Chapter 11 – Stormwater Management

TABLE OF CONTENTS

List of Tables

List of Figures

List of Appendices

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 can be used to store the increases in volume and control peak discharge rates.

11.1.1 Objective

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.

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.

11.2 Design Policy

11.2.1 Introduction

Acts of the General Assembly have resulted in the issuance of Stormwater Management Regulations and Erosion and Sediment Control Regulations. The Virginia Stormwater Management Program Regulations can be obtained at the Department of Environmental Quality (DEQ) website at http://law.lis.virginia.gov/admincode/title9/agency25/chapter870/. The Erosion and Sediment Control Regulations can also be obtained at their web site at http://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 IIB or Part IIC 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 IIB & IIC) governs a given project, the design elements for Best Management Practices (BMPs) will be different. For example, Part IIB includes 9VAC25-870-62 through 9VAC25-870-92, and Part IIC includes 9VAC25-870-93 through 9VAC25-870-99.

For those projects following Part IIC, 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.

For those projects following Part IIB, the design of BMPs will follow design criteria as identified in the Chapter 11 Appendices including the 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-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

SWM facilities may function as both quantity control and quality control facilities (also known as a Best Management Practice or BMP). Some facilities may only be needed for either quality or quantity control.

11.3.4.2 Temporary versus Permanent

Permanent SWM facilities may be utilized as temporary sediment basins during the construction phase of the project, and if so, the design of the SWM facility will need to address this dual function. The design that is needed for a permanent SWM facility 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 SWM facility 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 SWM facility 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 SWM facilities 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 SWM facilities 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 SWM facilities must address any mitigation needed to meet the water quality and quantity requirements of any known future VDOT projects within the contributing watershed. Regional SWM facilities 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 SWM facility 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 SWM facilities 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 SWM facilities 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 SWM facilities 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 SWM basins 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 SWM facilities.

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 SWM facility as a temporary sediment basin.
- Specify at each SWM facility 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 SWM facility.

11.3.9.3 Stormwater Management Summary

- All drainage items related to the construction of SWM facilities 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 SWM facilities 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 SWM facilities. 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 SWM facilities, 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 SWM facility. 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 SWM facility on an annual basis, and inspecting each SWM facility 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.

11.4 Part IIB 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.

    b. Portions of projects which lie on prior developed lands should be calculated separately from portions of projects which are not disturbing prior developed lands. Portions of projects not on prior developed lands are considered “new development”. Prior developed lands means land that has been previously utilized for residential, commercial, industrial, institutional, recreation, transportation or utility facilities or structures, and that will have the impervious areas associated with those uses altered during a land-disturbing activity.

    c. The designer can utilize the VRRM spreadsheet provided by DEQ (ensuring to utilize the most current, corrected version) to calculate compliance (use of separate spreadsheets for new development and prior developed portions of a project is recommended per above) for scenarios where the pre-development soil conditions are predominantly HSG A, B, or C. In these instances the spreadsheet places a floor of 0.41 lb/ac/yr in the reduction calculations.

    d. For sites where there is a mix of soil groups including some D soils, the designer should use caution to evaluate whether the aggregate “pre-development” loading is greater than 0.51 lb/ac/yr. In situations where the pre-development load exceeds 0.51 lb/ac/yr, it is suggested that a manual calculation utilizing this criteria (i.e., 20% below pre-development load) be used to determine the required load reduction.

e. For sites where the pre-development conditions are predominantly turf in HSG D soils, it is recommended that the designer perform a manual calculation utilizing the criteria in subdivision d to determine the required load reduction. For the D soil turf condition, this would result in a required load at 0.456 lb/ac/yr, corresponding to 20% below the D soil turf load of 0.57 lb/ac/yr.

All water quality requirements related to redevelopment projects shall be summarized on the Water Quality Redevelopment Tabulation Sheet in Appendix 11B-2.

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 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
   - Infiltration
   - Dry Swale
   - Wet Swale
   - Sheet Flow to Filter/Open Space
   - Extended Detention Pond
   - Filtering Practice
   - Constructed Wetland
   - Wet Pond

5) Manufactured or proprietary BMPs accepted by DEQ may be utilized, when accepted by VDOT, in accordance with the design guidance and efficiencies approved by DEQ.

6) Where a project drains to more than one 6^{th} Order Hydrologic Unit Code (HUC), the pollutant load reduction requirements shall be applied independently within each HUC unless reductions are achieved in accordance with a comprehensive watershed SWM plan.

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 VDOT or 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 watershed, offsite SWM facilities 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 TR-55 and TR-20; 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)*:

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

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

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

Where:

I.F. (Improvement Factor) = 0.8 for sites > 1 acre LDA or 0.9 for sites ≤ 1 acre LDA
$Q_{Developed}$ = the allowable peak flow rate of runoff from the developed site for the 1-yr 24-hour storm.
$RV_{Developed}$ = the volume of runoff from the site in the developed condition for the 1-yr 24-hour storm.
$Q_{Pre\text{-}Developed}$ = the peak flow rate of runoff from the site in the pre-developed condition for the 1-yr 24-hour storm.
$RV_{Pre\text{-}Developed}$ = the volume of runoff from the site in pre-developed condition for the 1-yr 24-hour storm.
Q_{Forest} = the peak flow rate of runoff from the site in a forested condition for the 1-yr 24-hour storm.
RV_{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. Channel Protection criteria under subdivisions 1, 2, or 3 will apply only when the regulated land-disturbing activity contributes more than 1% of the total

watershed area or existing peak discharge at the point of analysis. If the energy balance under subdivision 3 of this section is applied at the point of discharge, no further analysis is required. If analysis is required under subdivisions 1 or 2, the stormwater conveyance systems shall be analyzed for compliance with channel protection criteria to a point where either:

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.

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.

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 SWM facility 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.

11.5 Part IIB 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, SWM facilities 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.

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 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 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.

Compliance with the water quality criteria are evaluated generally using the entire site. However, where a site drains to more than one 6^{th} 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 (8).

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.

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 Drainage 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 Drainage 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

Conventional channel protection in Virginia focused on the control of runoff peak rate of flow for specific design events (2-yr storm for natural channels, and the 2- and 10-yr storms for manmade channels), as defined under Minimum Standard 19 of the Virginia Erosion and Sediment Control Regulations (9VAC25-840-40).

The application of Minimum Standard 19 has not always resulted in protection of channels and properties downstream of land development projects from erosion and sedimentation. As a result, the amended Virginia Stormwater Act (§62.2-44.15:28) requires the maintenance of runoff peak rate of flow and other characteristics (such as runoff volume, frequency, and duration) that replicate the existing pre-development runoff and site hydrology, or improve upon the existing pre-development conditions if stream channel erosion already exists. In order to address this requirement, a Technical Advisory Panel (TAP) was formed by the DEQ to develop new channel protection criteria in 2011-2012. The results of the TAP work is the new channel protection criteria in the Virginia Stormwater Management Program Regulation.

The new 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 to the limits of analysis. If inadequate for the post-development conditions, the designer must provide for detention or channel improvements, or apply the new Energy Balance method. For discharges to natural channels, an adequacy analysis is not required, but the new Energy Balance criteria must be applied.

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 SWM facility is required for compliance with the water quantity criteria, and to optimize the size of the necessary SWM facilities, 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 SWM facility versus the cost for the channel adequacy analysis. While the channel adequacy analysis may require additional field survey, hydrologic analysis, and hydraulic modeling, the cost for the analysis may be less than the cost of implementing a SWM facility 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 storm event without causing erosion of the system. The system must be shown adequate to the limits of analysis (see below). 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 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. 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" 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:

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

Where:
I.F. (Improvement Factor) = 0.8 for sites > 1 acre LDA or 0.9 for sites ≤ 1 acre LDA
$Q_{Developed}$ = the allowable peak flow rate of runoff from the developed site for the 1-yr 24-hour storm.
$RV_{Developed}$ = the volume of runoff from the site in the developed condition for the 1-yr 24-hour storm.
$Q_{Pre\text{-}Developed}$ = the peak flow rate of runoff from the site in the pre-developed condition for the 1-yr 24-hour storm.
$RV_{Pre\text{-}Developed}$ = 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. Form LD-445G may to be used for this purpose. The request shall include documentation of the need for the 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:

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

Where:

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

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

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

RV_{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 new channel protection criterion requires that manmade and restored channels be analyzed and demonstrated to be adequate downstream to the point in the drainage system where the site contributing drainage area is ≤1% of the total watershed area. Alternatively, the limits of analysis can be 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.

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 SWM facility 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:

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

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^3)/watershed drainage area (acres) x 12 (inches/foot)/43,560 (ft^2/acre)
P = rainfall (inches) for the 1-, 2-, or 10-yr 24-hour storm event

S_{adj} = 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^3, converted to watershed inches, and subtracted from Q; and the equation solved for an adjusted S value (S_{adj}). The S_{adj} value is then used to determine the adjusted CN using the following relationship from TR-55:

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

Where:
CN_{adj} = adjusted CN calculated individually for the 1-, 2-, and 10-yr 24-hour storm events

The solution for S_{adj} involves a quadratic equation, and multiple techniques are available to solve. However, **the Virginia RRM Spreadsheet and VDOT's BMP Optimization Software solve the equation for S_{adj} and provide CN_{adj} 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 SWM facility 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 IIC 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 <u>not</u> 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 to the point in the drainage system where the site contributing drainage area is ≤ 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.

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 SWM facility 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 sheetflows were evaluated and how no adverse impact on downstream waterways and properties were determined.

11.5.3 Part IIB Design Procedures and Sample Problems

11.5.3.1 SWM Plan Requirements

The following documentation will be required for SWM facility/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).
- SWM Facility 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.
- 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.

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

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.

<u>For this process, it is assumed that the VRRM Spreadsheets are being used to demonstrate compliance with the water quality criteria</u>. 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, <u>not</u> 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 conditions, as well as the post-development 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.

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^3)*
- *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.

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 <u>and</u> 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.

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 SWM facility 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, and 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 Rv, 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
Forest/Open Space (acres) -- undisturbed, protected forest/open space or reforested land					0.00
Managed Turf (acres) -- disturbed, graded for yards or other turf to be mowed/managed			0.90		0.90
Impervious Cover (acres)			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:

LAND COVER SUMMARY -- POST DEVELOPMENT

Land Cover Summary

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)	**1.70**
Site Rv	**0.56**

Treatment Volume and Nutrient Loads

Treatment Volume (acre-ft)	0.0798
Treatment Volume (cubic feet)	3,478
TP Load (lb/yr)	2.18
TN Load (lb/yr) (Informational Purposes Only)	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^3.

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
				Total	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^3)
5. Dry Swale (RR)				
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^3)	Remaining Runoff Volume (ft^3)	Total BMP Treatment Volume(ft^3)	Untreated Phosphorus Load to Practice (lb)	Phosphorus Removed By Practice (lb)	Remaining Phosphorus Load (lb)
5. Dry Swale (RR)						
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^3 of Runoff Reduction is required to reduce the runoff volume to pre-development condition, but only 1,391 ft^3 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^3)	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
5. Dry Swale (RR)					
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^3)	Runoff Reduction (ft^3)	Remaining Runoff Volume (ft^3)	Total BMP Treatment Volume (ft^3)	Phosphorus Load from Upstream Practices (lb)
11. Filtering Practices (no RR)					
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
RUNOFF REDUCTION VOLUME ACHIEVED (ft^3)	1,391
TP LOAD AVAILABLE FOR REMOVAL (lb/yr)	2.18
TP LOAD REDUCTION ACHIEVED (lb/yr)	1.76
TP LOAD REMAINING (lb/yr)	0.42

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)	1.76
TP LOAD REMAINING (lb/yr):	0.42
REMAINING TP LOAD REDUCTION REQUIRED (lb/yr):	0.00 **

** 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^3 of Runoff Reduction is required to reduce the runoff volume to pre-development condition, but only 1,391 ft^3 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 with a green font 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 SWM facility 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 SWM facilities 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
Forest/Open Space (acres) -- undisturbed, protected forest/open space or reforested land			1.20		1.20
Managed Turf (acres) -- disturbed, graded for yards or other turf to be mowed/managed			0.30		0.30
Impervious Cover (acres)			0.20		0.20
					1.70

Post-Development Land Cover (acres)

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

LAND COVER SUMMARY -- PRE-REDEVELOPMENT		
Land Cover Summary-Pre		
Pre-ReDevelopment	**Listed**	**Adjusted[1]**
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%
Total Site Area (acres)	**1.70**	**1.10**
Site Rv	**0.18**	**0.25**

[1]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 for linear development.

LAND COVER SUMMARY -- POST DEVELOPMENT					
Land Cover Summary-Post (Final)		***Land Cover Summary-Post***		***Land Cover Summary-Post***	
Post ReDev. & New Impervious		**Post-ReDevelopment**		**Post-Development New Impervious**	
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	New Impervious Cover (acres)	0.60
Rv(impervious)	0.95	Rv(impervious)	0.95	Rv(impervious)	0.95
% Impervious	47%	% Impervious	18%		
Final Site Area (acres)	**1.70**	**Total ReDev. Site Area (acres)**	**1.10**		
Final Post Dev Site Rv	**0.56**	**ReDev Site Rv**	**0.35**		

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

Treatment Volume and Nutrient Load		
Pre-ReDevelopment Treatment Volume (acre-ft)	0.0253	0.0233
Pre-ReDevelopment Treatment Volume (cubic feet)	1,104	1,016
Pre-ReDevelopment TP Load (lb/yr)	**0.69**	**0.64**
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.

The alternate TP Load allowed 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) x 0.69 lb/yr = 0.55 lb/yr. The TP Load required reduction from the post-development load of 2.18 lb/yr is (2.18 lb/yr – 0.55 lb/yr) = 1.63 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 lb/acre/yr x 1.70 acres) = 0.70 lb/yr. The TP Load required reduction in this case is the post-development load minus the minimum allowable, or (2.18 lb/yr – 0.70 lb/yr) = 1.48 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^3*?*

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 the project is a linear development and 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^3.

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
				Total	1.70	

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^3)
6. Bioretention (RR)				
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^3)	Remaining Runoff Volume (ft^3)	Total BMP Treatment Volume(ft^3)	Untreated Phosphorus Load to Practice (lb)	Phosphorus Removed By Practice (lb)	Remaining Phosphorus Load (lb)
6. Bioretention (RR)						
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^3 of Runoff Reduction is required to reduce the runoff volume to pre-development condition, but only 2,207 ft^3 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
RUNOFF REDUCTION VOLUME ACHIEVED (ft^3)	2,207
TP LOAD AVAILABLE FOR REMOVAL (lb/yr)	2.18
TP LOAD REDUCTION ACHIEVED (lb/yr)	1.56
TP LOAD REMAINING (lb/yr)	0.63

Total Phosphorus	LINEAR PROJECT:
FINAL POST-DEVELOPMENT TP LOAD (lb/yr)	2.18
TP LOAD REDUCTION REQUIRED (lb/yr)	1.49
TP LOAD REDUCTION ACHIEVED (lb/yr)	1.56
TP LOAD REMAINING (lb/yr):	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 in green font 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^3 of RR retention storage is required to reduce the runoff volume to pre-development conditions. Only 2,207 ft^3 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. The need for SWM facilities 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 cost comparison should include both the capital costs (R/W, easements, and construction) as well as the long-term maintenance costs.

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 SWM facility 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:

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

Where:
$DA_{outfall}$ = site drainage area at the outfall (acres or square miles)
DA_{system} = total drainage area for the system at the limits of analysis (use units consistent with DAoutfall)

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:

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

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

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 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.*

*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 SWM facilities/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 SWM facilities/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 SWM facilities 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 site 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:

$$\mathrm{DA_{outfall}} \le \frac{\mathrm{DA_{system}}}{100}$$

$$\mathrm{DA_{system}} \ge \mathrm{DA_{outfall}} \times 100$$

$$\mathrm{DA_{system}} \ge 0.5\ \mathrm{acres} \times 100 \ge 50\ \mathrm{acres}$$

The adequacy analysis can end at 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:

$$\mathrm{Q_{outfall}} \le \frac{\mathrm{Q_{system}}}{100}$$

$$\mathrm{Q_{system}} \ge \mathrm{Q_{outfall}} \times 100$$

$$\mathrm{Q_{system}} \ge 0.8\ \mathrm{cfs} \times 100 = 80\ \mathrm{cfs}$$

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)

$$105\ \mathrm{cfs} \ge 80\ \mathrm{cfs}$$

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?

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

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

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

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

Solve for CN_{adj}:

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

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 SWM facility 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 SWM facility 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:

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

Where:
I.F. = 0.80 (site > 1-acre)
$Q_{\text{Pre-Developed}}$ = 1.28 cfs
$RV_{\text{Pre-Developed}}$ = 0.603 in (0.0854 ac-ft)
$RV_{\text{Developed}}$ = 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 SWM facilities 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^3 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 cfs – 1.21 cfs) = 0.84 cfs or (0.84 cfs/3.22 cfs x 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:

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

Where:
$DA_{outfall}$ *= project drainage area at the outfall (acres or square miles)*
DA_{system} *= total drainage area for the system at the limits of analysis (use units consistent with* $DA_{outfall}$*)*

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:

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

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

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 <u>not</u> 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 <u>not</u> 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 <u>not</u> 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 <u>not</u> 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?

$$\mathrm{DA_{outfall}} \le \frac{\mathrm{DA_{system}}}{100}$$

$$\mathrm{DA_{system}} \ge \mathrm{DA_{outfall}} \times 100$$

$$\mathrm{DA_{system}} \ge 10.3\ \mathrm{acres} \times 100 \ge 1{,}030\ \mathrm{acres}$$

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

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?

$$\mathrm{Q_{outfall}} \le \frac{\mathrm{Q_{system}}}{100}$$

$$\mathrm{Q_{system}} \ge \mathrm{Q_{outfall}} \times 100$$

$$\mathrm{Q_{system}} \ge 187\ \mathrm{cfs} \times 100 \ge 18{,}700\ \mathrm{cfs}$$

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

As Q_{system} (15,895 cfs) is less than $Q_{outfall}$ x 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 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 $DA_{outfall}$ x 100, the flood prone area is located upstream. Therefore, the limits of analysis for flood protection can me moved upstream to the point in the system where flooding is mapped. This can be confirmed by using the FIRM to determining if the floodplain mapping begins before the point in the system where the drainage area is equal to $DA_{outfall}$ x 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 x 12.6 cfs = 1,260 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 it 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 it 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 <u>or</u> 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 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:

$$\mathrm{Tv} = 1.00\ \mathrm{inches} \times \mathrm{Site\ Area\ acres} \times \mathrm{Rv} \times \left(\frac{1\ \mathrm{foot}}{12\ \mathrm{inches}}\right)$$

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

$$Tv = 1.00\, inches \times 1.7\, acres \times 0.56 \times \left(\frac{1\, foot}{12\, inches}\right) = 0.0793\, acre - feet$$

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 Tv and draw down time. This is the VDOT preferred option.

- Using the **maximum hydraulic** head associated with the Tv, calculate the orifice size needed to achieve the required draw down time and route the Tv 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**
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

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 2½ 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 ft.
Tv = 8,720 ft^3

Step 1 - Calculate the average head:

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

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 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:

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

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

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

Step 5 - From Table 11-10, select a 2-inch orifice with A = 0.022 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 ft.
Q_{avg} = 0.101 cfs

Step 1 - Calculate the maximum Q_{max}:

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

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 ft^2

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

NOTE: The routing of the Tv 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 Tv Hydrograph

To develop a runoff hydrograph for the Tv, 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 TR-55 hydrograph will probably be the easiest hydrograph to provide the required treatment volume for an extended detention basin or other BMP. The land cover conditions should be based upon TR-55. Based upon the VRRM, "Forest/Open Space" uses the TR-55 CNs for "Woods, Good"; "Managed Turf" uses the CNs for "Open Space, Good" from TR-55; 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^3), 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.

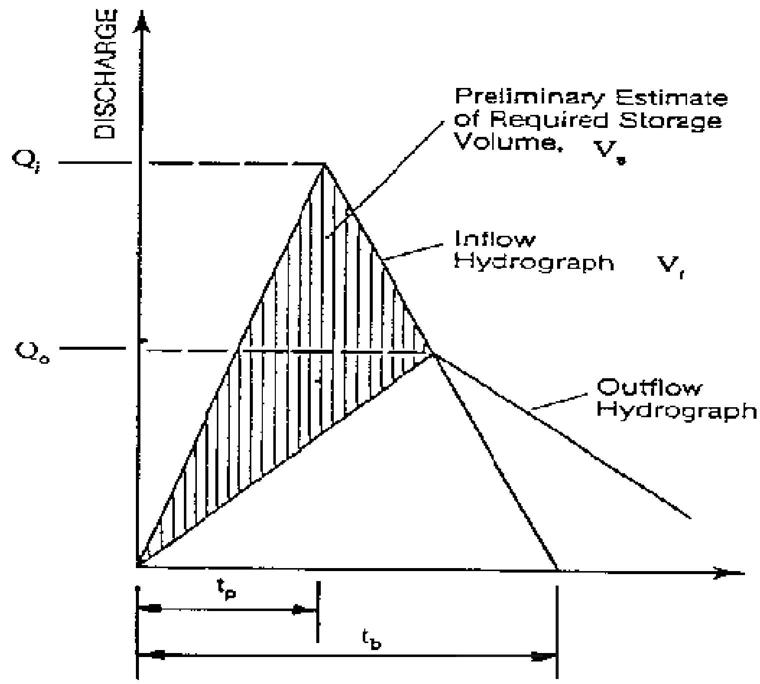


Figure 11-1. Simplified Triangular Hydrograph Method

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_s = Storage volume estimate, ft^3
Q_i = Peak inflow rate, cfs
Q_o = Peak outflow rate, cfs
T_b = 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:

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

Where:
V = Required storage volume, ft^3
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.

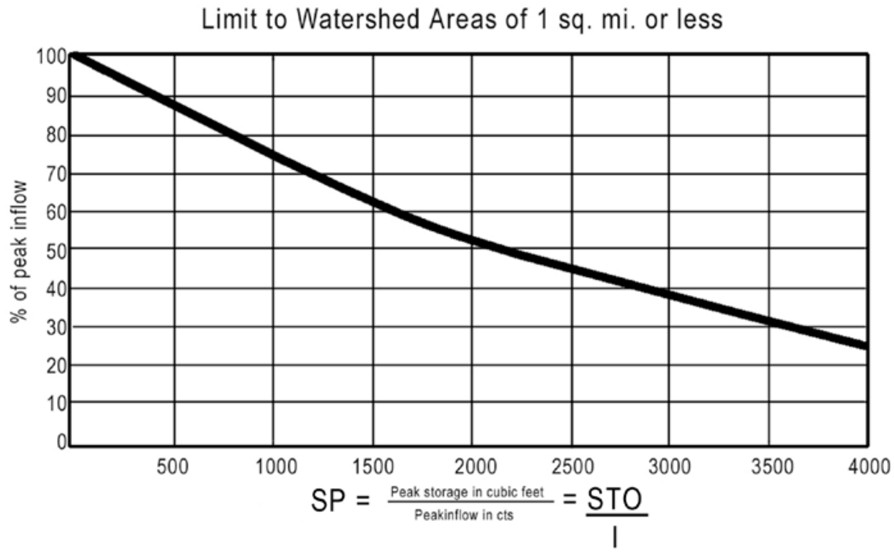


Figure 11-2. Pagan Method Curve

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):

$$\mathrm{STO} = \mathrm{SP}(\mathrm{I})$$

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:

	Rational Method			
Condition	**DA**	**C**	**T_c**	**Q**
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_s as follows:

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

Where:
V_s = Storage volume estimate, ft^3
Q_i = 65 cfs
Q_o = 24 cfs
T_b = 2 x T_c (post-development) = 2 x 21 min = 42 min = 2,520 sec

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

Method 2: Critical Storm Duration Method

Based on the critical storm duration method, determine the critical storm duration T_d as follows*:*

a = 189.2
b = 22.1
C = 0.59 (post-development)
A = 25 acres
t_c = 21 min (post-development)
q_o = 24 cfs (Allowable outflow based on pre-development)

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

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

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

Solve for the critical storm duration intensity (I):

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

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

$$\mathrm{Q = C_fCiA} = 1.0(0.59)(3.02)(25) = 44.5\ \mathrm{cfs}$$

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

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

$$V = \left[44.5(40.5) + \frac{44.5(21)}{4} - \frac{24(40.5)}{2} - \frac{3(24)(21)}{4}\right] 60$$

$$V = 70{,}313 \text{ ft}^3$$

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:

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

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^3)
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:

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

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

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

$$\boldsymbol{L} = \mathbf{2}(\boldsymbol{W})$$

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

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

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

Check the volume using the dimensions calculated:

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

Method 2:Critical Storm Duration Method

Calculate the footprint assuming a 4-ft depth:

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

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

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

$$\boldsymbol{LL} = \mathbf{2}(\boldsymbol{WW})$$

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

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

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

Check the volume using the dimensions calculated:

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

Method 3:Pagan Method

Calculate the footprint assuming a 4-ft depth:

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

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

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

$$\boldsymbol{L} = \mathbf{2}(\boldsymbol{W})$$

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

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

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

Check the volume using the dimensions calculated:

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

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

	Estimated Dimensions (ft)		
Method	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.*

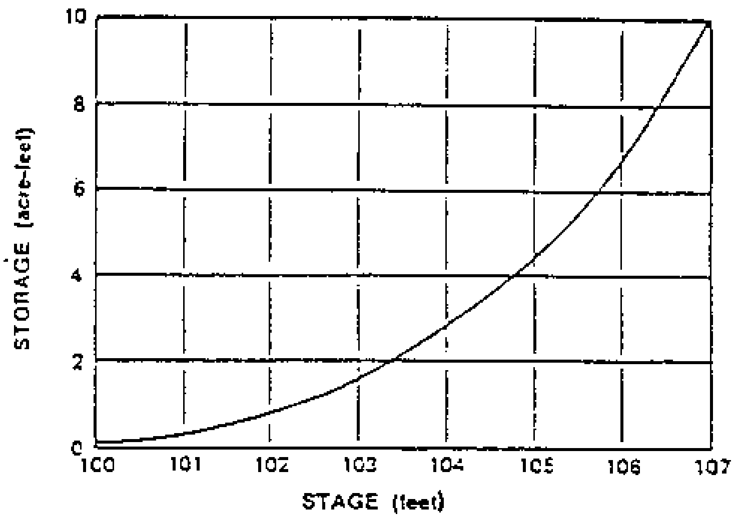


Figure 11-3. Stage-Storage Curve

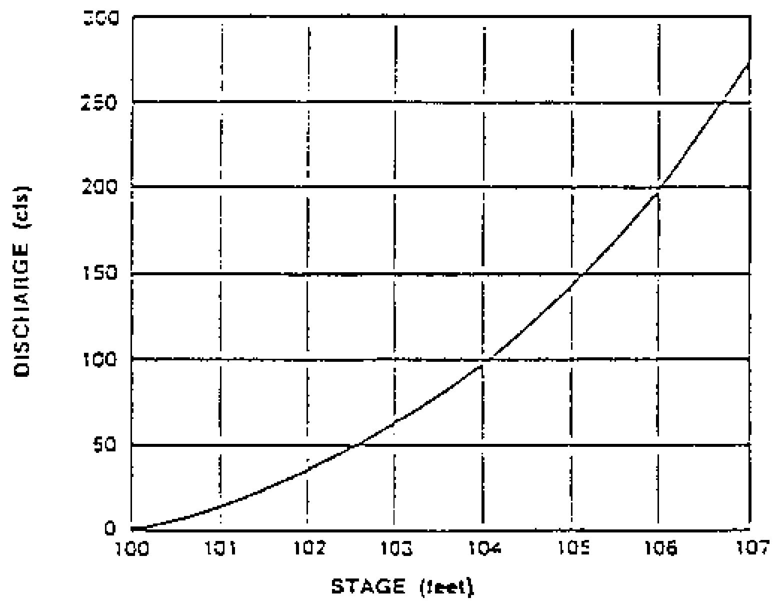


Figure 11-4. Stage-Discharge Curve

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

Step 3: *Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of* $S\Delta\frac{O}{2}$ *versus stage. An example tabulation of storage characteristics curve data is shown in* Table 11-11.

