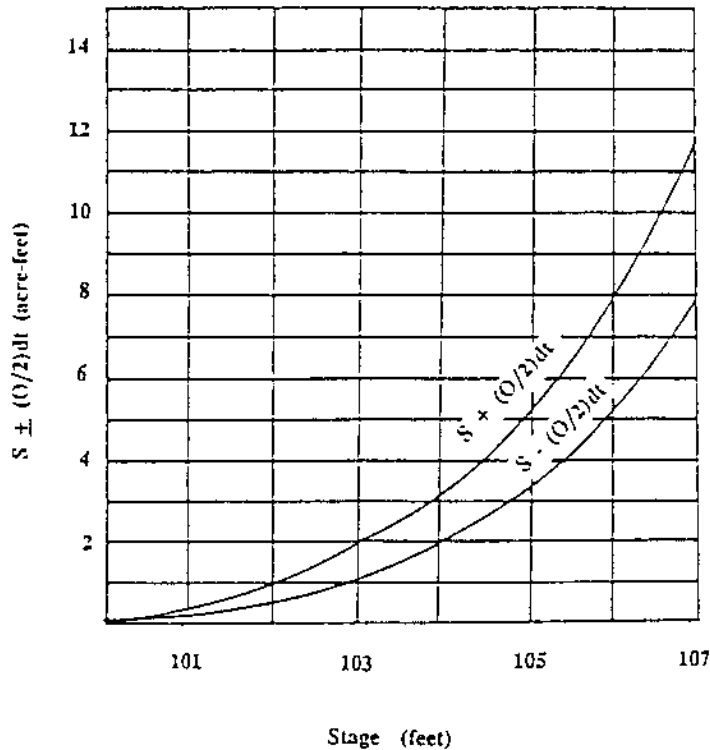
(1) Stage (H) (ft.)	(2) Storage <sup>1</sup> (S) (ac-ft)	(3) Discharge <sup>2</sup> (Q) (cfs)	(4) Discharge <sup>2</sup> (Q) (ac-ft/hr)	(5) $S - \frac{O}{2} \Delta T$ (ac-ft)	(6) $S + \frac{O}{2} \Delta T$ (ac-ft)
100	0.05	0	0	0.05	0.05
101	0.05	15	1.24	0.20	0.40
102	0.05	35	2.89	0.56	1.04
103	1.6	63	5.21	1.17	2.03
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.82	3.41	5.39
106	6.6	200	16.53	5.22	7.98

<sup>1</sup> Obtained from the Stage-Storage Curve.

<sup>2</sup> Obtained from the Stage-Discharge Curve.

Note:  $t = 10$  minutes = 0.167 hours and 1 cfs = 0.0826 ac-ft/hr.

Step 4. For a given time interval,  $I_1$  and  $I_2$  are known. Given the depth of storage or stage ( $H_1$ ) at the beginning of that time interval,  $S_1 - \frac{O_1}{2} \Delta T$  can be determined from the appropriate storage characteristics curve, Figure 11-14.



**Figure 11-5. Storage Characteristics Curve**



Step 5. Determine the value of  $S_2 + \frac{O_2}{2} \Delta T$  from the following equation:

**Equation 11.6**

$$S_2 + \frac{O_2}{2} \Delta T = S_1 - \frac{O_1}{2} \Delta T + \frac{I_1 + I_2}{2} \Delta T \quad (11.6)$$

Where:

$S_2$  = Storage volume at time 2, ft<sup>3</sup>

$O_2$  = Outflow rate at time 2, cfs.

$\Delta T$  = Routing time period, sec

$S_1$  = Storage volume at time 1, ft<sup>3</sup>

$O_1$  = Outflow rate at time 1, cfs

$I_1$  = Inflow rate at time 1, cfs

$I_2$  = Inflow rate at time 2, cfs

Other consistent units are equally appropriate.

Step 6. Enter the storage characteristics curve at the calculated value of  $S_2 + \frac{O_2}{2} \Delta T$  determined in Step 5 and read off a new depth of water ( $H_2$ ).

Step 7. Determine the value of  $O_2$ , which corresponds to a stage of  $H_2$  determined in Step 6, using the stage-discharge curve.

Step 8. Repeat Steps 1 through 7 by setting new values of  $I_1$ ,  $O_1$ ,  $S_1$ , and  $H_1$  equal to the previous  $I_2$ ,  $O_2$ ,  $S_2$ , and  $H_2$ , and using a new  $I_2$  value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

**11.5.9.2 Storage – Indication Method Routing Sample Problem #1**

This example demonstrates the application of the methodology presented for the design of a typical detention storage facility used for water quantity control.

Storage facilities shall be designed for allowable runoff from the applicable design storms. The analysis should also include the 100-yr design storm runoff to ensure that the structure can accommodate runoff from this storm without damaging adjacent and downstream property and structures due to flooding or overtopping the dam and causing it to fail.

For this sample problem, assume the receiving system is manmade, the energy balance is not being used for compliance, and there is no mapped floodplain or flood prone area immediately downstream. The peak discharges from the 2- and 10-yr 24-hour design storms are as follows:

- Allowable 2-yr 24-hour peak discharge = 150 cfs
- Allowable 10-yr 24-hour peak discharge = 200 cfs
- Post-development 2-yr 24-hour peak discharge = 190 cfs
- Post-development 10-yr 24-hour peak discharge = 250 cfs

Since the post-development peak discharge must not exceed the allowable peak discharge for channel and flood protection, the allowable design discharges are 150 cfs and 200 cfs for the 2- and 10-yr 24-hour design storms, respectively.

*Step 1. Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility.*

Runoff hydrographs are shown in Table 11-12 below. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 hours, respectively, for runoff from the 2- and 10-yr 24-hour storms.

**Table 11-3. Runoff Hydrographs**

(1)	Pre-Development Runoff		Post-Development Runoff	
	(2)	(3)	(4)	(5)
Time (hrs)	2-yr (cfs)	10-yr (cfs)	2-yr (cfs)	10-yr (cfs)
0	0	0	0	0
0.1	18	24	38	50
0.2	61	81	125	178
0.3	127	170	190 >150	250 >200
0.4	150	200	125	165
0.5	112	150	70	90
0.6	71	95	39	50
0.7	45	61	22	29
0.8	30	40	12	16
0.9	21	28	7	9
1.0	13	18	4	5
1.1	10	15	2	3
1.2	8	13	0	1

Preliminary estimates of required storage volumes are obtained using the simplified triangular hydrograph method outlined in Section 11.5.4.1. For runoff from the 2- and 10-yr storms, the required storage volumes,  $V_s$ , are computed using Equation 11.3:

$$V_s = \frac{1}{2} T_b (Q_i - Q_o)$$

$$V_{s2} = \frac{\frac{1}{2}(1.2)(3600)(190-150)}{43,560} = 1.98 \text{ ac. ft.}$$

$$V_{s10} = \frac{\frac{1}{2}(1.25)(3600)(250-200)}{43,560} = 2.58 \text{ ac. ft.}$$

Stage-discharge and stage-storage characteristics of a storage facility that should provide adequate peak flow attenuation for runoff from both the 2- and 10-yr design storms are presented below in Table 11-13. The storage-discharge relationship was developed and required that the preliminary storage volume estimates of runoff for both the 2- and 10-yr design storms to coincide with the occurrence of the corresponding allowable peak discharges.

Discharge values were computed by solving the broad-crested weir equation for head (H) assuming a constant discharge coefficient of 3.1, a weir length of 4', and no tailwater submergence. The capacity of storage relief structures was assumed to be negligible.

*Step 2. Select a routing time period ( $\Delta t$ ) to provide at least five points on the rising limb of the inflow hydrograph. Use  $t_p$  divided by 5 to 10 for  $\Delta t$ .*

$$\Delta T = \frac{t_p}{5} = \frac{0.5}{5} = 0.1 \text{ hr}$$

*Step 3. Use the storage-discharge data from Step 1 to develop storage characteristics curves (Stage-Discharge-Storage) that provide values of  $S + \frac{S}{2}\Delta T$  versus stage.*

**Table 11-4. Stage-Discharge-Storage Data**

(1)	(2)	(3)	(4)	(5)
Stage (H) (ft)	Discharge (Q) (cfs)	Storage (S) (ac-ft)	$S - \frac{O}{2} \Delta T$ (ac-ft)	$S + \frac{O}{2} \Delta T$ (ac-ft)
0.0	0	0.00	0.00	0.00
0.9	10	0.26	0.30	0.22
1.4	20	0.42	0.50	0.33
1.8	30	0.56	0.68	0.43
2.2	40	0.69	0.85	0.52
2.5	50	0.81	1.02	0.60
2.9	60	0.93	1.18	0.68
3.2	70	1.05	1.34	0.76
3.5	80	1.17	1.50	0.84
3.7	90	1.28	1.66	0.92
4.0	100	1.40	1.81	0.99
4.5	120	1.63	2.13	1.14
4.8	130	1.75	2.29	1.21
5.0	140	1.87	2.44	1.29
5.3	150	1.98	2.60	1.36
5.5	160	2.10	2.76	1.44
5.7	170	2.22	2.92	1.52
6.0	180	2.34	3.08	1.60

Storage routing was conducted for runoff from both the 2- and 10-yr design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results are shown below for runoff from the 2- and 10-yr design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-yr design storms.

*Step 4. For a given time interval,  $I_1$  and  $I_2$  are known. Given the depth of storage or stage ( $H_1$ ) at the beginning of that time interval,  $S_1 - \frac{O_1}{2} \Delta T$  can be determined from the appropriate storage characteristics curve.*

*Step 5. Determine the value of  $S_2 + \frac{O_2}{2} \Delta T$  from the following equation:*

$$S_2 + \frac{O_2}{2} \Delta T = S_1 - \frac{O_1}{2} \Delta T + \frac{I_1 + I_2}{2} \Delta T \quad (11.7)$$

Summarized in

Table 11-15 and 11-6 for the 2-yr and 10-yr storms.

Step 6. Enter the storage characteristics curve at the calculated value of  $S_2 + \frac{O_2}{2} \Delta T$  determined in Step 5 and read off a new depth of water ( $H_2$ ).

Summarized in

Table 11-15 and 11-6 for the 2-yr and 10-yr storms.

Step 7. Determine the value of  $O_2$ , which corresponds to a stage of  $H_2$  determined in Step 6, using the stage-discharge curve.

Summarized in Tables 11-5 and 11-6 for the 2-yr and 10-yr storms.

Step 8. Repeat Steps 1 through 7 by setting new values of  $I_1$ ,  $O_1$ ,  $S_1$ , and  $H_1$  equal to the previous  $I_2$ ,  $O_2$ ,  $S_2$ , and  $H_2$ , and using a new  $I_2$  value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

Summarized in

Table 11-15 and 11-6 for the 2-yr and 10-yr design storms.

**Table 11-5. Storage Routing for the 2-yr Storm**

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (T) (hrs)	Inflow (I) (cfs)	$\frac{I_1+I_2}{2} \Delta T$ (ac-ft)	Stage (H <sub>1</sub> ) (ft)	$S_1 - \frac{O_1}{2} \Delta T$ (6)-(8) (ac-ft)	$S + \frac{O}{2} \Delta T$ (3)+(5) (ac-ft)	Stage (H) (ft)	Outflow (O) (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	38	0.16	0.00	0.00	0.16	0.43	3
0.2	125	0.67	0.43	0.10	0.77	2.03	36
0.3	190	1.30	2.03	0.50	1.80	4.00	99
0.4	125	1.30	4.00	0.99	2.29	4.80	130<150 OK
0.5	70	0.81	4.80	1.21	2.02	4.40	114
0.6	39	0.45	4.40	1.12	1.57	3.60	85
0.7	22	0.25	3.60	0.87	1.12	2.70	55
0.8	12	0.14	2.70	0.65	0.79	2.02	37
0.9	7	0.08	2.08	0.50	0.58	1.70	27
1.0	4	0.05	1.70	0.42	0.47	1.03	18
1.1	2	0.02	1.30	0.32	0.34	1.00	12
1.2	0	0.01	1.00	0.25	0.26	0.70	7
1.3	0	0.00	0.70	0.15	0.15	0.40	3

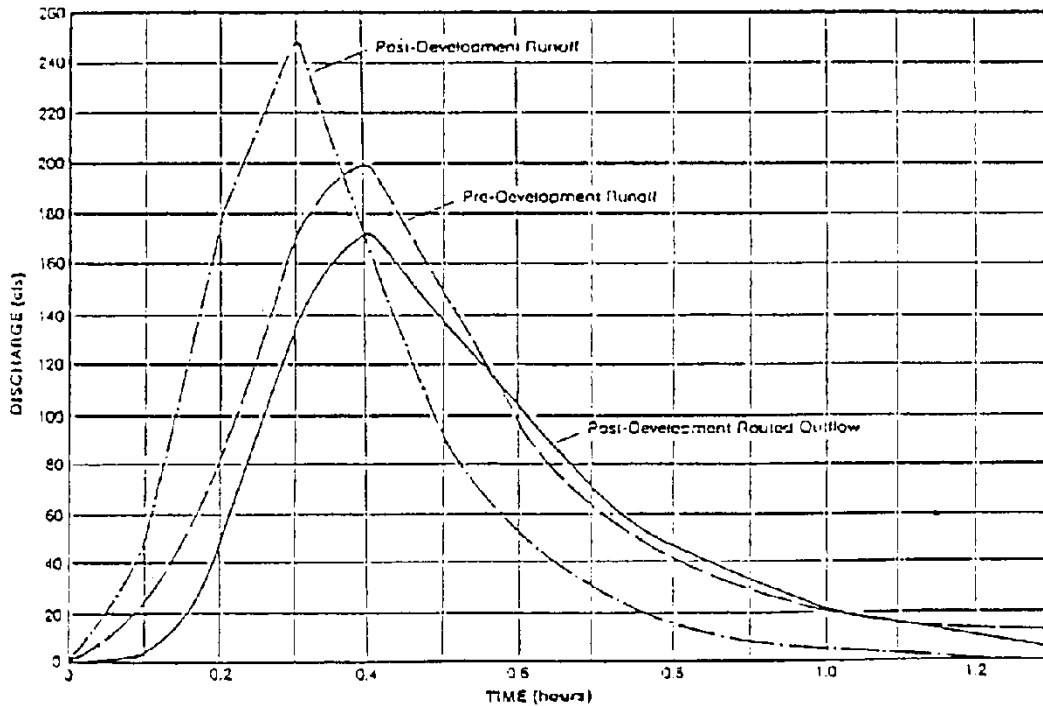
**Table 11-6. Storage Routing for the 10-yr Storm**

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (T) (hrs)	Inflow (I) (cfs)	$\frac{I_1+I_2}{2} \Delta T$ (ac-ft)	Stage (H <sub>1</sub> ) (ft)	$S_1 - \frac{O_1}{2} \Delta T$ (6)-(8) (ac-ft)	$S_2 + \frac{O_2}{2} \Delta T$ (3)+(5) (ac-ft)	Stage (H) (ft)	Outflow (O) (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	50	0.21	0.21	0.00	0.21	0.40	3
0.2	178	0.94	0.40	0.08	1.02	2.50	49
0.3	250	1.77	2.50	0.60	2.37	4.90	134
0.4	165	1.71	4.90	1.26	2.97	2.97	173<200 OK
0.5	90	1.05	5.80	1.30	2.35	4.00	137
0.6	50	0.58	4.95	1.25	1.83	4.10	103
0.7	29	0.33	4.10	1.00	1.33	3.10	68
0.8	16	0.19	3.10	0.75	0.94	2.40	46
0.9	9	0.10	2.40	0.59	0.69	1.90	32
1.0	5	0.06	1.90	0.44	0.50	1.40	21
1.1	3	0.03	1.40	0.33	0.36	1.20	16
1.2	1	0.02	1.20	0.28	0.30	0.90	11
1.3	0	0.00	0.90	0.22	0.22	0.60	6

Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storms, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations should be repeated.

Although not shown for this sample problem, runoff from the 100-yr frequency storm should be routed through the storage facility to establish freeboard requirements and to evaluate emergency overflow and stability requirements. In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth to water, side slope stability, maintenance, grading to prevent standing water, and provisions for public safety.

An estimate of the potential downstream effects (i.e., increased peak flow rate and recession time) of detention storage facilities may be obtained by comparing hydrograph recession limbs from the pre-development and routed post-development runoff hydrographs. Example comparisons are shown below.



**Figure 11-6. Runoff Hydrographs**

### 11.5.9.3 SWM Basin Design: Sample Problem

#### Step 1. Determine Stormwater Requirements:

- The receiving system is a manmade stormwater conveyance system that was determined not to be adequate for the uncontrolled post-development peak runoff, as the velocity and shear will exceed the allowable values for the channel materials and lining.
- In accordance with the water quantity flood protection criteria, the SWMF will need to attenuate the post-development  $Q_2$  such that the manmade system is not subject to erosion. The design of the dam and the emergency spillway will also need to provide protection of the dam for  $Q_{100}$ .

- The allowable peak discharge at which the channel is not expected to erode is  $Q_{2all} = 20.5$  cfs and the post development  $Q_{2post} = 29.6$  cfs (time of concentration,  $t_c = 0.333$  hr).
- A FEMA Zone A floodplain is mapped for the receiving stream immediately below the outfall.
- A VRRM Spreadsheet was developed for water quality and an Extended Detention Level 2 basin is required.

*Step 2. Determine the required Treatment Volume (Tv) and Design Treatment Volume:*

- An Extended Detention Level 2 design has been selected to meet the water quality criteria for the site.
- The Treatment Volume (Tv) for the drainage area contributing to the BMP was calculated in the VRRM Spreadsheet as 8,654 ft<sup>3</sup>.
- According to the VDOT BMP Design Manual of Practice, the total volume for an Extended Detention Level 2 design is 1.25 x Tv. With a Tv of 8,654 ft<sup>3</sup>, the total design volume is 1.25 x 8,654 ft<sup>3</sup> = 10,817 ft<sup>3</sup>.