Table 11-2. Storage Characteristics

(1)	(2)	(3)	(4)	(5)	(6)
Stage (H) (ft.)	Storage[1] (S) (ac-ft)	Discharge[2] (Q) (cfs)	Discharge[2] (Q) (ac-ft/hr)	$S - \frac{Q}{2}\Delta T$ (ac-ft)	$S + \frac{Q}{2}\Delta T$ (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

[1] Obtained from the Stage-Storage Curve.
[2] 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,* $S_1 - \frac{Q_1}{2}\Delta T$ *can be determined from the appropriate storage characteristics curve,* Figure 11-14.

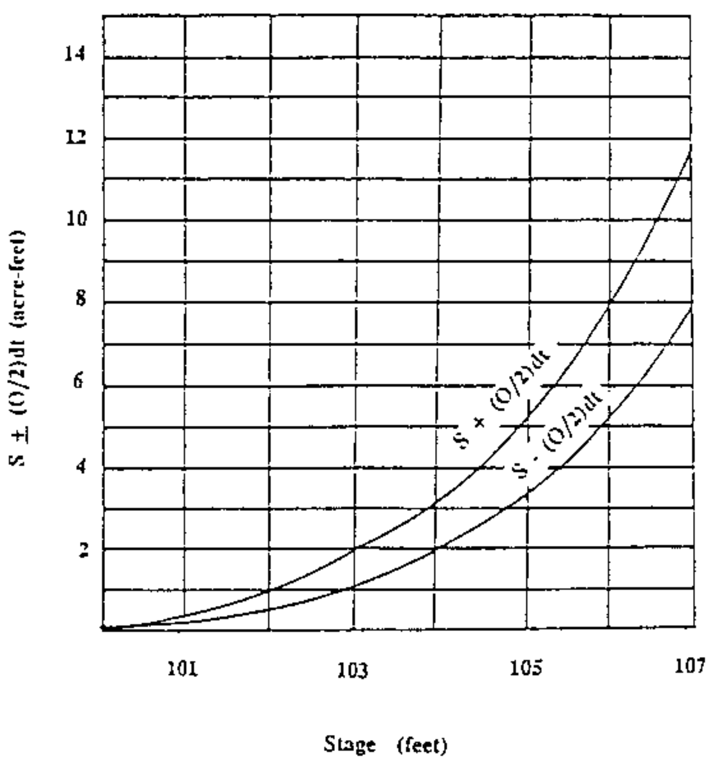


Figure 11-5. Storage Characteristics Curve

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

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

Where:

S_2 = Storage volume at time 2, ft^3
O_2 = Outflow rate at time 2, cfs.
ΔT = Routing time period, sec
S_1 = Storage volume at time 1, 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.*

11.5.9.2 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

	Pre-Development Runoff		**Post-Development Runoff**	
(1)	**(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:

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

$$V_{s_2} = \frac{\frac{1}{2}(1.2)(3600)(190\text{-}150)}{43{,}560} = 1.98 \text{ ac. ft.}$$

$$V_{S_{10}} = \frac{\frac{1}{2}(1.25)(3600)(250-200)}{43{,}560} = 2.58\ ac.\ ft.$$

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 (Δt) to provide at least five points on the rising limb of the inflow hydrograph. Use t_p divided by 5 to 10 for Δt.

$$\Delta T = \frac{t_p}{5} = \frac{0.5}{5} = 0.10 \text{ hr}$$

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

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

Summarized in Table 11-14and 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 11-14 and 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 Table 11-14 and Table 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 11-14 and Table 11-15 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_1) (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 11-6. 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_1) (ft)	$S_1 - \frac{O_1 \Delta T}{2}$ **(6)-(8)** (ac-ft)	$S_2 + \frac{O_2 \Delta T}{2}$ **(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.

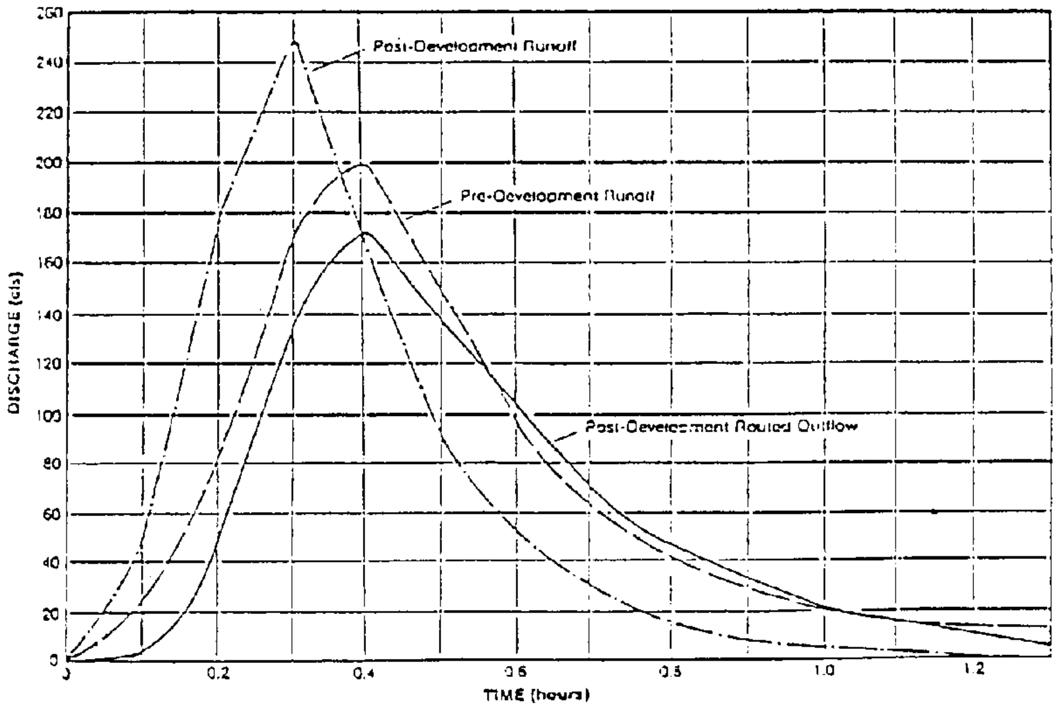


Figure 11-6. Runoff Hydrographs

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 SWM facility 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^3.
- 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^3, the total design volume is 1.25 x 8,654 ft^3 = 10,817 ft^3.

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:

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

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

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

- 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^3.
- The estimated forebay volume is 4,327 ft^3.

Step 4: Determining the Water Quality Volume Elevation

- Required treatment volume (for Extended Detention Level 2) = 10,817 ft^3
- From the Stage-Discharge-Storage table:
    - o The design Tv required satisfied @ Elev. 423.25

- Water depth @ Elev. 423.25 = 1.95 ft
- Actual Volume = 11,051 ft^3 @ 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:

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

- 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:

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

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

- Orifice sizing computations:

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

- 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 11-4, use a 1½-inch orifice with an area = 0.012 ft^2. 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^3
$Q_i = Q_{2post}$ = 29.6 cfs
$Q_o = Q_{2all}$ = 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:

      $$V_{total} = 11{,}051 \text{ ft}^3 + 10{,}911 \text{ ft}^3 = 21{,}962 \text{ ft}^3$$
    - Based on the Stage-Storage table, the volume provided at Elev. 425.50 = 22,783 ft^3, which is > 21,962 ft^3 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.

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

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

  Weir equation:

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

Where:
$Q = Q_{2all}$ = weir discharge, cfs
C = weir coefficient of discharge (use 3.0 for sharp-crested weir)
L = weir length, ft
h_{avg} = average head, ft

Rearranged weir equation to solve for weir length:

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

- 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 x Tv)
    - 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 fume 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.

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

- Weir sizing computation

  Weir equation:

$$\mathrm{Q_{avg}} = \mathrm{CL}\left(\mathrm{h_{avg}}\right)^{1.5}$$

  Where:
  Q = Q_{100} = weir discharge, cfs
  C = weir coefficient of discharge (use 2.6 for broad crested weir)
  L = weir length, ft
  h_{avg} = average head, ft

  Rearranged weir equation to solve for weir length:

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

- 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 x Tv)
  - Treatment volume provided at Elev. 423.25
  - 40% of the total treatment volume (1.25 c Tv) is in a permanent pool in a forebay
  - 60% of total treatment volume (1.25 c 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 x Tv), as 40% of (1.25 x Tv) 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/SWM facility 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/SWM facility and should be avoided.

11.6 Part IIC 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

Water Quality BMP	Treatment Volume	Target Phosphorus Removal Efficiency	Percent Impervious Cover
Vegetated filter strip Grassed swale		10% 15%	16-21%
Constructed wetlands Extended detention Retention basin I	2xWQV 2xWQV 3xWQV	20% 35% 40%	22-37%
Bioretention basin Bioretention filter Extended detention enhanced Retention basin II Infiltration	1xWQV 1xWQV 2xWQV 4xWQV 1xWQV	50% 50% 50% 50% 50%	38-66%
Bioretention basin Bioretention filter Sand filter Infiltration Retention basin III with aquatic bench	2xWQV 2xWQV 2xWQV 2xWQV 4xWQV	65% 65% 65% 65% 65%	67-100%
Manufactured BMP Systems Hydrodynamic Structures *		20%	
Manufactured BMP Systems Filtering Structures *		50%	
Filterra™ 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 SWM facilities 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 SWM facility 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^2 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 SWM facilities at the treated outfall are designed to account for the water quality volumes for those areas where SWM facilities 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 SWM facilities 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 design 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.

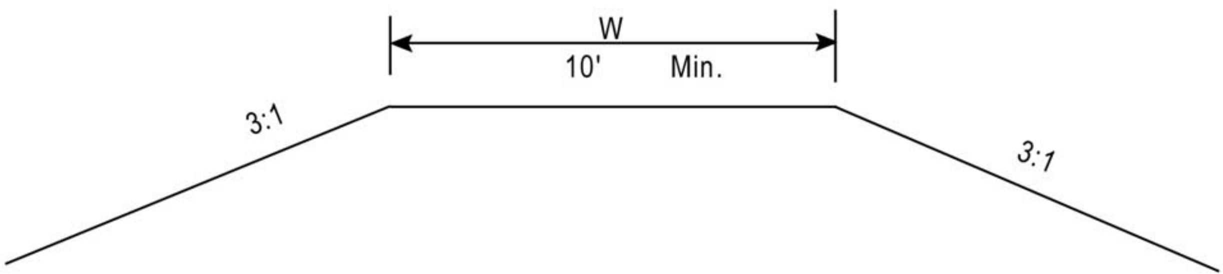


Figure 11-7. Typical SWM Basin Dam

- 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 SWM facility will operate effectively for its intended purpose during the passage of the 10-yr flood event on the flood plain. All SWM facility 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 SWM facility 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.

* Rev.1/17

- 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 SWM facilities within a sinkhole is prohibited. If SWM facilities 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 SWM facilities 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.

[*] Rev.1/17

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

Design Requirement	Dry Basin Design	Wet Basin Design
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).

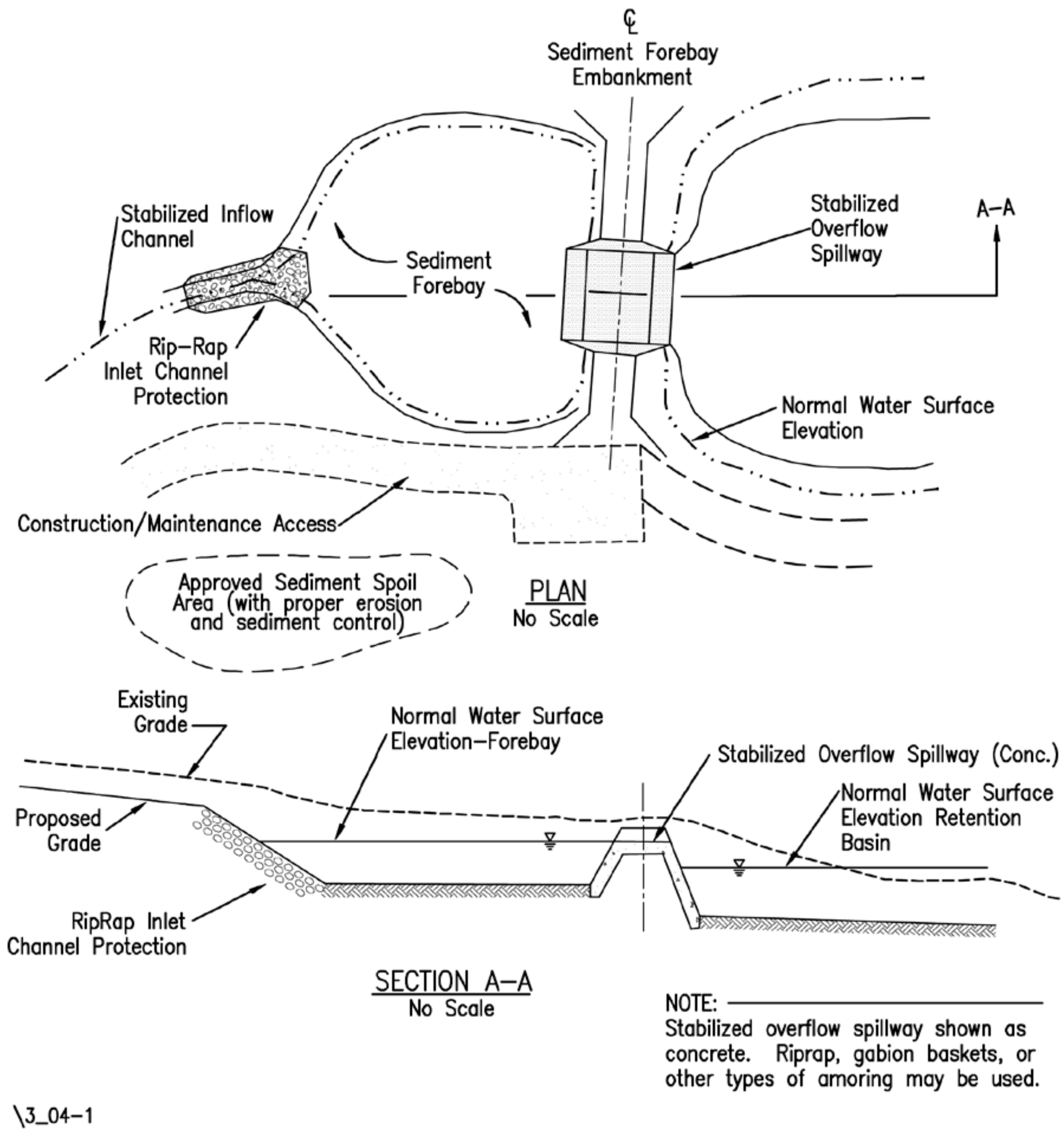


Figure 11-8. Typical Sediment Forebay Plan and Section

11.7 Part IIC 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

Stormwater BMP	Quality	Stream Channel Erosion	Quantity/ Flooding
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, 1st 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 SWM facilities.

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.

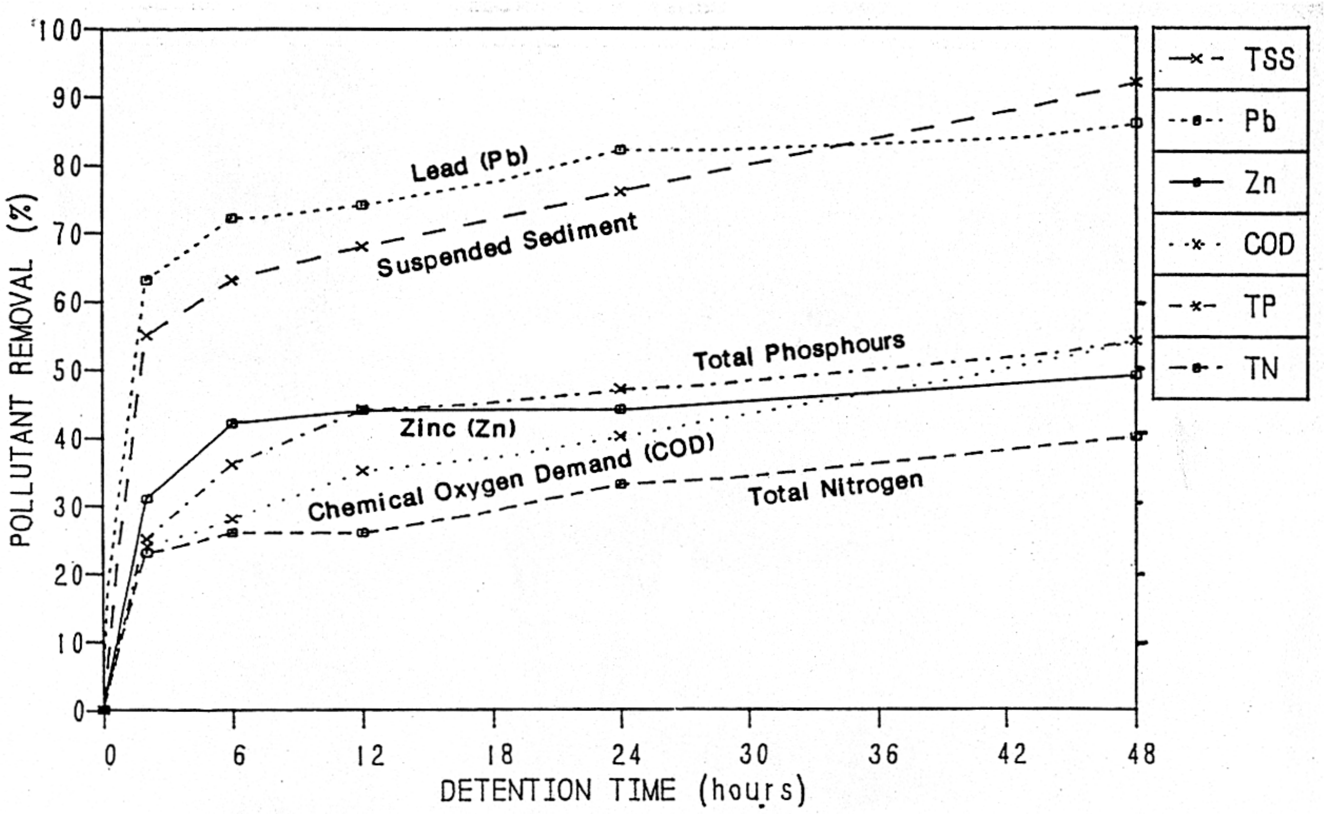


Figure 11-9. Removal Rates vs. Detention Time

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 SWM facility. 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_1 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 IIC 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^{th} 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 (see Section 11.5.9.4 for applicability of this requirement to current VDOT projects).

<u>Alternative BMPs</u>
BMPs included on the Virginia SWM BMP Clearing House website http://vwrrc.vt.edu/swc/ 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 LID and 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 SWM facility).
- 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 6th Order HUC or upstream HUC of the land-disturbing activity or within the same watershed as determined by DEQ, SWM facilities 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:
    - o Alternative site designs have been considered that may accommodate onsite BMPs, and
    - o Onsite BMPs have been considered in alternative site designs to the maximum extent practicable, and
    - o Appropriate onsite BMPs will be implemented, and
    - o 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:

$$Q = CA\sqrt{2gh} \qquad (11.1)$$

Where:

- Q = Discharge, cfs
- C = Orifice entrance coefficient (generally 0.6)
- A = Cross-sectional area of orifice, ft^2
- g = Acceleration due to gravity, 32.2 ft/s^2
- 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:

$$Q = CLH^{\frac{3}{2}} \qquad (11.2)$$

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 SWM facilities, 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 SWM facilities 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 SWM facility
- 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 IIC Design Procedures and Sample Problems

11.8.1 Documentation Requirements

The following documentation will be required for SWM facility 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
- SWM Facility 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.

11.8.1.1 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.*

WQV = ½ inch x Impervious Area

½ inch x (1 ft/12 inches) = 0.04126 ft

1 acre = 43,560 ft^2

WQV = 0.04167 x 43,560 x 2.4 ac. = 4,356 ft^3 (say 4,360 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.*

Required Treatment Volume = 2 x WQV = 2(4360) = 8720 cu.ft.

11.8.1.2 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	0.0034
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.

11.8.1.2.1 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 ft.

Volume = 8,720 ft^3 (from Sample Problem 11.5.2.1)

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

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.

$$Q_{avg} = \frac{\text{Treatment Volume}}{\text{Time}} = \frac{8720 \text{ cu.ft.}}{30 \text{ hr } (3600 \frac{\text{sec}}{\text{hr}})} = 0.081 \text{ cfs}$$

Calculate the orifice area by rearranging Equation 11.1.

$$A = \frac{Q}{C\sqrt{2gh_{avg}}} = \frac{0.081}{0.6\sqrt{2(32.2)(0.55)}} = 0.0223 \text{ sq.ft.}$$

From Table 11-10, select a 2-inch orifice with A = 0.022 ft^2.

11.8.1.2.2 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_{avg} x 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 ft.

$$Q_{max} = 2Q_{avg} = 2(0.081) = 0.16 \text{ cfs}$$

Calculate the orifice area by rearranging Equation 11.1.

$$A = \frac{Q}{C\sqrt{2gh_{max}}} = \frac{0.16}{0.6\sqrt{2(32.2)(1.1)}} = 0.0320\,sq.\ ft.$$

From Table 11-10, select a 2½-inch orifice with A = 0.034 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.

11.8.1.2.3 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 TR-55 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.

11.8.1.2.4 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.

11.8.1.3 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})*

$$V_{ce} = 25\text{ ac.}(0.8\text{ in.})\left(\frac{1\text{ ft.}}{12\text{ in.}}\right) = 1.67\text{ ac.ft.}$$

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:

$$A = \frac{\pi d^2}{4}$$

$$d = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4(0.104)}{\pi}} = 0.364 \text{ sq ft}$$

d = 4.4-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.

11.8.1.4 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.

11.8.1.4.1 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.

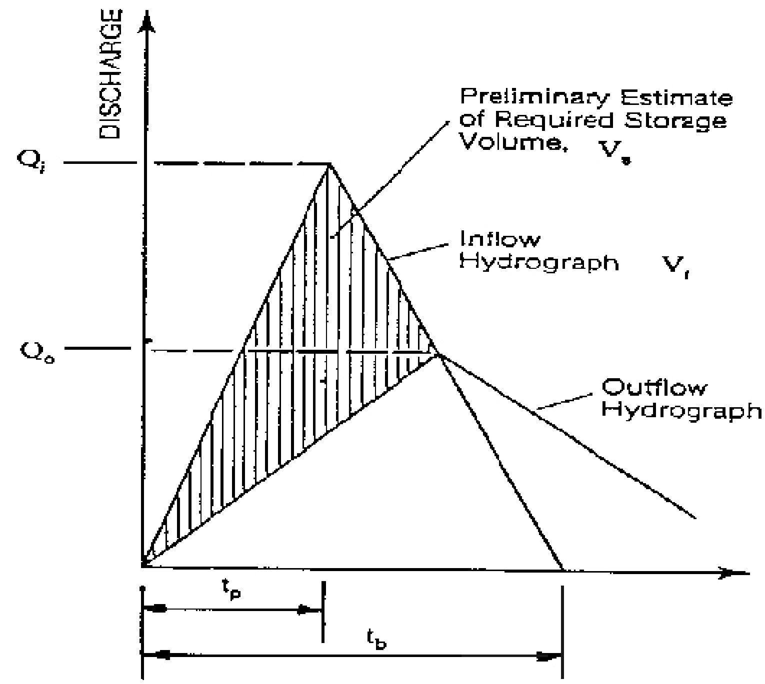


Figure 11-10. Simplified Triangular Hydrograph Method

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

$$V_s = \frac{1}{2} T_b (Q_i - Q_o) \qquad (11.3)$$

Where:

V_s = Storage volume estimate, ft^3
Q_i = Peak inflow rate, cfs
Q_o = Peak outflow rate, cfs
T_b = Duration of basin inflow, sec.

11.8.1.4.2 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:

$$V = \left[Q_i T_d + \frac{Q_i t_c}{4} - \frac{q_o T_d}{2} - \frac{3 q_o t_c}{4} \right] 60 \qquad (11.4)$$

Where:

- V = Required storage volume, ft^3
- 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 Equation 11.4 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. Equation 11.5 for the critical storm duration is:

$$T_d = \sqrt{\frac{2CAa(b-\frac{t_c}{4})}{q_o}}-b \qquad (11.5)$$

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.

11.8.1.4.3 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.

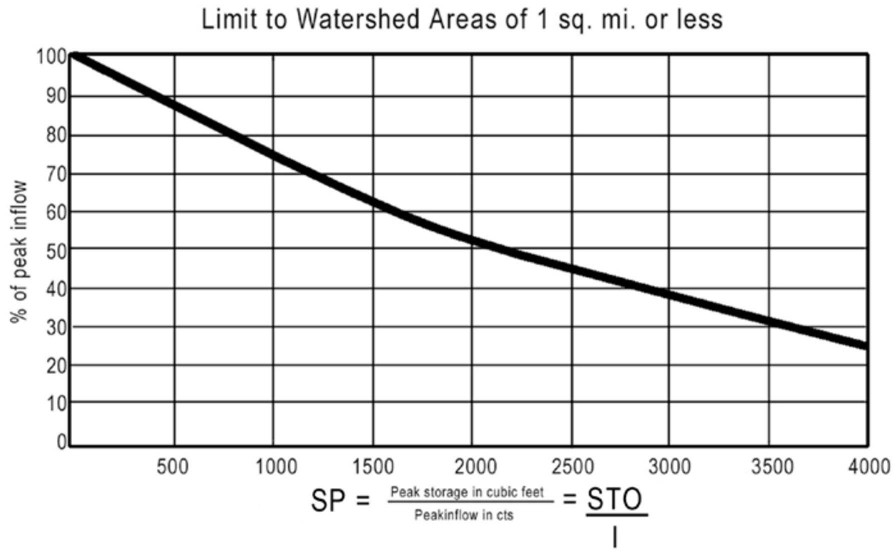


Figure 11-11. Pagan Method Curve

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):

$$STO = SP(\mathbf{I})$$

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

	Rational Method			
Condition	**D.A**	**C**	**T_c**	**Q_{10}**
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:

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

Where:

V_{s10} = Storage volume estimate, ft^3
Q_i = 65 cfs
Q_o = 24 cfs
T_b = 2520 sec. = 42 min.

$$V_s = \frac{1}{2}(2520)(65\text{-}24)$$
$$= 51{,}660 \text{ cu.ft.}$$

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*:*

a = 189.2
b = 22.1
C = 0.59 (Post-development)
A = 25 acres
t_c = 21 min (Post-development)
q_{o10} = 24 cfs (Allowable outflow based on pre-development)

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

$$T_{d_{10}} = \sqrt{\frac{2(0.59)(25.0)(189.2)\left(22.1\text{-}\frac{21}{4}\right)}{24}}\text{-}22.1$$

$$T_{d_{10}} = 40.5 \text{ min}$$

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})

$$Q = C_f CiA$$
$$Q_{10} = 1.0(0.59)(3.02)(25) = 44.5 \text{ cfs}$$

Determine the required 10-yr storage volume (V_{10}) for the 10-yr critical storm duration (T_{d10})

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

$$V_{10} = \left[(44.5)(40.5) + \frac{(44.5)(21)}{4} - \frac{(24)(40.5)}{2} - \frac{3(24)(21)}{4} \right] 60$$

= 70,313 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:

$$\frac{Q_o}{Q_i} = \frac{24}{65} = 0.37\ (37\%)$$

SP = 3100 seconds.

$$\begin{aligned} STO &= SP(I) \\ &= 3100(65) \\ &= 201{,}500\ \text{cu.ft.} \end{aligned}$$

11.8.1.5 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 ft^3

For a 4-ft depth, $\frac{51{,}660}{4} = 12{,}915\ \text{sq.ft.}$

About 80'x160'

From Method 2: Critical Storm Duration Method

V_{10} = 70,300 ft^3

For 4' depth, $\frac{70{,}300}{4} = 17{,}575\ \text{sq.ft.}$

About 90'x195'

From Method 3: Pagan Method

V_{10} = 201,500 ft^3

For a 4' deep, $\frac{201{,}500}{4} = 50{,}375\,\text{sq.ft.}$

About 150'x335'

Summary: Preliminary trial size basin would be recommended about 100'x200'

11.8.1.6 Final Basin Sizing-Reservoir Routing

11.8.1.6.1 Storage – Indication Method Routing Procedure

The following procedure presents the basic principles of performing routing through SWM 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 SWM facility. Example stage-storage and stage-discharge curves are shown in Figure 11-12 *and* Figure 11-13 *respectively.*

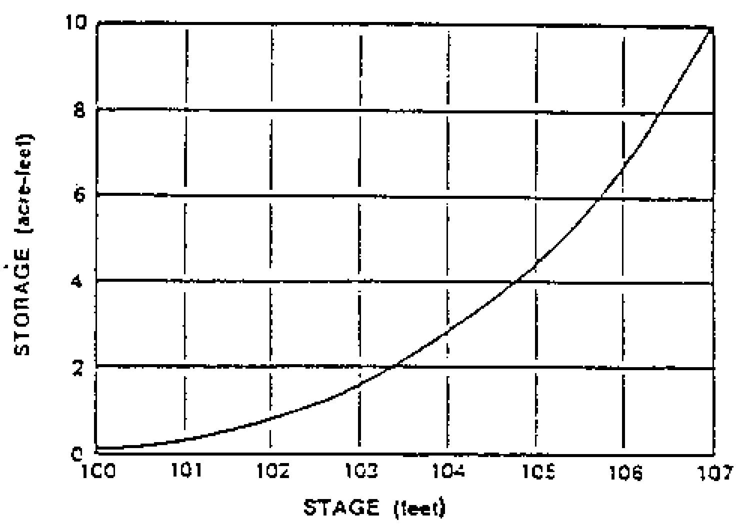


Figure 11-12. Stage-Storage Curve

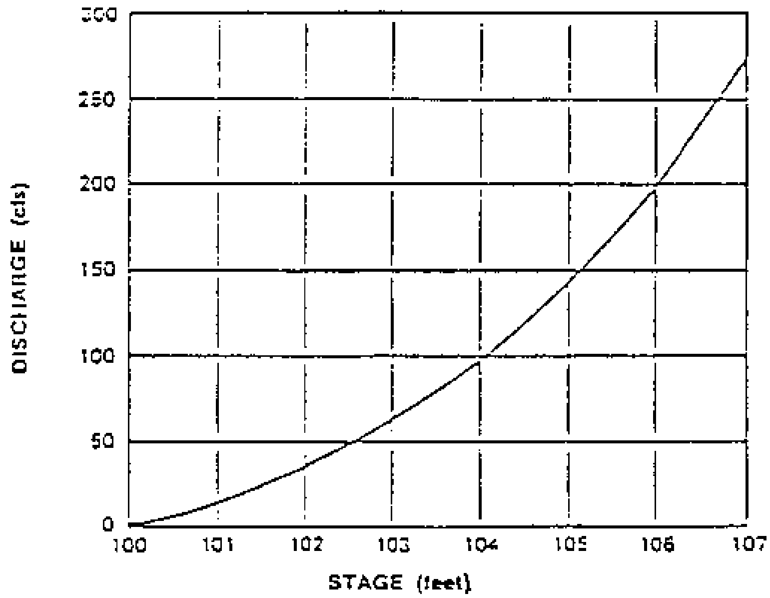


Figure 11-13. Stage-Discharge Curve

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

Step 3: *Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of* $S \pm \Delta t \frac{Q}{2}$ *versus stage. An example tabulation of storage characteristics curve data is shown in* Table 11-11*.*

Table 11-11. Storage Characteristics

(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**
Stage (H) (ft.)	Storage[1] (S) (ac-ft)	Discharge[2] (Q) (cfs)	Discharge[2] (Q) (ac-ft/hr)	$S - \Delta t \frac{Q}{2}$ (ac-ft)	$S + \Delta t \frac{Q}{2}$ (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

[1] Obtained from the Stage-Storage Curve.
[2] 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*.*

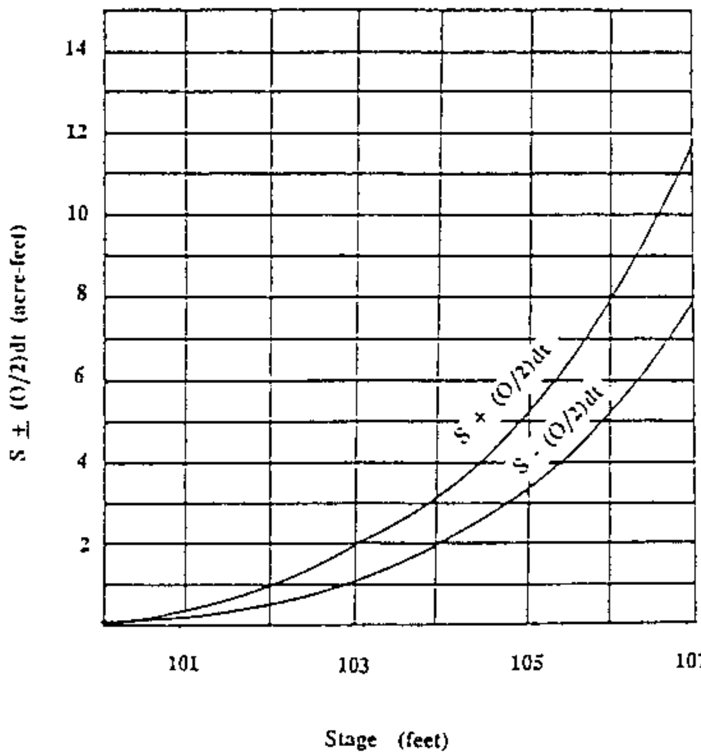


Figure 11-14. Storage Characteristics Curve

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

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

Where:

S_2 = Storage volume at time 2, ft^3
O_2 = Outflow rate at time 2, cfs.
ΔT = Routing time period, sec
S_1 = Storage volume at time 1, 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 $S_2 \Delta t + \frac{O_2}{2}$ *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.

11.8.1.6.2 Storage – Indication Method Routing Sample Problem

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

SWM facilities 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 SWM 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 storms.

Table 11-12. Runoff Hydrographs

	Pre-Development Runoff		Post-Development Runoff	
(1)	**(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:

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

$$V_{s_2} = \frac{\frac{1}{2}(1.2)(3600)(190\text{-}150)}{43{,}560} = 1.98 \text{ ac.ft.}$$

$$V_{S_{10}} = \frac{\frac{1}{2}(1.25)(3600)(250-200)}{43{,}560} = 2.58 \; ac.\; ft.$$

Stage-discharge and stage-storage characteristics of a SWM 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 (Δt) to provide at least five points on the rising limb of the inflow hydrograph. Use t_p divided by 5 to 10 for Δt.

$$\Delta T = \frac{t_p}{5} = \frac{0.5}{5} = 0.10 \text{ hr}$$

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

Table 11-13. Stage-Discharge-Storage Data

(1)	**(2)**	**(3)**	**(4)**	**(5)**
Stage (H) (ft)	Discharge (Q) (cfs)	Storage (S) (ac-ft)	$S - \Delta T \frac{Q}{2}$ (ac-ft)	$S + \Delta T \frac{Q}{2}$ (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, $S_1 \Delta T \frac{O_1}{2}$ can be determined from the appropriate storage characteristics curve.*

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

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

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

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

Summarized in and 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 Table 11-14 and Table 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 11-14 and 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_1) (ft)	$S_1 - \frac{O_1 \Delta T}{2}$ **(6)-(8)** (ac-ft)	$S_2 + \frac{O_2 \Delta T}{2}$ **(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_1) (ft)	$S_1 - \frac{O_1 \Delta T}{2}$ **(6)-(8)** (ac-ft)	$S_2 + \frac{O_2 \Delta T}{2}$ **(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 SWM 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 SWM facilities 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.

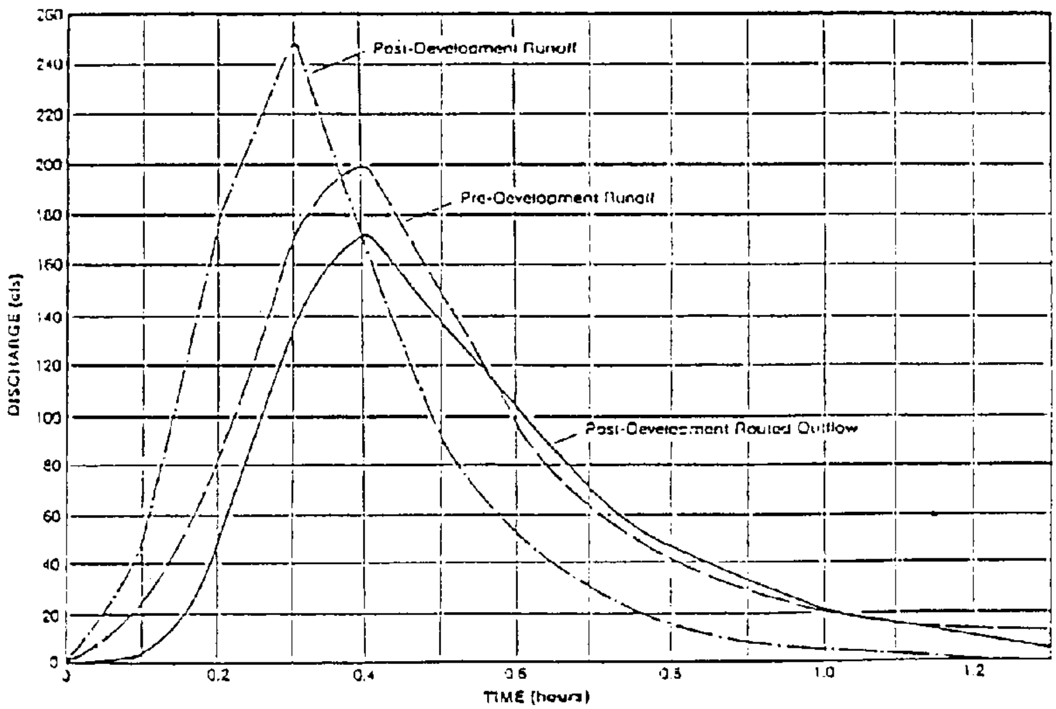


Figure 11-15. Runoff Hydrographs

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.

11.8.1.6.3 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 $\frac{2.98}{9.03} = 0.33\ (33\%)$

  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 x WQV. The WQV being equal to ½ inches x New Impervious area. 2 x ½ inch = 1 in or 0.083 ft
- New pavement within the drainage area for this outfall = 2.98 ac or 129,809 ft^2

$$\begin{aligned}\text{Treatment Volume} &= 2\text{xWQV}\\ &= 2\left[\frac{0.5(2.98)(43560)}{12}\right]\\ &= 10{,}817\ \text{cu.ft.}\end{aligned}$$

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 cu. yd. x 13 ac = 23,517 ft^3 for wet storage

  67 cu. yd. x 13 ac = 23,517 ft^3 for dry storage

  The total volume required for temporary sediment storage, wet plus dry = 47,034 ft^3. This is much larger than the 10,817 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 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

1. An extended detention basin is required for this site.
2. QUANTITY CONTROL FOR Q_2 PEAK IS REQUIRED. The required volume will be estimated in the design process.
3. Alternative Q_1 control is not needed.
4. The required WQV is 10,817 ft^3
5. The temporary sediment volume (if needed) is 47,034.
6. The estimated forebay volume is 1,082 to 2,704 ft^3

Determining the Water Quality Volume

Calculate required WQV (for extended detention) = 10,817 ft^3

From Preliminary Elevation/Storage Table:

The WQV required is met @ Elev. 423.25

Depth = 1.95 ft.

Actual Volume = 11,051 ft^3 @ 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:*

$$Q_{avg} = \frac{\text{Treatment Volume}}{\text{Time}} = \frac{11{,}051\,\text{cu. ft.}}{30\,\text{hr}\,(3600\,\frac{\text{sec}}{\text{hr}})} = 0.102\,\text{cfs}$$

- *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^2

Q_1 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_1 instead of the Q_2 as required by MS-19. Control of the Q_1 requires containing the entire volume of the Q_1 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_1 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_1 Control Volume:
Use Method #2 – Average Hydraulic Head (Recommended Method)

Find the Q_1 Control volume.

Given from design computations:

DA = 12.98 ac
C = 0.67
T_c = 16 min
Q_2 = 29.6 cfs.

- *Use TR-55 to find the volume for Q_1:*
- *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 inches.

- *Find the runoff depth for CN= 80 and RF = 2.8 inches using TR-55.*

  Runoff (RO) = 1.1 inches

- *Compute the Q_1 Control volume:*

$$V_{ce} = 12.98\text{ ac.}(1.1\text{ in.})\left(\frac{1\text{ ft.}}{12\text{ in.}}\right)\left(\frac{43{,}560\text{ sq. ft.}}{1.0\text{ ac.}}\right) = 51{,}829\text{ cu. ft.}$$

To account for the routing effect, reduce the channel erosion control volume by 60%:

$$V_{ce} = 0.60(51{,}829) = 31{,}097\text{ cu. ft.}$$

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 cu. yd. x 13 ac = 23,517 ft^3 for wet storage

  67 cu. yd. x 13 ac = 23,517 ft^3 for dry storage

  The total volume required for temporary sediment storage, wet plus dry = 47,034 ft^3. This is much larger than the 10,817 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 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

1. An extended detention basin is required for this site.
2. QUANTITY CONTROL FOR Q_2 PEAK IS REQUIRED. The required volume will be estimated in the design process.
3. Alternative Q_1 control is not needed.
4. The required WQV is 10,817 ft^3
5. The temporary sediment volume (if needed) is 47,034.
6. The estimated forebay volume is 1,082 to 2,704 ft^3

Determining the Water Quality Volume

Calculate required WQV (for extended detention) = 10,817 ft^3

From Preliminary Elevation/Storage Table:

The WQV required is met @ Elev. 423.25

Depth = 1.95 ft.

Actual Volume = 11,051 ft^3 @ 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:*

$$Q_{avg} = \frac{\text{Treatment Volume}}{\text{Time}} = \frac{11{,}051\,\text{cu.ft.}}{30\,\text{hr}\,(3600\,\frac{\text{sec}}{\text{hr}})} = 0.102\,\text{cfs}$$

- *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^2

Q_1 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_1 instead of the Q_2 as required by MS-19. Control of the Q_1 requires containing the entire volume of the Q_1 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_1 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_1 Control Volume:
Use Method #2 – Average Hydraulic Head (Recommended Method)

Find the Q_1 Control volume.

Given from design computations:

DA = 12.98 ac
C = 0.67
T_c = 16 min
Q_2 = 29.6 cfs.

- *Use TR-55 to find the volume for Q_1:*
- *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 inches.

- *Find the runoff depth for CN= 80 and RF = 2.8 inches using TR-55.*

  Runoff (RO) = 1.1 inches

- *Compute the Q_1 Control volume:*

$$V_{ce} = 12.98\text{ ac.}(1.1\text{ in.})\left(\frac{1\text{ ft.}}{12\text{ in.}}\right)\left(\frac{43{,}560\text{ sq.ft.}}{1.0\text{ ac.}}\right) = 51{,}829\text{ cu.ft.}$$

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 inches
     RF_2 = 3.5 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 cfs

     $$\begin{aligned} Q_1 &= 0.80Q_2 \\ &= 0.80(29.6) \\ &= 23.7\text{ cfs} \end{aligned}$$

   - Compute the volume from a triangular HYG:

     Using t_c = 16 min., $T_b = 2t_C$ = 32 min.

     $$\begin{aligned} V_1 &= 0.5(Q_1)(T_b)(60\frac{\text{sec}}{\text{min}}) \\ &= 0.5(23.7\text{ cfs})(32\text{ min.})(60\frac{\text{sec}}{\text{min}}) \\ &= 22{,}752\text{ cu. ft.} \end{aligned}$$

   - Compute the volume from a trapezoidal HYG:

     Using t_c = 16 min. and determining the critical storm duration, T_d = 22 min.

     $T_b = t_c + T_d$= 38 min.

     $$\begin{aligned} V_1 &= 0.5(Q_1)[(T_d - t_c) + T_b]60\frac{\text{sec}}{\text{min}} \\ &= 0.5(23.7\,cfs)[(22\text{ min} - 16\text{ min}) + 38\text{ min}]60\frac{\text{sec}}{\text{min}} \\ &= 31{,}284\,cu.ft. \end{aligned}$$

     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 cfs with t_c = 16 minutes and the duration = 22 minutes, the volume of the HYG = 31,284 ft^3 which is very close to the volume of 31,097 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:

$$Q_{avg} = \frac{V_{ce}}{Time} = \frac{31{,}097\ \text{cu.ft.}}{24\ \text{hr.}(3600\frac{\text{sec}}{\text{hr}})} = 0.360\ \text{cfs}$$

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^2.

11.9 References

American Association of State Highway and Transportation Officials (2014), *AASHTO Drainage Manual (*First Edition), Washington, D.C.: American Association of State Highway and Transportation Officials.*

American Association of State Highway and Transportation Officials (2007), *AASHTO Highway Drainage Guidelines (*Fourth Edition), Washington, D.C.: American Association of State Highway and Transportation Officials.

Center for Watershed Protection, *Virginia Runoff Reduction Method: Instructions & Documentation*, March 28, 2011

Virginia Department of Environmental Quality (1999), *Virginia Stormwater Management Handbook*, *Volumes I & II* (First Edition) Richmond, VA: Virginia Department of Environmental Quality.

Virginia Department of Environmental Quality *Technical Bulletin #6: Minimum Standard 3.11C Filterra Bioretention Filter System* (Revised 11/01/02)

Virginia Department of Environmental Quality, Guidance Memo No. 16-2001 - Updated Virginia Runoff Reduction Method Compliance Spreadsheets - Version 3.0, May 2016
Brater, E. F. and H. W. King. *Handbook of Hydraulics*, 6th Edition, New York: McGraw Hill Book Company, 1976

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

Sandvik, A. March, Proportional Weirs for Stormwater Pond Outlets, *Civil Engineering, ASCE*, pp. 54-56, March 1985

Spangler, M.G. and R. L. Handy, *Geotechnical Engineering*, 5th Edition, McGraw-Hill Professional Publishing, October 2006

Metropolitan Government of Nashville and Davidson County, *Stormwater Management Manual – Volume 2 Procedures*, Camp Dresser & McKee, Inc May 2000

Wycoff, R. L. and U. P. Singh, *Preliminary Hydrologic Design of Small Flood Detention Reservoirs*, JAWRA, Vol. 12, No. 2, pp 337-344, 11 June 2007

* Rev. 1/17

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1.1 Introduction

This Appendix was prepared for the Virginia Department of Transportation by Virginia Tech under contract for the Virginia Center for Transportation Innovation & Research. It provides guidance in the design of Best Management Practices capable of contributing to the goal of stormwater management as defined in Instructional and Informational Memorandum of General Subject *"Post Development Stormwater Management"* (IIM-LD-195), which states:

"Stormwater Management – to inhibit the deterioration of the aquatic environment by maintaining the post development water quantity and quality run-off characteristics, as nearly as practicable, equal to or better than pre-development run-off characteristics."

Additionally, the design examples apply the BMP design methodologies found in the *Virginia Stormwater Management Handbook* (DCR/DEQ, 1999, Et seq.) to the site conditions and constraints typically encountered in linear development projects.

It is assumed that the readers of this document are knowledgeable in the engineering disciplines of hydrology and hydraulics and will understand fundamental fluid flow principles used in this manual.

This Appendix does not constitute a standard, specification, or regulation.

1.2 Project Site

The *project site* is defined as:

> The area of actual proposed land disturbance (i.e., construction limits) plus any right of way acquired in support of the proposed land disturbance activity/project. Any staging areas within existing or proposed VDOT right of way associated with the proposed land disturbance activity/project and identified in the pre-construction SWPPP for the proposed land disturbance activity/project shall also be considered a part of the site. Permanent easements and/or other property acquired through the right of way acquisition process in support of the proposed land disturbance activity/project may be considered a part of the site and utilized in the determination of the post development water quality requirements provided such property will remain under the ownership/control of the VDOT and providing such property is so identified/designated on the proposed land disturbance activity/project plans and legally encumbered for the purpose of stormwater management.

1.3 Water Quality Standards

As stated in Section 11.6.1 of the VDOT Drainage Manual, water quality requirements use the “Performance Based” criteria. The BMP selection table is shown in Table 1.1.

Table 1.1 BMP Selection Table for VDOT Projects*

Water Quality BMP	Target Phosphorus Removal Efficiency (%)	Percent Impervious Cover Cover (%)**
Vegetated Filter Strip Grassed Swale	10 15	16-21
Constructed Wetlands Extended Detention (2xWQV) Retention Basin I (3xWQV)	30 35 40	22-37
Bioretention Basin Bioretention Filter Extended Detention - Enhanced Retention Basin II (4xWQV) Infiltration (1xWQV)	50 50 50 50 50	38-66
Sand Filter Infiltration (2xWQV) Retention Basin III (4xWQV with aquatic bench)	65 65 65	67-100

*Innovative or alternate BMPs not included in this table may be allowed at the discretion of DCR/DEQ and with the concurrence of the VDOT State MS4/Stormwater Management Engineer, as stated in IIM-LD-195.

Source: Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.)

1.4 Water Quantity Standards

Although it is recognized that some BMPs used for water quality control implicitly have the ability to partially, or in some cases, fully meet the requirements for stormwater quantity control, this manual is not intended to cover Commonwealth of Virginia requirements for flooding or erosion control. The user is directed to the Virginia Erosion and Sediment Control Handbook (Third Edition, 1992), the Virginia Stormwater Management Handbook (First Edition, 1999), the Virginia Stormwater Management Program (VSMP) Permit Regulations, the VDOT Drainage Manual, and any applicable VDOT Instructional and Information Memoranda (specifically IIM-LD-195) for further discussion of specific state requirements and sample calculations related to stormwater quantity control.

2.1 Dry Extended Detention Basin - Overview of Practice

A dry extended detention basin is defined as an impoundment which temporarily detains runoff and releases that runoff at a controlled rate over a specified period of time. By definition, extended dry detention basins are dry structures during non-precipitation periods. Extended dry detention basins are capable of providing water quality improvement, downstream flood control, channel erosion control, and mitigation of post-development runoff to pre-development levels. The primary mechanism by which a dry extended detention facility improves runoff quality is through the gravitational settling of pollutants.

Extended dry detention basins are most effective as water quality improvement practices when the new impervious cover of their total contributing drainage area ranges between 22 and 37%. Additionally, as shown, extended dry detention facilities should be designed to provide 30-hour drawdown storage for *twice* the site's computed water quality volume (2 X WQV), equivalent to a total of 1" of runoff from the project site's new impervious area.

Figure 2.1 presents the schematic layout of a dry extended detention basin presented in the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.). Of note is that the low flow rip rap-lined channel *has been removed from the drawing*. Please note that this channel is not recommended due to maintenance concerns.

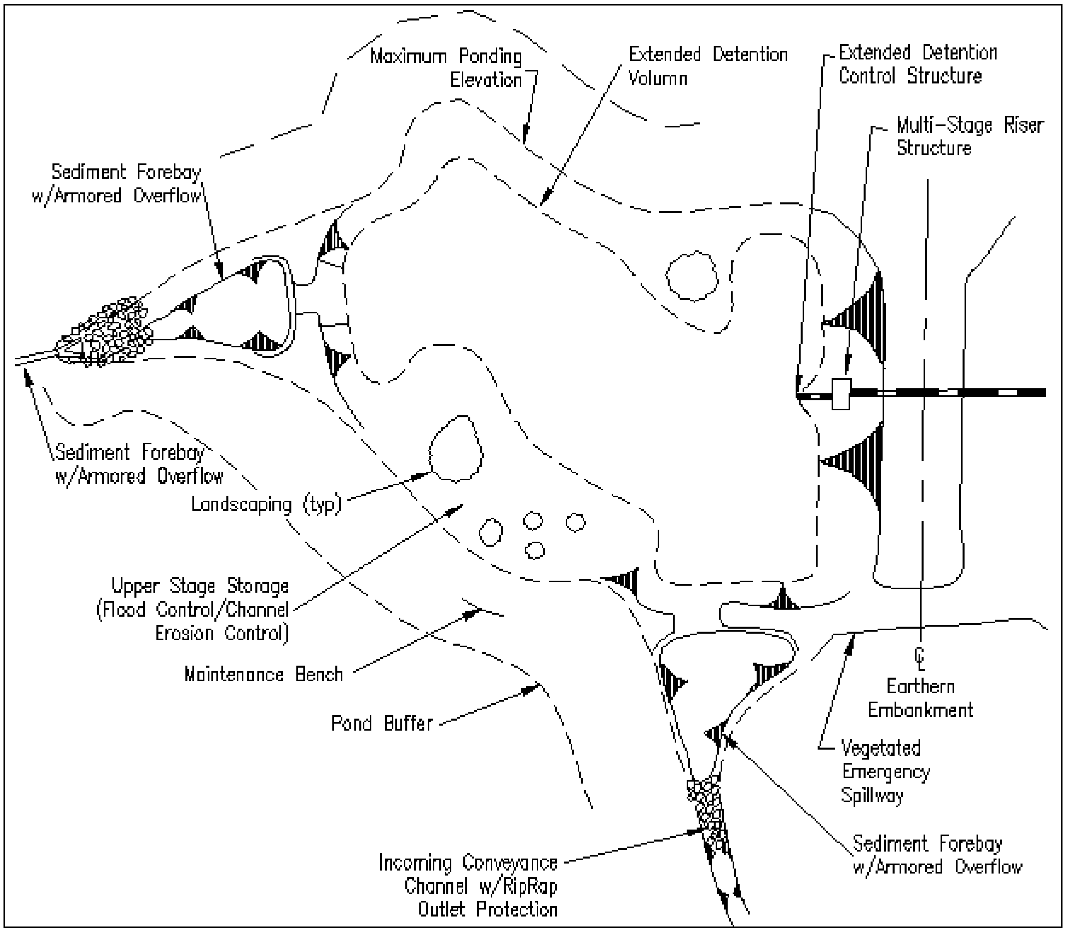


Figure 2.1 – DCR/DEQ Schematic Dry Extended Detention Basin Plan View
(_Virginia Stormwater Management Handbook_, 1999, Et seq.)

2.1.1 Site Constraints and Siting of the Facility

In addition to the new impervious cover in the contributing drainage area, the designer must consider additional site constraints when the implementation of a dry extended detention basin is proposed. These constraints are discussed as follows.