*Step 3. Determine the size of the sediment forebay:*

- For an Extended Detention Level #2 design, a minimum of 40% of the Tv should be in a permanent pool, such as a forebay, micropool, deep pool, or wetlands. For the sample project, assume a forebay is selected for pretreatment and to meet the permanent pool requirements.
- Compute the sediment forebay volume and determine its dimensions:

$$V_{\text{forebay}} = \left( \frac{40\%}{100\%} \right) \times 10,817 \text{ ft}^3 = 4,327 \text{ ft}^3$$

If forebay is 4 ft deep, then the area of the forebay (assuming vertical walls) is calculated as:

$$\frac{4,327 \text{ ft}^3}{4 \text{ ft}} = 1,082 \text{ ft}^2$$

- The shape of the forebay does not need to be square and should be shaped to fit the site.



The established design parameters for the basin:

- An Extended Detention Level 2 with a 36-hour drawdown time is required for water quality.
- Quantity control for the  $Q_2$  is required for channel protection. The required volume will be estimated in the design process.
- Quantity control for the  $Q_{10}$  is not required for flood protection, as a floodplain is mapped immediately below the outfall.
- The required design treatment volume for water quality is 10,817 ft<sup>3</sup>.
- The estimated forebay volume is 4,327 ft<sup>3</sup>.

*Step 4. Determining the Water Quality Volume Elevation*

- Required treatment volume (for Extended Detention Level 2) = 10,817 ft<sup>3</sup>
- From the Stage-Discharge-Storage table:
  - The design  $T_v$  required satisfied @ Elev. 423.25
  - Water depth @ Elev. 423.25 = 1.95 ft
  - Actual Volume = 11,051 ft<sup>3</sup> @ Elev. 423.25

*Step 5. Determining the Extended Detention Orifice Size Required for Water Quality Using Method #2 Average Hydraulic Head (VDOT Preferred)*

- As 40% of the total treatment volume is contained in a permanent pool in the forebay, the remaining 60% must be detained for 36-hours. Calculate the remaining 60% volume for extended detention,  $V_{ED}$ :

$$V_{ED} = \left( \frac{60\%}{100\%} \right) \times 11,051 \text{ ft}^3 = 6,631 \text{ ft}^3$$

- Compute the  $Q_{avg}$  for the remaining volume ( $V_{ED}$ ) using the required 36-hour drawdown time:

$$Q_{avg} = \frac{T_v}{\text{Time}} = \frac{6,631 \text{ ft}^3}{36 \text{ hr (3600 sec/hr)}} = 0.051 \text{ cfs}$$

- While the storage depth for the total treatment volume is  $h = 1.95$  ft, the storage depth for 40% of the total treatment volume is at Elev. 422.05. This will be the invert for the extended detention orifice.

- Calculate the head for detaining 60% of the total storage volume for 36-hours:

$$h = 423.25 - 422.05 = 1.20 \text{ ft}$$

$$h_{\text{avg}} = \frac{1.2 \text{ ft}}{2} = 0.6 \text{ ft}$$

- Orifice sizing computations:

$$A = \frac{Q_{\text{avg}}}{C\sqrt{2gh_{\text{avg}}}} = \frac{0.051}{0.6\sqrt{2(32.2)(0.6)}} = 0.014 \text{ ft}^2$$

- The depth (h) used in the orifice equation would normally be measured from the center of the orifice. Due to the small size of the water quality orifice it is acceptable to consider h as the depth to the invert of the orifice.
- From Table 11-10, use a 1½-inch orifice with an area = 0.012 ft<sup>2</sup>. This is slightly smaller than calculated, so reservoir routing should be conducted confirm the allowable peak discharge and required extended detention time are achieved.

*Step 6. Determining the Storage Volume and Orifice Size Required for Channel Protection Using Method #2 Average Hydraulic Head (VDOT Preferred)*

The uncontrolled post-development peak discharge for the site in the 2-yr storm is 29.6 cfs, with a  $t_c$  of 0.333 hr.

The allowable 2-yr storm peak discharge for channel protection is 20.5 cfs.

*Step 6a. Sizing the Storage Volume for the Channel Control*

Use the Modified Triangular Hydrograph method to estimate the volume needed:

$$V_s = \frac{1}{2} T_b(Q_i - Q_o)$$

Where:

$V_s$  = Detention storage volume estimate, ft<sup>3</sup>

$Q_i$  =  $Q_{2\text{post}}$  = 29.6 cfs

$Q_o$  =  $Q_{2\text{all}}$  = 20.5 cfs

$T_b$  = 2 x  $t_c$  (post-development) = 2 x 0.333 hr = 0.666 hr = 2,398 sec

$$V_s = \frac{1}{2} (2,398)(29.6 - 20.5) = 10,911 \text{ ft}^3$$

- From the Stage-Discharge-Storage table:
  - The volume to Elev. 423.25 is reserved for water quality.
  - Determine the total volume required for water quality and channel protection:
 
$$V_{\text{total}} = 11,051 \text{ ft}^3 + 10,911 \text{ ft}^3 = 21,962 \text{ ft}^3$$
  - Based on the Stage-Storage table, the volume provided at Elev. 425.50 = 22,783 ft<sup>3</sup>, which is > 21,962 ft<sup>3</sup> required
  - The storage volume from Elev. 423.25 to 425.50 is for channel protection storage.
  - Water depth for channel protection storage = 425.50 – 432.25 = 2.25 ft

*Step 6b. Determining the Weir Size Required for Channel Protection Using Method #2 Average Hydraulic Head (VDOT Preferred)*

- Assume depth, h = 2.25 ft.

$$h_{\text{avg}} = \frac{2.25 \text{ ft}}{2} = 1.13 \text{ ft}$$

- Select a weir as the hydraulic control structure for channel protection. Weir sizing computation

Weir equation:

$$Q_{\text{avg}} = CL(h_{\text{avg}})^{1.5}$$

Where:

Q = Q<sub>2all</sub> = weir discharge, cfs

C = weir coefficient of discharge (use 3.0 for sharp-crested weir)

L = weir length, ft

h<sub>avg</sub> = average head, ft

Rearranged weir equation to solve for weir length:

$$L = \frac{Q_{2\text{all}}}{C(h_{\text{avg}})^{1.5}} = \frac{20.5}{3.0(1.13)^{1.5}} = 5.69 \text{ ft}$$

- Use a 5.7 ft long sharp crested weir for channel protection. Reservoir routing should confirm that the allowable peak discharge is not exceeded for the 2-yr storm.

- The invert for the weir should be at the Elev. 423.25 where the water quality storage volume ends and the channel protection storage volume begins.

Summary of basin design for water quality and water quantity:

- Water Quality
  - Extended Detention Level 2 ( $1.25 \times T_v$ )
  - Treatment volume provided at Elev. 423.25
  - 40% of total treatment volume in permanent pool in a forebay
  - 60% of total treatment volume in extended detention for a minimum of 36-hours
  - Extended Detention orifice 1½-inches diameter with invert at Elev. 422.05.
- Water Quantity
  - Channel Protection
    - Allowable peak discharge for 2-yr storm to manmade system is 20.5 cfs.
    - Detention volume achieved at Elev. 425.50
    - Control weir 5.7 ft long at Elev. 423.25
  - Flood Protection is not required as the system immediately below the outfall is a mapped FEMA floodplain.

*Step 7: Determining the Elevation and Sizing of an Auxiliary Spillway to Convey the 100-year Storm Using Method #2 Average Hydraulic Head (VDOT Preferred)*

- The post-development peak discharge for the 100-yr storm was calculated to be 237 cfs.
- The criterion for a facility with an auxiliary spillway is conveyance of the 100-year peak rate of runoff with a freeboard of 1 ft.
- To be conservative, the designer can assume that the principal spillway orifice (extended detention) and weir (channel protection) are blocked during the 100-yr event and size the auxiliary spillway to pass the full 100-yr storm. Routing the 100-yr event will help the designer optimize the auxiliary spillway design.

*Step 7a: Determine the Invert Elevation for the Auxiliary Spillway*

- The elevation for the water quality and water quantity storage volumes is estimated to be at Elev. 425.50
- The auxiliary spillway invert should be designed to have a minimum freeboard of 1.0 ft above the water quantity storage volume = Elev. 425.5 + 1.0 ft = Elev. 426.5

*Step 7b: Determine the Sizing for the Auxiliary Spillway Using Method #2 Average Hydraulic Head (VDOT Preferred)*

Auxiliary (sometimes called “emergency”) spillways are generally designed as weirs with a flume or channel lined with appropriate material to resist erosion. The weir section is the hydraulic control structure.

- The final embankment height (including freeboard) has not been computed, but there are often site constraints that drive it. For the preliminary design, assume the storage depth for conveying the 100-yr storm = 3.00 ft.

$$h_{\text{avg}} = \frac{3.00 \text{ ft}}{2} = 1.50 \text{ ft}$$

- Weir sizing computation

Weir equation:

$$Q_{\text{avg}} = CL(h_{\text{avg}})^{1.5}$$

Where:

$Q = Q_{100}$  = weir discharge, cfs

$C$  = weir coefficient of discharge (use 2.6 for broad crested weir)

$L$  = weir length, ft

$h_{\text{avg}}$  = average head, ft

Rearranged weir equation to solve for weir length:

$$L = \frac{Q_{100}}{C(h_{\text{avg}})^{1.5}} = \frac{237}{2.6(1.50)^{1.5}} = 49.6 \text{ ft}$$

- Use a 50 ft long broad crested weir to safely convey the 100-yr storm without overtopping the embankment.
- The final top elevation for the embankment should be set 1 ft above the peak water surface elevation for the routed 100-yr storm. Reservoir routing should be used to confirm that the 100-yr storm peak discharge is conveyed in the auxiliary spillway with a minimum of 1 ft of freeboard to the top of the embankment.
- Note that the flume or channel below the broad crested weir must be designed to adequately convey the 100-yr storm to the system without causing erosion of the embankment or flooding of property above the BMP. If the BMP is located within a mapped 100-yr floodplain, then the final design must not have an adverse effect on the mapped floodplain and base flood elevations (where present).

Summary of basin design for water quality and water quantity:

- Water Quality
  - Extended Detention Level 2 ( $1.25 \times Tv$ )
  - Treatment volume provided at Elev. 423.25
  - 40% of the total treatment volume ( $1.25 \times Tv$ ) is in a permanent pool in a forebay
  - 60% of total treatment volume ( $1.25 \times Tv$ ) is in extended detention for a minimum of 36-hours
  - Extended Detention orifice is 1½-inches diameter with invert at Elev. 422.05
  
- Water Quantity
  - Channel Protection
    - Allowable peak discharge for 2-yr storm to manmade system is 20.5 cfs
    - Detention volume achieved at Elev. 425.50
    - Control weir 5.7 ft long at Elev. 423.25
  - Flood Protection is not required as the system immediately below the outfall is a mapped FEMA floodplain
  
- 100-yr Storm Conveyance
  - An auxiliary spillway is proposed to convey the 100-yr storm event
  - The invert for the auxiliary spillway is set 1 ft above the SWM storage at Elev. 426.50
  - The auxiliary spillway design consists of a broad crested weir with a length of 50 ft
  - The peak water surface elevation for the 100-yr storm is Elev. 426.50 + 3.00 ft = Elev. 429.50
  - The low point in the embankment should be a minimum of 1 ft above the peak water surface elevation for the 100-yr storm to provide freeboard = Elev. 429.50 + 1.00 ft = Elev. 430.50

*Step 8: Route the Water Quality and Quantity Design Storms through the Basin*

- To confirm that the water quality and quantity control criteria are satisfied for the design, a reservoir routing should be conducted for the basin and spillways to confirm that maximum peak discharges, minimum detention times, and adequate freeboard is provided.
- Use the Puls Method or another acceptable level-pool routing method to route the water quality storm, the 2-yr storm, and the 100-yr storm through the basin. Note that the water quality volume for an Extended Detention Level 2 is 60% of  $(1.25 \times T_v)$ , as 40% of  $(1.25 \times T_v)$  is included in a permanent pool in the forebay.
- Use the results to confirm that:
  - The water quality storm is detained for a minimum of 36-hours for brim draw down.
  - The 2-yr storm peak runoff is less than or equal to the allowable peak discharge.
  - The maximum water surface elevation in the 100-yr storm is at least 1 ft below the invert of the embankment crest.
- Note that the initial design for a BMP/SWMF may not be the optimal design, providing too little or too much control, especially for extended detention and detention. This may not be apparent until the reservoir routing is complete. The designer should adjust the design to provide the control required for compliance with the Part II.B stormwater management criteria, making sure that the requirements are met. A designer may choose to apply some factor of safety based upon professional judgment and documented in the design, but an excessively large design will increase the cost to construct and maintain the BMP/SWMF and should be avoided.

## **11.6 Part II C Design Criteria**

### **11.6.1 Water Quality**

SWM design for water quality control is to be in accordance with the latest revisions to the Virginia Stormwater Management Regulations. The regulations state that the water quality volume (WQV) is equal to the first 0.5” of runoff multiplied by the area of impervious surface associated with the land development project.

The following comments represent the significant points of the current regulations (the page numbers referenced are those in the DEQ SWM Handbook):

- SWM requirements for water quality control are “Performance Based” (9VAC25-870-96). The type of BMP required is determined by the comparison of the pre-developed, post-developed, and average cover conditions (% impervious area) of the site or stormwater planning area to classify the project as Situation 1, 2, 3, or 4. Unless otherwise defined by a local Chesapeake Bay Preservation Act program, the average cover condition is assumed to be 16% impervious.
  - Situation 1 occurs when the site pre-developed and developed conditions both result in a % imperviousness area < the average cover condition. No additional water quality controls are required as the low density development is considered the best management practice.
  - Situation 2 occurs when the site pre-developed % impervious area is ≤ average cover condition, but the developed condition % impervious area is > average cover condition. In this situation, water quantity controls are provided to reduce the developed pollutant load to the pre-developed condition.
  - Situation 3 occurs when both the pre-developed and developed & impervious areas are > the average cover condition. In this case, controls are provided to reduce the developed pollutant loading to 10% below the pre-developed pollutant loading or to the pollutant loading associated with the average cover condition, whichever requires less pollutant removal.
  - Situation 4 occurs when the project site discharges to an existing stormwater Best Management Practice (BMP) and the existing BMP was designed to treat the developed project site.
- A BMP is selected from Table 11-1 below that provides the necessary removal rate to satisfy the Performance Based calculations and Situation applicable to the project.
- BMP requirements for water quantity control are determined by the ESC Regulation MS-19 (9VAC25-840-40) for adequate receiving channels.
- Extended Detention Basins and Enhanced Extended Detention Basins require 2 times the Water Quality Volume (WQV), or a total of 1” of runoff from the developed projected site within VDOT R/W or easement draining to the BMP.



- Extended Detention Basins and Enhanced Extended Detention Basins require a 30 hour drawdown time for the required WQV. If the required orifice size is found to be significantly less than 3", an alternative outlet design or alternative BMP should be investigated for use, such as a linear facility that treats the first flush and allows larger storms to bypass. The calculation procedure for drawdown time and orifice sizing is shown on in the Virginia SWM Handbook Volume II, Pages 5-33 through 5-38. Alternative outlet designs for Extended Detention and Enhanced Extended Detention are presented in the Virginia SWM Handbook Volume I, Figures 3.07-3a to 3.08-3c, Pages 3.07-8 to 3.07-10.
- Suggested details for the Extended Detention Basin are shown on Pages 3.07-4 and 5 (Virginia SWM Handbook). The riprap-lined low-flow channel through the basin is not recommended due to maintenance concerns.
- Suggested details for the Enhanced Extended Detention Basin are shown on Pages 3.07-6 and 7 (Virginia SWM Handbook). The geometric design may need to be more symmetrical than that shown in order to facilitate construction of the basin to the dimensions needed.

**Table 11-7. BMP Selection Table**

<b>Water Quality BMP</b>	<b>Treatment Volume</b>	<b>Target Phosphorus Removal Efficiency</b>
Vegetated filter strip		10%
Grassed swale		15%
Constructed wetlands	2xWQV	20%
Extended detention	2xWQV	35%
Retention basin I	3xWQV	40%
Bioretention basin	1xWQV	50%
Bioretention filter	1xWQV	50%
Extended detention enhanced	2xWQV	50%
Retention basin II	4xWQV	50%
Infiltration	1xWQV	50%
Bioretention basin	2xWQV	65%
Bioretention filter	2xWQV	65%
Sand filter	2xWQV	65%
Infiltration	2xWQV	65%
Retention basin III with aquatic bench	4xWQV	65%
Manufactured BMP Systems Hydrodynamic Structures *		20%
Manufactured BMP Systems Filtering Structures *		50%
Filtterra™ Bioretention Filter System **		74%

\*Innovative or alternate BMPs not included in this table may be allowed at the discretion of DEQ and VDOT.

\*\*See Technical Bulletin No.6 in the Virginia SWM Handbook.

### **11.6.2 Water Quantity**

The Virginia Erosion and Sediment Control Regulations Minimum Standard 19 (9VAC25-840-40 section 19) and Virginia Stormwater Management Program Regulations (9VAC25-870-97) shall govern water quantity control. Linear development projects shall not be required to control post-developed stormwater runoff for flooding, except in accordance with a watershed or regional SWM plan (9VAC25-870-98). The following general criteria apply:

- Determination of flooding and channel erosion impacts to receiving streams due to land-disturbing activities shall be measured at each point of discharge from the land disturbance and such determination shall include any runoff from the balance of the watershed that also contributes to that point of discharge.
- The specified design storms shall be defined as either a 24-hour storm using the rainfall distribution recommended by the U.S. Department of Agriculture's Natural Resources Conservation Service (NRCS) when using NRCS methods or as the storm of critical duration that produces the greatest required storage volume at the site when using a design method such as the Modified Rational Method.

- For purposes of computing runoff, all pervious lands in the site shall be assumed prior to development to be in good condition (if the lands are pastures, lawns, or parks), with good cover (if the lands are woods), or with conservation treatment (if the lands are cultivated); regardless of conditions existing at the time of computation.
- Construction of SWMFs or modifications to channels shall comply with all applicable laws, regulations, and ordinances. Evidence of approval of all necessary permits shall be presented.
- Pre-development and post-development runoff rates shall be verified by calculations that are consistent with good engineering practices.
- Outflows from a SWMF or stormwater conveyance system shall be discharged to an adequate channel.
- Hydrologic parameters shall reflect the ultimate land disturbance and shall be used in all engineering calculations.
- Natural channel characteristics shall be preserved to the maximum extent practicable.
- Pre-development conditions should be that which exist at the time the road plans are approved for R/W acquisition.
- An adequate receiving channel is required for stormwater outflows from all projects with more than 10,000 ft<sup>2</sup> of land disturbance.
- The receiving channel at a pipe or storm drain outlet should be analyzed by use of a 2-yr storm for natural channel capacity and erosion protection; while the 10-yr storm shall be used for man-made channel capacity, with the 2-yr storm for man-made channel erosion protection.

### **11.6.3 Compensatory Treatment**

Compensatory treatment for water quality requirements (over treating at one outfall in a local watershed to compensate for not treating at an adjacent outfall in the same watershed) can be considered for meeting the requirements provided:

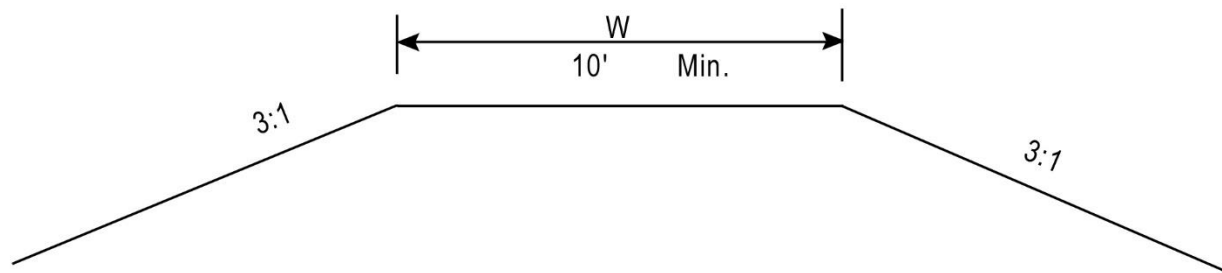
- The SWMFs at the treated outfall are designed to account for the water quality volumes for those areas where SWMFs are determined to be impractical or unacceptable.
- The downstream impacts, if any, which would occur as a result of discharging untreated runoff at the untreated outfall, must be documented. The documentation should note that compensating treatment of SWMFs has been incorporated.
- The channel at the untreated outfall must be analyzed to determine its adequacy to convey the additional runoff in accordance with the requirements of MS-19 of the VESCR and any necessary channel protection or improvements must be provided.
- The project is to be reviewed either by the State Hydraulics Engineer or his assistant when the project reaches the Field Inspection stage.

#### 11.6.4 Embankment (Dam)

The following details are to be incorporated into the design of dams for VDOT SWM basins.

- The design of the dam and the basin should provide only a relatively shallow depth of ponded water in order to prevent the basin from being a hazard. It is desirable to have the ponded depth no more that about 2' for water quality and about 4' for the 10-yr storm ( $Q_{10}$ ) quantity control.
- Foundation data for the base of the dam should be secured from the Materials Division for all SWM basins in order to determine if the native material will support the dam and not allow ponded water to seep under the dam. An additional boring near the center of the basin should also be requested if:
  - Excavation from the basin may, potentially, be used to construct the dam, or
  - There is potential for rock to be encountered in the area of excavation, or
  - A high water table is suspected that may alter the performance of the SWM basin.
- For large basins, more than one boring for the dam and one boring for the area of the basin shall be needed. The number and locations of the borings are to be determined by the VDOT SWM Plan Designer/Hydraulics Engineer and/or the VDOT District Materials Engineer.
- The foundation material under the dam and the material used for the embankment of the dam should be an ML or CL Type in accordance with the Unified Soil Classification System (ASTM D2487) (type A-4 or finer in accordance with the AASHTO Classification System M145) and/or meet the approval of the Materials Division. If the native material is not adequate, the foundation of the dam is to be undercut a minimum of 4' or to a depth recommended by the Materials Division. The backfill and embankment material must meet the above soil classification, or the design of the dam may incorporate a trench lined with a membrane such as bentonite penetrated fabric, HDPE, or LDPE liner, to be approved by the Materials Division.
- The pipe culvert under or through the dam is to be concrete pipe with rubber gaskets, and the joint must be leak-resistant as per AASHTO PP-63, and shall be included in the Department's Approved List No. 14.
- The foundation data for the SWM basin should be requested by the VDOT SWM Plan Designer/Hydraulics Engineer at the same time that the request for culvert foundation data is initiated.
- A concrete cradle is to be used under the pipe to prevent seepage through the dam barrel. The concrete cradle extends from the riser or inlet end of the pipe to the outlet of the pipe, and extend the full length of the pipe. For details of the concrete cradle, see Std. SWM-DR of the 2016 VDOT Road & Bridge Standards."

- If the height of the dam is greater than 15', or if the basin includes a permanent water pool, the design of the dam is to include a homogenous embankment with seepage controls or zoned embankment or similar design conforming to DEQ design standards for earth dams and is to be approved by the Materials Division.
- The minimum top width should be 10'. This helps facilitate both construction and maintenance and allows the embankment to be used for access. The side slopes should also be a minimum of 3:1, to permit mowing and maintenance access. The design of the dam should include a seepage analysis as well as a slope stability analysis (minimum Factor of Safety of 1.5 (checked with normal pool level on storage side and rapid drawdown conditions). A typical cross-section of a SWM basin dam is shown in Figure 11-7.



**Figure 11-7. Typical SWM Basin Dam**

- Construction of SWM impoundment structures within a FEMA designated 100-yr flood plain shall be avoided whenever possible. When this is unavoidable, a thorough review shall be made to ensure that the SWMF will operate effectively for its intended purpose during the passage of the 10-yr flood event on the flood plain. All SWMF construction within a designated 100-yr flood plain shall be in compliance with all applicable regulations under the FEMA's National Flood Insurance Program. The SWMF shall be reviewed for any potential impacts to the 100-yr flood event characteristics of the floodplain and designed for structural stability during the passage of the 100-yr flood event on the flood plain.
- Impounding structures (dams) that are not covered by the Virginia Dam Safety Regulations shall be designed in accordance with this manual and reviewed for floodplain impacts during the passage of the 100-yr storm event.

### **11.6.5 Basin Grading**

The layout and grading of a basin has a major influence on how effective the basin will be in removing pollutants. The designer should try to blend the basin into the surrounding topography while keeping several criteria in mind. First, the basin should be designed and graded so that the desirable length-to-width ratio is about 3:1 with a minimum ratio

of 2:1. This helps prevent short-circuiting of the basin's storage areas. The basin's longest dimension should run parallel to the contours, which helps minimize cut and fill. The wider dimension should also be located at the outlet end. If the length to width ratio is less than about 2:1, and there is concern that the velocity of flow through the basin is high, the designer should consider using baffles within the basin to reduce velocity and prevent short-circuiting by increasing travel length. Baffles should be constructed of a pervious type material such as snow fence, rather than earth berms, which do not reduce the velocity.

- Basin side slopes should be no steeper than 3:1 to permit mowing and maintenance access
- The bottom slope of dry detention basins should be no more than 2% and no less than 0.5%
- Where safety is a concern, and fencing is not practical, use 4:1 side slope
- The depth of basin from the bottom to the primary outflow point (crest of riser, or invert of weir) should be no more than 3' if possible, in order to reduce the hazard potential. If the depth needs to be more than 3', fencing should be considered and a safety ledge considered around the perimeter to prevent people from falling in, and to facilitate their escape from the basin.
- Construction of SWMFs within a sinkhole is prohibited. If SWMFs are required along the periphery of a sinkhole, the design of such facilities shall comply with the guidelines in Chapter 5 of this manual and the DEQ's Technical Bulletin No. 2 (Hydrologic Modeling and Design in Karst) and applicable sections of the Virginia SWM Handbook.
- Design of any SWMFs with permanent water features (proposed or potential) located within five (5) miles of a public use or military airport is to be reviewed and coordinated in accordance with Section A-6 of the VDOT Road Design Manual.

Table 11-8 summarizes the design criteria for dry and wet basin designs:

**Table 11-8. Summary of Design Criteria for Dry and Wet Basins**

<b>Design Requirement</b>	<b>Dry Basin Design</b>	<b>Wet Basin Design</b>
Quality control	Detain WQV for 30-hour minimum	Permanent pool volume is a function of the BMP selected (see Table 11-7)
Quantity control	Control 2- and 10-yr (when applicable) peak flows and maintain a non-erosive outfall velocity	Control 2- and 10-yr (when applicable) peak flows and maintain a non-erosive outfall velocity
Shape	3:1 length-to-width ratio; wedge shaped (wider at the outlet)	3:1 length-to-width ratio*; wedge shaped (wider at the outlet); permanent pool depth to 3' max, if possible
Safety		Fence around basin if depth is greater than 3'; shallow safety ledge around basin. See following notes on fencing. (Section 11.3.8)
Other Considerations	3:1 side slopes for easy maintenance access; 0.5-2% bottom slope to prevent ponding; sediment forebay to reduce maintenance requirements	3:1 side slopes for easy maintenance access; sediment forebay to reduce maintenance requirements; provide valve to drain pond for maintenance

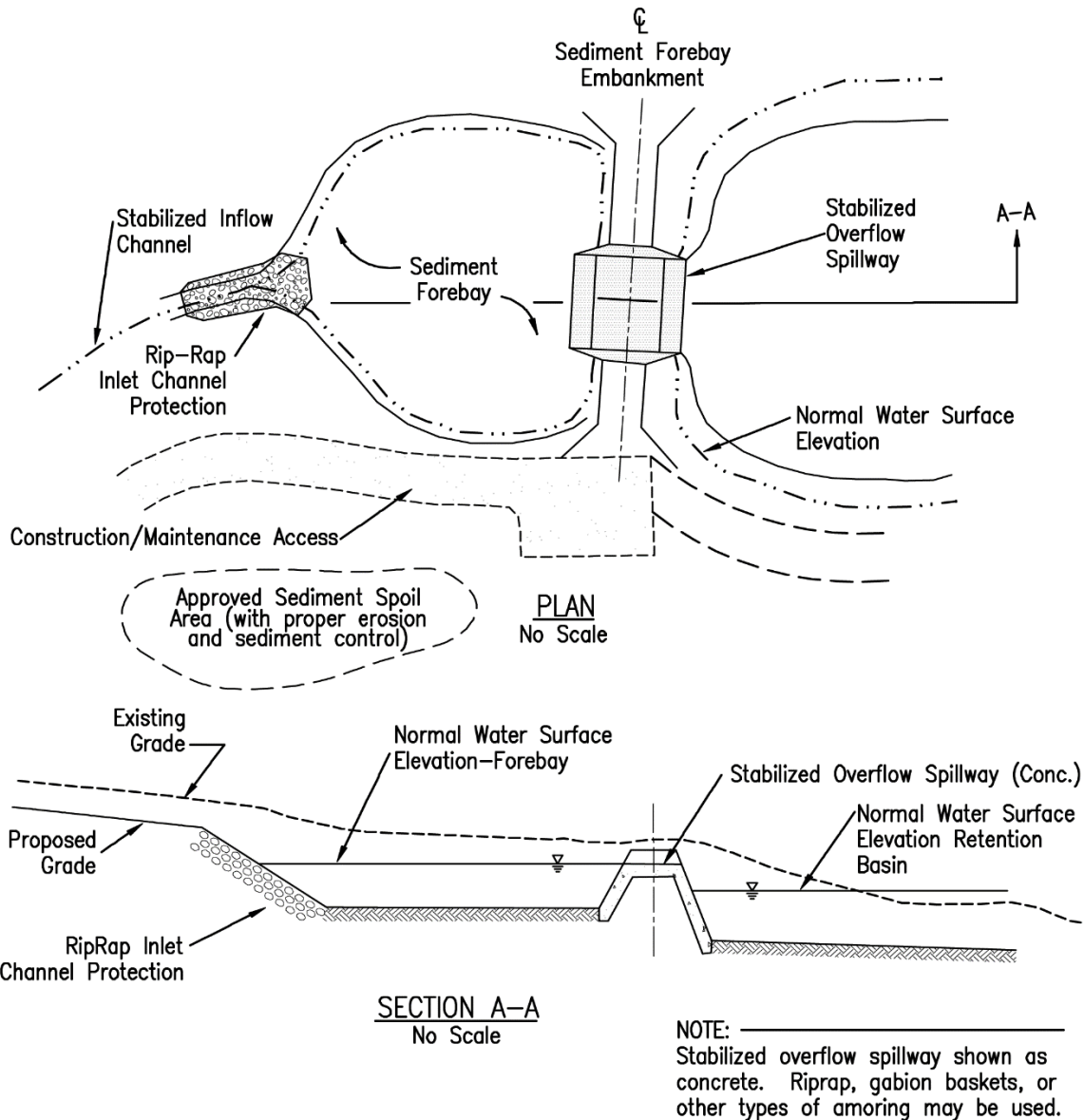
\* If this is not possible, every effort should be made to design the basin with no less than a 2:1 length to width ratio.

### **11.6.6 Sediment Forebay**

A sediment forebay is a settling basin or plunge pool constructed at the incoming discharge points of a stormwater BMP. The purpose of a sediment forebay is to allow sediment to settle from the incoming stormwater runoff before it is delivered to the balance of the BMP. It is an essential component of most impoundment and infiltration BMPs including retention, detention, extended-detention, constructed wetlands, and infiltration basins. A sediment forebay also helps to isolate the sediment deposition in an accessible area, which facilitates BMP maintenance efforts.

A sediment forebay shall be located at each inflow point in the stormwater BMP. Storm drain piping or other conveyances may be aligned to discharge into one forebay or several, as appropriate for the particular site. Sediment forebays should always be installed in a location that is accessible by maintenance equipment. Figure 11-8 shows a typical sediment forebay.

In order to facilitate maintenance activities, sediment forebays are to be incorporated into the design of Extended Detention Basins and Extended Detention Basins Enhanced. The volume of the forebay should be 0.1" – 0.25" x the impervious area treated by the facility or 10% of the required detention volume. See Pages 3.04-1 through 5 (SWM Handbook) for details. Where the overflow (emergency) spillway is incorporated as part of the dam/embankment, it shall be stabilized utilizing rip rap, concrete, or other non-erodible material (such as EC-3).



\3\_04-1

**Figure 11-8. Typical Sediment Forebay Plan and Section**



## **11.7 Part II C Design Concepts**

### **11.7.1 Water Quality**

Control of stormwater quality offers the following potential benefits:

- Control of sediment deposition
- Improved water quality through stormwater filtration
- Settling out of roadway runoff pollutants

Ideally, the pollutant removal mechanism should dictate the treatment volume or storm frequency for water quality BMPs. The sizing of BMPs, which uses gravitational settling of pollutants as the removal mechanism, can be based on a volume of runoff. The Virginia Stormwater Management Regulations require that the first flush of runoff be captured and treated to remove pollutants. The first flush, or water quality volume (WQV) is generally defined as the first one-half inch of runoff from impervious surfaces. Table 11-1 specifies the required treatment volume for each type of BMP based upon the WQV.

One of the first considerations in selecting a stormwater BMP is the functional goal of the BMP. The main components of SWM are: quality, stream channel erosion, and stormwater quantity or flooding. Any one or a combination of these components will dictate the functional goal of the BMP. In general, stormwater BMPs can be categorized into water quality BMPs and water quantity (stream channel erosion and flooding) BMPs.

Table 11-9 provides a general categorization of BMPs by functional goal. Note that some BMPs can be designed to satisfy both quality and quantity goals while others are specifically suited for only one.

The use of some BMPs is limited by site or watershed feasibility factors such as environmental impacts, drainage area or watershed size, and topographic constraints.

The BMPs designed for water quality control provide varying levels of pollutant removal and are suitable for specific development densities. Table 11-1 also provides a generic list of water quality BMPs and their target phosphorus removal efficiency. Phosphorus is the keystone pollutant targeted for removal in Virginia.

**Table 11-9. Functional Goals of Stormwater BMPs**

<b>Stormwater BMP</b>	<b>Quality</b>	<b>Stream Channel Erosion</b>	<b>Quantity/ Flooding</b>
Vegetated filter strip	+++		
Grasses Swale (w/check dams)	+++	+	
Constructed wetlands	+++	+	
Extended detention	++	+++	+
Extended detention enhanced	+++	++	+
Bioretention	+++		
Retention basin	+++	++	+++
Sand filter	+++		
Infiltration	+++		
Infiltration Basin	++	+	+
Detention		++	+++
Manufactured BMPs (Water Quality Structures)	+++		

Legend:     +++     Primary functional goal  
               ++     Potential secondary functional goal  
               +     Potential secondary functional goal with design modifications or additional storage

Source: Virginia Stormwater Management Handbook, Vol. 1, 1<sup>st</sup> Ed.

### **11.7.2 Water Quantity**

Controlling the quantity of stormwater can provide the following potential benefits:

- Prevention or reduction of peak runoff rate increases caused by urban development
- Decrease downstream channel erosion
- Mitigation of downstream drainage capacity problems
- Recharge of groundwater resources
- Reduction or elimination of the need for downstream outfall improvements
- Maintenance of historic low flow rates by controlled discharge from storage

One concept that can be used to control the quantity of stormwater is to consider the use of offsite improvements or regional SWMFs.

### **11.7.3 Extended Detention vs. Retention**

When evaluating the relative merits of extended dry detention versus wet retention basins, there are several factors to consider. Extended detention basins generally require much less storage volume than retention basins. However, wet basins generally provide more pollutant removal and are usually considered an amenity if designed properly. Wet basins require a reliable water/groundwater source and sometimes a significant size drainage area in order to maintain the desired permanent pool level and to prevent the basin from being objectionable. A typical extended detention basin plan is shown in the Appendices of Chapter 6. A typical retention basin plan is shown in the Appendices of Chapter 6.