2.1.1 Minimum Drainage Area

The minimum drainage area contributing to a dry extended detention facility is not restricted. However, careful attention must be given to the water quality volume generated from this area. When this water quality volume is particularly low, the computed orifice size required to achieve the desired drawdown time may be small (less than 3" in diameter). These small openings are vulnerable to clogging by debris. Generally, the minimum area contributing runoff to a dry extended detention pond should be selected such that the desired water quality drawdown time is achieved with an orifice of at least 3" in diameter. In instances when this is unavoidable, provisions must be made to prevent clogging. Figure 3.07-3 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.) illustrates recommended outlet configurations for the control of sediment, trash, and debris. For convenience, these details are provided as Figures 2.2, 2.3, and 2.4. Note that Figures 2.2, 2.3, and 2.4 include a *shallow marsh* area. This permanent marsh area is not part of a dry extended detention basin, and shall only be provided if the basin is to be "enhanced" – reference *Section Three – Dry Extended Detention Basin – Enhanced*. If the required water quality orifice size is significantly less than 3", an alternative water quality BMP should be considered, such as a practice which treats the first flush volume and bypasses large runoff producing events.

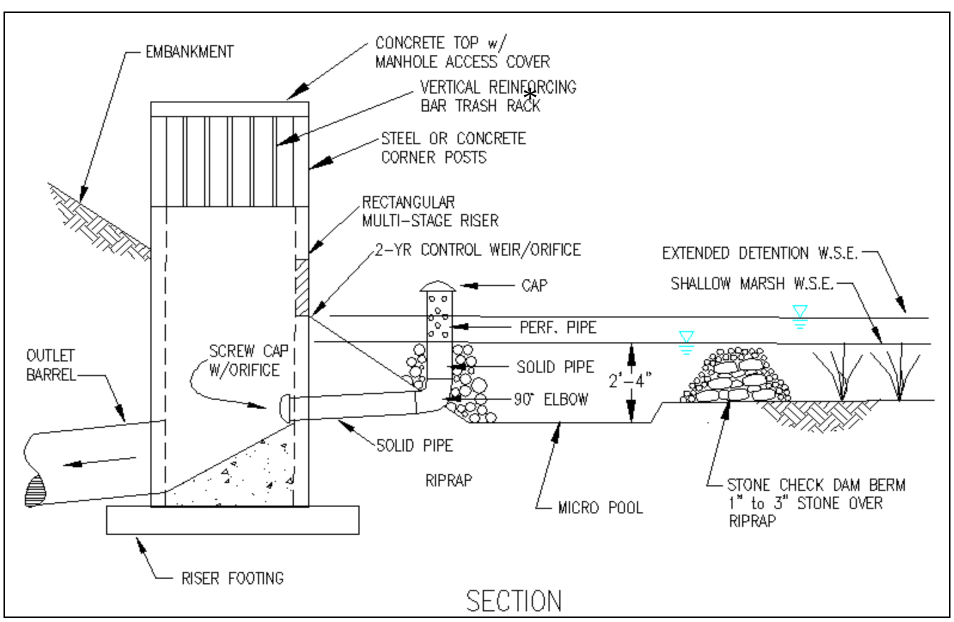


Figure 2.2 – DCR/DEQ Recommended Outlet Configuration 1 for the Control of Trash, Sediment and Debris
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

* Recommended minimum bar spacing of 2”, maximum bar spacing of 3”.

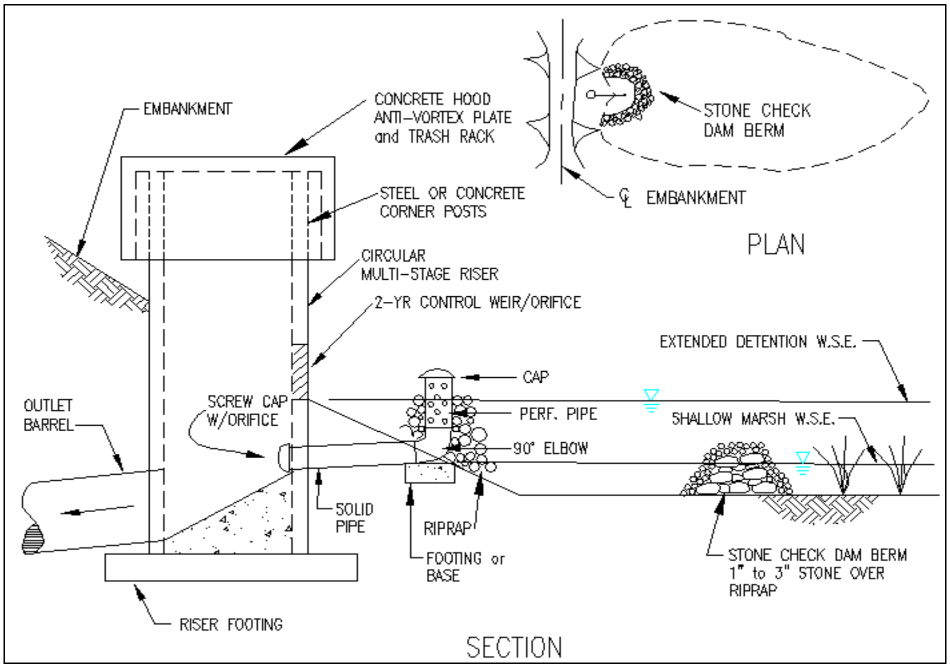


Figure 2.3 – DCR/DEQ Recommended Outlet Configuration 2 for the Control of Trash, Sediment and Debris
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

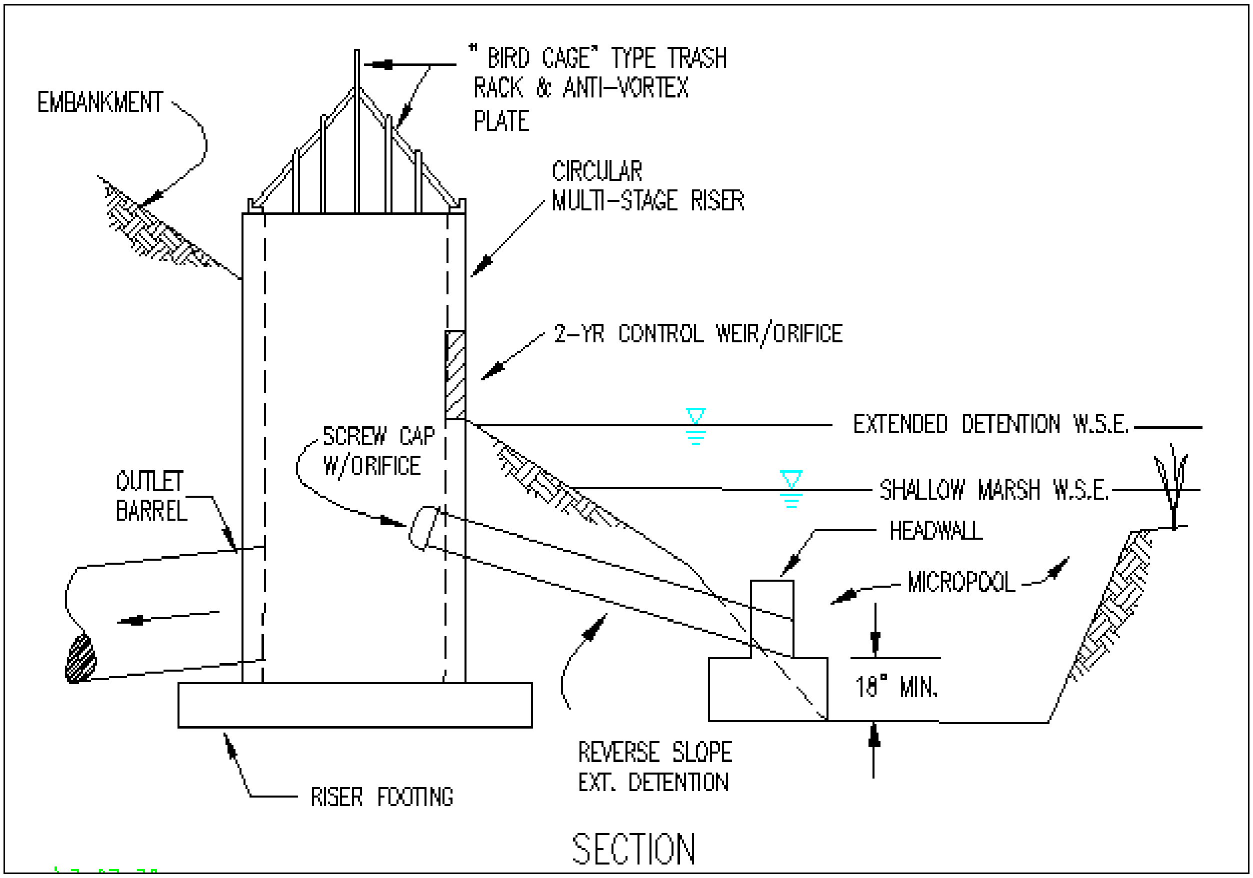


Figure 2.4. DCR/DEQ Recommended Outlet Configuration 3 for the Control of Trash, Sediment and Debris
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

2.1.2 Maximum Drainage Area

The maximum drainage area to an extended dry detention facility is frequently restricted to no more than 50 acres. When larger drainage areas are directed to a single facility, often there is a need to accommodate base flow through the facility. When no permanent pool is proposed, as with a dry extended detention basin, the presence of this base flow is a nuisance that presents a complex set of design challenges. The most notable concern is the “choking” of base flow conveyance such that a permanent pool volume accumulates and encroaches upon the volume of dry storage allocated to extended detention. A reduced extended detention volume results in ineffectively low hydraulic residence times for the water quality volume generated from significant rainfall events. Contrasting this problem is the situation occurring when the orifice allocated to pass-through of the base flow is sized too large to provide the desired minimum draw down time for the site’s water quality volume.

2.1.3 Separation Distances

Extended dry detention facilities should be kept a minimum of 20' from any permanent structure or property line, and a minimum of 100' from any septic tank or drainfield.

2.1.4 Site Slopes

Generally, extended detention basins should not be constructed within 50' of any slope steeper than 15%. When this is unavoidable, a geotechnical report is required to address the potential impact of the facility in the vicinity of such a slope.

2.1.5 Site Soils

The implementation of a dry extended detention basin can be successfully accomplished in the presence of a variety of soil types. However, when such a facility is proposed, *a subsurface analysis and permeability test is required*. Soils exhibiting excessively high infiltration rates are not suited for the construction of a dry extended detention facility, as they will behave as an infiltration facility until clogging occurs. The designer should also keep in mind that as the ponded depth within the basin increases, so does the hydraulic head. This increase in hydraulic head results in increased pressure, which leads to an increase in the observed rate of infiltration. To combat excessively high infiltration rates, a clay liner, geosynthetic membrane, or other material (as approved by the Materials Division) may be employed. The basin's embankment material must meet the specifications detailed later in this section and/or be approved by the Materials Division. Embankment design shall be in accordance with DCR dam safety regulations.

2.1.6 Rock

The presence of rock within the proposed construction envelope of a dry extended detention basin should be investigated during the aforementioned subsurface investigation. When blasting of rock is necessary to obtain the desired basin volume, a liner should be used to eliminate unwanted losses through seams in the underlying rock.

2.1.7 Existing Utilities

Basins should not be constructed over existing utility rights-of-way or easements. When this situation is unavoidable, permission to impound water over these easements must be obtained from the utility owner *prior* to design of the basin. When it is proposed to relocate existing utility lines, the costs associated with their relocation should be considered in the estimated overall basin construction cost.

2.1.8 Karst

The presence of Karst topography places even greater importance on the subsurface investigation. Implementation of dry extended detention facilities in Karst regions may greatly impact the design and cost of the facility, and must be evaluated early in the planning phases of a project. *Construction of stormwater management facilities within a sinkhole is prohibited.* When the construction of such facilities is planned along the periphery of a sinkhole, the facility design must comply with the guidelines found in Chapter 5 of this Manual and DCR/DEQ's Technical Bulletin #2 *"Hydrologic Modeling and Design in Karst."*

2.1.9 Wetlands

When the construction of a dry extended detention facility is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify the wetlands' boundaries, their protected status, and the feasibility of BMP implementation in their vicinity. In Virginia, the Department of Environmental Quality (DEQ) and the U.S. Army Corps of Engineers (USACOE) should be contacted when such a facility is proposed in the vicinity of known wetlands.

2.1.10 Upstream Sediment Considerations

Close examination should be given to the flow velocity at all basin inflow points. When entering flows exhibit erosive velocities, they have the potential to greatly increase the basin's maintenance requirements by transporting large amounts of sediment. Additionally, when a basin's contributing drainage area is highly pervious, there is a potential hindrance to the basin's performance by the transport of excessive sediment.

2.1.11 Floodplains

The construction of dry extended detention facilities within floodplains is strongly discouraged. When this situation is deemed unavoidable, critical examination must be given to ensure that the proposed basin remains functioning *effectively* during the 10-year flood event. The structural integrity and safety of the basin must also be evaluated thoroughly under 100-year flood conditions as well as the basin's impact on the characteristics of the 100-year floodplain. When basin construction is proposed within a floodplain, construction and permitting must comply with all applicable regulations under FEMA's National Flood Insurance Program.

2.1.12 Basin Location

When possible, dry extended detention facilities should be placed in low profile areas. When such a basin must be situated in a high profile area, care must be given to ensure that the facility empties completely within a 72 hour maximum, and that no stagnation occurs (see DCR/DEQ Reg. 44 CFR Part 5). The location of a dry extended detention basin in a high profile area places a great emphasis on facility maintenance.

"Design of any stormwater management facilities with permanent water features (proposed or potential) located within 20,000 feet of a public use or military airport is to be reviewed and coordinated in accordance with Appendix A Section A-6 of the VDOT Road Design Manual."

2.2 *General Design Guidelines*

The following presents a collection of broad design issues to be considered when designing a dry extended detention basin. Many of these items are expanded upon later in this document within the context of a full design scenario.

2.2.1 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam.

"The foundation material under the dam and the material used for the embankment of the dam should be an AASHTO Type A-4 or finer and/or meet the approval of the Materials Division. If the native material is not adequate, the foundation of the dam is to be excavated and backfilled a minimum of 4' or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."

If the basin embankment height exceeds 15', or if the basin includes a permanent pool, the design of the dam should employ a homogenous embankment with seepage controls or zoned embankments, or similar design in accordance with the Virginia SWM Handbook and recommendations of the VDOT Materials Division.

During the initial subsurface investigation, additional borings should be made near the center of the proposed basin when:

- Excavation from the basin will be used to construct the embankment
- There is a potential of encountering rock during excavation
- A high or seasonally high water table, generally 2' or less, is suspected

2.2.2 Outfall Piping

The pipe culvert under or through the basin's embankment shall be reinforced concrete equipped with rubber gaskets. Pipe: Specifications Section 232 (AASHTO M170), Gasket: Specification Section 212 (ASTM C443).

A concrete cradle shall be used under the pipe to prevent seepage through the dam. The cradle shall begin at the riser or inlet end of the pipe, and run the full length of the pipe.

2.2.3 Embankment

The top width of the embankment should be a minimum of 10’ in width to provide ease of construction and maintenance.

To permit mowing and other maintenance, the embankment slopes should be no steeper than 3H:1V.

2.2.4 Embankment Height

A detention basin embankment may be regulated under the Virginia Dam Safety Act, Article 2, Chapter 6, Title 10.1 (10.1-604 et seq.) of the Code of Virginia and Dam Safety Regulations established by the Virginia Soil and Water Conservation Board (VS&WCB). A detention basin embankment may be excluded from regulation if it meets any of the following criteria:

- is less than 6’ in height
- has a capacity of less than 50 acre-ft and is less than 25’ in height
- has a capacity of less than 15 acre-ft and is more than 25’ in height
- will be owned or licensed by the Federal Government

When an embankment is not regulated by the Virginia Dam Regulations, it must still be evaluated for structural integrity when subjected to the 100-year flood event.

2.2.5 Prevention of Short-Circuiting

Short circuiting of inflow occurs when the basin floor slope is excessive and/or the pond’s length to width ratio is not large enough. Short circuiting of flow can greatly reduce the hydraulic residence time within the basin, thus negatively impacting the desired water quality benefit.

To combat short-circuiting, and reduce erosion, the maximum longitudinal slope of the basin floor shall be no more than 2%. To maintain minimal drainage within the facility, the floor shall be no less than 0.5% slope from entrance to discharge point.

It is preferable to construct the basin such that the length to width ratio is 3:1 or greater, with the widest point observed at the outlet end. 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. When this minimum ratio is not possible, consideration should be given to pervious baffles.

2.2.6 Ponded Depth

The basin depth, measured from basin floor to primary outflow point (riser top or crest of orifice or weir) should not exceed 3’, if practical, to reduce hazard potential and liability issues.

2.2.7 Principal Spillway Design

The basin outlet should be designed in accordance with Minimum Standard 3.02 of the Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.). *The primary control structure (riser or weir) should be designed to operate in weir flow conditions for the full range of design flows*. If this is not possible, and orifice flow regimes are anticipated, the outlet must be equipped with an anti-vortex device, consistent with that described in Minimum Standard 3.02. The riser and barrel shall be designed to prevent surging or other adverse hydraulic conditions.

2.2.8 Emergency Spillway Stabilization

The emergency spillway shall be stabilized with rip rap, concrete, or any other non-erodible material approved by the VDOT Material Division.

2.2.9 Fencing

Fencing is typically *not required or recommended* on most VDOT detention facilities. However, exceptions do arise, and the fencing of a dry extended detention facility may be needed. Such situations include:

- Ponded depths greater than 3' and/or excessively steep embankment slopes
- The basin is situated in close proximity to schools or playgrounds, or other areas where children are expected to frequent
- It is recommended by the VDOT Field Inspection Review Team, the VDOT Residency Administrator, or a representative of the City or County who will take over maintenance of the facility

"No Trespassing" signs should be considered for inclusion on all detention facilities, whether fenced or unfenced.

2.2.10 Sediment Forebays

Each basin inflow point should be equipped with a sediment forebay. The forebay volume should range between 0.1" and 0.25" over the individual outfall's new impervious area or 10% of the required WQV (whichever is greater).

2.2.11 Discharge Flows

All basin outfalls must discharge into an adequate receiving channel per the most current Virginia Erosion and Sediment Control (ESC) laws and regulations. Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

2.3 Design Process

This section presents the design process applicable to dry extended detention basins serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered during linear development projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 4 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.) for expanded hydrologic methodology.

The following example basin design will provide the water quality and quantity needs arising from the construction of a section of two lane divided highway situated in Montgomery County. The total project site, including right-of-way and all permanent easements, consists of 17.4 acres. Pre and post-development hydrologic characteristics are summarized below in Tables 2.1 and 2.2. Peak rates of runoff for both pre and post-development conditions were computed by the Rational Method and the regional NOAA Atlas 14 factors (B, D, and E) recommended in the VDOT Drainage Manual.

Table 2.1 - Hydrologic Characteristics of Example Project Site

	Pre-Development	**Post-Development**
Project Area (acres)	17.4	17.4
Land Cover	Unimproved Grass Cover	4.75 acres *new* impervious cover
Rational Runoff Coefficient	0.30	0.50*
Time of Concentration (min)	45	10

*Represents a weighted runoff coefficient reflecting undisturbed site area and new impervious cover

Table 2.2 - Peak Rates of Runoff (cfs)

	Pre-Development	**Post-Development**
2-Year Return Frequency	7.97	15.7
10-Year Return Frequency	11.37	21.0

Step 1 - Compute the Required Water Quality Volume

The project site's water quality volume is a function of the development's new impervious area. This basic water quality volume is computed as follows:

$$WQV = \frac{NIA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

NIA= New Impervious Area (ft^2)

Dry extended detention basins should be designed to provide extended draw down for *two times the computed water quality volume (2xWQ$_V$).* If the basin is to be implemented as a *water quality basin*, this computed volume of twice the WQ_V must be detained and released over a period of not less than 30-hours. The basin must completely drawdown within 72 hours.

When the proposed basin is to function as a *channel erosion control* basin, the extended draw down volume is computed as the volume of runoff generated from the basin's contributing drainage area by the 1-year return frequency storm. This channel protection volume must be detained and released over a period of not less than 24 hours.

When the 1-year return frequency storm is detained for a minimum of 24 hours there is no need to provide additional or separate storage for the WQ_V provided it can be demonstrated that the WQ_V will be detained for approximately 24 hours.

It is noted that providing extended 24 hour (or longer) detention for the 1-year runoff volume may require the basin size to be 1.5 to 2 times the volume required to simply mitigate the 2 and 10-year runoff events to pre-development levels.

The basis of this example lies in the design of Best Management Practices for *water quality improvement*. Therefore, the example basin is sized as a water quality control basin and not a channel erosion control basin.

The demonstration project site has a total drainage area of 17.4 acres. The total new impervious area within the project site is 4.75 acres. Therefore, the water quality volume is computed as follows:

$$WQV = 4.74ac \times 43{,}560 \frac{ft^2}{ac} \times \frac{1}{2} in \times \frac{1 ft}{12 in} = 8{,}603\, ft^3$$

The total extended draw down volume for a dry extended detention basin is 2 x WQ_V, calculated as follows:

$$V = 2 \times 8{,}603\, ft^2 = 17{,}206 \text{ft}^3$$

The basin will be designed to provide a minimum 30-hour draw down time for a volume of 0.40 acre-ft.

Step 2 - Estimate the Volume Required for Mitigation of Post-Development Runoff Peaks to Equal or Less than Pre-Development Levels

Chapter 4 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.) details a number of different methods for estimating the peak rate of runoff from a watershed. Adhering to standard VDOT practice, we will employ the Modified Rational Method in this section to both size and model the example basin.

The Modified Rational Method is a hydrograph generating variation of the Rational formula of runoff peak estimation. It is used on small sites for the sizing of impoundment / detention facilities. The fundamental difference between the Rational Method and the Modified Rational Method lies in the application of fixed rainfall duration. The Rational Method generates a peak discharge that occurs when the entire drainage area is contributing runoff to the point of interest (storm duration equal to watershed time of concentration). The Modified Rational Method considers not only this situation, but also examines storms exhibiting a longer duration than the watershed time of concentration. Such storms may exhibit lower *peak rates* of runoff but higher *volumes* of runoff. The fixed rainfall duration is generally selected as that which requires the greatest storage volume to mitigate post-development runoff for the return frequency of interest. Hydrographs generated by the Modified Rational Method may be triangular or trapezoidal in shape. Figure 2.5 presents the two types of runoff hydrographs that can arise from the Modified Rational Method. Note that the first type of hydrograph is that computed by the simple Rational Method.

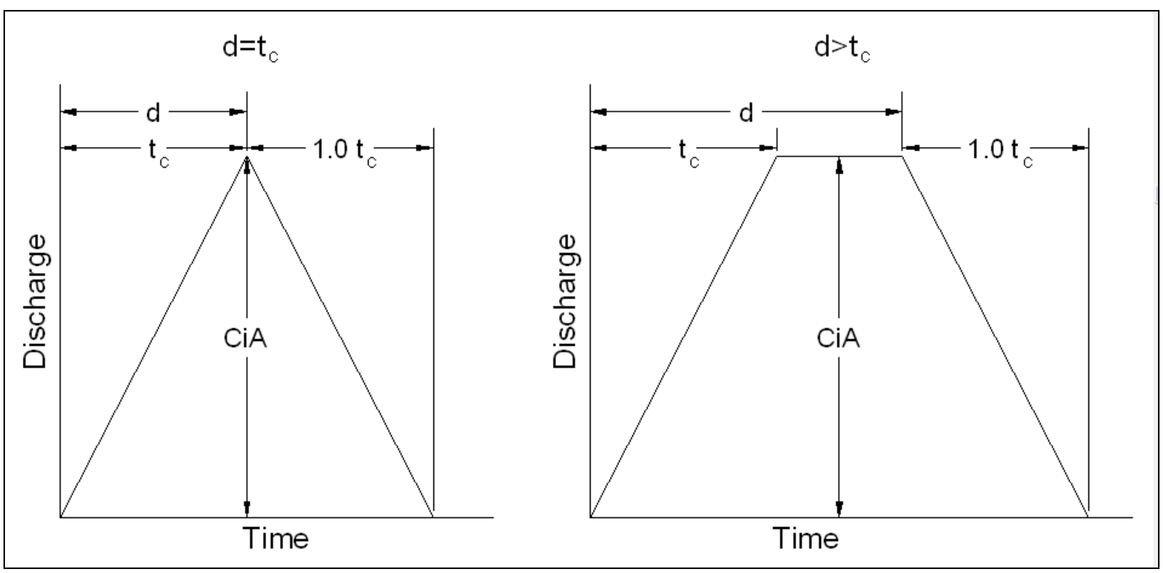


Figure 2.5 - Modified Rational Runoff Hydrographs

Selection of the critical rainfall intensity averaging period can be accomplished by an iterative graphical approach or a simpler, direct, analytical approach.

The graphical approach requires the user to construct a plot, to some scale, of a family of hydrographs and an allowable release rate. The family of hydrographs will be generated by first selecting various rainfall intensity averaging periods. These periods should be such that their corresponding rainfall intensities are readily available (i.e. 10, 20, 30 min., etc.). The allowable release rate will generally be established as the pre-development runoff rate for the return frequency storm of interest. The critical rainfall averaging period may differ among various return frequency storms, and thus requires the construction of individual plots for *each return frequency* for which detention is proposed. Graphically, the basin outflow hydrograph is represented as a straight line which starts at time zero and rises linearly to the intersection of the hydrograph's receding limb and the allowable release rate. Figure 2.6 illustrates a typical plot for determining the critical rainfall intensity duration.

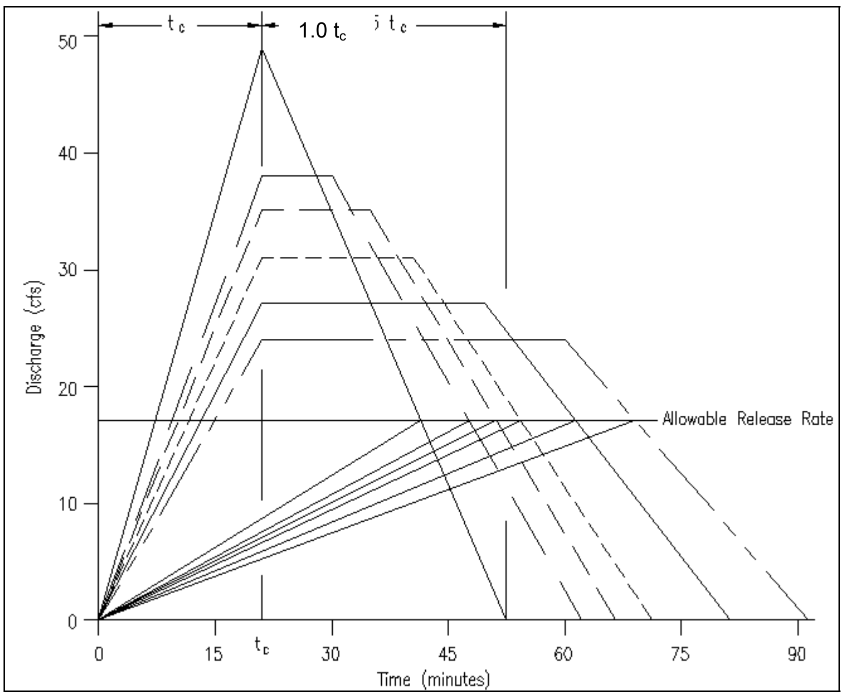


Figure 2.6 – DCR/DEQ Graphical Determination of Critical Rainfall Intensity Duration

(<u>*Virginia Stormwater Management Handbook*</u>, 1999, Et seq.)