### 11.7.4 Detention Time

Settling or sedimentation is limited to particulate pollutants that drop out of the water column by means of gravitational settling. Pollutants attach themselves to heavier sediment particles or suspended solids and settle out of the water. Laboratory and field studies indicate that significant settling of urban pollutants occurs in the first 6 to 12 hours of detention. Figure 11-9 shows removal rate versus detention time for selected pollutants.

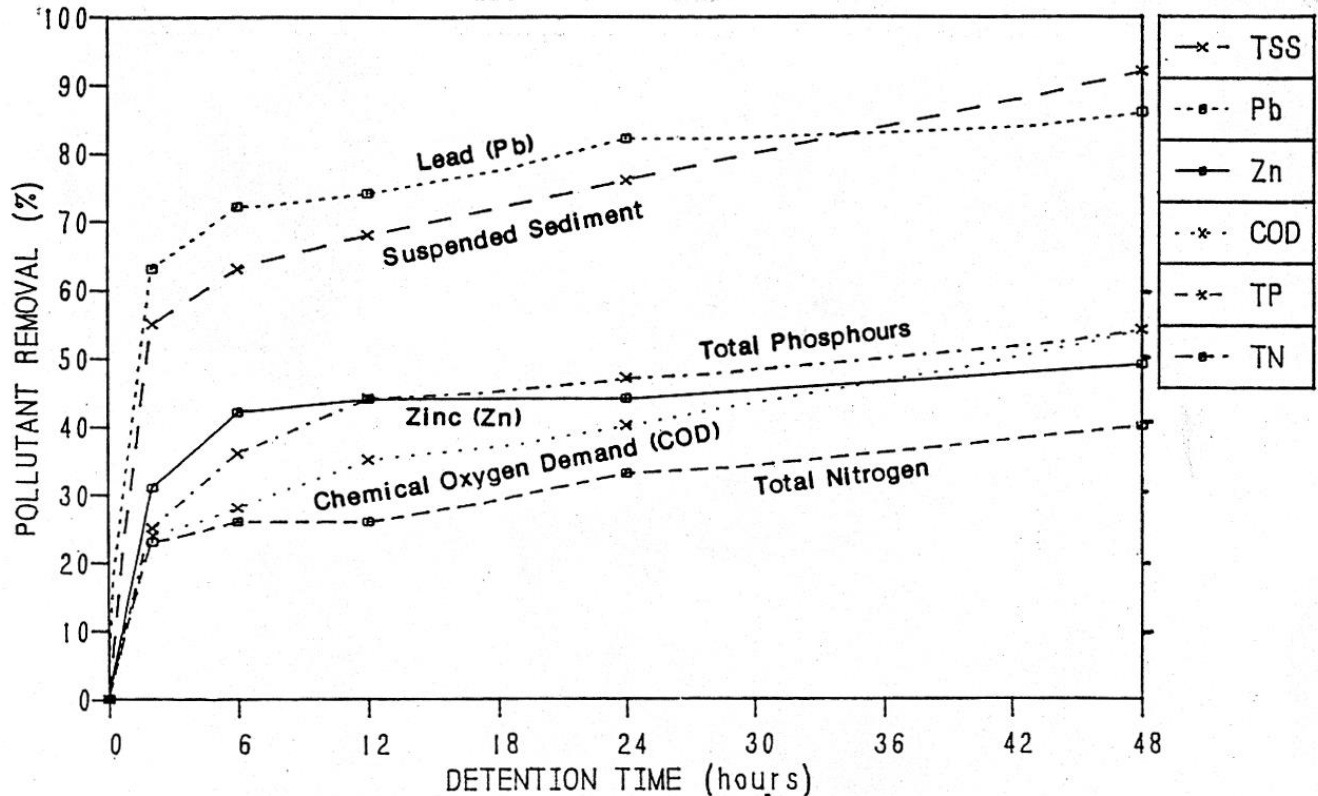


Figure 11-9. Removal Rates vs. Detention Time

The brim drawdown requirement for water quality for extended detention design is 30 hours. The additional time is required to allow for ideal settling conditions to develop within the SWMF. In addition, the added time will allow for settling of smaller particle sizes and nutrients, as well as increasing the opportunity for biological processes to take place. Stormwater BMPs that utilize settling are usually suited for dual purposes that include providing storage volume for peak rate control, channel erosion, and flood control.

### 11.7.5 Release Rates

Control structure release rates are usually designed to approximate pre-developed peak runoff rates for the 2- and 10-yr design storms with an emergency spillway capable of

handling the 100-yr peak discharge. Design calculations are required to demonstrate that the post-development release rates for the 2- and 10-yr design storms are equal to or less than the pre-development release rates. If it can be shown that the 2- and 10-yr design storms are controlled, then runoff from intermediate storm frequencies are assumed to be adequately controlled as well.

Multi-stage control structures may be required to control runoff from both the 2- and 10-yr storms. This can be accomplished through the use of orifices and weirs and is discussed in Section 11.4.7.

#### **11.7.5.1 Channel Erosion Control – Q<sub>1</sub> Control**

Water quantity control for the 1-year design storm (in lieu of the 2-yr design storm required by MS-19) may be needed if there is existing or anticipated erosion downstream. Control of the 1-year design storm requires detaining the volume of runoff from the entire drainage area and releasing that volume over a 24-hour period.

When the 1-year design storm is detained for 24 hours there will be no need to provide additional or separate storage for the WQV if it can be demonstrated that the WQV will be detained for approximately 24 hours. The control of the 1-year design storm may require a basin size that is 1.5 to 2 times larger than a basin used to control the increase in runoff from a 2- or 10-yr design storm.

Properties and receiving waterways downstream of any land-disturbing activity shall be protected from erosion and damage due to changes in stormwater flows and hydrologic characteristics, including but not limited to, changes in runoff volume, velocity, frequency, duration, and peak flow rate.

Requirements for stream channel erosion control shall be governed by the Virginia ESC Regulation MS19 for an adequate receiving channel for stormwater discharges.

Receiving channels shall be reviewed for adequacy based upon the following criteria:

- Natural channels shall be analyzed by the use of a post-development peak discharge from a 2-yr storm to verify that stormwater will not cause erosion of the channel bed and banks, and
- All previously constructed man-made channels shall be analyzed by the use of a post-development peak discharge from a 2-yr storm to verify that the stormwater will not cause erosion of the channel bed or banks.

When utilizing an existing culvert or storm sewer pipe as the outfall for stormwater runoff from the project site, the receiving channel at the outlet end of the existing culvert or storm sewer pipe shall be analyzed for adequacy based on the type of receiving channel (natural or man-made).

If existing natural or previously constructed man-made receiving channels are not adequate, then one of the following measures must be implemented:

- Improve the receiving channel to a condition where the post-development peak runoff rate from a 2-yr storm will not cause erosion to the channel bed or banks or to the point where the drainage area within the channel complies with the requirements, or
- Develop a site design that will not cause the pre-development peak runoff rate from a 2-yr storm to increase (i.e., post development 2 year peak discharge is equal to or less than the pre-development 2 year peak discharge) when runoff discharges into a natural channel or will not cause the post-development peak runoff rate from a 10-year storm to increase (i.e., post development 10-yr peak discharge is equal to or less than pre-development 10-yr peak discharge) when runoff discharges into a man-made channel, or
- Provide a combination of channel improvements, stormwater detention or other measures to prevent downstream erosion.

Where determined necessary by the SWM Plan Designer or requested by DEQ, water quantity control for the 1-year storm may be required if there is existing or anticipated erosion concerns downstream of the project site. Such determination or request shall be made prior to the public participation phase of the project (or other such phase when no public participation process is required). Control of the 1-year storm requires detaining the volume of runoff from the entire drainage area and releasing that volume over a 24-hour period. See the Virginia SWM Handbook, Volume I, Page 1-23 and Volume II, Pages 5-38 thru 5-41 for additional information.

Post-development conditions for both offsite and onsite areas shall be those that exist at the time when the final receiving channel analysis is performed. All land cover shall be assumed to be in "good" condition regardless of actual conditions existing at the time the analysis is performed.

Post-development conditions for offsite areas shall be determined the same as for Pre-development conditions. Post-development conditions for the on-site areas shall be determined based on the proposed project plans and any known future plans of development within the project site.

**One Percent (1%) Rule** - If it can be demonstrated that the total drainage area to the point of analysis within the receiving channel is 100 times greater than the contributing drainage area from within the project site, the receiving channel may be considered adequate, with respect to the stability (erosion) requirements, without further analysis.

### **11.7.5.2 Flooding**

Properties and receiving waterways downstream of any land-disturbing activity shall be protected from localized flooding due to changes in stormwater flows and hydrologic characteristics including, but not limited to, changes in runoff volume, velocity, frequency, duration, and peak flow rate.

For non-linear projects, the 10-yr post-development peak rate of runoff from the site shall not exceed the 10-yr pre-development peak rate of runoff. For linear projects, requirements for downstream flooding control shall be governed by the Virginia ESC Regulation MS19 for adequate receiving channel for stormwater discharges.

Receiving channels shall be reviewed for adequacy based upon the following criteria:

- Natural channels shall be analyzed by the use of a post-development peak discharge rate from 2-yr storm to verify that stormwater will not overtop the channel banks, and
- All previously constructed man-made channels shall be analyzed by the use of a post-development peak discharge rate from a 10-yr storm to verify that the stormwater will not overtop the channel banks, and
- Existing culvert and storm sewer systems, utilized as stormwater outfalls for the development site, shall be analyzed by the use of a post-development peak discharge rate from a 10-yr frequency storm to verify that the stormwater will be contained within the pipe or storm sewer system.

When utilizing an existing culvert or storm sewer pipe as the outfall for stormwater runoff from the project site, the receiving channel at the outlet end of the existing culvert or storm sewer pipe shall be analyzed for adequacy based on the type of receiving channel (natural or man-made).

If existing natural or previously constructed man-made receiving channels or existing culvert or storm sewer pipe systems are not adequate, then one of the following measures must be implemented:

- Improve the channel to a condition where the post-development peak runoff rate from a 10-yr storm will not overtop the channel banks or to the point where the drainage area within the channel complies with the requirements, or
- Improve the culvert or storm sewer system to a condition where the post-development peak runoff rate from a 10-yr storm is contained within the appurtenances, or
- Develop a site design that will not cause the pre-development peak run-off rate from a 2-yr storm to increase (i.e., post development 2-yr peak discharge is equal to or less than pre-development 2-yr peak discharge) when runoff from the site discharges into

a natural channel or will not cause the pre-development peak runoff rate from a 10-yr storm to increase (i.e., post development 10-yr peak discharge is equal to or less than pre-development 10-yr peak discharge) when runoff from the site discharges into a man-made channel or a culvert/storm sewer system, or

- Provide a combination of channel/culvert/storm sewer system improvements, stormwater detention or other measures in order to prevent downstream flooding.

**One Percent (1%) Rule** - If it can be demonstrated that the total drainage area to the point of analysis within the receiving channel is 100 times greater than the contributing drainage area from within the project site, the receiving channel may be considered adequate, with respect to the flooding requirements, without further analysis.

Pre-development conditions for both the offsite and onsite areas shall be those that exist at the time when the final receiving channel analysis is performed. All land cover shall be assumed to be in good condition regardless of actual conditions existing at the time the analysis is performed.

Post-development conditions for offsite areas shall be determined the same as for Pre-development conditions. Post-development conditions for the on-site areas shall be determined based on the proposed project plans and any known future plans of development within the project site.

### **11.7.5.3 Water Quality Control**

Unless otherwise exempt, a water quality control plan that provides compliance with the VSMP Regulations Part II C technical criteria shall be developed for each grandfathered VDOT land-disturbing activity exceeding the land disturbance thresholds noted in IIM-LD-195 (see Section 11.5.9 for additional information on grandfathered projects).

Compliance with the water quality criteria may be achieved by applying the performance-based criteria (see below for discussion and application of this methodology). Additional discussion and application of this methodology can also be found in Volumes I and II of the Virginia SWM Handbook.

Evaluation of water quality requirements may be performed considering the site area at each individual stormwater discharge (outfall) point from the proposed land-disturbing-activity/project or may be performed considering the site area for the entire limits of the proposed land-disturbing activity/project.

Where the proposed land-disturbing activity/project drains to more than one 6<sup>th</sup> Order HUC, the required pollutant load reductions shall be applied independently within each HUC unless reductions are proposed to be achieved under a project specific or a comprehensive SWM plan developed in accordance with Section 9VAC25-870-92 of the VSMP Regulations.

#### Performance-Based Criteria

- The calculated post-development pollutant load from the site shall be compared to the calculated pre-development pollutant load from the site based upon the average land cover condition or the existing site condition as related to the site's percent impervious.
- The site's percent impervious shall be determined as follows:
  - For pre-development conditions - The amount of pre-development impervious area within the site divided by the total area of the site times 100.
  - For post-development conditions - The amount of post-development impervious area within the site divided by the total area of the site times 100.
- A BMP shall be located, designed, and maintained to achieve the target pollutant removal efficiencies specified in Table 11-1 for the purposes of reducing the post-development pollutant load from the site to the required level based upon the following four applicable land development situations for which the performance-based criteria apply:
  - Situation 1 consists of land-disturbing activities where the pre-development percent impervious cover of the site is less than or equal to the average land cover condition (16%) and the proposed improvements will create a total post-development percent impervious cover of the site which is less than the average land cover condition (16%).
    - Water Quality Requirement: No reduction in the post-development pollutant discharge from the site is required.
  - Situation 2 consists of land-disturbing activities where the pre-development percent impervious cover of the site is less than or equal to the average land cover condition (16%) and the proposed improvements will create a total post-development percent impervious cover of the site which is greater than the average land cover condition (16%).
    - Water Quality Requirement: The post-development pollutant discharge from the site shall not exceed the pre-development pollutant discharge from the site based on the average land cover condition (16%).
  - Situation 3 consists of land-disturbing activities where the pre-development percent impervious cover of the site is greater than the average land cover condition (16%).
    - Water Quality Requirement: The post-development pollutant discharge from the site shall not exceed (a) the pre-development pollutant discharge from the site less 10% or (b) the pollutant discharge based on the average land cover condition (16%), whichever is greater.
  - Situation 4 consists of land-disturbing activities where the pre-development impervious cover of the site is served by an existing BMP that addresses water quality.
    - Water Quality Requirement: The post-development pollutant discharge from the site shall not exceed the pre-development pollutant discharge from the site based on the existing percent impervious cover of the area being served by the existing BMP. The existing BMP shall be shown to have been designed and



constructed in accordance with proper design standards and specifications, and to be in proper functioning condition.

When the applicable percent impervious cover of the site is less than the statewide “average land cover condition” of 16%, no water quality BMPs are required. (Exception - Where a locality has established a lower “average land cover condition” than the statewide average, the provisions of IIM-LD-195 shall govern.)

The applicable post-development percent impervious cover of the site shall be as follows:

- For linear development projects:
  - “Old” criteria - The net increase in impervious area of the site (total post-development impervious area of the site minus the total pre-development impervious area of the site) divided by the total post-development area of the site times 100.
  - “New” criteria – See Performance-Based Criteria
- For Non- Linear Projects – See Performance-Based Criteria

The water quality volume for any required BMP shall be based on the total post-development impervious area draining to the BMP from within the R/W of the proposed project/activity and from within any VDOT R/W adjacent to the proposed project/activity.

#### Alternative BMPs

BMPs included on the Virginia SWM BMP Clearing House website <https://www.swbmp.vwrrc.vt.edu/> may be used with the Performance-Based water quality criteria. Unless otherwise approved by DEQ, the maximum removal efficiency allowed for the BMP will be that shown for phosphorus removal by treatment and any removal efficiency associated with phosphorus removal by runoff reduction will not be allowed.

Other alternative BMPs not included in Table 11-1 or the Virginia SWM BMP Clearing House website may be allowed at the discretion and approval of DEQ.

Approval to use alternative BMPs is to be coordinated between the VDOT District or Central Office SWM Plan Designer and the DEQ Regional Stormwater Program Manager. The VDOT State Stormwater Management Program Administrator and the DEQ Central Office Director of the Office of Water Permits shall be copied on any correspondence related to a request for approval of the use of any alternative BMPs.

Use of Low Impact Development (LID) and Better Site Design (BSD) practices are encouraged to the maximum extent practicable in order to reduce the stormwater runoff impacts of the proposed development. LID practices include, but are not limited to, the preservation/protection of riparian buffers, wetlands, steep slopes, mature trees, flood plains, woodlands and highly permeable soils. BSD practices include, but are not limited

to, reduction of impervious cover, conservation of natural areas and the more effective use of pervious areas to treat stormwater runoff.

When the 1-year storm is detained for 24 hours, there will be no need to provide additional or separate storage for the WQV if it can be demonstrated that the WQV will be detained for approximately 24 hours.

#### Offsite Water Quality Compliance Options

Where the water quality requirements for the land development activity cannot be satisfied onsite, offsite options may be used to achieve compliance with the requirements of the VSMP Regulations.

Offsite compliance options allowed for use in meeting required phosphorus load reductions include one or more of the following:

- Offsite controls utilized in accordance with a comprehensive SWM plan adopted pursuant to Section 9VAC25-870-95 of the VSMP regulations for the local watershed within which a project is located (e.g., a regional SWMF).
- A locality pollutant loading pro rata share program established pursuant to § 15.2-2243 of the Code of Virginia or similar local funding mechanism (e.g., a stream restoration fund).
- The Nonpoint Nutrient Offset Program established pursuant to § 62.1-44.15:35 of the Code of Virginia (i.e., the purchase of phosphorus credits from a Nutrient Credit Bank).
- Any other offsite option approved by DEQ.
- When VDOT has additional properties located within the same 6<sup>th</sup> Order HUC or upstream HUC of the land-disturbing activity or within the same watershed as determined by DEQ, SWMFs located on those properties may be utilized to meet the required phosphorus load reductions from the land-disturbing activity.