The triangular hydrograph shown in Figure 2.6 is generated from a rainfall averaging period equal to the watershed time of concentration. Its peak discharge is computed as the product *Q=CiA,* with *"i"* derived from the rainfall intensity corresponding to the time of concentration. By contrast trapezoidal-shaped hydrographs exhibit a peak discharge also computed as the product of *CiA*, but with the *"i"* parameter derived from the rainfall intensity corresponding to the selected duration.

The critical rainfall intensity averaging period is the one which produces the greatest storage volume. The required detention volume for each of the various rainfall intensity averaging periods is a function of the area lying between the inflow hydrograph and the corresponding basin outflow. For an intra-hydrograph area computed in square inches (as in Figure 2.6 for example), a typical conversion is shown as follows:

$$V = in^2 \times A\left(\frac{\min}{in}\right) \times \frac{60\sec}{\min} \times B\left(\frac{cfs}{in}\right)$$

Variables *"A"* and *"B"* scaling factors measured respectively in minutes per inch and cfs per inch from the plot scales.

The iterative graphical approach to determining the critical rainfall duration is time intensive, cumbersome, and provides numerous opportunities for error. A direct analytical approach to determining the critical rainfall duration is recommended, and demonstrated as follows.

The critical storm duration is determined from the following equation, with variables as defined:

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

T_d= critical storm duration for the return period of interest
C= rational runoff coefficient (developed conditions)
A= drainage area (acres)
t_c= post-development time of concentration
q_o= allowable peak rate of outflow from basin
a= geographic rainfall regression constant
b= geographic rainfall regression constant

Regression constants *"a"* and *"b"* can be found in Appendix 5A of the Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.). The coefficients for the example project site, located in Montgomery County, are presented below.

Table 2.3 - Rainfall Regression Constants
Montgomery County

	2-Year	10-Year
a	118.78	177.0
b	19.21	22.39

Setting the allowable release rates equal to the respective pre-developed peak rates of runoff for the 2 and 10-year return frequency events, the critical storm durations are computed as follows:

$$T_2 = \sqrt{\frac{(2)(0.50)(17.4ac)(118.78)(19.21 - \frac{10\min}{4})}{7.97cfs}} - 19.21 = 46.6\min$$

$$T_{10} = \sqrt{\frac{(2)(0.50)(17.4ac)(176.95)(22.39 - \frac{10\min}{4})}{11.37cfs}} - 22.39 = 51.0\min$$

The next step is to apply the computed critical durations to determine the corresponding rainfall intensities. This intensity is defined as follows, with variables as previously defined.

$$I = \frac{a}{b + T_d}$$

The 2 and 10-year return intensities are computed as follows:

$$I_2 = \frac{118.78}{19.21 + 46.6} = 1.80\frac{in}{hr}$$

$$I_{10} = \frac{176.95}{22.39 + 51.0} = 2.41\frac{in}{hr}$$

The peak rate of runoff from the post-development site under the critical storm is then determined using the Rational Method equation.

$$Q = CiAC_f$$

Q= runoff rate (cfs)
i= rainfall intensity (in/hr) corresponding to the critical duration
C= post-development runoff coefficient
A= drainage area (acres)
C_f= Correction factor for ground saturation (1.0 for storm return frequency of 10 years or less)

$$Q_2 = (0.50)(1.80)(17.4)(1.0) = 15.7cfs$$

$$Q_{10} = (0.50)(2.41)(17.4)(1.0) = 21.0cfs$$

Finally, the volume of detention storage required to reduce the post-development runoff rates to pre-development levels can be estimated from the following equation.

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

V= required storage volume (ft^3)
Q_i= peak inflow for critical storm (cfs)
t_c= post-development time of concentration
q_o= allowable release rate from basin
T_d= critical storm duration

The estimated detention volumes required to mitigate the peak rate of runoff from the 2 and 10-year post-development events to pre-development levels are computed as follows.

$$V_2 = \left[(15.7)(46.6) + \frac{(15.7)(10)}{4} - \frac{(7.97)(46.6)}{2} - \frac{(3)(7.97)(10)}{4} \right] 60 = 31{,}523.6\, ft^3$$

$$V_{10} = \left[(21.0)(51.0) + \frac{(21.0)(10)}{4} - \frac{(11.37)(51.0)}{2} - \frac{(3)(11.37)(10)}{4} \right] 60 = 44{,}897.4\, ft^3$$

Step 3 - Development of Runoff Hydrographs

Having determined the critical storm durations and their corresponding peak runoff rates, it is now possible to construct full inflow hydrographs by the Modified Rational Method. The general shape of these hydrographs is shown in Figure 2.7.

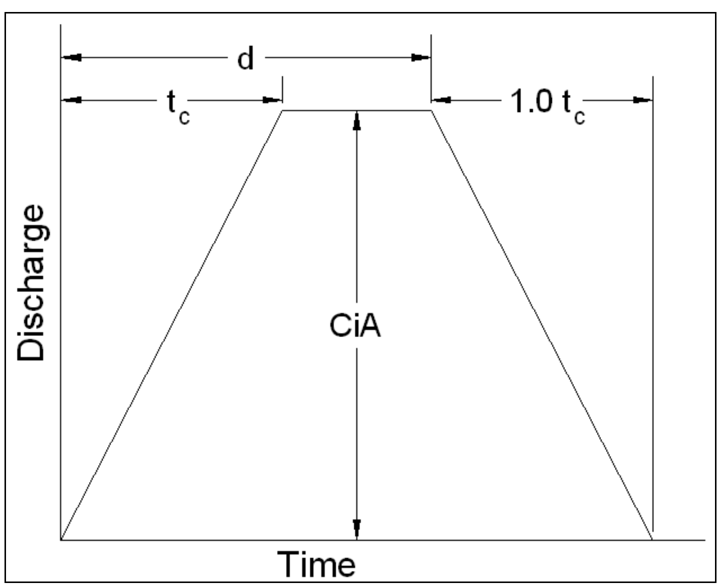


Figure 2.7 - Modified Rational Hydrograph Shape

The hydrographs developed with the previously computed parameters are presented below as Figures 2.8 and 2.9. These hydrographs subsequently will be routed by the storage indication method to verify pond sizing and outlet structure design.

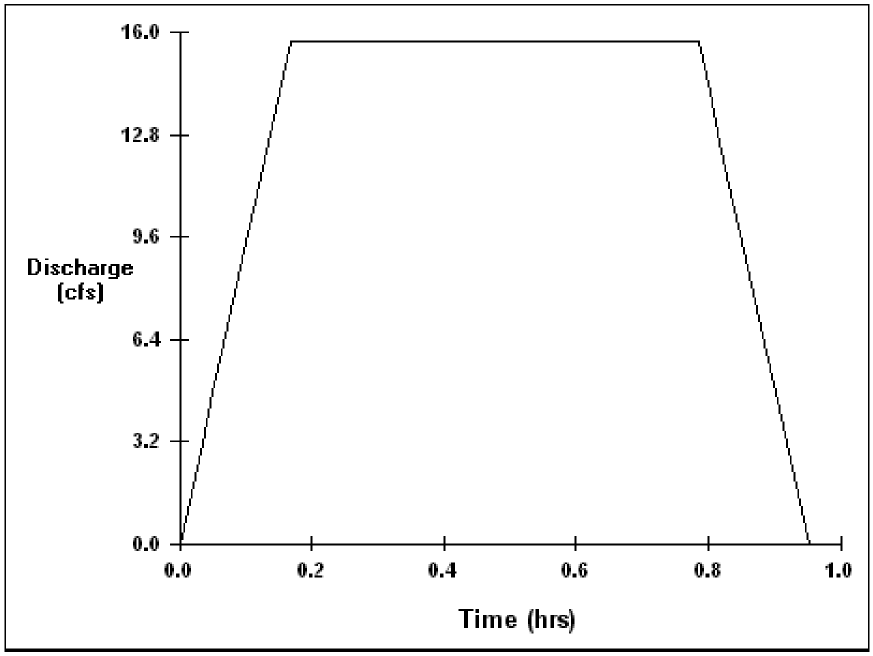


Figure 2.8 - 2-Year Post-Development Modified Rational Hydrograph

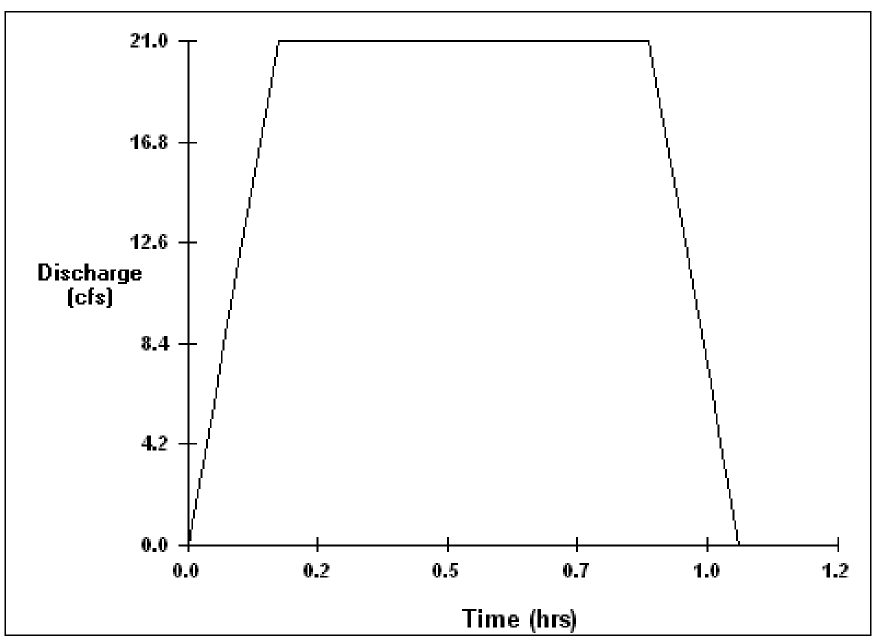


Figure 2.9 - 10-Year Post-Development Modified Rational Hydrograph

Step 4 - Development of Storage Versus Elevation Data

Having determined the required storage volumes, we now turn to developing the preliminary basin grading plan in order to establish the relationship between ponded depth and storage volume. Site geometry and topography must be carefully examined during the siting and grading of the basin. As well as providing the peak mitigation volumes estimated previously, the pond grading must also provide safe passage of the 100-year runoff producing event without breaching the basin embankment. The required freeboard depths under 100-year conditions are as follows:

- When equipped with an emergency spillway, the basin must provide a minimum of 1' of freeboard from the maximum water surface elevation arising from the 100-year event and the lowest point in the embankment.

- When no emergency spillway is provided, a minimum of 2' of freeboard should be provided between the maximum water surface elevation produced by the 100-year runoff event and the lowest point in the embankment.

In addition to considering site geometry and topography, the previously discussed *"General Design Guidelines"* should also be closely integrated into the proposed basin grading. Side slope steepness, length-to-width ratio, and desirable ponded depth must be considered. The total storage volume is computed from the lowest stage outlet.

Pond sizing is, generally, an iterative process. A typical storage versus elevation data table and curve are presented in Table 2.4 and Figure 2.10. The data presented represents a basin of rectangular orientation with an approximate length-to-width ratio of 3:1 and variable side slopes (minimum 3H:1V). Note that the computed water quality volume is provided at a depth of less than 3'. This will permit the invert of the principal outlet or weir to be placed at a depth of less than 3'. This condition should be met when practically possible. The storage – elevation data presented below is intended only to serve as a means of illustrating the outlet structure design and storm routing steps of the design procedure. It does not reflect an actual grading plan.

Table 2.4 - Basin Storage Versus Elevation Data

Elevation (ft)	Storage (CF)	Storage (AF)
2100	0	0
2100.5	3,920	0.09
2101	7,841	0.18
2101.5	12,197	0.28
2102	16,553	0.38
2102.5	21,780	0.50
2103	27,007	0.62
2103.5	37,026	0.85
2104	52,272	1.20
2104.5	69,696	1.60
2105	91,476	2.10
2105.5	113,256	2.60
2106	139,392	3.20
2106.5	169,884	3.9

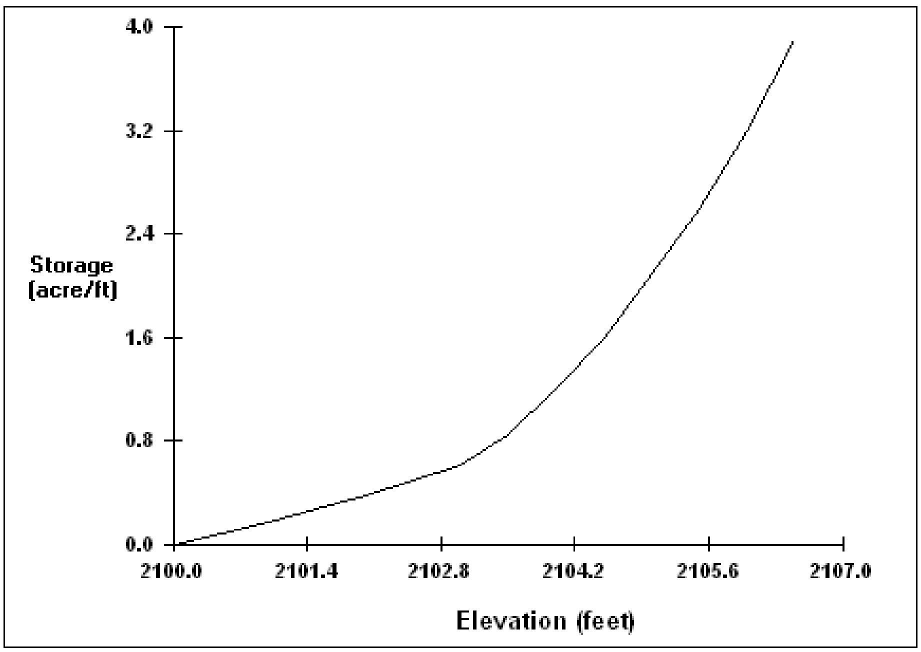


Figure 2.10 - Basin Storage Versus Elevation Curve

Step 5 - Design of the Water Quality Control Orifice

The previously computed water quality volume of 0.40 acre-ft (17,424 ft^3) must be detained and released over a period of not less than 30 hours. This requires the design of a controlling orifice.

The first step is to determine the ponded depth within the basin that provides the extended draw down volume of 0.40 acre-ft. Linearly interpreting the storage – elevation table presented as Table 2.4, we see that this volume is provided at a ponded depth of 2.1', or at elevation 2102.1.

The Virginia Stormwater Management Handbook identifies two methods for sizing a water quality release orifice. The VDOT preferred method is the “average head/average discharge” approach as presented below.

The water quality volume is attained at a ponded depth of 2.1’, therefore the average discharge and head associated with this volume are computed as:

$$h_{avg} = \frac{2.1\,ft}{2} = 1.05\,ft$$

$$Q_{avg} = \frac{WQV}{(30hr)(3{,}600\sec/hr)} = \frac{17{,}424\,ft^3}{(30hr)(3{,}600\sec/hr)} = 0.16cfs$$

Next, the orifice equation is rearranged and used to compute the required orifice diameter.

$$Q = Ca\sqrt{2gh}$$

Q= discharge (cfs)
C= orifice Coefficient (0.6)
a= orifice Area (ft^2)
g= gravitational acceleration (32.2 ft/sec^2)
h= head (ft)

The head is estimated as that acting upon the *invert* of the water quality orifice when the total water quality volume of 17,424 ft^3 is present in the basin. While the orifice equation should employ the head acting upon the center of the orifice, the orifice diameter is presently unknown. Therefore, the head acting upon the orifice invert is used. As demonstrated in the water quality draw down verification later in this section, the error incurred from this assumption does not compromise the usefulness of the results.

Rearranging the orifice equation, the orifice area is computed as

$$a = \frac{Q_{avg}}{C\sqrt{2gh}} = \frac{0.16}{0.6\sqrt{(2)(32.2)(1.05)}} = .03\,ft^2$$

The diameter is then computed as:

$$d = \sqrt{\frac{4a}{\pi}} = \sqrt{\frac{(4)(0.03)}{3.14}} = 0.20\,ft = 2.4in$$

The computed orifice diameter is less than 3”. However, a 3” diameter will be chosen, and later verified for adequacy by storage indication routing.

Step 6 - Design of the Principal Spillway

The basin principal spillway controls the rate at which storms are released from the basin. To control the release rate for multiple return frequency storms, the spillway will typically need to be *multi-staged*. A multi-stage riser employs various precisely located outlets such that the desired target release rates are achieved for all chosen return frequencies. Hydraulic modeling of a basin's principal spillway is termed "Reservoir Routing" or "Storage Indication Routing." The basic input parameters for this modeling are:

- o Stage – Storage Relationship
- o Stage – Discharge Relationship
- o Inflow Hydrograph(s)

The design of a principal spillway to control multiple return frequency storms is usually iterative. A design which attains target release rates along with minimized storage volume and ponded depth will often require several iterations and the subsequent refinement of stage – discharge and/or stage – storage data. A number of proprietary desktop computing programs are available to assist in principal spillway design process. A non-exhaustive list of these programs includes Hydraflow, PondPack, HydroCAD, and the Virginia Tech Penn State Urban Hydrology Model (VTPSUHM). Each of these programs employ the same basic methodology of routing, which includes subjecting a given pair of stage – storage and stage – discharge relationships to some inflow hydrograph. The following steps will demonstrate the fundamental process of designing a basin's principal spillway. The routing operations are conducted using the Virginia Tech/Penn State Urban Hydrology Model (VTPSUHM). In the absence of acceptable hydraulic computing software, the calculations shown here can be done by hand. Refer to Section 5-9 of the *Virginia Stormwater Management Handbook*, 1999, Et seq. or any standard textbook on water resources engineering for information on manual storage indication routing.

Step 6A - Size Basin Outfall Culvert

Before proceeding to the design of various outlets in the multi-stage riser structure, we must first size the outfall conduit conveying pond releases through the embankment and into the receiving channel. The first step is to determine the outlet conduit's maximum discharge and corresponding ponded depth in the basin. Flows in excess of the 10-year runoff producing event will be conveyed through an emergency spillway. Therefore, the design discharge for the culvert is that of the routed 10-year event. The 10-year post-development runoff must be detained and released at a rate equal to or less than the 10-year pre-development runoff. This value was computed previously as *11.37 cfs*.

Step 2 of this example detailed the Modified Rational approach to estimating the detention volume necessary to reduce the 10-year peak runoff rate to that of pre-development conditions. This volume was found to be 44,897ft^3. Linearly interpreting the stage – storage data (Table 2.4), we find this volume at basin elevation *2103.7*. This ponded depth corresponds to an *approximate* head of 3.7' acting upon the outfall culvert during 10-year conditions.

The next step is to employ FWHA culvert rating charts like the one shown on the following page. This chart is taken from FHWA HDS 5, "Design of Highway Culverts" (1985, revised 2001). The use of the inlet control chart for sizing the culvert is done only to develop a first trial value of the culvert diameter. Once this is done, the elevation-discharge rating table for the culvert will be computed by VTPSUHM (or other software), whereby the selected culvert is checked for inlet versus outlet control at each water surface elevation in the outer pond. In other words, for a given water surface elevation in the pond, the headwater depth in the riser box will be computed under inlet control and then under outlet or friction control to determine which condition controls the discharge capacity at that elevation. The larger of the two headwaters will dictate the hydraulic control. Once the rating table is generated in VTPSUHM (or other software), the designer can then route the design hydrograph through the outlet structure (which includes the outfall culvert) to determine if the design has met the outflow target. If it does not, the designer must select a larger or smaller culvert size and repeat the rating table development and routing steps until a satisfactory design solution is achieved. Selecting a RCP outfall culvert with a finished concrete entrance, and making the initial assumption of a headwater depth to pipe diameter ratio of 1.5, we observe that an 18" culvert appears to be adequate for a discharge of 11.4 cfs at headwater depths exceeding 2.25' (1.5D). Note that the 18" RCP outfall culvert is attached to the back of the riser box assembly and represented in all subsequent design calculations.

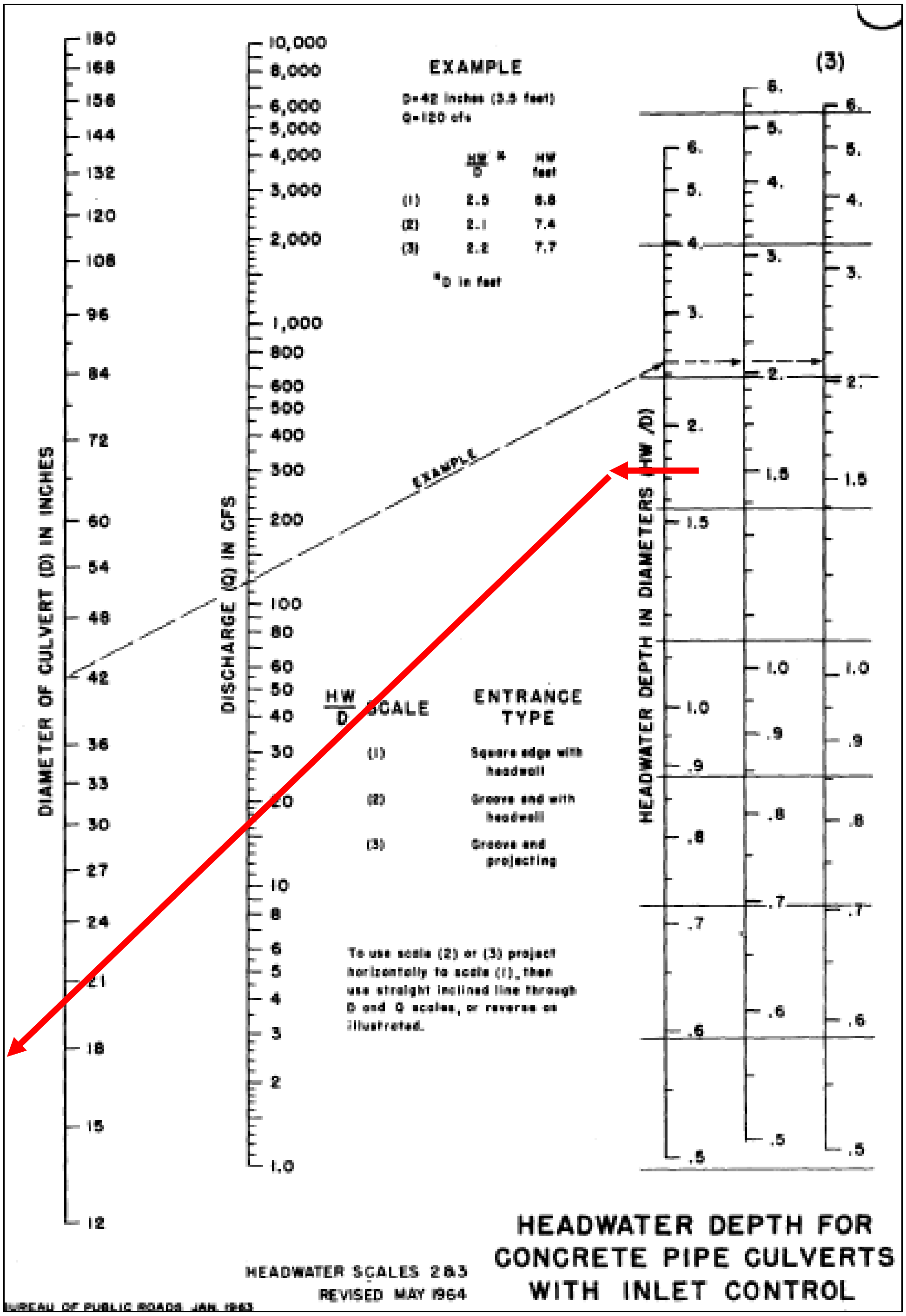


Figure 2.11 - Culvert Design Chart (FHWA, 2001)

For an 18" diameter pipe acting under the available 3.7' of hydraulic head during 10-year discharge, the estimated HW/D is:

$$\frac{HW}{D} = \frac{3.7\,ft}{(18in)\left(\frac{1\,ft}{12in}\right)} = 2.5$$

By aligning HW/D = 2.5 and D = 18”, we see that the estimated capacity is about 29 cfs. This is certainly conservative. For purposes of this design, we will employ an 18” culvert placed on a 1% slope leaving the proposed riser structure. Note that this culvert will be submitted to full testing in subsequent flood routings by VTPSUHM, as described later.

Step 6B. - Design the 2-Year Control Outlet

The first step in sizing the 2-year control outlet is to determine the basin water surface elevation at which the estimated 2-year detention volume is provided. Step 2 detailed the Modified Rational approach to estimating the 2-year detention volume required to reduce the 2-year peak runoff rate to the pre-development level. This volume was found to be 31,523.6 ft^3. Linearly interpreting the stage – elevation data (Table 2.4), we find this volume at basin elevation *2103.2’*.

The next step is to estimate the maximum hydraulic head acting on the 2-year control outlet. The crest/invert of the 2-year control outlet should be set just above the surface of the ponded water quality volume. The water quality volume was found to occur at basin elevation 2102.1’. Therefore, the crest of the 2-year control outlet is set at elevation *2102.2*’, and the maximum estimated head acting upon the 2-year outlet is the difference between the ponded water surface elevation and the crest of the outlet:

$$h_{2-year} = 2103.2\,ft - 2102.2\,ft = 1.0\,ft.$$

The designer has an essentially unlimited number of weir and orifice shapes, geometries, and sizes from which to choose. However, unless unique site restraints prohibit such a design, the outlets comprising the principal spillway should function in weir flow for *all* design storms. When site conditions are such that weir flow cannot be maintained, an anti-vortex device must be provided in accordance with the specifications detailed in the Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.).

Regardless of the shape and size chosen, the outlet will function under weir flow conditions until the entire opening is submerged. Therefore, the weir equation is very useful in selecting control outlet sizes and shapes. The weir equation is shown as follows:

$$Q = C_W L h^{1.5}$$

Q= Weir flow discharge (cfs)
C_W= Weir coefficient (3.1 for most sharp-crested weirs)
L= Weir crest length (ft)
h= Head measured from the water surface elevation to the crest of the weir (ft)

When rearranged, the weir equation can be used to compute weir lengths necessary to meet basin release targets. The rearranged form of the weir equation, with variables as previously defined, is shown as follows:

$$L = \frac{Q}{C_W h^{1.5}}$$

Another useful approach in the sizing of *circular* orifices is to select an orifice diameter that is just slightly larger than that required under orifice flow. Sizing the orifice in this manner will ensure that, for the available storage volume, the orifice provides the minimal release from the basin that is possible while remaining under weir flow conditions. This approach utilizes the orifice equation, shown as follows:

$$Q = Ca\sqrt{2gh}$$

Q= Discharge (cfs)
C= Orifice coefficient (0.6)
a= Orifice area (ft^2)
g= Gravitational acceleration (32.2 ft/sec^2)
h= Head (ft)

The previously estimated head acting upon the 2-year control outlet is 1.1', and the target 2-year release from the basin is 7.97 cfs. Rearranging the orifice equation and applying these values, we compute the diameter as follows:

$$a = \frac{Q}{C\sqrt{2gh}} = \frac{7.97}{0.6\sqrt{(2)(32.2)(1.0)}} = 1.65\,ft^2$$

The diameter is then computed as:

$$d = \sqrt{\frac{4a}{\pi}} = \sqrt{\frac{(4)(1.65)}{3.14}} = 1.4\,ft = 16.8in$$

To ensure that the orifice does not become submerged, thus inducing orifice flow, the orifice diameter is increased to the nominal size of 18".