VDOT may utilize offsite options if the project meets any one of the following conditions:

- The activity will disturb less than five acres of land (100% offsite compliance allowed).
- The activity's post-developed phosphorus load reduction requirement is less than 10 pounds per year (100% offsite compliance allowed).
- At least 75% of the required phosphorus load reductions can be achieved onsite (up to 25% offsite compliance allowed).
- If at least 75% of the activity's required phosphorus load reductions cannot be achieved onsite, then the required phosphorus load reductions may be achieved, in whole or in part, through the use of offsite compliance options (up to 100% offsite compliance may be allowed) provided VDOT can demonstrate to the satisfaction of the DEQ that:

- Alternative site designs have been considered that may accommodate onsite BMPs, and
- Onsite BMPs have been considered in alternative site designs to the maximum extent practicable, and
- Appropriate onsite BMPs will be implemented, and
- Full compliance with post-development nonpoint nutrient runoff compliance requirements cannot practicably be met onsite,

Offsite options shall not be allowed:

- Unless the selected offsite option achieves the necessary phosphorus load reductions prior to the commencement of the construction of the proposed project. Where the offsite option will be constructed as a part of the proposed VDOT project, the offsite option must be completed and functional prior to the completion of the VDOT project, or
- In violation of local water quality-based limitations at the point of discharge that are consistent with the determinations made pursuant to a TMDL Implementation Plan, contained in a MS4 Program Plan approved by DEQ or as otherwise may be established or approved by DEQ.

Non-structural practices including, but not limited to, minimization of impervious areas and curbing requirements, open space acquisition, floodplain management, and protection of wetlands may be utilized as appropriate in order to at least partially satisfy water quality requirements. Approval to use such non-structural measures is to be secured in advance from DEQ and is to be coordinated between the VDOT State Stormwater Management Program Administrator and the DEQ Central Office Director of the Office of Water Permits.

## 11.7.6 Outlet Hydraulics

### 11.7.6.1 Orifice

An orifice is an opening into a standpipe, riser, weir, or concrete structure. Openings smaller than 12 inches may be analyzed as a submerged orifice if the headwater to depth ratio (HW/D) is greater than 1.5. An orifice for water quality is usually small (less than 6 inches) and round. VDOT has determined that the orifice is less prone to clogging when located in a steel plate rather than a 6- or 8-inch hole in a concrete wall. Details are shown in the latest version of VDOT Location & Design Instructional & Informational Memorandum IIM-LD-195. For square-edged entrance conditions, the orifice equation is expressed as:

$$Q = CA\sqrt{2gh} \quad (11.1)$$

Where:

Q = Discharge, cfs

C = Orifice entrance coefficient (generally 0.6)

A = Cross-sectional area of orifice, ft<sup>2</sup>

g = Acceleration due to gravity, 32.2 ft/s<sup>2</sup>

h = Head on orifice, ft.

### 11.7.6.2 Weirs

The most common type of weir associated with SWM is the broad-crested weir as is defined by Equation 11.2:

$$Q = CLH^{\frac{3}{2}} \quad (11.2)$$

Where:

Q = Discharge, cfs

C = Broad-crested weir coefficient (Range from 2.67 to 3.33 and is generally assumed to be 3.0.) For additional information, refer to King and Brater, Handbook of Hydraulics, 1976, which lists coefficients and instructions on determining an appropriate coefficient.

L = Broad-crested weir length, ft.

H = Head above weir crest, ft.

If the upstream edge of a broad-crested weir is rounded so as to prevent contraction and if the slope of the crest is as great as the headless due to friction, flow will pass through critical depth at the weir crest; this gives the maximum entrance coefficient (C) of 3.00. For sharp corners on the broad-crested weir; however, a minimum (C) of 2.67 should be used. The designer should also check to make certain the weir or orifice is not submerged by the downstream tailwater.

### **11.7.6.3 Types of Outlet Structures**

#### **11.7.6.3.1 General**

Outlet structures typically include a principal spillway and an emergency overflow, and must accomplish the design functions of the facility. Outlet structures can take the form of combinations of drop inlets, pipes, weirs, and orifices. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet. If site restrictions prevent the use of an emergency spillway, then the principal spillway should be sized to safely pass the 100-yr design storm without overtopping the facility. The designer should consider partial clogging (50%) of the principal spillway during the 100-yr design storm to ensure the facility would not be overtopped. For large SWMFs, selecting a flood magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The minimum flood to be used to size the emergency spillway is the 100-yr design storm flood. The sizing of a particular outlet structure should be based on results of hydrologic routing calculations.

A principal spillway system that controls the rate of discharge from a stormwater facility will often use a multi-stage riser for the drop inlet structure, such as the VDOT standard SWM-1. A multi-stage riser is a structure that incorporates separate openings or devices at different elevations to control the rate of discharge from a stormwater basin during multiple design storms. Permanent multi-stage risers are typically constructed of concrete to help increase their life expectancy. The geometry of risers will vary from basin to basin. The designer can be creative to provide the most economical and hydraulically efficient riser design possible.

The primary control structure (riser or weir) should be designed to operate in weir flow conditions for the full range of design flows. Where this is not possible or feasible and the control structure will operate in orifice flow conditions at some point within the design flow range, an anti-vortex device, consistent with the design recommendations in the SWM Handbook, shall be utilized.

In a SWM basin design, the multi-stage riser is of utmost importance because it controls the design water surface elevations. In designing the multi-stage riser, many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. Each iterative routing requires that the facility's size and outlet shape be designed and tested for performance.

Two types of outlet structures are discussed below.

#### **11.7.6.3.2 SWM-1 (VDOT Standard)**

The VDOT standard riser outlet structure is identified as a SWM drainage structure (VDOT Standard SWM-1). This structure should be used at all applicable locations where a drop inlet type control structure is desired. Water quality orifices and additional orifices and weirs can be designed for use with the SWM-1. In addition, the SWM-1 can be modified during construction to serve as the outlet for a temporary sediment basin. The subsurface base of a SWM-1 is typically loaded with Class I stone to counter buoyancy forces.

Anti-vortex vanes are usually not needed on risers for SWM basins due to the VDOT practice of designing relatively shallow basins with emergency spillways. A small trash rack in front of the water quality orifice is included in the SWM-1 details. SWM-1 details can be found in VDOT Road and Bridge Standards Volume I.

Culverts under or through the dam of a SWM Basin are to be reinforced concrete pipe with rubber gaskets, and the joint must be leak-resistant as per AASHTO PP-63, and shall be included in the Department's Approved List No. 14. A concrete cradle is to be used under the pipe to prevent seepage through the dam. The concrete cradle is to extend the full length of the pipe. (See Road and Bridge Standard SWM-DR)

#### **11.7.6.3.3 Weir Wall (Stormwater Management Dam)**

Another type of outlet structure that can be used is a weir wall. The weir wall may be constructed either in place of a riser or as part of a pipe culvert's wingwalls.

A weir wall in lieu of a riser may be used in areas of shallow basins where the weir wall is no higher than about 5'. The weir wall will have an outlet channel instead of a pipe and will operate efficiently with fewer maintenance concerns than a riser and pipe configuration.

In conjunction with a culvert, the weir is created by building a wall between the culvert's wingwalls. A concrete apron extends from the pipe to the weir wall at a distance of approximately 1.5 times the culvert diameter. The top of the wall is used to provide the required storage volume and flow attenuation. Notches can also be used in the weir wall to attenuate various storms, and a water quality orifice can be installed at the base in order to drain the basin and provide quality treatment. In addition, the weir wall can be modified during construction to serve as the outlet for a sediment basin. Weir wall outfall structures have proven useful in providing online SWMFs at culvert crossings with dry, intermittent drainage swales by providing the required storage on the upstream side of the crossing. Online facilities should not be used in live streams.

#### **11.7.6.4 Routing**

The following data is needed to complete storage design and routing calculations using the appropriate computer program:

- Inflow hydrographs for all selected design storms
- Allowable release rates
- Stage-storage curve or data for proposed SWMF
- Stage-discharge curve or data for the outlet control structures based upon the preliminary design of the outlet control structure and emergency spillway
- Receiving channel performance curve or data

## **11.8 Part II C Design Procedures and Sample Problems**

### **11.8.1 Documentation Requirements**

The following documentation will be required for SWMF design:

- Documentation requirements presented in Chapter 6, Hydrology
- Computations for determination of the pre- and post-development peak runoff rates for the design storms
- Receiving channel adequacy to include  $Q_2$  velocity and  $Q_{10}$  capacity
- Water quality volume based on new impervious area calculation and BMP selection
- WQV orifice size
- Drawdown time for WQV
- Compensatory treatment for uncontrolled new impervious areas  
The designer will complete the SWM and TSB Summary Sheet as provided in Appendix 11B-1
- SWMF Tabulation Sheet when submitting final plans
- Provide all documentation from routing. This would generally include inflow and outflow hydrographs and storage computations for sizing the primary spillway. This information would be generated by various computer modeling software
- Basin grading and primary spillway details and specifications
- Complete (C) and Minimum (M) plan projects shall show SWM measures in the plan assembly as directed in the VDOT Drainage Manual and the VDOT Road Design Manual.
- No-plan (N) and other types of projects (including maintenance activities) that have an abbreviated plan assembly must conform to the requirements of the VSMP Regulations and VPDES General Construction Permit where the land disturbance value exceeds the applicable land disturbance thresholds for such. For the definition of these types of projects, and the procedures for addressing the SWM plan details for such projects, see the VDOT Drainage Manual and the VDOT Road Design Manual.

The plan design details for BMPs shall be appropriately sealed and signed by a person registered in the Commonwealth of Virginia as a professional architect, engineer, land surveyor or landscape architect.

### **11.8.2 Water Quality Volume Computation and BMP Selection Procedure**

*Step 1. Determine the new impervious area within that area at the outfall being evaluated.*

*Step 2. Determine the area within the R/W and easement(s) at the outfall being evaluated.*

- Step 3. *Compute the percentage new impervious (Step 1/Step 2)*
- Step 4. *Compute the WQV by multiplying ½ inch by the new impervious area and convert the units to cubic feet.*
- Step 5. *Refer to Table 11-7 to determine which type of BMP is best suited for the percentage of impervious area*
- Step 6. *Multiply the WQV by the basin treatment factor based (Table 11-1) on the BMP determined from Step 5. This provides the required treatment volume.*

Water Quality Volume Computation and BMP Selection Sample Problem:

Assume the basin is to be an extended detention basin based upon 35% new impervious area within the R/W.

- Step 1. *Determine the new impervious area within that area at the outfall being evaluated.*

New Impervious Area = 2.4 acres

- Step 2. *Determine the area within the R/W and easement(s) at the outfall being evaluated.*

- Step 3. *Compute the percentage new impervious (Step 1/Step 2).*

Given in the problem statement as 35%.

- Step 4. *Compute the WQV by multiplying ½ inch by the impervious area and convert the units to cubic feet.*

$$\text{WQV} = \frac{1}{2} \text{ inch} \times \text{Impervious Area}$$

$$\frac{1}{2} \text{ inch} \times (1 \text{ ft}/12 \text{ inches}) = 0.04126 \text{ ft}$$

$$1 \text{ acre} = 43,560 \text{ ft}^2$$

$$\text{WQV} = 0.04167 \times 43,560 \times 2.4 \text{ ac.} = 4,356 \text{ ft}^3 \text{ (say } 4,360 \text{ ft}^3)$$

- Step 5. *Refer to Table 11-7 to determine which type of BMP is best suited for the percentage of impervious area*

For 35% impervious cover, an extended detention basin will be used.

- Step 6. *Multiply the WQV by the basin treatment factor based on the BMP determined from Step 4. This provides the treatment volume.*

$$\text{Required Treatment Volume} = 2 \times \text{WQV} = 2(4360) = 8720 \text{ cu. ft.}$$



### 11.8.3 Detention Time Computation and Orifice Sizing

A water quality extended-detention basin treats the water quality volume by detaining it and releasing it over a specified amount of time. In theory, extended-detention of the water quality volume will allow the particulate pollutants to settle out of the first flush of runoff, functioning similarly to a permanent pool. Virginia's Stormwater Management Regulations pertaining to water quality specify a 30-hour draw down time for the water quality volume. This is a brim draw down time, beginning at the time of peak storage of the water quality volume. Brim drawdown time means the time required for the entire calculated volume to drain 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. The extended detention orifice can be sized using either of the following methods:

- Using the average hydraulic head associated with the water quality volume (WQV) and the required drawdown time. This is the VDOT preferred option.
- Using the maximum hydraulic head associated with the water quality volume (WQV), calculate the orifice size needed to achieve the required draw down time and route the water quality volume through the basin to verify the actual storage volume used and the drawdown time.

**Table 11-10. WQV Orifice Sizes**

Diameter	
Inches	Square Feet
½	0.0013
¾	0.003
1	0.005
1 ½	0.012
2	0.022
2 ½	0.034
3	0.049
3½	0.067
4	0.087
4 ½	0.110
5	0.136
5 ½	0.165
6	0.196

After calculating the needed orifice size the designer should select the nearest nominal size opening from Table 11-10.

#### 11.8.3.1 Average Hydraulic Head Method (Method #2) - VDOT Preferred Method

The average hydraulic head method is the preferred method for determining the required orifice size. It is quicker and easier than the maximum hydraulic head method, which

requires a routing to verify the drawdown time. It is also noted that the difference in orifice size produced by the two different methods is insignificant, (i.e. 2 inches versus 2½ inches).

Average Hydraulic Head Sample Problem:

Find the orifice size for the required treatment volume using the average hydraulic head method.

$$h_{\max} = 1.1 \text{ ft.}$$

Volume = 8,720 ft<sup>3</sup> (from Sample Problem 11.5.2.1)

$$h_{\text{avg}} = \frac{1.1}{2} = 0.55 \text{ ft.}$$

Note: Actual h on orifice is to the center of the orifice. Since the size of this orifice is unknown and assumed small, use  $h_{\max} = 1.1$  ft.

Calculate the discharge through the orifice based on the required treatment volume.

$$Q_{\text{avg}} = \frac{\text{Treatment Volume}}{\text{Time}} = \frac{8720 \text{ cu. ft.}}{30 \text{ hr} (3600 \frac{\text{sec}}{\text{hr}})} = 0.081 \text{ cfs}$$

Calculate the orifice area by rearranging Equation 11.1.

$$A = \frac{Q}{C\sqrt{2gh_{\text{avg}}}} = \frac{0.081}{0.6\sqrt{2(32.2)(0.55)}} = 0.0223 \text{ sq. ft.}$$

From Table 11-10, select a 2-inch orifice with  $A = 0.022$  ft<sup>2</sup>.

### 11.8.3.2 Maximum Hydraulic Head Method (Method #1)

The maximum hydraulic head method uses the maximum discharge and results in a slightly larger orifice than the same procedure using the average hydraulic head method. The routing allows the designer to verify the performance of the calculated orifice size. As a result of the routing effect however, the actual basin storage volume used to achieve the drawdown time will be less than the computed brim drawdown volume.

Maximum Hydraulic Head Sample Problem:

Using the data provided in sample problem 11.5.3.1.1, determine the orifice size using the maximum hydraulic head method: Use the maximum hydraulic head (not the average) and the maximum Q ( $Q_{\text{avg}} \times 2$ ). The WQV hydrograph (HYG) should then be routed through the basin to determine if the residence time is approximately 30 hours.

Find the orifice size for the required treatment volume using the maximum hydraulic head method.

$$h_{\max} = 1.1 \text{ ft.}$$

$$Q_{\max} = 2Q_{\text{avg}} = 2(0.081) = 0.16 \text{ cfs}$$

Calculate the orifice area by rearranging Equation 11.1.

$$A = \frac{Q}{C\sqrt{2gh_{\text{avg}}}} = \frac{0.16}{0.6\sqrt{2(32.2)(1.1)}} = 0.0320 \text{ sq. ft.}$$

From Table 11-10, select a 2½-inch orifice with  $A = 0.034 \text{ ft}^2$

Next step: Route the WQV hydrograph thru the basin using the 2½-inch orifice.

COMMENTS: The routing of the WQV hydrograph thru a basin may not be possible with some routing software. The problem can be due to the need for using a hydrograph for a minimum of about 30 hours and with possibly the last 29-hours inflow of 0.0 or 0.01 cfs. The problem could also be due to the need for small orifice sizes < 3-inches in diameter.

### 11.8.3.3 WQV Hydrograph (HYG)

To develop a hydrograph for the WQV following the sample problem in Section 11.5.3.2.1, you need only to calculate the hydrograph for the new impervious area and use the time of concentration that applies to the new impervious area and its proximity to the basin. The NRCS Method hydrograph will probably be the easiest hydrograph to provide the required treatment volume of 1 inch of runoff for an extended detention basin. The time of concentration ( $t_c$ ) may be found by methods discussed in Chapter 6, Hydrology, since the  $t_c$  has the same definition in the Rational Method as in TR-55. The process will involve using a  $CN = 98$  for the impervious area, Rainfall (RF) = 1.2 inches to produce RUNOFF (RO) = 1 inch and the NRCS 24-hour Type II storm distribution. All VDOT designers should have the TR-55 software and the above values can be used to produce the hydrograph.

### 11.8.3.4 Alternative Method of Routing WQV to Find Drawdown Time

The Stormwater Management Handbook, Vol. II, defines brim drawdown time as from the time the WQV elevation is reached until the basin is emptied. This is based upon a treatment volume storm producing only the amount of runoff required for the WQV.

The normally required routing of the 2-yr storm for quantity control can also be used for drawdown time with some slight adjustment providing that the routing software will accommodate a 30-hour duration and a small size orifice. The receding limb of the inflow hydrograph will need to be showing either 0.0 or 0.01 cfs inflow up to a time of about 30 hours. By this method the drawdown time for WQV is actually from the time that the ponded depth recedes to the treatment volume elevation (with no more inflow) until the basin is empty. For practical purposes, if the routing shows that the basin is empty at about 30 hours, the design is adequate.

## 11.8.4 Channel Erosion Control Volume – $Q_1$ Control

Extended detention of a specified volume of stormwater runoff can also be incorporated into a basin design to protect downstream channels from erosion. Virginia's Stormwater Management Regulations recommend 24-hour extended detention of the runoff from the

1-year frequency storm as an alternative to the 2-yr peak discharge reduction required by MS-19 of the VESCR.

The design of a channel erosion control extended-detention orifice is similar to the design of the water quality orifice in that previous orifice sizing methods can be used:

1. Using the average hydraulic head method (VDOT Preferred Method), approximate the orifice size associated with the channel erosion control volume ( $V_{ce}$ ) and the drawdown time.
2. Using the maximum hydraulic head method, approximate the orifice size associated with the channel erosion control volume ( $V_{ce}$ ) and the required drawdown time and route the 1-year frequency storm through the basin to verify the storage volume and drawdown time.

The routing procedure takes into account the discharge that occurs before maximum or brim storage of the channel erosion control volume ( $V_{ce}$ ). The routing procedure provides a more accurate accounting of the storage volume used while water is flowing into and out of the basin, and may result in less storage volume being used than the calculated brim storage volume associated with the maximum hydraulic head. The actual storage volume needed for extended detention of the runoff generated by the 1-year frequency storm will be approximately 60 percent of the calculated volume ( $V_{ce}$ ) of runoff for curve numbers between 75 and 95 and with times of concentration between 0.1 and 1 hour.

Channel Erosion Control Volume, (Q1 Control) Sample Problem:

The following sample problem illustrates the design of the extended-detention orifice for channel erosion control volume using the average hydraulic head method.

Drainage Area = 25 ac.

1-year rainfall = 2.7"

CN = 75

1-year rainfall depth of runoff = 0.8"

- Step 1. Determine the rainfall amount (inches) of the 1-year frequency storm for the local area where the project is located.*
- Step 2. With the rainfall amount and the runoff curve number (CN), determine the corresponding runoff depth using the runoff equation.*
- Step 3. Calculate the channel erosion control volume ( $V_{ce}$ )*

$$V_{ce} = 25\text{ac.} (0.8 \text{ in.}) \left( \frac{1 \text{ ft.}}{12 \text{ in.}} \right) = 1.67 \text{ ft.}$$

To account for the routing effect, reduce the channel erosion control volume by 60%:

$$V_{ce} = 0.60(1.67) = 1.0 \text{ ac. ft. or } 43,560 \text{ cu. ft.}$$

*Step 4. Determine the average hydraulic head ( $h_{avg}$ ) corresponding to the required channel erosion control volume.*

$$h_{avg} = \frac{2 - 0}{2} = 1.0 \text{ ft.}$$

Note: When considering the maximum depth of ponding, the WQV is generally limited to 2'.

*Step 5. Determine the average discharge ( $Q_{avg}$ ) resulting from the 24-hour drawdown requirement.*

$$Q_{avg} = \frac{43,560 \text{ cu ft}}{(24 \text{ hr})(3,600 \frac{\text{sec}}{\text{hr}})} = 0.50 \text{ cfs}$$

*Step 6. Determine the required orifice diameter by rearranging the Equation 11.1.*

$$A = \frac{Q}{C\sqrt{2gh_{avg}}} = \frac{0.50}{0.6\sqrt{2(32.2)(1.0)}} = 0.104 \text{ sq. ft.}$$

Calculate the orifice diameter:

$$A = \frac{\pi d^2}{4}$$

$$d = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4(0.104)}{\pi}} = 0.364 \text{ sw ft}$$

$$d = 4.4 \text{ in (Say 4.5 in)}$$

The designer can also use Table 11-10 to determine a 4½-inch diameter extended detention orifice for channel erosion control.

## 11.8.5 Preliminary Detention Volume Computation

Three methods are presented for estimating the volume of storage needed for peak flow attenuation (quantity control). The estimated storage volumes are approximate and the designer will need to select the most appropriate volume in order to determine the preliminary basin size.

### 11.8.5.1 Modified Rational Method, Simplified Triangular Hydrograph Routing

Information needed includes the hydrology and hydrographs for the watershed or drainage area to be controlled, calculated by using one of the methods as outlined in

Chapter 6, and the allowable release rates for the facility, as established by ordinance or downstream conditions.

*Step 1. Determine BMP requirements*

Determine the percent of new impervious area within the R/W. Select the type of BMP needed from Table 11-7. Calculate the water quality volume.

Some considerations for BMP selection include:

- Water Quality Extended-Detention Basin: The water quality volume must be detained and released over 30 hours. The established pollutant removal efficiency is based on a 30-hour drawdown.
- Water Quality Retention Basin: The volume of the permanent pool is established by the site impervious cover or the desired pollutant removal efficiency.
- Channel Erosion Control Extended-Detention Basin: The channel erosion control volume based upon  $Q_1$ , for the entire drainage area, must be detained and released over 24 hours.

*Step 2. Compute allowable release rates*

Compute the pre- and post-developed hydrology for the watershed. Sometimes, the pre-developed hydrology will establish the allowable release rate from the basin. Other times, the release rate will be established by downstream conditions. In either case, the post-developed hydrology will provide the peak discharge into the basin, as a peak discharge (cfs) or a runoff hydrograph. Refer to Chapter 6, Hydrology, on developing runoff hydrographs and peak discharge.

*Step 3. Estimate the required storage volume*

The information required includes the developed condition peak rate of runoff, or runoff hydrograph, and the allowable release rates for each of the appropriate design storms. These methods provide a preliminary estimate of the storage volume required for peak flow attenuation.

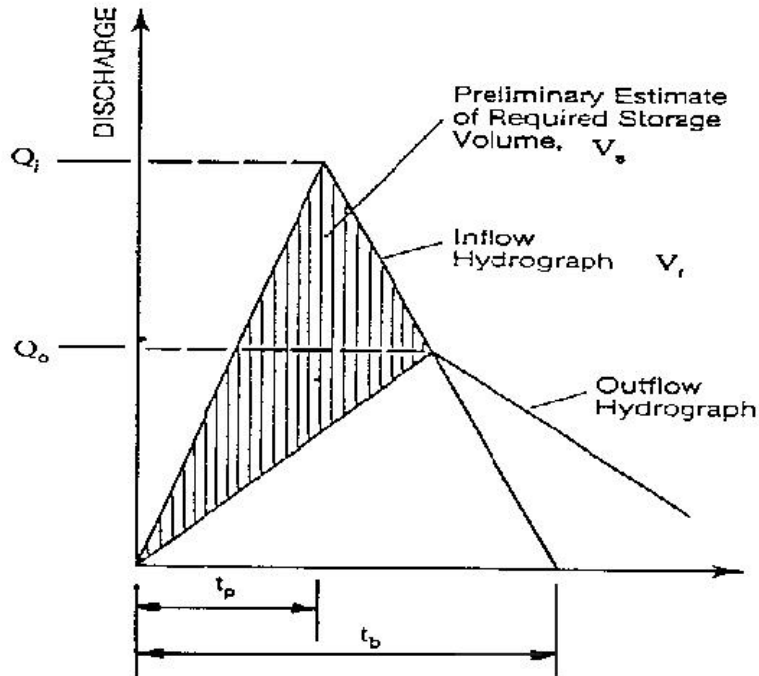


Figure 11-10. Simplified Triangular Hydrograph Method

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

$$V_s = \frac{1}{2} T_b (Q_i - Q_o)$$

Where:

- $V_s$  = Storage volume estimate,  $\text{ft}^3$
- $Q_i$  = Peak inflow rate, cfs
- $Q_o$  = Peak outflow rate, cfs
- $T_b$  = Duration of basin inflow, sec.

#### 11.8.5.2 Critical Storm Duration Method

The critical storm duration method is used to calculate the maximum storage volume for a detention facility. This critical storm duration is the storm duration that generates the greatest volume of runoff and, therefore, requires the most storage. The required storage volume is represented by the area between the inflow hydrograph and the outflow hydrograph. The area can be approximated using the following equation:

$$V = \left[ Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3q_o t_c}{4} \right] 60$$

Where:

- V = Required storage volume, ft<sup>3</sup>
- Q<sub>i</sub> = Inflow peak discharge, cfs, for the critical storm duration, T<sub>d</sub>
- T<sub>c</sub> = Time of concentration, min.
- q<sub>o</sub> = Allowable peak outflow, cfs
- T<sub>d</sub> = Critical storm duration, min.

The first derivative of the critical storage volume equation with respect to time is an equation that represents the slope of the storage volume curve plotted versus time. When Equation 11.4 is set to equal zero, and solved for T<sub>d</sub>, it represents the time at which the slope of the storage volume curve is zero, or at a maximum. Equation 11.5 for the critical storm duration is:

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

Where:

- T<sub>d</sub> = Critical storm duration, min.
- C = Runoff coefficient
- A = Drainage area, ac.
- a & b\* = Rainfall constants developed for storms of various recurrence intervals and various geographic locations†
- t<sub>c</sub> = Time of concentration, min.
- q<sub>o</sub> = Allowable peak outflow, cfs

\*The a & b rainfall constants are not to be used for any other purpose.

The Department has developed a computer program entitled “CRITSTRM” for performing these computations. Access is available upon request at the following web address: <http://www.virginiadot.org/business/locdes/notification.asp>.

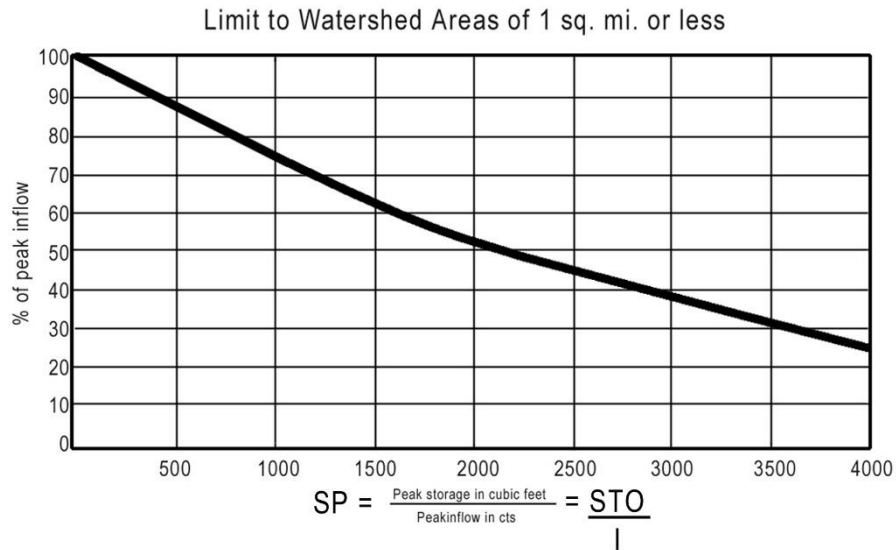
### 11.8.5.3 Pagan Volume Estimation Method

This method is appropriate for use with small basins serving watersheds of 200 acres or less. For this method, data from many small basins was compiled and the curve in Figure 11-11 was developed. This curve is used to determine the storage volume for a given drainage area by dividing the pre-development peak inflow by the post-development peak inflow.

Knowing the percentage of peak inflow, the storage parameter (peak storage in cubic feet over peak inflow in cubic feet per second) can be found by moving horizontally over the y-axis to the curve and down to the x-axis.

By multiplying the storage parameter by the peak inflow, the approximate peak storage can be found. This method should be used only as a first trial. Experience has shown that this method is conservative.





**Figure 11-11. Pagan Method Curve**

*Step 1. Determine pre- and post-development peak discharges.*

*Step 2. Determine the Storage Parameter (SP).*

SP is determined from Figure 11-11 drawing a line from the percentage of peak inflow ( $Q_o/Q_i$ ) to the line and reading the factor along the base of the figure.

*Step 3. Compute the Maximum Storage Volume (STO):*

$$STO = SP(I)$$

#### 11.8.5.4 Sample Problems – Using 3 Methods to Estimate Volume of Storage for Quantity Control

Condition	Rational Method			$Q_{10}$
	D.A	C	$T_c$	
Pre-developed	25ac.	0.38	52 min.	24 cfs
Post-developed	25 ac	0.59	21 min.	65 cfs

*Method 1. Modified Triangular Hydrograph Method*

Based on the methodology from 11.5.4.1, solve for  $V_{s10}$  as follows:

$$V_s = \frac{1}{2}T_b(Q_i - Q_o)$$

Where:

$$\begin{aligned}V_{s10} &= \text{Storage volume estimate, ft}^3 \\Q_i &= 65 \text{ cfs} \\Q_o &= 24 \text{ cfs} \\T_b &= 2520 \text{ sec.} = 42 \text{ min.}\end{aligned}$$

$$V_s = \frac{1}{2}(2520)(65 - 24) = 51,660 \text{ cu. ft.}$$

*Method 2. Critical Storm Duration Method*

Based on the methodology in 11.5.4.2, determine the 10-yr critical storm duration  $T_{d10}$  as follows:

$$\begin{aligned}a &= 189.2 \\b &= 22.1 \\C &= 0.59 \text{ (Post-development)} \\A &= 25 \text{ acres} \\t_c &= 21 \text{ min (Post-development)} \\q_{o10} &= 24 \text{ cfs (Allowable outflow based on pre-development)}\end{aligned}$$

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

$$T_{d10} = \sqrt{\frac{2(0.59)(25.0)(189.2)(22.1 - \frac{21}{4})}{24}} - 22.1$$

$$T_{d10} = 40.5 \text{ min}$$

**Solve for the 10-yr critical storm duration intensity ( $I_{10}$ )**

$$I_{10} = \frac{189.2}{22.1 + 40.5} = 3.02 \text{ in/hr}$$

Determine the 10-yr peak inflow ( $Q_{10}$ ) using the Rational Equation and the critical storm duration intensity ( $I_{10}$ )

$$Q = C_f C_i A$$

$$Q_{10} = 1.0(0.59)(3.02)(25) = 44.5 \text{ cfs}$$

Determine the required 10-yr storage volume ( $V_{10}$ ) for the 10-yr critical storm duration ( $T_{d10}$ )

$$V = \left[ Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3q_o t_c}{4} \right] 60$$

$$V = \left[ (44.5)(40.5) + \frac{(44.5)(21)}{4} - \frac{(24)(40.5)}{2} - \frac{3(24)(21)}{4} \right] 60$$

$$V = 70,313 \text{ cu. ft. (Say 70,300 cu. ft.)}$$

### Method 3. Pagan Method

Based on the methodology in 11.5.4.3, solve for the storage volume as follows:

$$\frac{Q_o}{Q_i} = \frac{24}{65} = 0.37 \text{ (37\%)}$$

$$SP = 3100 \text{ seconds}$$

$$STO = SP(I) = 3100(65) = 201,500 \text{ cu. ft.}$$

## 11.8.6 Determine Preliminary Basin Size

Based upon the estimated storage volume requirements calculated by the three methods in Section 11.5.4.4, determine the preliminary size of the basin. Assume the basin will have a rectangular shaped base, about 2:1 length to width ratio and optimum depth for  $Q_{10}$  about 4'. The basin will have 3:1 side slopes, but for the first size estimate, the size of the base using vertical sides will provide an adequate first estimate.

From Method 1: Simplified Triangular Hydrograph Method

$$V_{10} = 51,660 \text{ ft}^3$$

$$\text{For a 4-ft depth, } \frac{51,660}{4} = 12,915 \text{ sq. ft.}$$

About 80' x 160'

From Method 2: Critical Storm Duration Method

$$V_{10} = 70,300 \text{ ft}^3$$

$$\text{For 4' depth, } \frac{70,300}{4} = 12,915 \text{ sq. ft.}$$

About 90' x 195'

From Method 3: Pagan Method

$$V_{10} = 201,500 \text{ ft}^3$$

$$\text{For a 4' deep, } \frac{201,500}{4} = 50,375 \text{ sq. ft.}$$

About 150' x 335'

Summary: Preliminary trial size basin would be recommended about 100' x 200'

## 11.8.7 Final Basin Sizing-Reservoir Routing

### 11.8.7.1 Storage – Indication Method Routing Procedure

The following procedure presents the basic principles of performing routing through SWMF (Puls Method of storage routing). Routing is most often completed with computer software, which develops the stage-discharge and stage-storage curves within the program.

- Step 1. *Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed SWMF. Example stage-storage and stage-discharge curves are shown in Figure 11-12 and Figure 11-13 respectively.*

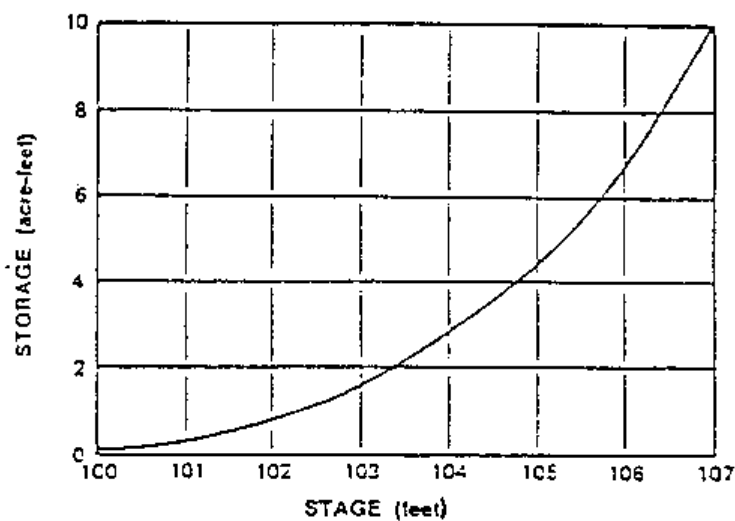
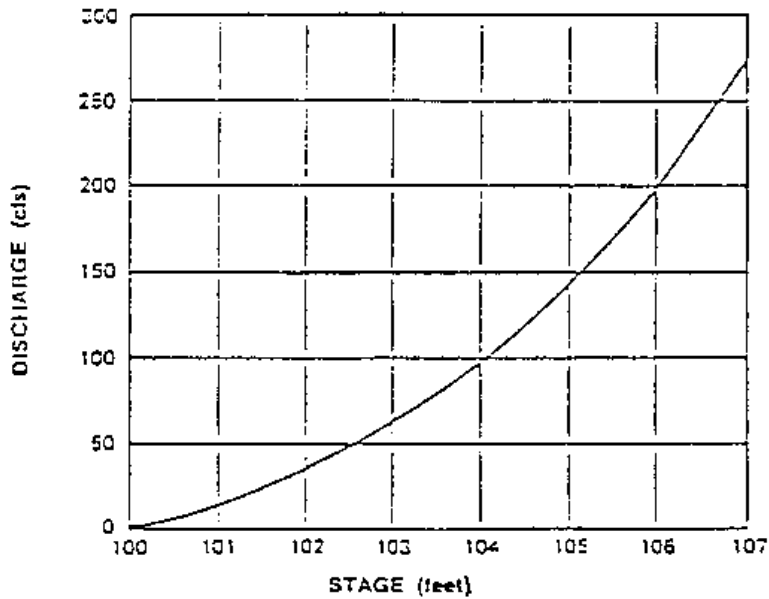


Figure 11-12. Stage-Storage Curve



**Figure 11-13. Stage-Discharge Curve**

Step 2. Select a routing time period ( $\Delta t$ ) to provide at least five points on the rising limb of the inflow hydrograph. Use  $t_p$  divided by 5 to 10 for  $\Delta t$ .