Next, the designer must construct the stage – discharge relationship for the chosen outlet. It is noted that the stage – discharge curve should reflect not only the 2-year control outlet, but also the 18" concrete outfall culvert. Typically, on VDOT projects, the water quality orifice is not considered in the flood control rating curve(s). Table 2.5 presents the stage – discharge relationship for the 2-year control orifice, and the 18" concrete outfall culvert.

Stage 1: Circular Orifice
Invert = 2102.2
Discharge Coefficient = 0.6
Diameter = 18 in

Stage 2: Outfall Culvert (RCP)
Invert = 2100.0
Diameter = 18 in

Table 2.5 - Preliminary Stage – Discharge Relationship

Basin Water Elevation (ft)	Basin Outflow (cfs)
2100.00	0.00
2100.50	0.00
2101.00	0.00
2101.50	0.00
2102.00	0.00
2102.50	0.35
2103.00	2.27
2103.50	5.55
2104.00	8.72
2104.50	10.59
2105.00	11.46
2105.50	12.33
2106.00	13.34
2106.50	14.35
2107.00	15.03

Next, using the stage – storage and stage – discharge data, along with the 2-year return frequency post-development Modified Rational hydrograph, we apply storage indication routing to determine the actual peak discharge and maximum storage volume used during this event. The results of this routing are shown on the following page.

Modified Puls Output

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
0.73	15.70	15.70	0.8165	2090.32	2.10	2.10
0.75	15.70	15.70	0.8352	2097.68	2.20	2.20
0.77	15.70	15.70	0.8515	2103.50	5.56	5.56
0.78	15.70	15.70	0.8654	2103.52	5.69	5.69
0.80	14.13	14.13	0.8781	2103.54	5.81	5.81
0.82	12.56	12.56	0.8884	2103.56	5.90	5.90
0.83	10.99	10.99	0.8965	2103.57	5.97	5.97
0.85	9.42	9.42	0.9023	2103.58	6.02	6.02
0.87	7.85	7.85	0.9059	2103.58	6.06	6.06
0.89	6.28	6.28	0.9072	2103.58	6.07	6.07
0.90	4.71	4.71	0.9065	2103.58	6.06	6.06
0.92	3.14	3.14	0.9035	2103.58	6.04	6.04
0.94	1.57	1.57	0.8985	2103.57	5.99	5.99
0.95	0.00	0.00	0.8913	2103.56	5.93	5.93

Total Routing Mass Balance Discrepancy is 0.40%

Save Outflow Hydrograph | Print | Print Summary | Perform Another Routing | Done

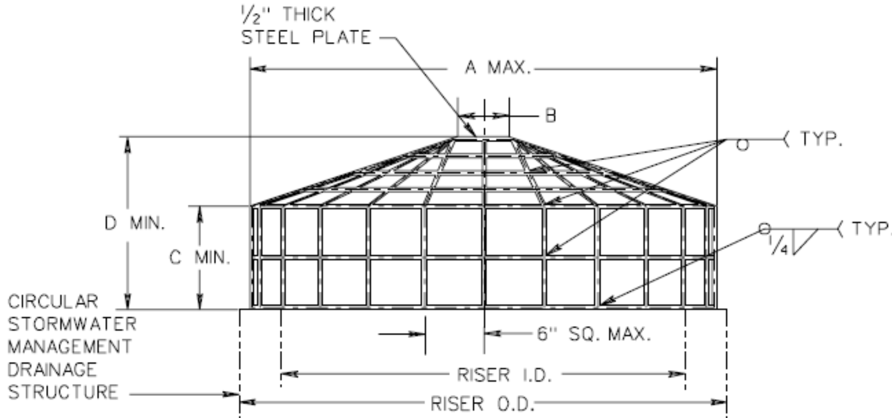
Figure 2.12 - Preliminary Routing Results – 2-Year Inflow Hydrograph

The results reveal a peak discharge from the basin of 6.07cfs, a value below the maximum allowable release rate of 7.97 cfs. Additionally, the maximum observed water surface elevation is 2103.58', 1.38' above the invert of the 2-year control orifice. This indicates that the 18" circular orifice is never completely submerged, and thus *does not support orifice flow conditions*.

The use of a smaller diameter outlet would subject the outlet to more hydraulic head. This increased hydraulic head could raise the maximum discharge from the basin. In doing so, the release rate could be brought closer to the target rate of 7.97 cfs. However, this would likely` place the outlet in an orifice flow regime – a condition which should be avoided when possible.

Step 6C - Design the 10-Year Control Outlet

As with the 2-year control outlet, the designer has a multitude of options for the control of larger runoff producing events. These options range from circular riser tops equipped with a "bird cage" trash rack to various types of grated inlet tops. Regardless of the type of riser top selected, the effective weir length and total flow area of the configuration must be known in order to design and model the structure. This design example will employ a "bird cage" trash rack top consistent with the SWM-DR, 114.07 structure detailed in the Virginia Department of Transportation Road and Bridge Standards, (VDOT, 2016). A detail of this type of inlet top is shown in Figure 2.13.



RISER		DIMENSION						APPROX.
I.D.	O.D.	A	B	C	D	E	SEGMENT	WT. (LBS.)
24	30	30	6	7	10	27	15	46
36	44	42	6	13	18	40	22	82
48	58	55	9	13	20	53	29	120
60	72	68	9	17	26	66	36	169
72	86	81	9	23	34	79	42	227
84	100	94	12	25	37	92	49	290
96	114	107	12	22	36	105	56	341

Figure 2.13 - VDOT SWM-DR Inlet Top (Metal)
<u>*VDOT Road and Bridge Standards*</u> (2016)

In this example, we will employ a square riser with interior dimensions (I.D.) of 48", consistent with structure SWM-1 shown below in Figure 2.14.

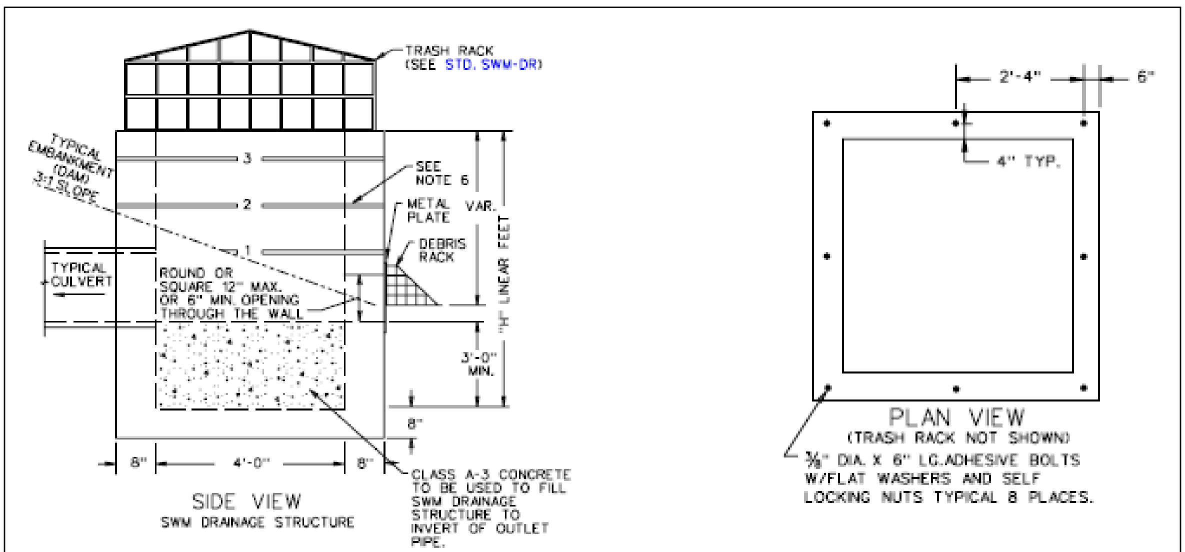


Figure 2.14 - VDOT SWM-1 Riser
<u>*VDOT Road and Bridge Standards*</u>

For the SWM-1 square riser, the effective weir length and flow area are 16’ and 16 square feet respectively.

Examining the estimate of required detention volume developed in Step 2, we see that 44,897.4 ft^3 of storage is required to mitigate the 10-year post-development runoff event. This storage volume occurs at a basin elevation of *2103.8’*. Linearly interpolating the previously developed stage – discharge data, at this water surface elevation we can see that the 2-year control outlet is discharging approximately 7.45 cfs. Therefore the design flow for the riser top is computed as the difference between the allowable pre-development release rate and the flow being discharged through the 2-year control outlets:

$$Q_{Design} = 11.37cfs - 7.45cfs = 3.92cfs$$

The outlet should be designed to operate under weir flow conditions. This assumption will be made to establish the riser crest elevation. Verification of the weir flow assumption will later be made. Placement of the riser crest is determined as follows:

Weir equation: $Q = CPh^{1.5}$

C = discharge coefficient (3.1)
P = effective perimeter (ft)
h = head acting on weir (ft)

$$h = \left(\frac{Q}{CP}\right)^{\frac{2}{3}} = \left(\frac{3.92}{(3.1)(16)}\right)^{\frac{2}{3}} = 0.18\,ft$$

Crest elevation of riser: $2103.8\,ft - 0.18\,ft = 2103.6\,ft$

This elevation, however, coincides with the top of the 18” orifice controlling the 2-year storm flows. Therefore, to provide a minimum separation, the crest elevation of the riser is set at 2103.9’.

Next, a stage – discharge relationship is built for the 2-year control outlet, the riser weir top, and the outfall culvert. This relationship is shown in Table 2.6.

Stage 1: Circular Orifice
Invert = 2102.2’
Discharge Coefficient = 0.6
Diameter = 18 in

Stage 2: SWM-1 Riser
Crest Elev. = 2103.9’

Stage 3: Outfall Culvert (RCP)
Invert = 2100.0’
Diameter = 18 in

Table 2.6 - Final Stage – Discharge Relationship

Basin Water Elevation (ft)	18” Orifice Outflow (cfs)	SWM-1 Riser Outflow (cfs)	Total Basin Outflow (cfs)
2100.00	0.00	0.00	0.00
2100.50	0.00	0.00	0.00
2101.00	0.00	0.00	0.00
2101.50	0.00	0.00	0.00
2102.00	0.00	0.00	0.00
2102.50	0.35	0.00	0.35
2103.00	2.27	0.00	2.27
2103.50	5.55	0.00	5.55
2104.00	8.72	1.57	10.29
2104.50	10.59	23.06	33.65
2105.00	11.46	57.95	69.41
2105.50	12.33	98.71	111.04
2106.00	13.34	113.16	126.50
2106.50	14.35	125.90	140.25
2107.00	15.03	137.74	152.77

Next, using the stage – storage and revised stage – discharge data, along with the 10-year return frequency post-development Modified Rational hydrograph, we will conduct storage indication routing to determine the actual peak discharge and maximum storage volume used during this event. The results of this routing are shown on the following page.

Modified Puls Output

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
0.80	21.00	21.00	1.0942	2103.85	8.86	8.86
0.82	21.00	21.00	1.1108	2103.87	9.08	9.08
0.83	21.00	21.00	1.1271	2103.90	9.30	9.30
0.85	21.00	21.00	1.1431	2103.92	9.52	9.52
0.87	18.90	18.90	1.1574	2103.94	9.71	9.71
0.89	16.80	16.80	1.1685	2103.96	9.86	9.86
0.90	14.70	14.70	1.1766	2103.97	9.97	9.97
0.92	12.60	12.60	1.1816	2103.97	10.04	10.04
0.94	10.50	10.50	1.1837	2103.98	10.07	10.07
0.95	8.40	8.40	1.1828	2103.98	10.05	10.05
0.97	6.30	6.30	1.1791	2103.97	10.00	10.00
0.99	4.20	4.20	1.1726	2103.96	9.92	9.92
1.00	2.10	2.10	1.1634	2103.95	9.79	9.79
1.02	0.00	0.00	1.1514	2103.93	9.63	9.63

Total Routing Mass Balance Discrepancy is 0.45%

Save Outflow Hydrograph | Print | Print Summary | Perform Another Routing | Done

Figure 2.15 - Routing Results – 10-Year Inflow Hydrograph

The results reveal a peak discharge from the basin of 10.07 cfs, a value below the maximum allowable release rate of 11.37 cfs.

Now, the weir flow assumption must be verified for accuracy. This is done by computing both the weir and orifice flow values for the observed head. The lower of the two values is the controlling condition.

From Figure 2.15, the actual head acting on the grate = 2103.98' – 2103.9' = *0.08'*. Using the orifice equation, the discharge is computed as follows:

$$Q = CA\sqrt{2gh}$$

$$Q = (0.6)(16)\sqrt{(2)(32.2)(0.08)} = 21.79cfs$$

The discharge computed for weir conditions acting under the same head:

$$Q = CPH^{1.5}$$

$$Q = (3.1)(16)(0.08)^{1.5} = 1.12cfs$$

Therefore, it is verified that the initial weir flow assumption was correct.

Step 6D - Evaluate the Performance of the Principal Spillway Under 100-Year Runoff Conditions

All stormwater impoundment facilities should be equipped with an armored emergency spillway. However, site conditions occasionally make the construction of such a spillway impractical. When this occurs, the 100-year runoff must be safely passed through the basin's principal spillway.

In an effort to provide an increased level of safety against embankment breaching, *the routed 100-year water surface elevation must be a minimum of 2' below the embankment's lowest point when no emergency spillway is provided.*

Evaluation of the 100-year inflow event is performed in the same manner as the 10-year event. The post-development 100-year runoff hydrograph is routed by the storage indication method using the stage – storage and stage – discharge relationships previously developed. See Step 7 for Q_{100} hydrograph development.

Step 6E - Verify Target Draw Down Time for Water Quality Volume

Many of the proprietary hydraulic modeling programs discussed on page 1-25 possess some version of a basin draw-down calculator. Generally, the input parameters will be the stage – discharge data curve representing only the water quality orifice and a specified beginning water surface elevation coinciding with the ponded water quality volume. In the example basin, the water quality volume is attained at a water surface elevation of 2102.07'. Employing the basin draw down calculator in VTPSUHM reveals a water quality draw down-time of 30.4 hours, as seen in Figure 2.16.

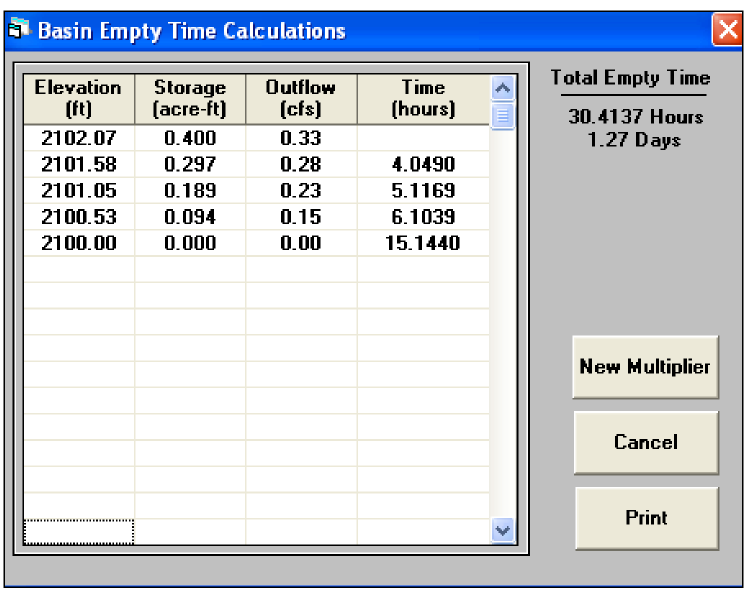


Figure 2.16 - Water Quality Draw Down Calculator

When no draw-down software aid is available, the engineer can verify the water quality draw-down time by storage indication routing. The water quality volume, beginning at pool elevation 2102.07', is assumed to be present in the basin at the onset of the routing operation. Then, a null hydrograph exhibiting all zeroes is routed through the basin. The results of this calculation are shown in Figure 2.17.

Modified Puls Output

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
25.50	0.00	0.00	0.0244	2100.14	0.030	0.030
26.00	0.00	0.00	0.0232	2100.13	0.027	0.027
26.50	0.00	0.00	0.0221	2100.12	0.025	0.025
27.00	0.00	0.00	0.0211	2100.12	0.023	0.023
27.50	0.00	0.00	0.0202	2100.11	0.021	0.021
28.00	0.00	0.00	0.0194	2100.11	0.019	0.019
28.50	0.00	0.00	0.0187	2100.11	0.017	0.017
29.00	0.00	0.00	0.0180	2100.10	0.016	0.016
29.50	0.00	0.00	0.0174	2100.10	0.015	0.015
30.00	0.00	0.00	0.0168	2100.09	0.014	0.014
30.50	0.00	0.00	0.0162	2100.09	0.014	0.014
31.00	0.00	0.00	0.0157	2100.09	0.013	0.013
31.50	0.00	0.00	0.0151	2100.09	0.013	0.013
32.00	0.00	0.00	0.0146	2100.08	0.012	0.012

Total Routing Mass Balance Discrepancy is 0.06%

Save Outflow Hydrograph | Print | Print Summary | Perform Another Routing | Done

Figure 2.17 - Verification of Water Quality Draw Down by Storage Indication Routing

At time event 30-hours, there is a very small amount of water in the basin. Since the inflow hydrograph has no flow, the volume of water shown in the "Storage Used" column of the routing table is part of the initial water quality volume. The elevation of the water in the WQ pool at time event 30-hours is only 0.09' above the basin floor elevation of 2100.0', a negligible amount.

Step 7 - Design of the Emergency Spillway

The design of an vegetated emergency spillway should conform to that outlined in Minimum Standard 3.03, Vegetated Emergency Spillways, found in the Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.).

The location of a vegetated emergency spillway must always be on native, undisturbed material, or "cut." Under no circumstances should a vegetated emergency spillway be constructed on embankment fill material. When site conditions prohibit the location of an emergency spillway on cut material, an armored or oversized spillway may be considered. Design of such a spillway is very site-specific, and when any spillway is considered, it must be designed by a qualified professional.

The spillway itself is comprised of three distinct elements – the entrance channel, the level section, and the exit channel. Flow exits the basin in a sub-critical flow regime through the spillway's entrance channel. The level section may serve as a control section with flows becoming super-critical upon entering the exit channel. As flow exits the basin through the emergency spillway, the upstream end of the entrance channel will function much like a broad-crested weir. At the entrance point, unless the spillway is constructed in rock, the maximum side slopes of the spillway are 3H:1V. Figure 2.18 illustrates the schematic layout of a vegetated emergency spillway.

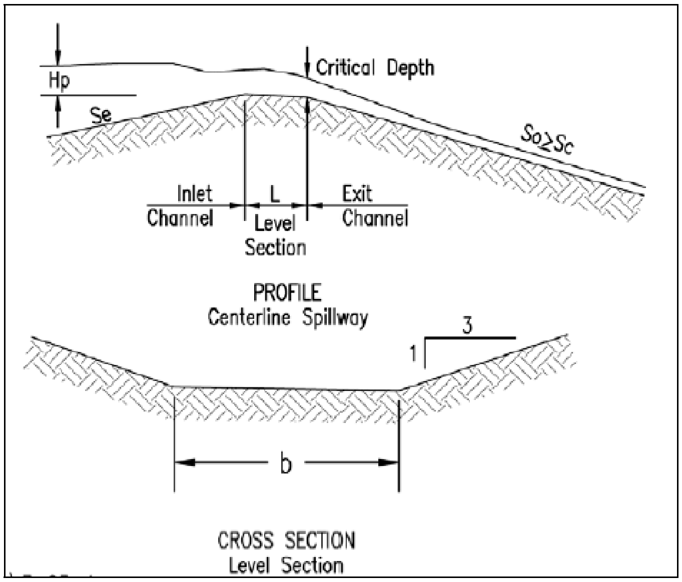


Figure 2.18 – DCR/DEQ Profile and Cross Section of Typical Vegetated Emergency Spillway
(Virginia Stormwater Management Handbook, 1999, Et seq.)

The first step in the design of a vegetated emergency spillway is to determine the peak inflow for the 100-year return frequency event. Applying the Rational Method and the regional NOAA NW-14 factors recommended in the VDOT Drainage Manual, we obtain the post-development 100-year peak rate of runoff shown in Table 2.7.

Table 2.7 - 100-Year Post-Development Runoff Parameters

Area	17.4 ac
C_w	0.5
t_c	10 min
B	27.24
D	5
E	0.55
Intensity	6.14 in/hr
Q (*CiA*)	53.4 cfs

Conservative design of a vegetated emergency spillway assumes that the principal spillway is damaged, clogged, or otherwise not operating during the 100-year storm event. Therefore, the peak design discharge for the emergency spillway is set equal to the peak inflow of the 100-year event, *53.4 cfs*.

The crest of the emergency spillway should be set at a small increment above the surface of the routed 10-year event. This will ensure that only those runoff events in excess of a 10-year return frequency will result in discharge through the emergency spillway. Minimizing the frequency of flows through the emergency spillway will reduce required maintenance and prolong the facility lifespan. Figure 2.15 shows the routed 10-year water surface to be 2103.98'. Therefore the crest of the emergency spillway will be set at 2104.1'. Table 2.4 shows the embankment top at elevation 2106.5'. Maintaining the required 1' of freeboard, we can compute the maximum allowable head acting on the emergency spillway as:

$$h = (2106.5 - 1.0) - 2104.1 = 1.4\,ft$$

Next, the required base width of the spillway is determined from Figure 2.19 on the following page. This figure, taken from the USDA – SCS *Design Data for Earth Spillways*, relates available head to spillway base width, exit channel slope, exit channel length, and exit channel velocity.

STAGE (Hp) IN FEET	SPILLWAY VARIABLES	BOTTOM WIDTH (b) IN FEET																
		8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0.5	Q	6	7	8	10	11	13	14	15	17	18	20	21	22	24	25	27	28
	V	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	S	3.9	3.9	3.9	3.9	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	X	32	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
0.6	Q	8	10	12	14	16	18	20	22	24	26	28	30	32	34	35	37	39
	V	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	S	3.7	3.7	3.7	3.7	3.6	3.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	X	36	36	36	36	36	36	37	37	37	37	37	37	37	37	37	37	37
0.7	Q	11	13	16	18	20	23	25	28	30	33	35	38	41	43	44	46	48
	V	3.2	3.2	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
	S	3.5	3.5	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
	X	39	40	40	40	41	41	41	41	41	41	41	41	41	41	41	41	41
0.8	Q	13	16	19	22	26	29	32	35	38	42	45	46	48	51	54	57	60
	V	3.5	3.5	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	S	3.3	3.3	3.3	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
	X	44	44	44	44	45	45	45	45	45	45	45	45	45	45	45	45	45
0.9	Q	17	20	24	28	32	35	39	43	47	51	53	57	60	64	68	71	75
	V	3.7	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	S	3.2	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
	X	47	47	48	48	48	48	48	48	48	48	49	49	49	49	49	49	49
1.0	Q	20	24	29	33	38	42	47	51	56	61	63	68	72	77	81	86	90
	V	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	S	3.1	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	X	51	51	51	51	52	52	52	52	52	52	52	52	52	52	52	52	52
1.1	Q	23	28	34	39	44	49	54	60	65	70	74	79	84	89	95	100	105
	V	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	S	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	55	55	55	55	55	55	55	56	56	56	56	56	56	56	56	56	56
1.2	Q	28	33	40	45	51	58	64	69	76	80	86	92	98	104	110	116	122
	V	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
	S	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	58	58	59	59	59	59	59	59	60	60	60	60	60	60	60	60	60
1.3	Q	32	38	46	53	58	65	73	80	86	91	99	106	112	119	125	133	140
	V	4.5	4.6	4.6	4.6	4.6	4.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
	S	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	X	62	62	62	63	63	63	63	63	63	63	63	64	64	64	64	64	64
1.4	Q	37	44	51	59	66	74	82	90	96	103	111	119	127	134	142	150	158
	V	4.7	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9
	S	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
	X	65	66	66	66	66	67	67	67	67	67	67	68	68	68	68	68	69
1.5	Q	41	50	58	66	75	85	92	101	108	116	125	133	142	150	160	169	178
	V	4.8	4.9	4.9	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.1	5.1	5.1
	S	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.5	2.5	2.5
	X	69	69	70	70	71	71	71	71	71	71	71	72	72	72	72	72	72
1.6	Q	46	56	65	75	84	94	104	112	122	132	142	149	158	168	178	187	197
	V	5.0	5.1	5.1	5.1	5.1	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2
	S	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	72	74	74	75	75	76	76	76	76	76	76	76	76	76	76	76	76
1.7	Q	52	62	72	83	94	105	115	126	135	145	156	167	175	187	196	206	217
	V	5.2	5.2	5.2	5.3	5.3	5.3	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4
	S	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	76	78	79	80	80	80	80	80	80	80	80	80	80	80	80	80	80
1.8	Q	58	69	81	93	104	116	127	138	150	160	171	182	194	204	214	226	233
	V	5.3	5.4	5.4	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.6	5.6	5.6	5.6	5.6	5.6
	S	2.5	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	80	82	83	84	84	84	84	84	84	84	84	84	84	84	84	84	84
1.9	Q	64	76	88	102	114	127	140	152	164	175	188	201	213	225	235	248	260
	V	5.5	5.5	5.5	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7
	S	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	84	85	86	87	88	88	88	88	88	88	88	88	88	88	88	88	88
2.0	Q	71	83	97	111	125	138	153	164	178	193	204	218	232	245	256	269	283
	V	5.6	5.7	5.7	5.7	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.9	5.9
	S	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	88	90	91	91	91	91	92	92	92	92	92	92	92	92	92	92	92
2.1	Q	77	91	107	122	135	149	162	177	192	207	220	234	250	267	276	291	305
	V	5.7	5.8	5.9	5.9	5.9	5.9	5.9	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
	S	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	92	93	95	95	95	95	95	95	95	96	96	96	96	96	96	96	96
2.2	Q	84	100	116	131	146	163	177	194	210	224	238	253	269	288	301	314	330
	V	5.9	5.9	6.0	6.0	6.0	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.2	6.2	6.2	6.2
	S	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	96	98	99	99	99	99	99	100	100	100	100	100	100	100	100	100	100
2.3	Q	90	108	124	140	158	175	193	208	226	243	258	275	292	306	323	341	354
	V	6.0	6.1	6.1	6.1	6.2	6.2	6.2	6.2	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3
	S	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	100	102	102	103	103	103	104	104	104	105	105	105	105	105	105	105	105
2.4	Q	99	116	136	152	170	189	206	224	241	260	275	294	312	327	346	364	378
	V	6.1	6.2	6.2	6.3	6.3	6.3	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4
	S	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	105	105	106	107	107	108	108	108	108	109	109	109	109	109	109	109	109

Figure 2.19 – DCR/DEQ Design Data for Earth Spillways

(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

Interpolating Figure 2.19 with an available head (stage) value of 1.4' and a design discharge of 53.4 cfs, we obtain the following spillway parameters:

Table 2.8 - Armored Emergency Spillway Parameters (1.4' of Head Acting on Crest)

Minimum Base Width	13'
Minimum Exit Channel Slope	.027 ft/ft
Minimum Length of Level Section	66'
Exit Channel Velocity	4.8 ft/sec

Figure 2.19 (of the USDA / SCS document) can be employed to determine the required head to convey the design storm discharge if site constraints restrict the available base width of the spillway, thus making it the known variable.

The computed base width of the channel should not exceed 35 times the depth of flow acting upon the spillway. Compliance with this ratio is shown as follows:

$$\frac{13\,ft}{1.4\,ft} = 9.3 < 35$$

Additionally, the cross-sectional area of the exit channel must be equal to or greater than the cross-sectional area of the control section.