Step 3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of  $s \pm \frac{0}{2} \Delta T$  versus stage. An example tabulation of storage characteristics curve data is shown in Table 11-11.

**Table 11-11. Storage Characteristics**

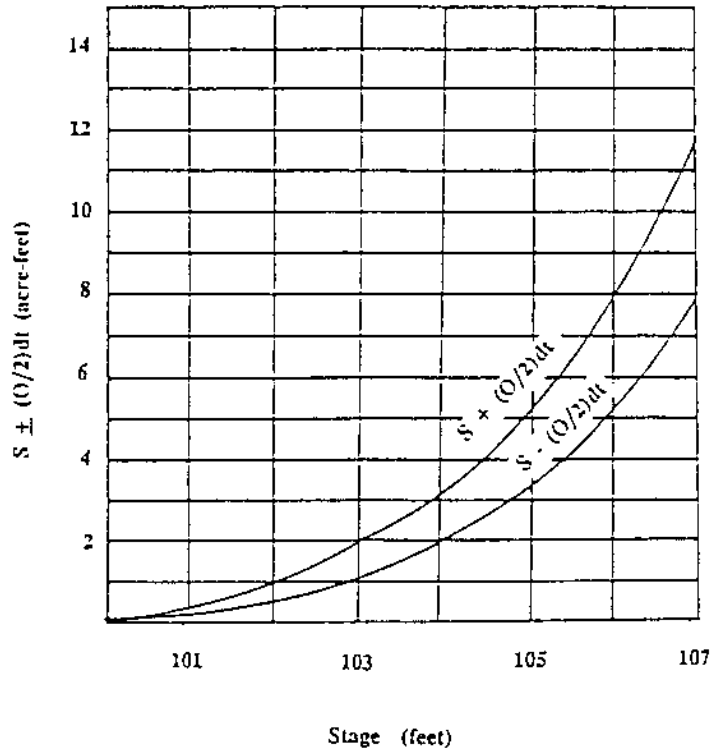
(1) Stage (H) (ft.)	(2) Storage <sup>1</sup> (S) (ac-ft)	(3) Discharge <sup>2</sup> (Q) (cfs)	(4) Discharge <sup>2</sup> (Q) (ac-ft/hr)	(5) $s - \frac{0}{2} \Delta T$ (ac-ft)	(6) $s \pm \frac{0}{2} \Delta T$ (ac-ft)
100	0.05	0	0	0.05	0.05
101	0.05	15	1.24	0.20	0.40
102	0.05	35	2.89	0.56	1.04
103	1.6	63	5.21	1.17	2.03
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.82	3.41	5.39
106	6.6	200	16.53	5.22	7.98

<sup>1</sup> Obtained from the Stage-Storage Curve.

<sup>2</sup> Obtained from the Stage-Discharge Curve.

Note:  $t = 10$  minutes = 0.167 hours and 1 cfs = 0.0826 ac-ft/hr.

Step 4. For a given time interval,  $I_1$  and  $I_2$  are known. Given the depth of storage or stage ( $H_1$ ) at the beginning of that time interval,  $s - \frac{O_1}{2}\Delta T$  can be determined from the appropriate storage characteristics curve, Figure 11-14.



**Figure 11-14. Storage Characteristics Curve**

Step 5. Determine the value of  $s_2 + \frac{O_2}{2}\Delta T$  from the following equation:

$$S_2 + \frac{O_2}{2}\Delta T = S_1 - \frac{O_1}{2}\Delta T + \frac{I_1 + I_2}{2}\Delta T$$

Where:

- $S_2$  = Storage volume at time 2,  $\text{ft}^3$
- $O_2$  = Outflow rate at time 2, cfs.
- $\Delta T$  = Routing time period, sec
- $S_1$  = Storage volume at time 1,  $\text{ft}^3$
- $O_1$  = Outflow rate at time 1, cfs
- $I_1$  = Inflow rate at time 1, cfs
- $I_2$  = Inflow rate at time 2, cfs

Other consistent units are equally appropriate.

Step 6. Enter the storage characteristics curve at the calculated value of  $s_2 + \frac{O_2}{2} \Delta T$  determined in Step 5 and read off a new depth of water ( $H_2$ ).

Step 7. Determine the value of  $O_2$ , which corresponds to a stage of  $H_2$  determined in Step 6, using the stage-discharge curve.

Step 8. Repeat Steps 1 through 7 by setting new values of  $I_1$ ,  $O_1$ ,  $S_1$ , and  $H_1$  equal to the previous  $I_2$ ,  $O_2$ ,  $S_2$ , and  $H_2$ , and using a new  $I_2$  value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

#### 11.8.7.2 Storage – Indication Method Routing Sample Problem

This example demonstrates the application of the methodology presented for the design of a typical detention SWMF used for water quantity control.

SWMFs shall be designed for runoff from both the 2- and 10-yr design storms and an analysis done using the 100-yr design storm runoff to ensure that the structure can accommodate runoff from this storm without damaging adjacent and downstream property and structures.

The peak discharges from the 2- and 10-yr design storms are as follows:

- Pre-developed 2-yr peak discharge = 150 cfs
- Pre-developed 10-yr peak discharge = 200 cfs
- Post-development 2-yr peak discharge = 190 cfs
- Post-development 10-yr peak discharge = 250 cfs

Since the post-development peak discharge must not exceed the pre-development peak discharge, the allowable design discharges are 150 and 200 cfs for the 2- and 10-yr design storms, respectively.

Step 1. Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed SWMF.

Runoff hydrographs are shown in Table 11-12 below. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 hours, respectively, for runoff from the 2- and 10-yr storms.

**Table 11-12. Runoff Hydrographs**

(1)	Pre-Development Runoff		Post-Development Runoff	
	(2)	(3)	(4)	(5)
Time (hrs)	2-yr (cfs)	10-yr (cfs)	2-yr (cfs)	10-yr (cfs)
0	0	0	0	0
0.1	18	24	38	50
0.2	61	81	125	178
0.3	127	170	190 >150	250 >200
0.4	150	200	125	165
0.5	112	150	70	90
0.6	71	95	39	50
0.7	45	61	22	29
0.8	30	40	12	16
0.9	21	28	7	9
1.0	13	18	4	5
1.1	10	15	2	3
1.2	8	13	0	1

Preliminary estimates of required storage volumes are obtained using the simplified triangular hydrograph method outlined in Section 11.5.4.1. For runoff from the 2- and 10-yr storms, the required storage volumes,  $V_s$ , are computed using Equation 11.3:

$$V_s = \frac{1}{2} T_b (Q_i - Q_o)$$

$$V_{s_2} = \frac{\frac{1}{2} (1.2)(3600)(190 - 150)}{43,560} = 1.98 \text{ ac. ft.}$$

$$V_{s_{10}} = \frac{\frac{1}{2} (1.2)(3600)(250 - 200)}{43,560} = 2.58 \text{ ac. ft.}$$

Stage-discharge and stage-storage characteristics of a SWMF that should provide adequate peak flow attenuation for runoff from both the 2- and 10-yr design storms are presented below in Table 11-13. The storage-discharge relationship was developed and required that the preliminary storage volume estimates of runoff for both the 2- and 10-yr design storms to coincide with the occurrence of the corresponding allowable peak discharges.

Discharge values were computed by solving the broad-crested weir equation for head (H) assuming a constant discharge coefficient of 3.1, a weir length of 4', and no tailwater submergence. The capacity of storage relief structures was assumed to be negligible.



Step 2. Select a routing time period ( $\Delta t$ ) to provide at least five points on the rising limb of the inflow hydrograph. Use  $t_p$  divided by 5 to 10 for  $\Delta t$ .

$$\Delta T = \frac{t_p}{5} = \frac{0.5}{5} = 0.10 \text{ hr}$$

Step 3. Use the storage-discharge data from Step 1 to develop storage characteristics curves (Stage-Discharge-Storage) that provide values of  $s \pm \frac{O}{2} \Delta T$  versus stage.

**Table 11-13. Stage-Discharge-Storage Data**

(1)	(2)	(3)	(4)	(5)
Stage (H) (ft)	Discharge (Q) (cfs)	Storage (S) (ac-ft)	$S - \frac{O}{2} \Delta T$ (ac-ft)	$S + \frac{O}{2} \Delta T$ (ac-ft)
0.0	0	0.00	0.00	0.00
0.9	10	0.26	0.30	0.22
1.4	20	0.42	0.50	0.33
1.8	30	0.56	0.68	0.43
2.2	40	0.69	0.85	0.52
2.5	50	0.81	1.02	0.60
2.9	60	0.93	1.18	0.68
3.2	70	1.05	1.34	0.76
3.5	80	1.17	1.50	0.84
3.7	90	1.28	1.66	0.92
4.0	100	1.40	1.81	0.99
4.5	120	1.63	2.13	1.14
4.8	130	1.75	2.29	1.21
5.0	140	1.87	2.44	1.29
5.3	150	1.98	2.60	1.36
5.5	160	2.10	2.76	1.44
5.7	170	2.22	2.92	1.52
6.0	180	2.34	3.08	1.60

Storage routing was conducted for runoff from both the 2- and 10-yr design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results are shown below for runoff from the 2- and 10- year design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-yr design storms.

Step 4. For a given time interval,  $I_1$  and  $I_2$  are known. Given the depth of storage or stage ( $H_1$ ) at the beginning of that time interval,  $S_1 - \frac{O_1}{2} \Delta T$  can be determined from the appropriate storage characteristics curve.

*Step 5. Determine the value of  $s_2 + \frac{O_2}{2} \Delta T$  from the following equation:*

$$S_2 + \frac{O_2}{2} \Delta T = S_1 - \frac{O_1}{2} \Delta T + \frac{I_1 + I_2}{2} \Delta T$$

Summarized in

Table 11-15 for the 2-yr and 10-yr storms.

*Step 6. Enter the storage characteristics curve at the calculated value of  $s_2 + \frac{O_2}{2} \Delta T$  determined in Step 5 and read off a new depth of water ( $H_2$ ).*

Summarized in

Table 11-15 for the 2-yr and 10-yr storms.

*Step 7. Determine the value of  $O_2$ , which corresponds to a stage of  $H_2$  determined in Step 6, using the stage-discharge curve.*

Summarized in and

Table 11-154 and 11-15 for the 2-yr and 10-yr storms.

*Step 8. Repeat Steps 1 through 7 by setting new values of  $I_1$ ,  $O_1$ ,  $S_1$ , and  $H_1$  equal to the previous  $I_2$ ,  $O_2$ ,  $S_2$ , and  $H_2$ , and using a new  $I_2$  value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.*

Summarized in

Table 11-15 for the 2-yr and 10-yr design storms.

**Table 11-14. Storage Routing for the 2-yr Storm**

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (T) (hrs)	Inflow (I) (cfs)	$\frac{I_1 + I_2}{2} \Delta T$ (ac-ft)	Stage (H <sub>1</sub> ) (ft)	$S_1 - \frac{O_1}{2} \Delta T$ (6)-(8) (ac-ft)	$s_2 + \frac{O_2}{2} \Delta T$ (3)+(5) (ac-ft)	Stage (H) (ft)	Outflow (O) (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	38	0.16	0.00	0.00	0.16	0.43	3
0.2	125	0.67	0.43	0.10	0.77	2.03	36
0.3	190	1.30	2.03	0.50	1.80	4.00	99
0.4	125	1.30	4.00	0.99	2.29	4.80	130<150 OK
0.5	70	0.81	4.80	1.21	2.02	4.40	114
0.6	39	0.45	4.40	1.12	1.57	3.60	85
0.7	22	0.25	3.60	0.87	1.12	2.70	55
0.8	12	0.14	2.70	0.65	0.79	2.02	37
0.9	7	0.08	2.08	0.50	0.58	1.70	27
1.0	4	0.05	1.70	0.42	0.47	1.03	18
1.1	2	0.02	1.30	0.32	0.34	1.00	12
1.2	0	0.01	1.00	0.25	0.26	0.70	7
1.3	0	0.00	0.70	0.15	0.15	0.40	3

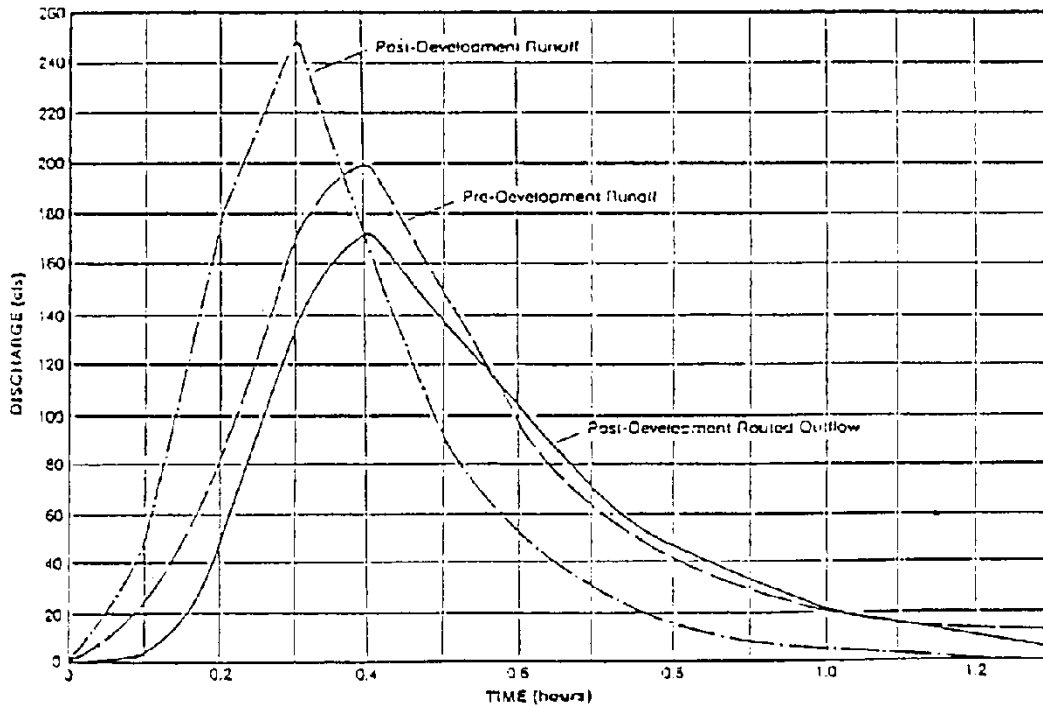
**Table 11-15. Storage Routing for the 10-yr Storm**

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (T) (hrs)	Inflow (I) (cfs)	$\frac{I_1 + I_2}{2} \Delta T$ (ac-ft)	Stage (H <sub>1</sub> ) (ft)	$S_1 - \frac{O_1}{2} \Delta T$ (6)-(8) (ac-ft)	$s_2 + \frac{O_2}{2} \Delta T$ (3)+(5) (ac-ft)	Stage (H) (ft)	Outflow (O) (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	50	0.21	0.21	0.00	0.21	0.40	3
0.2	178	0.94	0.40	0.08	1.02	2.50	49
0.3	250	1.77	2.50	0.60	2.37	4.90	134
0.4	165	1.71	4.90	1.26	2.97	2.97	173<200 OK
0.5	90	1.05	5.80	1.30	2.35	4.00	137
0.6	50	0.58	4.95	1.25	1.83	4.10	103
0.7	29	0.33	4.10	1.00	1.33	3.10	68
0.8	16	0.19	3.10	0.75	0.94	2.40	46
0.9	9	0.10	2.40	0.59	0.69	1.90	32
1.0	5	0.06	1.90	0.44	0.50	1.40	21
1.1	3	0.03	1.40	0.33	0.36	1.20	16
1.2	1	0.02	1.20	0.28	0.30	0.90	11
1.3	0	0.00	0.90	0.22	0.22	0.60	6

Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storms, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations should be repeated.

Although not shown for this sample problem, runoff from the 100-yr frequency storm should be routed through the SWMF to establish freeboard requirements and to evaluate emergency overflow and stability requirements. In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth to water, side slope stability, maintenance, grading to prevent standing water, and provisions for public safety.

An estimate of the potential downstream effects (i.e., increased peak flow rate and recession time) of detention SWMFs may be obtained by comparing hydrograph recession limbs from the pre-development and routed post-development runoff hydrographs. Example comparisons are shown below for the 10-yr design storms.



**Figure 11-15. Runoff Hydrographs**

Potential effects on downstream facilities should be minor when the maximum difference between the recession limbs of the pre-developed and routed outflow hydrographs is less than about 20%. As shown in Figure 11-15, the sample problem results are well below 20%; downstream effects can thus be considered negligible and downstream flood routing or  $Q_1$  control omitted.

### 11.8.7.3 SWM Basin Design: Sample Problem

*Step 1. Determine the type of BMP required:*

- New impervious area draining to this outfall = 2.98 ac.
- Total drainage area at the outfall within the R/W and easements = 9.03 ac.
- Percentage Impervious Cover =  $2.98/9.03 = 0.33$  (33%)  
From Table 11-7 select an extended detention basin

*Step 2. Determine Quantity Control Requirements:*

- The receiving channel is a natural channel that was determined not to be adequate. The post construction  $Q_2$  will overtop the banks of the channel. The pre-construction  $Q_2$  is also above the banks of the channel, but that is not a factor.
- In accordance with MS-19 of the VESCR, the BMP will need to attenuate the post-development  $Q_2$  to not be greater than pre-development  $Q_2$ . The design of the dam and the emergency spillway will need to provide protection of the dam for  $Q_{100}$ .
- The  $Q_{2pre} = 20.5$  cfs and the  $Q_{2post} = 29.6$  cfs. The usual design process would be to now estimate the quantity control volume needed for the basin.

*Step 3. Determine if quantity control for  $Q_1$  is required:*

- Flood control for the 1-year frequency storm in lieu of the 2-yr frequency storm may be needed if there is existing or anticipated erosion downstream.
- A field review of the receiving channel has shown no significant erosion and none is anticipated.

Therefore, the alternative  $Q_1$  control is not needed.

*Step 4. Determine the required water quality volume and treatment volume:*

- From Table 11-7 the required treatment volume for an extended detention basin is  $2 \times \text{WQV}$ . The WQV being equal to  $\frac{1}{2}$  inches  $\times$  New Impervious area.  $2 \times \frac{1}{2}$  inch = 1 in or 0.083 ft
- New pavement within the drainage area for this outfall = 2.98 ac or 129,809  $\text{ft}^2$

$$\begin{aligned}\text{Treatment Volume} &= 2 \times \text{WQV} \\ &= 2 \left[ \frac{0.5(2.98)(43560)}{12} \right] = 10,817 \text{ cu. ft.}\end{aligned}$$

*Step 5. Determine the temporary sediment storage requirements:*

- The total drainage area to this outfall from a storm drain system is 12.98 ac.

- All of the drop inlets in the storm drain will have erosion control measures.
- Temporary sediment storage is not required because all of the inlets can be protected from sediment. However, temporary sediment storage will be provided with the volume equal to the treatment value due to the convenience of the basin and as a supplement to the erosion and sediment controls.
- If a temporary sediment basin were needed, the quantities would be:  
 $67 \text{ cu. yd.} \times 13 \text{ ac} = 23,517 \text{ ft}^3$  for wet storage  
 $67 \text{ cu. yd.} \times 13 \text{ ac} = 23,517 \text{ ft}^3$  for dry storage  
 The total volume required for temporary sediment storage, wet plus dry =  $47,034 \text{ ft}^3$ . This is much larger than the  $10,817 \text{ ft}^3$  required for the WQV.

*Step 6. Determine the size of the sediment forebay:*

- A sediment/debris forebay is recommended for extended detention basins and the volume should be between 0.1 to 0.25 in/ac of new impervious area or 10% of the required detention volume. This range establishes the minimum to maximum desirable sediment storage volumes needed. The actual size of the forebay is dependent upon the site conditions. It is desirable to size the forebay as near to the maximum sediment storage volume as possible.
- Compute the sediment forebay volume and determine its dimensions:

$$\text{Vol.} = 0.1 \text{ in.} \left( \frac{1 \text{ ft.}}{12 \text{ in.}} \right) 2.98 \text{ ac.} \left( \frac{43560 \text{ sq. ft.}}{1 \text{ ac.}} \right) = 1082 \text{ cu. ft.}$$

If forebay is 1 ft. deep:      Size = 33 ft. x 33 ft.

For 0.25 inch, volume =  $2,704 \text{ ft}^3$

If basin is 1 ft. deep: Size = 50 ft. x 50 ft.

The shape of the forebay does not need to be square and should be shaped to fit the site. The volume of the forebay that cannot be drained should not be considered as part of the required storage volume for the basin.

The established design parameters for the basin:

- An extended detention basin is required for this site.
- QUANTITY CONTROL FOR Q<sub>2</sub> PEAK IS REQUIRED. The required volume will be estimated in the design process.
- Alternative Q<sub>1</sub> control is not needed.
- The required WQV is 10,817 ft<sup>3</sup>
- The temporary sediment volume (if needed) is 47,034.
- The estimated forebay volume is 1,082 to 2,704 ft<sup>3</sup>

### Determining the Water Quality Volume

Calculate required WQV (for extended detention) = 10,817 ft<sup>3</sup>

From Preliminary Elevation/Storage Table:

The WQV required is met @ Elev. 423.25

Depth = 1.95 ft.

Actual Volume = 11,051 ft<sup>3</sup> @ Elev. 423.25

### WQV Computations – Determining the Orifice Size Required Using Method #2 Average Hydraulic Head (VDOT Preferred)

- Assume depth,  $h = 1.95$  ft. (Say 2.0 ft)

$$h_{avg} = \frac{2.0}{2} = 1.0 \text{ ft.}$$

- Compute the  $Q_{avg}$  for the WQV using the required 30-hour drawdown time:

$$Q_{avg} = \frac{\text{Treatment Volume}}{\text{Time}} = \frac{11,051 \text{ cu. ft.}}{30 \text{ hr} \left( \frac{3600 \text{ sec}}{\text{hr}} \right)} = 0.102 \text{ cfs}$$

- Orifice sizing computations:

$$A = \frac{Q_{avg}}{C\sqrt{2gh_{avg}}} = \frac{0.102}{0.6\sqrt{2(32.2)(1.0)}} = 0.021 \text{ sq. ft.}$$

- The depth ( $h$ ) used in the orifice equation would normally be measured from the center of the orifice. Due to the small size of the water quality orifice it is acceptable to consider the  $h$  as the depth to the invert of the orifice.

From Table 11-10, use a 2-inch orifice with an area = 0.022 ft<sup>2</sup>

## Q<sub>1</sub> Control – Alternative Quantity Control

Assume that a field review of the receiving channel shows that there is significant erosion and it has been decided that the channel should be protected from the Q<sub>1</sub> instead of the Q<sub>2</sub> as required by MS-19. Control of the Q<sub>1</sub> requires containing the entire volume of the Q<sub>1</sub> from the total drainage area and releasing that volume over a 24-hour period. The computations are similar to those used for WQV storage and released over a 30-hour period. When Q<sub>1</sub> is detained and released over the 24-hour period, there will be no need to provide additional or separate storage for the WQV if it can be demonstrated that the treatment volume will be detained for approximately 24 hours.

### Determine the Q<sub>1</sub> Control Volume:

#### Use Method #2 – Average Hydraulic Head (Recommended Method)

Find the Q<sub>1</sub> Control volume.

Given from design computations:

$$DA = 12.98 \text{ ac}$$

$$C = 0.67$$

$$T_c = 16 \text{ min}$$

$$Q_2 = 29.6 \text{ cfs}$$

- Use TR-55 to find the volume for Q<sub>1</sub>:
- Convert the runoff coefficient,  $C = 0.67$  from the Rational Method to  $CN = 80$ .
- Find the 1-year frequency 24-hour rainfall (RF) using the site-specific rainfall precipitation frequency data recommended by the U.S. National Oceanic and Atmospheric Administration (NOAA) Atlas 14. Partial duration time series shall be used for the precipitation data..

$$RF = 2.8 \text{ inches.}$$

- Find the runoff depth for  $CN = 80$  and  $RF = 2.8$  inches using TR-55.

$$\text{Runoff (RO)} = 1.1 \text{ inches}$$

- Compute the Q<sub>1</sub> Control volume:

$$V_{ce} = 12.98 \text{ ac.} (1.1 \text{ in.}) \left( \frac{1 \text{ ft.}}{12 \text{ in.}} \right) \left( \frac{43,560 \text{ sq. ft.}}{1.0 \text{ ac.}} \right) = 51,829 \text{ cu. f. t.}$$

- To account for the routing effect, reduce the channel erosion control volume by 60%:

$$V_{ce} = 0.60(51,829) = 31,097 \text{ cu. ft.}$$



## Sizing the Basin for the $Q_1$ Volume

1. Use the Rational Method triangular hydrograph (HYG) to estimate the volume needed:

- From 24 hour rainfall (RF) table

$$RF_1 = 2.8 \text{ inches}$$

$$RF_2 = 3.5 \text{ inches}$$

- $\frac{RF_1}{RF_2} = \frac{2.8}{3.5} = 0.80$  (80%)

Thus  $Q_1 = 80\%$  of  $Q_2$

$$Q_2 = 29.6 \text{ cfs}$$

$$Q_1 = 0.80 Q_2 = 0.80(29.6) = 23.7 \text{ cfs}$$

- Compute the volume from a triangular HYG:

Using  $t_c = 16 \text{ min.}$ ,  $T_b = 2t_c = 32 \text{ min.}$

$$V_1 = 0.5(Q_1)(T_b) \left(60 \frac{\text{sec}}{\text{min}}\right)$$

$$V_1 = 0.5(23.7 \text{ cfs})(32 \text{ min.}) \left(60 \frac{\text{sec}}{\text{min}}\right) = 22,752 \text{ cu. ft.}$$

- Compute the volume from a trapezoidal HYG:

Using  $t_c = 16 \text{ min.}$  and determining the critical storm duration,  $T_d = 22 \text{ min.}$

$$T_b = t_c + T_d = 38 \text{ min.}$$

$$V_1 = 0.5(Q_1)[(T_d - t_c) + T_b] 60 \frac{\text{sec}}{\text{min}}$$

$$V_1 = 0.5(23.7 \text{ cfs})[(22 \text{ min} - 16 \text{ min}) + 38 \text{ min}] 60 \frac{\text{sec}}{\text{min}} = 31,284 \text{ cu. ft.}$$

NOTE: Calculation is for entire volume of hydrograph

It is noted that this drainage area is sensitive to the critical storm duration of 22 minutes. For the  $Q_1 = 23.7 \text{ cfs}$  with  $t_c = 16 \text{ minutes}$  and the duration = 22 minutes, the volume of the HYG = 31,284  $\text{ft}^3$  which is very close to the volume of 31,097  $\text{ft}^3$  as calculated using the average hydraulic head method.

2. Determine the required orifice size:

- To achieve the  $Q_1$  volume at a safe ponded depth, assume a depth,  $h = 3.0$  ft.
- Find  $Q_{avg}$  for the required 24-hour drawdown for  $Q_1$  Control:

$$Q_{avg} = \frac{V_{ce}}{\text{Time}} = \frac{31,097 \text{ cu. ft.}}{24 \text{ hr} \left(3600 \frac{\text{sec}}{\text{hr}}\right)} = 0.360 \text{ cfs}$$

3. Determine the orifice size:

- Determine  $h_{avg}$

$$h_{avg} = \frac{3.0}{2} = 1.5 \text{ ft.}$$

- Using the rearranged orifice equation:

$$A = \frac{Q_{avg}}{C\sqrt{2gh_{avg}}} = \frac{0.360}{0.6\sqrt{2(32.2)(1.5)}} = 0.061 \text{ sq. ft.}$$

- From Table 11-10, use a 3 ½-inch orifice with an area = 0.067 ft<sup>2</sup>.

## 11.9 References

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**Appendix 11B-2 Redevelopment/Surplus Credit Tracking Form**

Sample sheet:

<b>Redevelopment/Surplus Credit Tracking Form</b>						page 1 of 3
						Rev. 4/27/16
<b>Project Information</b>						
<b>Point of Contact Information</b>						
Name	Address		Phone Number	Email		
Title:						
<b>Project Information</b>						
Project Name	ID #	UPC	District			
	GPIN	Lat/Long	County			
	River Basin	Trib Name	HUC			
<b>Narrative / Project Summary:</b>						
<b>Qualifying Criteria</b>						
Was the redevelopment project implemented after 7/1/09			Yes <input type="checkbox"/>	No <input type="checkbox"/>		
Did the project result in net reductions in pollutants?			Yes <input type="checkbox"/>	No <input type="checkbox"/>		
Is the credit for treatment of VDOT ROW exclusively?			Yes <input type="checkbox"/>	No <input type="checkbox"/>		
Stormwater Quality Criteria Utilized			Part IIB (new) <input type="checkbox"/>	Part IIC (old) <input type="checkbox"/>		
Is the project located in an urbanized area			Yes <input type="checkbox"/>	No <input type="checkbox"/>		
<b>Notes:</b>						
<b>Crediting Information</b>						
(*Include only the creditable portion associated with redevelopment or surplus that was not offsetting new development)						
<b>Summary of Nutrient Crediting</b>						
TP Credit (lb/yr)*:			DA Entirely in MS4 UA?	Yes <input type="checkbox"/>	No <input type="checkbox"/>	
TN Credit (lb/yr)*:						
TSS Credit (lb/yr)*:						
* These values represent load reductions creditable for CB TMDL purposes						

# Chapter 11 – Stormwater Management

BMP Information*			page 2 of 3		
NOTE: There are 8 GS L1, therefore D.A.'s are added together (1+2+3+4+5+6+7+8)					
Type of Permanent BMP	Grass Swale Level 1		TP (CBPO)	TN (CBPO)	TSS (CBPO Established)
Total DA to BMP (ac)		CDA Load Deliv. To BMPs			
Total Imp to BMP (ac)		% Vol Red			
Total Pervious to BMP (ac)		% Rem Eff			
		% Total Mass Rem			
		Load Removed (CBPO)			
		Load Removed (VRRM)			
Regional BMP (Y/N)		CBPO BMP Classification			
If Yes, provide details on regional facility:					
*Replicate above information for each BMP as needed.					
<b>SITE SUMMARY</b>		<b>Creditable Reduction Computations</b>			
Total Disturbed Site Area (ac)					
Total Existing Impervious (ac)					
Total Existing Pervious (ac)					
New Impervious (ac)					
Existing TP Load, CBPO (lbs/yr)					
Required TP Reduction, Redev (lbs/yr)					
Allowable TP Load, New IC (lbs/yr)					
Required TP Total Reductions, (lbs/yr)					
TP Load Removed by BMPs (lbs/yr)					
TP Creditable Reduction, Adjusting for New IC Req'd (lbs/yr)					
Proportion of TP Creditable Reduction to Total Load					
TN Creditable Reduction (lbs/yr)					
TSS Creditable Reduction (lbs/yr)					
Location of Crediting Forms:					
Link to Crediting Forms:					
Notes:					
Preparer Name:		Title:		Date:	
Address:		Email:		Phone:	

## Chapter 11 – Stormwater Management

<b>Implementation Information</b>				Page 3 of 3	
Estimated Implementation Date(mo/yr):	<input type="text"/>	Est. Time to Construct (mo):	<input type="text"/>		
Actual Implementation Date:	<input type="text"/>	Cost (\$):	<input type="text"/>		
Project Accepted and Completed*		Yes <input type="checkbox"/>		No <input type="checkbox"/>	
<i>*includes planting and initial maintenance</i>					
Inspection/Acceptance Form Location:	<input type="text"/>				
Notes:	<input type="text"/>				
Certified by:	<input type="text"/>	Title:	<input type="text"/>	Date:	<input type="text"/>
Address:	<input type="text"/>	Email:	<input type="text"/>	Phone:	<input type="text"/>
<b>Maintenance/Inspection Information*</b>					
Maintenance ID:	<input type="text"/>	Date Transferred to Maintenance:	<input type="text"/>		
<i>* All ongoing maintenance and verification is assumed to be associated with the VDOT Maintenance database and inspection/O&amp;M program. Note the date xferred.</i>					
<b>Documents/Tracking Information</b>					
Location of Plans on File:	<input type="text"/>				
Location of Photos on File:	<input type="text"/>				
Location of Permits on File:	<input type="text"/>				

Appendix 11C-1 Equivalent Runoff Curve Number (RCN) for Rational 'C'

RATIONAL METHOD "C" VALUES		N.R.C.S. "TR-55" METHOD "CN" VALUES					
LAND COVER	RUNOFF COEFFICIENT "C"	COVER TYPE & HYDROLOGIC CONDITION	Avg. % Imp.	Curve Numbers for Hydrologic Soil Group*			
				A	B	C	D
Business, industrial and commercial	0.80 to 0.90	Commercial and business Industrial	85 72	92 88	94 91	95 93	
Residential lots 10,000 sq. ft. - lots 12,000 sq. ft. - lots 17,000 sq. ft. - lots 1/2 ac. or more	0.40 to 0.50 0.40 to 0.45 0.35 to 0.45 0.30 to 0.40	Residential area by lot size: 1/8 acre or less (town houses) 1/4 acre 1/3 acre 1/2 acre 1 acre 2 acres Farmsteads – buildings, lanes, driveways, and surrounding lots	65 38 30 25 20 12 n.a.	85 77 61 57 54 51 46 59	90 83 81 80 79 77 82 82	92 87 86 85 84 82 86	
Parks, cemeteries and unimproved areas Lawns	0.20 to 0.35 0.20 to 0.40	Open space (lawns, parks, golf courses Cemeteries, etc.) grass cover > 75%	n.a.	39	74	80	
Paved and roof areas	0.9	Streets & roads: Paved parking lots, roofs, driveways, etc. Paved: open ditches (excluding R/W) Gravel (including R/W) Dirt (including R/W)	n.a. n.a. n.a. n.a.	98 83 76 72	98 92 89 87	98 93 91 89	
Cultivated areas	0.50 to 0.70	Cultivated areas (combination of straight & Row crops)	n.a.	71	87	90	
Pasture	0.35 to 0.45	Pasture, grassland, or range Meadow – continuous grass Brush-brush-weed-grass mixture with brush the major element	n.a. n.a. n.a.	39 30 30	74 71 65	80 78 73	
Forest	0.20 to 0.30	Woods Woods/grass combination	n.a. n.a.	30 32	70 72	77 79	

If the accurate soil information is not available use Soil Group B.