The values obtained from the USDA / SCS Design Data for Earth Spillways table are minimum values only. It should be noted that exit channel slopes less than those found in the table will restrict the conveyance, Q, through the spillway. Also of note is that the exit channel velocities presented in the table correspond directly to the *minimum* exit channel slope from the table. If the slope of the exit channel is increased above the minimum value, the flow velocity will also increase. However, *increasing this minimum exit channel slope, for a given head or stage, will not increase conveyance through the spillway itself.*

Assuming that the minimum exit channel slope is used, the flow velocity in the exit channel is now known. The final step is to ensure that this exit channel velocity is below the velocity deemed erosive for the type of vegetation present. Table 2.9 presents permissible exit channel velocities as a function of vegetation type, soil erosion potential, and exit channel slope.

Table 2.9 – DCR/DEQ Exit Channel Permissible Velocities
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

Vegetative Cover	Permissible Velocity [2] (*ft/s*)			
	Erosion Resistant Soils [3]		Easily Erodible Soils [4]	
	Slope of Exit Channel		Slope of Exit Channel	
	0-5%	*5-10%*	*0-5%*	*5-10%*
Bermuda Grass Bahiagrass	8	7	6	5
Buffalograss Kentucky Bluegrass Smooth Bromegrass Tall Fescue Reed Canary Grass	7	6	5	4
Sod Forming Grass-Legume Mixtures	5	4	4	3
Lespedeza Weeping Lovegrass Yellow Bluestem Native Grass Mixtures	3.5	3.5	2.5	2.5

[1] SCS-TP-61
[2] Increase values 25 percent when the anticipated average use of the spillway is not more frequent than once in 10 years.
[3] Those with a high clay content and high plasticity. Typical soil textures are silty clay, sandy clay, and clay.
[4] Those with a high content of fine sand or silty and lower plasticity or non-plastic. Typical soil textures are fine sand, silt, sandy loam, and silty loam.

If the exit channel velocity exceeds the permissible value for the type of vegetation present, the base width of the spillway may be increased. This increase in base width will result in less head acting on the spillway, in turn reducing the observed velocity in the exit channel.

The example basin embankment, principal spillway, emergency spillway, and various water surface elevations are shown schematically in Figure 2.20.

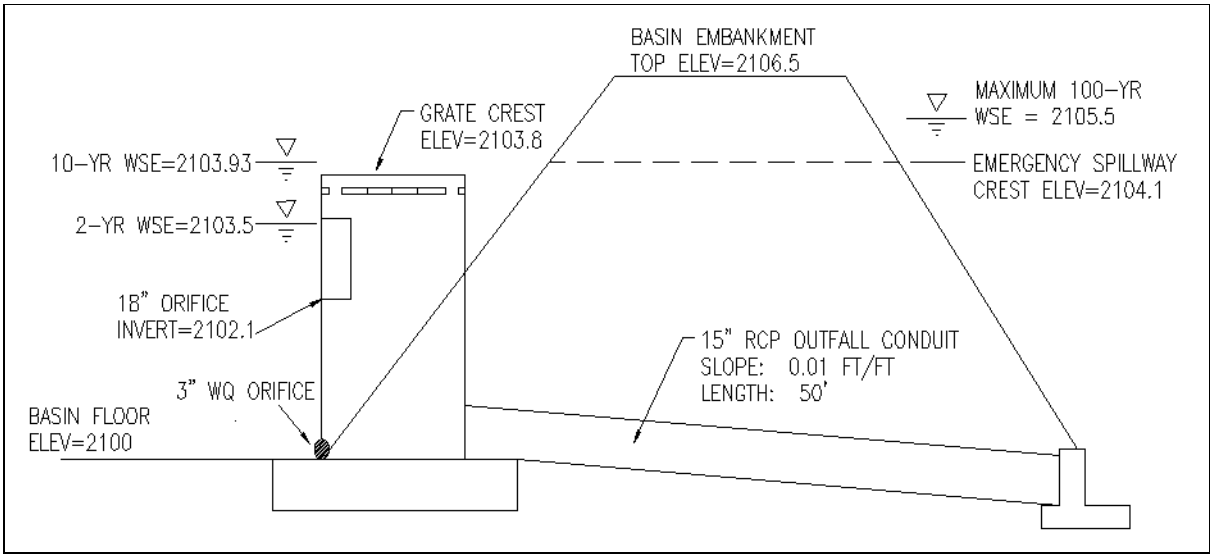


Figure 2.20 - Schematic Illustration of Principal and Emergency Spillway Configuration and Resulting Water Surface Elevations

Step 8 - Provision for Seepage Control

A primary cause of failure in earthen embankments arises from piping/seepage along the principal spillway's outfall conduit. Traditionally, an attempt to reduce the severity of piping has been made through the use of anti-seep collars. These collars attempt to lengthen the percolation path along the conduit, thus reducing the available hydraulic gradient. This, in effect, discourages piping along the conduit. In 1987, the U.S. Army Corps of Engineers released Technical Memorandum No. 9 stating:

"When a conduit is selected for a waterway through an earth or rockfill embankment, cutoff collars will not be selected as the seepage control measure."

As an alternative to anti-seep collars, a variety of anti-seepage controls have been developed for *major* impoundments. By their nature, linear highway projects typically do not require large impoundment facilities for control of runoff. Therefore, per Instructional and Informational Memorandum of General Subject *"Management of Stormwater,"* dated February 12, 2003, concrete cradles are recommended for seepage control on VDOT stormwater management basins. These cradles are to extend the *entire length of all outfall conduits penetrating earthen embankments*.

A cross-section of the size and type of concrete cradle to be used on VDOT stormwater impoundment facilities is presented in Figure 2.21.

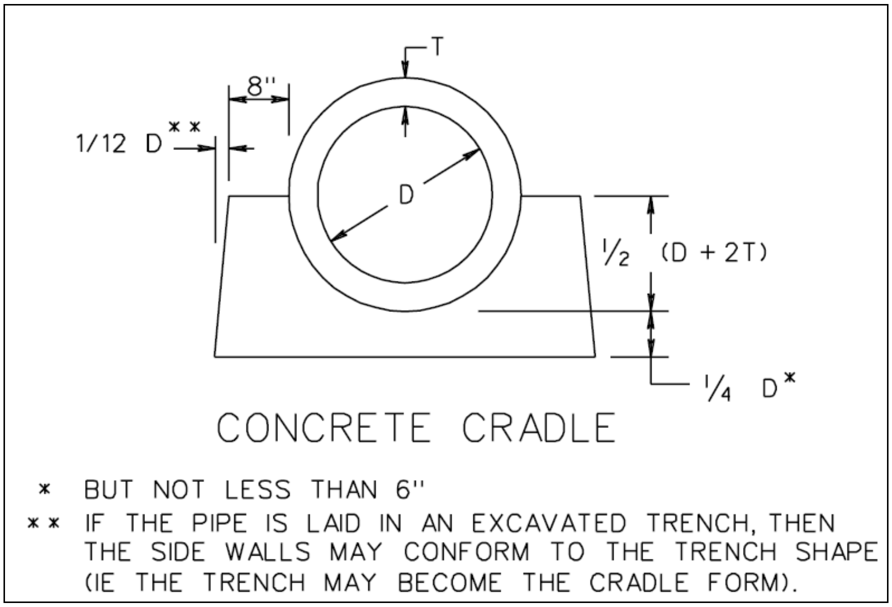


Figure 2.21 - Typical Concrete Cradle for Minimization of Piping Along Outfall Conduits
(*VDOT Drainage Manual*)

Step 9 - Embankment Design

Proper design and construction of the earthen impounding structure is of critical importance to the long-term performance of a stormwater detention basin.

Early in the design stages of a project for which a detention basin is proposed, foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam.

"The foundation material under the dam and the material used for the embankment of the dam should be an AASHTO Type A-4 or finer and/or meet the approval of the Materials Division. If the native material is not adequate, the foundation of the dam is to be excavated and backfilled a minimum of 4' or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."

If the basin embankment height exceeds 15', or if the basin includes a permanent pool, the design of the dam should employ a homogenous embankment with seepage controls or zoned embankments, or similar design in accordance with the Virginia SWM Handbook and recommendations of the VDOT Materials Division.

During the initial subsurface investigation, additional borings should be made near the center of the proposed basin when:

- Excavation from the basin will be used to construct the embankment
- There is a potential of encountering rock during excavation
- A high or seasonally high water table, generally 2' or less, is suspected

On larger projects, multiple borings for the dam and/or basin may be deemed necessary. The number and location of these borings should be determined by the Hydraulics and/or Materials Engineer.

If the basin embankment height exceeds 15', or if the basin includes a permanent pool, the design of the dam should employ a homogenous embankment with seepage controls or zoned embankments. Embankment height is largely dictated by freeboard requirements. The required freeboard depths under 100-year conditions are as follows:

- When equipped with an emergency spillway, the basin must provide a minimum of 1' of freeboard from the maximum water surface elevation arising from the 100-year event and the lowest point in the embankment (excluding the emergency spillway itself).

- When no emergency spillway is provided, a minimum of 2' of freeboard should be provided between the maximum water surface elevation induced by the 100-year runoff event and the lowest point in the embankment.

This example embankment does not exceed 15' in height, nor does the basin hold a permanent pool. Reference *Design Example 3 – Retention Basin* for a zoned embankment design example.

The top width of the embankment should be a minimum of 10' in width to provide ease of construction and maintenance. Additionally, the top of the embankment should be graded to promote positive drainage and prevent the ponding of water on the embankment top.

To permit mowing and other maintenance, the embankment slopes should be no steeper than 3H:1V.

All earthen impounding structures should be equipped with a foundation cutoff trench. Figure 2.22 illustrates the general configuration of such a trench.

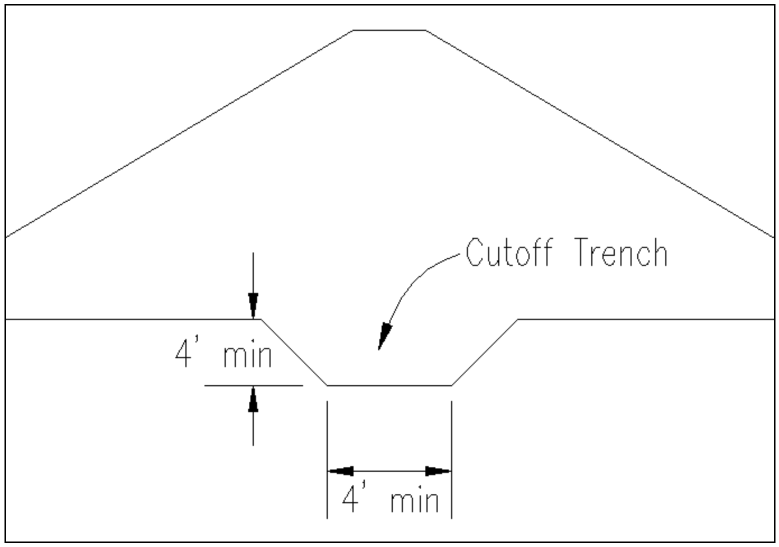


Figure 2.22 - Typical Cutoff Trench Configuration

The trench bottom width and depth should be no less than 4', and the trench slopes should be no steeper than 1H:1V. The cutoff trench should be situated along the centerline of the embankment, or slightly upstream of the centerline. Along the width of the embankment, the trench should extend up the embankment abutments to a point coinciding with the 10-year water surface elevation.

The cutoff trench material should be that of the embankment, provided the Materials Division has approved such material. When the embankment is "zoned," the cutoff trench material shall be that of the embankment core.

Step 10 - Buoyancy Calculation

A buoyancy calculation should be performed on every proposed riser structure. A minimum factor of safety of 1.25 should be provided between the weight of the structure and the uplifting buoyant force when the riser is submerged and the ground is saturated. When the summation of downward forces, including the riser's weight, are less than this buoyant force, *flotation will occur*.

The first step is to compute the buoyant force acting on the riser. The buoyant force is a function of the volume of water displaced by the riser. The calculation presented here also assumes that the basin ground is saturated, thus including the buoyant force of the volume of water displaced below grade by the riser footing. A VDOT SWM-1 is used in this design example. The side view of a SWM-1 riser is shown below in Figure 2.23:

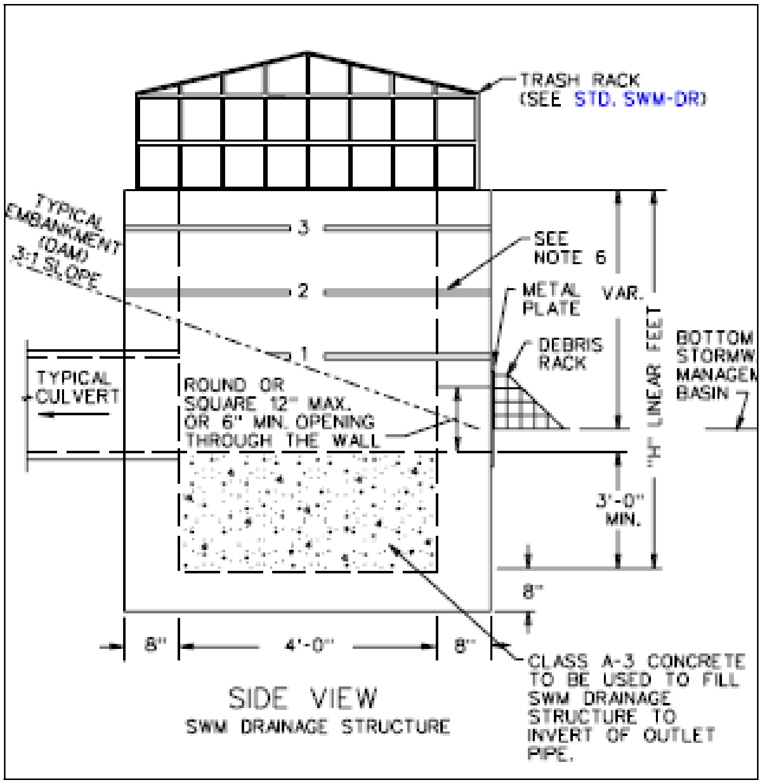


Figure 2.23 - VDOT SWM-1 Side View
VDOT Road and Bridge Standards

The outside dimensions of the SWM-1 are 5'-4" x 5'-4". The above-ground height, H, of the riser designed in Step 6 of this example is the difference between the grate top's crest elevation and the bottom of the basin floor. The total riser height calculation is as follows:

$$H_{Displaced} = 2103.9 - 2100 + 3\,ft + \frac{8in}{12\frac{in}{ft}} = 7.6\,ft$$

Therefore, the volume of water displaced is computed as:

$$\left(5\,ft + \frac{4in}{12\frac{in}{ft}}\right)^2 \times 7.6\,ft = 216.2\,ft^3$$

The unit weight of water is 62.4 lb/ft^3, with the buoyant force computed as:

$$F_{Buoyant} = 216.2\,ft^3 \times 62.4\frac{lb}{ft^3} = 13{,}491lb$$

Applying the 1.25 factor of safety:

$$1.25 \times 13{,}491lb = 16{,}864lb$$

The sum of all downward forces acting upon the riser must be greater than 16,864 lb.

First, consider the weight of the riser walls. The SWM-1 has reinforced concrete walls that are 8" thick. The "plan-view" area of the walls is computed as:

$$A_{Wall} = \left(5\,ft + \frac{4in}{12\frac{in}{ft}}\right)^2 - (4\,ft)^2 = 12.4\,ft^2$$

The height of the riser walls was computed previously as 7.6'. The volume of concrete represented in the walls of the riser is computed as:

$$V_{Walls} = 12.4\,ft^2 \times 7.6\,ft = 94.2\,ft^3$$

The unit weight of reinforced concrete is 150 lb/ft^3, with the weight of the riser walls computed as:

$$F_{Walls} = 94.2\,ft^3 \times 150\frac{lb}{ft^3} = 14{,}130lb$$

We must subtract the weight of concrete lost to the 18" diameter 2-year control outlet:

$$F_{Orifice} = \left[\left(\frac{1.5\,ft}{2}\right)^2 \times \pi \times \frac{8in}{12\frac{in}{ft}}\right] \times 150\frac{lb}{ft^3} = 177lb$$

The weight of the riser bottom (which excludes the wall sections already considered) is computed as follows:

$$F_{Bottom} = (4\,ft)^2 \times \frac{8in}{12\frac{in}{ft}} \times 150\frac{lb}{ft^3} = 1{,}600lb$$

The weight of the metal "bird cage" trash rack, per Figure 2.13 is 120 lbs.

The unit weight of riprap is 165 lb/ft^3, with the weight of riprap computed as:

$$F_{Riprap} = 3\,ft \times 4\,ft \times 165\frac{lb}{ft^3} = 1{,}980lb$$

The downward force of the riser weight is computed as:

$$F_{Walls} - F_{Orifice} + F_{Bottom} + F_{Top} + F_{Riprap} =$$
$$14{,}130lb - 177lb + 1{,}600lb + 120lb + 1{,}980lb = 17{,}653lb > 1.25F_{Buoyant}\,(16{,}864\,lb)$$

Step 11 - Design of Sediment Forebays

A sediment forebay must be provided at any point in the basin that receives concentrated discharge from a pipe, open channel, or other means of stormwater conveyance. The inclusion of a sediment forebay in these locations assists maintenance efforts by isolating the bulk of sediment deposition in well-defined, easily accessible locations.

In addition to serving a vital maintenance function, sediment forebays are an integral component of the BMPs water quality improvement performance. The phosphorus removal percentages expressed in the BMP Selection Table for VDOT Projects consider that a sediment forebay is provided at all basin inflow points.

The volume of storage provided at each forebay should range between 0.1 and 0.25" of runoff over the outfall's new contributing impervious area, with the sum of all forebay volumes not less than 10% of the total extended detention volume.

The storage volume in the sediment forebay is provided by separating the forebay from the rest of the basin. This separation is accomplished by means of an earthen berm, gabion baskets, concrete, or riprap. In a dry facility, the forebay outlet crest should be set at the elevation corresponding to the basin's water quality extended detention pool. Depending on the type of material employed to construct the forebay embankment, the flows captured in the forebay may be detained over very long periods, with losses occurring only by means of infiltration and evaporation. Because the volume may be inundated at the onset of a runoff producing event, in a dry extended detention basin the forebay volume *should not be considered part of the extended detention water quality volume*.

The forebay outlet crest should be stabilized and capable of conveying the 10-year inflow event into the basin in a non-erosive manner.

The example project site is comprised of a post-development runoff area of 17.4 acres, with 4.75 acres of new impervious cover. For the example forebay design, we consider two entrance points into the basin, each exhibiting the following characteristics:

Table 2.10 - Summary of Pond Inflow Points

Entrance Point 1		
Acreage	**New Impervious Acreage**	**Peak 10-Year Inflow (cfs)**
6.96	2.25	16
Entrance Point 2		
Acreage	**New Impervious Acreage**	**Peak 10-Year Inflow (cfs)**
10.44	2.5	21

First, the forebays will be sized to provide storage of 0.1” of runoff from the new impervious area contributing runoff to each entrance point:

$$V_1 = 2.25ac \times \frac{43{,}560\,ft^2}{ac} \times \frac{0.1in}{12\frac{in}{ft}} = 817\,ft^3$$

$$V_2 = 2.5ac \times \frac{43{,}560\,ft^2}{ac} \times \frac{0.1in}{12\frac{in}{ft}} = 908\,ft^3$$

The sum of the forebay storage volumes:

$$817\,ft^3 + 908\,ft^3 = 1{,}725\,ft^3$$

The project site water quality volume is 0.20 acre-ft. The sum of all forebay volumes must be at least 10% of this volume, computed as follows:

$$0.10 \times 0.20ac - ft \times \frac{43{,}560\,ft^2}{ac} = 862\,ft^3 < V_{Forebay} = 1{,}725\,ft^3$$

The calculation confirms that adequate sediment forebay volumes are provided. A permanent gage shall be provided to indicate the level of sediment accumulation and to provide visible indication of when maintenance is required.

To combat against particle resuspension in the forebay, The Center for Watershed Protection (1995) recommends depths ranging between 4’ and 6’. However, these depths may be considered excessive on smaller basins, particularly when the forebay depth would exceed the ponded depth of the 10-year or greater storm. Furthermore, as with the basin itself, extended ponding (> 72 hours) of depths exceeding 3' gives rise to undesirable nuisance and liability issues. When practical, greater forebay depths should be used. When shallower depths (<4') are used, it is critical that the forebay's accumulated sediment is removed at regular intervals. The use of properly sized outlet protection at the point of concentrated discharge will assist in dissipating the energy of incoming flows, thus reducing the severity of pollutant resuspension.

The geometric layout of the forebay is dictated by site constraints and the designer's preference. The required forebay volume for entrance point 1 was found to be 817 ft^3. Figures 2.24 and 2.25 illustrate the respective plan and cross-sectional view of a forebay providing this volume.

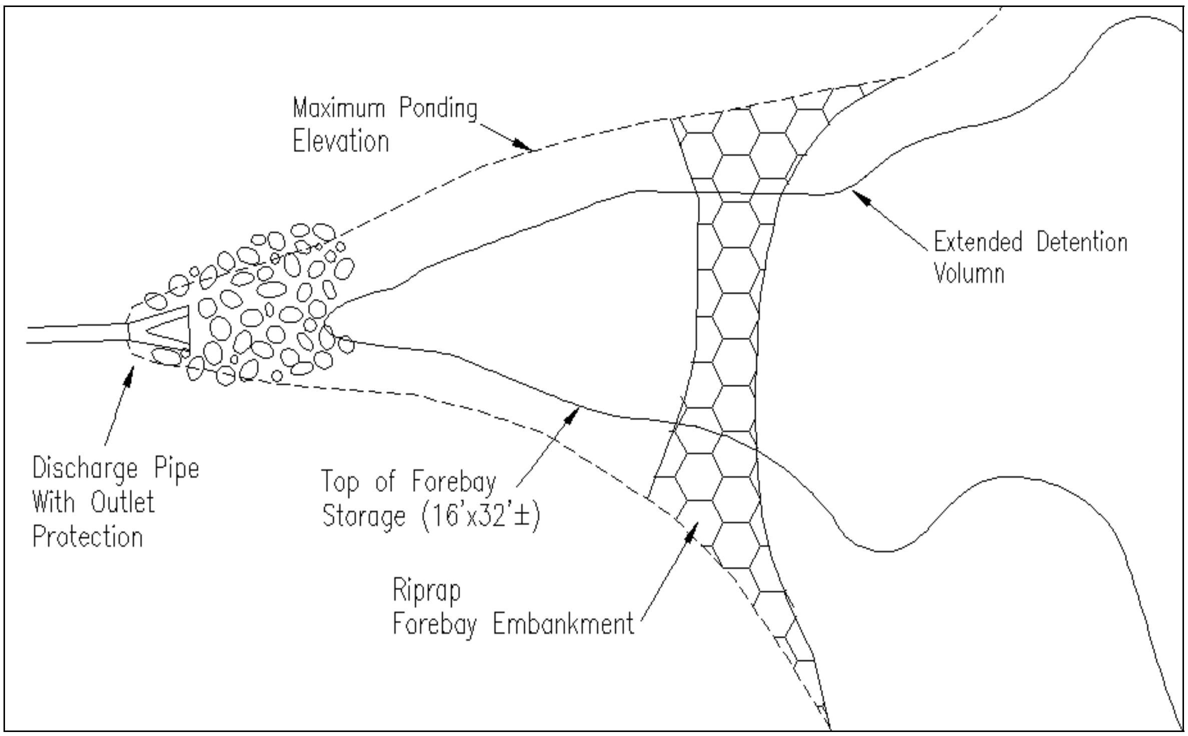


Figure 2.24 - Plan View Sediment Forebay 1
(No Scale)

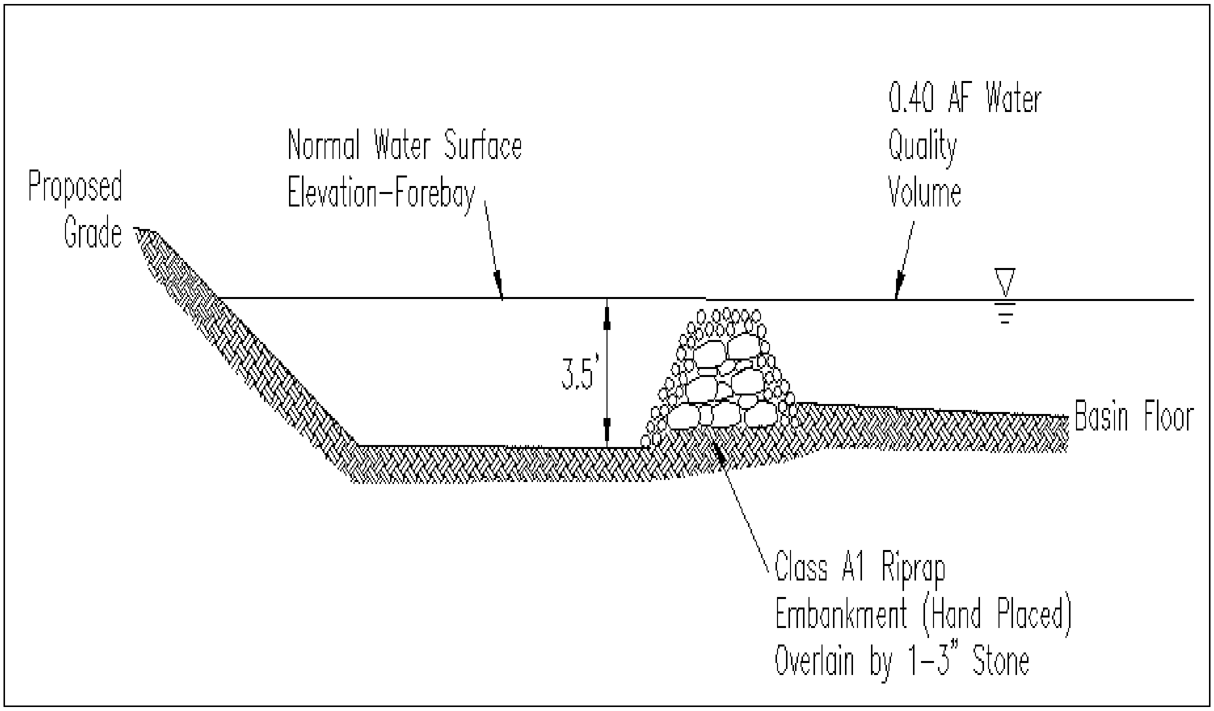


Figure 2.25 - Cross-Section View Sediment Forebay 1
(No Scale)

Step 12 - Landscaping

Stormwater management basins should be permanently seeded within 7 days of attaining final grade. This seeding should comply with all applicable VDOT standards for erosion and sediment control.

The permanent vegetative stabilization of an extended dry detention basin entails meeting planting requirements for four distinct zones. These zones are discussed as follows.

The *shoreline fringe* encompasses all basin area located below the high water mark of the extended detention water quality volume. This zone is subject to frequent inundation, but also lengthy dry periods during the summer months. Species suitable for planting in this zone, as identified in Chapter 3-05 of the Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.) include soft-stem bulrush, pickerelweed, rice cutgrass, sedges, shrubs such as chokeberry, and trees such as black willow and river birch.

The *Riparian Fringe Zone* is an area of the basin that only becomes inundated during runoff producing events, and only then for relatively brief periods. This zone encompasses the basin area above the extended detention volume. A wide array of planting species are acceptable in this zone, and should be chosen based on ability to prevent erosion and pollutant resuspension.

The *Floodplain Terrace* is the basin area that is only inundated during severe runoff producing events such as the 100-year storm. Native floodplain species generally grow well in this zone. The species selected for this zone should exhibit the ability to provide erosion resistance, grow in compacted soil, and require minimal maintenance.

Upland Areas are comprised of the vegetated areas adjacent to stormwater impoundments. Their chosen planting species should be based on prevailing native soil and hydrologic conditions.

The choice of planting species should be largely based on the project site's physiographic zone classification. Additionally, the selection of plant species should match the native plant species as closely as possible. Surveying a project site's native vegetation will reveal which plants have adapted to the prevailing hydrology, climate, soil, and other geographically-determined factors. Figure 3.05-4 of the Virginia Stormwater Management Handbook provides guidance in plant selection based on project location.

All chosen plant species should conform to the American Standard for Nursery Stock, current issue, and be suited for USDA Plant Hardiness Zones 6 or 7, see Figure 2.26 below.

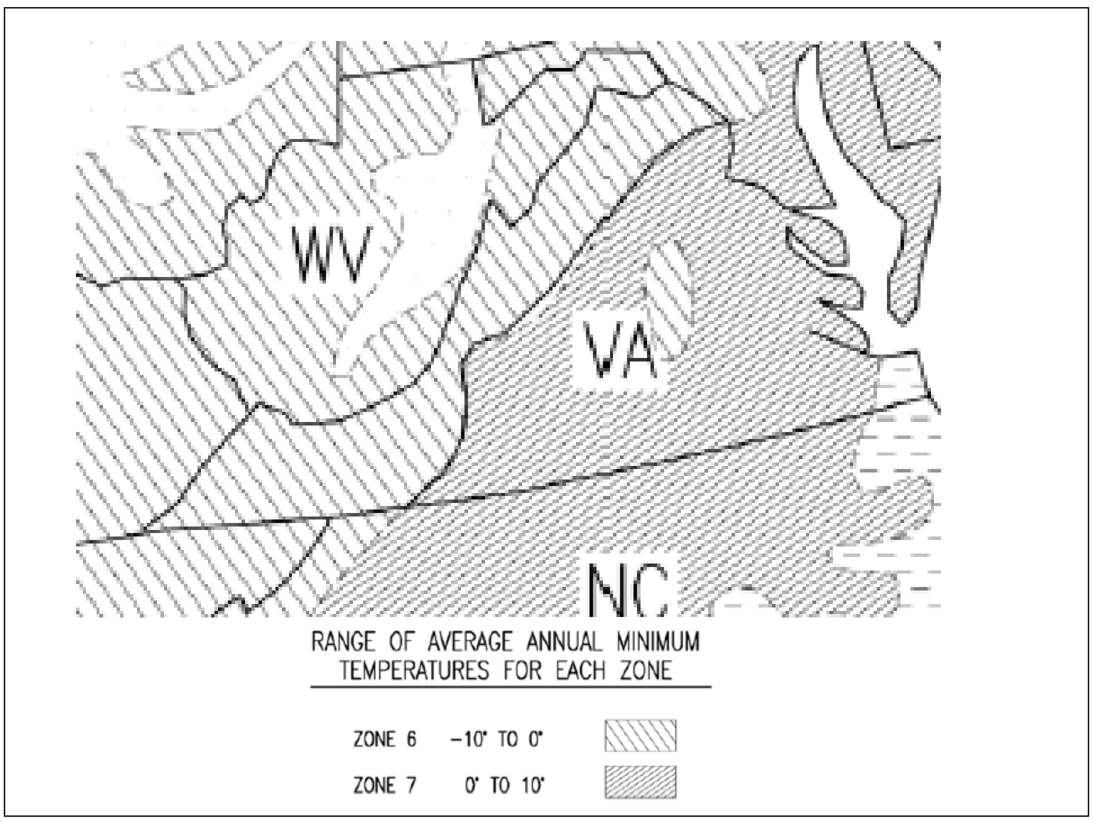


Figure 2.26 - USDA Plant Hardiness Zones

Under no circumstances should trees or shrubs be planted on a basin's embankment. The large root structure may compromise the structural integrity of the embankment.

3.1 Enhanced Dry Extended Detention Basin - Overview of Practice

An “enhanced” dry extended detention basin is a variation of a conventional dry extended detention basin. The methods and calculations demonstrated in this example should be used in conjunction with *Section 2 – Dry Extended Detention Basin.* Like dry detention basins, an enhanced basin is capable of temporarily detaining runoff and releasing that runoff at a controlled rate over a specified period of time. However, unlike dry facilities, enhanced facilities are equipped with an engineered permanent marsh area. This marsh area functions to improve the pollutant removal performance of the facility beyond that which is possible in a traditional dry detention basin. Enhanced extended dry detention basins are capable of providing water quality improvement, downstream flood control, channel erosion control, and mitigation of post-development runoff to pre-development levels. Enhanced extended detention facilities improve runoff quality through the gravitational settling of pollutants as well as through wetland uptake, absorption, and decomposition. Also aiding in pollutant removal performance, the marsh area of the basin helps to prevent the resuspension of captured pollutants.

Figure 3.1 presents the schematic layout of a dry extended detention basin – enhanced presented in the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.).

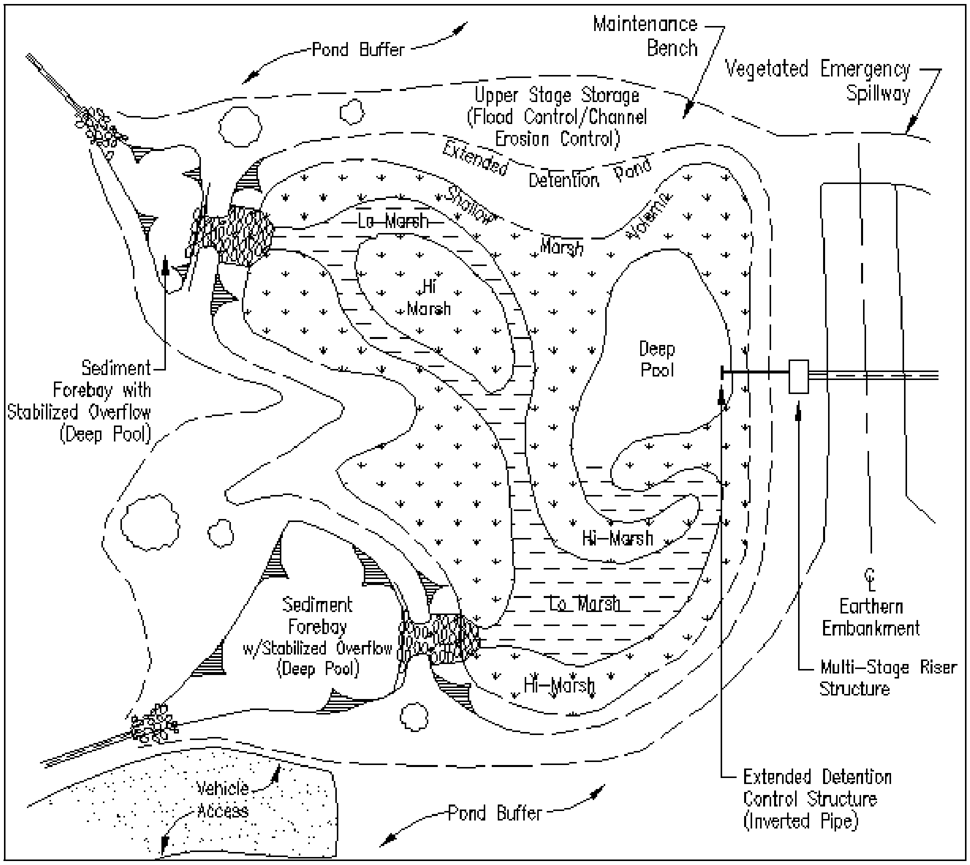


Figure 0.1 – DCR/DEQ Schematic Dry Extended Detention Basin – Enhanced Plan View
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

As evidenced in Figure 3.1, the marsh area is comprised of three distinct zones – "low marsh," "high marsh," and "deep pool." These varying-depth zones introduce *microtopography* to the basin floor. Detailed surface area and depth requirements of the various marsh zones are discussed later in this section.

3.2 Site Constraints and Siting of the Facility

In addition to the contributing drainage area's new impervious cover, a number of site constraints must be considered when the implementation of an enhanced dry extended detention basin is proposed. The marsh area requirements of an enhanced basin are similar to those of a constructed stormwater wetland (*Section Five)*, and introduce planning considerations beyond those that must be considered for conventional dry detention facility.

3.2.1 Minimum Drainage Area

The minimum drainage area contributing to an enhanced dry extended detention facility is not restricted. However, careful attention must be given to the water quality volume generated from this area. When this water quality volume is particularly low, the computed orifice size required to achieve the desired drawdown time may be small (less than 3" in diameter). These small openings are vulnerable to clogging by debris. Generally, the minimum area contributing runoff to a dry extended detention pond should be selected such that the desired water quality drawdown time is achieved with an orifice of at least 3" in diameter. In instances when the use of a smaller orifice is unavoidable, provisions must be made to prevent clogging. Figure 3.07-3 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.) illustrates recommended outlet configurations for the control of sediment, trash, and debris. For convenience, these details are provided as Figures 3.2, 3.3, and 3.4. If the required water quality orifice size is significantly less than 3", the designer may wish to examine alternative water quality BMPs, such as practices which treat the first flush volume and bypass large runoff producing events.

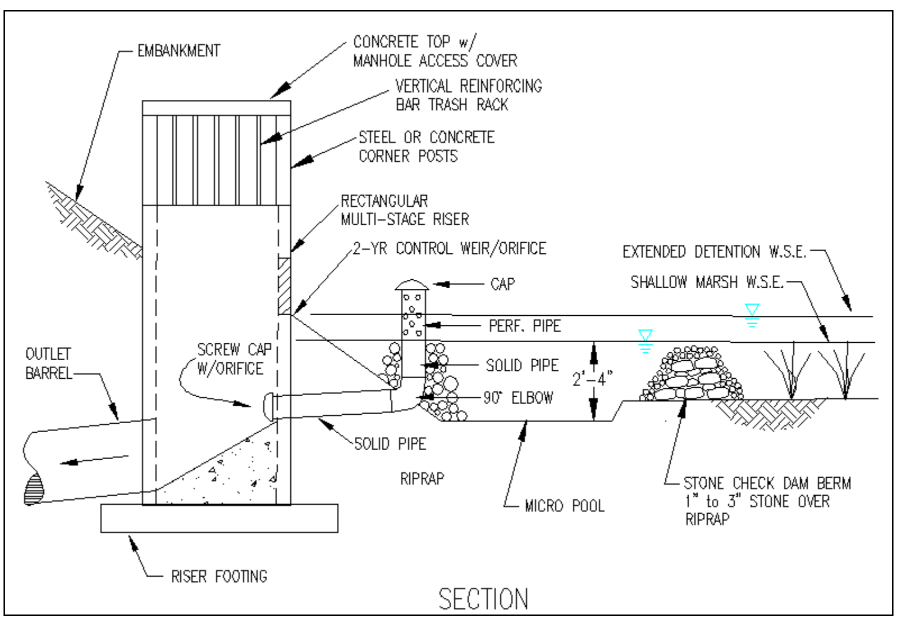


Figure 3.2.1 – DCR/DEQ Recommended Outlet Configuration 1 for the Control of Trash, Sediment and Debris
(*Virginia Stormwater Management Handbook,* 1999, Et seq.*)*

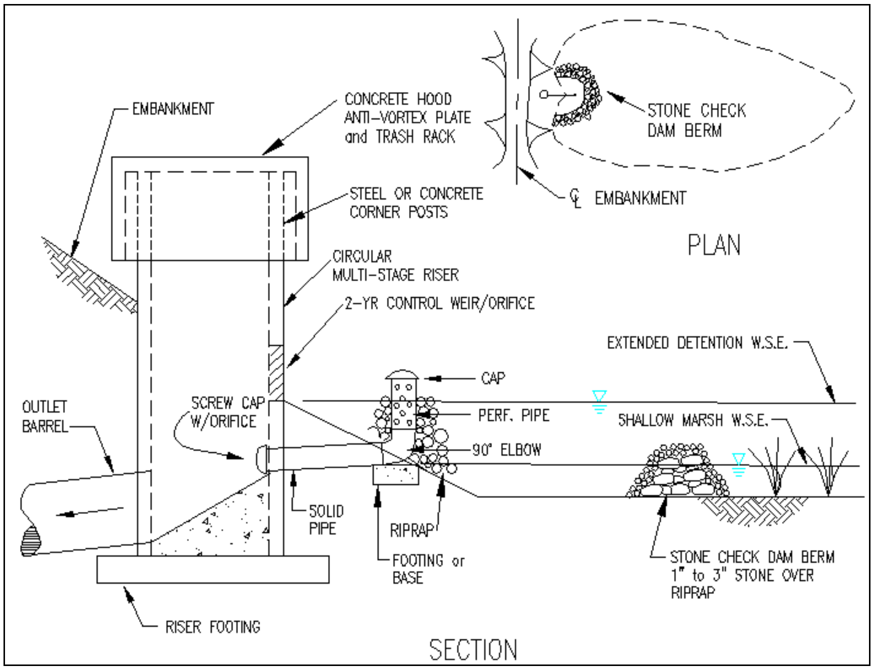


Figure 3.2.2 – DCR/DEQ Recommended Outlet Configuration 2 for the Control of Trash, Sediment and Debris
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

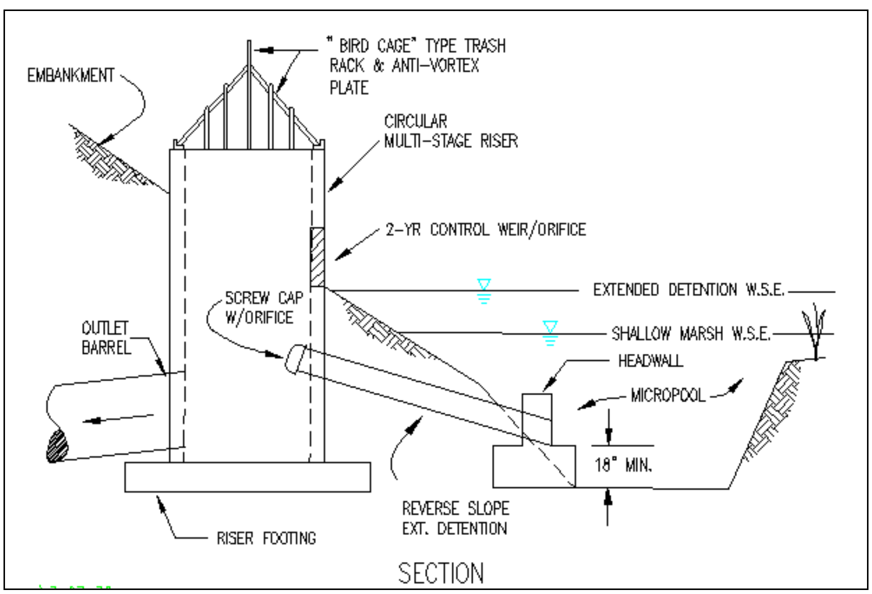


Figure 3.2.3. DCR/DEQ Recommended Outlet Configuration 3 for the Control of Trash, Sediment and Debris
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

3.2.2 Maximum Drainage Area

The maximum drainage area to an enhanced extended dry detention facility is frequently restricted to no more than 50 acres. When larger drainage areas are directed to a single facility, often there is a need to accommodate base flow through the facility. The most notable difficulty in accommodating base flow in the facility lies in sizing the low-flow/water quality control orifice. Undersizing of the orifice will lead to the "choking" of base flow conveyance such that a permanent pool volume accumulates and encroaches upon the volume of dry storage dedicated to extended detention. The loss of this volume will result in excessively low hydraulic residence times for the water quality volume generated from significant rainfall events. Contrasting this problem is the situation occurring when the orifice allocated to pass-through of the base flow is sized too large to provide the desired minimum draw down time for the site's water quality volume.

3.2.3 Separation Distances

Extended dry detention facilities should be kept a minimum of 20' from any permanent structure or property line, and a minimum of 100' from any septic tank or drainfield.

3.2.4 Site Slopes

Generally, extended detention basins should not be constructed within 50' of any slope steeper than 15%. When this is unavoidable, a geotechnical report is required to address the potential impact of the facility in the vicinity of such a slope.

3.2.5 Site Soils

The implementation of an enhanced extended detention basin can be successfully accomplished in the presence of a variety of soil types. However, when such a facility is proposed, *a subsurface analysis and permeability test is required*. This data must be provided to the Materials Division early in the project planning stages to determine if an enhanced basin is feasible on native site soils. Soils exhibiting excessively high infiltration rates are not suited for the construction of extended detention facilities, as they will behave as an infiltration facility until clogging occurs. Furthermore, enhanced facilities must be constructed on soils capable of supporting the shallow marsh *at the time of stabilization and seeding*. The designer should also keep in mind that as the ponded depth within the basin increases, so does the hydraulic head. This increase in hydraulic head results in increased pressure, which leads to a potential increase in the observed rate of infiltration. To combat excessively high infiltration rates, a clay liner, geosynthetic membrane, or other material (as approved by the Materials Division) may be employed. The basin's embankment material must meet the specifications detailed later in this section and/or be approved by the Materials Division.

3.2.6 Rock

The presence of rock within the proposed construction envelope of an enhanced extended detention basin should be examined during the aforementioned subsurface investigation. When blasting of rock is necessary to obtain the desired basin volume, a liner (of material approved by the Materials Division) should be used to eliminate unwanted losses through seams in the underlying rock.

3.2.7 Existing Utilities

Basins should not be constructed over existing utility rights-of-way or easements. When this situation is unavoidable, permission to impound water over these easements must be obtained from the utility owner *prior* to design of the basin. When it is proposed to relocate existing utility lines, the costs associated with their relocation should be included in the overall basin construction cost.

3.2.8 Karst

The presence of Karst topography places even greater importance on the subsurface investigation. Implementation of extended detention facilities in Karst regions may greatly impact the design and cost of the facility, and must be evaluated early in the planning phases of a project. *Construction of stormwater management facilities within a sinkhole is prohibited.* When the construction of such facilities is planned along the periphery of a sinkhole, the facility design must comply with the guidelines found in Chapter 5 of this Manual and DCR/DEQ's Technical Bulletin #2 *"Hydrologic Modeling and Design in Karst."*

3.2.9 Existing Wetlands

When the construction of an enhanced dry extended detention facility is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify the wetlands' boundaries, their protected status, and the feasibility of BMP implementation in their vicinity. In Virginia, the Department of Environmental Quality (DEQ) and the U.S. Army Corps of Engineers (USACOE) should be contacted when such a facility is proposed in the vicinity of known wetlands.

3.2.10 Upstream Sediment Considerations

Close examination should be given to the flow velocity at all basin inflow points. When entering flows exhibit erosive velocities, they have the potential to greatly increase the basin maintenance requirements by depositing large amounts of sediment. Additionally, when a basin contributing drainage area is highly pervious, it may hinder basin performance by the deposition of excessive sediment. Enhanced basins are even more vulnerable to sediment loading than their dry counterparts, as excessive sediment loading has the potential to greatly alter the microtopography of the basin floor. The negative impacts associated with excessive sediment loading reinforce the need for sediment forebays as discussed in Section 3.3.

3.2.11 Floodplains

The construction of extended detention facilities within floodplains is strongly discouraged. When this situation is deemed unavoidable, critical examination must be given to ensure that the proposed basin remains functioning *effectively* during the 10-year flood event. The structural integrity and safety of the basin must also be evaluated thoroughly under 100-year flood conditions as well as the basin's impact on the characteristics of the 100-year floodplain. When basin construction is proposed within a floodplain, construction and permitting must comply with all applicable regulations under FEMA's National Flood Insurance Program.

3.2.12 Basin Location

When possible, enhanced extended detention facilities should be placed in low profile areas. When such a basin must be situated in a high profile area, care must be given to ensure that the facility empties completely, save for the marsh area, within a 72 hour maximum. The location of an extended detention basin in a high profile area places a great emphasis on the facility's ongoing maintenance.

3.2.13 Hydrology

The marsh area of an enhanced extended detention basin must support aquatic and emergent plant species in order for the basin to support the pollutant removal efficiencies expressed in Table 3.1. While a quantified volumetric flow rate is not explicitly required, the basin's contributing watershed should supply enough runoff to ensure that the marsh pools of varying depth are maintained as intended.

3.3 General Design Guidelines

The following presents a collection of broad design issues to be considered when designing an enhanced extended detention basin.

3.3.1 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam.

> *"The foundation material under the dam and the material used for the embankment of the dam should be an AASHTO Type A-4 or finer and/or meet the approval of the Materials Division. If the native material is not adequate, the foundation of the dam is to be excavated and backfilled a minimum of 4' or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."*

If the basin embankment height exceeds 15', or if the basin includes a permanent pool (excluding the shallow marsh area), the design of the dam should employ a homogenous embankment with seepage controls or zoned embankments.

During the initial subsurface investigation, additional borings should be made near the center of the proposed basin when:

- Excavation from the basin will be used to construct the embankment
- The likelihood of encountering rock during excavation is high
- A high or seasonally high water table, generally 2' or less, is suspected

3.3.2 Outfall Piping

The pipe culvert under or through the basin embankment shall be reinforced concrete equipped with rubber gaskets. Pipe: Specifications Section 232 (AASHTO M170), Gasket: Specification Section 212 (ASTM C443).

A concrete cradle shall be used under the pipe to prevent seepage through the dam. The cradle shall begin at the riser or inlet end of the pipe, and extend the pipe's full length.

3.3.3 Embankment

The top width of the embankment should be a minimum of 10' in width to provide ease of construction and maintenance. Positive drainage should be provided along the embankment top.

The embankment slopes should be no steeper than 3H:1V to permit mowing and other maintenance.

3.3.4 Embankment Height

A detention basin embankment may be regulated under the Virginia Dam Safety Act, Article 2, Chapter 6, Title 10.1 (10.1-604 Et seq.) of the Code of Virginia and Dam Safety Regulations established by the Virginia Soil and Water Conservation Board (VS&WCB). A detention basin embankment may be excluded from regulation if it meets any of the following criteria:

- is less than 6' in height
- has a capacity of less than 50 acre-ft and is less than 25' in height
- has a capacity of less than 15 acre-ft and is more than 25' in height
- will be owned or licensed by the Federal Government

When an embankment is not regulated by the Virginia Dam Regulations, it must still be evaluated for structural integrity when subjected to the 100-year flood event.

3.3.5 Prevention of Short-Circuiting

Short circuiting of inflow occurs when the basin floor slope is excessive and/or the pond's length to width ratio is not large enough. Short circuiting of flow can greatly reduce the hydraulic residence time within the basin, thus negatively impacting the observed water quality benefit.

To combat short-circuiting, and reduce erosion, the maximum longitudinal slope of the basin floor shall be no more than 2%. To maintain minimal drainage within the facility, the floor shall be no less than 0.5% slope from entrance to discharge point.

It is preferable to construct the basin such that the length to width ratio is 3:1 or greater, with the widest point observed at the outlet end. 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. When this minimum ratio is not possible, consideration should be given to pervious baffles.

3.3.6 Ponded Depth

The basin depth, measured from basin floor to the principal spillway's lowest discharge outlet (excluding the water quality orifice) should not exceed 3', if practical, to reduce hazard potential and liability issues. This depth restriction necessarily excludes deep pool zones, which range in depth between 1.5 and 4'.

3.3.7 Principal Spillway Design

The basin outlet should be designed in accordance with Minimum Standard 3.02 of the Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.). *The primary control structure (riser or weir) should be designed to operate in weir flow conditions for the full range of design flows*. If this is not possible, and orifice flow regimes are anticipated, the outlet must be equipped with an anti-vortex device, consistent with that described in Minimum Standard 3.02.

3.3.8 Fencing

Fencing is typically *not required or recommended* on most VDOT detention facilities. However, exceptions do arise, and the fencing of a dry extended detention facility may be needed. Such situations include:

- Ponded depths greater than 3' and/or excessively steep embankment slopes
- The basin is situated in close proximity to schools or playgrounds, or other areas where children are expected to frequent
- It is recommended by the VDOT Field Inspection Review Team, the VDOT Residency Administrator, or a representative of the City or County who will take over maintenance of the facility

"No Trespassing" signs should be considered for inclusion on all detention facilities, whether fenced or unfenced.

3.3.9 Sediment Forebays

Each basin inflow point should be equipped with a sediment forebay. Individual forebay volumes should range between 0.1" and 0.25" over the outfall's contributing new impervious area with the sum of all forebay volumes not less than 10% of the total WQ_V. When properly constructed, the forebay volumes can be considered a portion of the deep pool zone volume requirement.

3.3.10 Discharge Flows

All basin outfalls must discharge into an adequate receiving channel per the most current Virginia Erosion and Sediment Control (ESC) laws and regulations. Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

3.4 Design Process

Many of the design elements in an enhanced extended detention basin are identical to those of a dry extended detention basin. For those design items, the reader is referred to *Section 2 – Dry Extended Detention Basin*. The design items presented in detail in this section are exclusive to *enhanced* extended detention basins.

This section presents the design process applicable to enhanced extended detention basins serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered in linear development projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full discussion of hydrologic principles is beyond the scope of this report, and the user is referred to Chapter 4 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, ET SEQ.) for expanded hydrologic methodology.

The following example basin design will provide the water quality and quantity needs arising from the construction of a small interchange and new section of two lane divided highway in Staunton. The total project site, including right-of-way and all permanent easements, consists of 24.8 acres. Pre and post-development hydrologic characteristics are summarized below in Table 3.1. Initial geotechnical investigations reveal a soil infiltration rate of 0.01 in/hr.

Table 0.1 - Hydrologic Characteristics of Example Project Site

	Pre-Development	Post-Development
Project Area (acres)	24.8	24.8
Land Cover	Unimproved Grass Cover	11.2 acres *new* impervious cover
Impervious Percentage	0	45

Step 1 - Compute the Required Water Quality Volume

The project site's water quality volume is a function of the developed new impervious area. This basic water quality volume is computed as follows:

$$WQV = \frac{NIA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

NIA= New Impervious Area (square feet)

An enhanced dry detention basin must be sized to provide an extended detention volume of no less than *twice* the computed water quality volume.

This volume should be distributed equally between the permanent marsh area and a separate extended detention volume.

When the proposed basin is to be implemented as a *channel erosion control* basin, the extended draw down volume is computed as the volume of runoff generated from the basin's contributing drainage area by the 1-year return frequency storm. This channel protection volume must be detained and released over a period of not less than 24 hours.

When the 1-year return frequency storm is detained for a minimum of 24 hours there is no need to provide additional or separate storage for the WQ_V provided it can be demonstrated that the WQ_V will be detained for approximately 24 hours. It is noted that providing extended 24+ hour detention for the 1-year runoff volume may require the basin size to be 1.5 to 2 times the volume required to simply reduce the 2 and 10-year runoff events to pre-development levels.

The basis of this example lies in the design of Best Management Practices for *water quality improvement*. Therefore, the example basin is sized as a water quality control basin and not a channel erosion control basin.

The demonstration project site is comprised of a total drainage area of 24.8 acres. The total new impervious area within the project site is 11.2 acres. Therefore, the water quality volume is computed as follows:

$$WQV = \frac{11.2ac \times 43{,}560\frac{ft^2}{ac} \times \frac{1}{2}in}{12\frac{in}{ft}} = 20{,}328\,ft^3$$

The total volume provided by summing each of the three marsh zones must be at least 20,328 ft^3, and an additional 20,328 ft^3 of storage must be provided for a 30-hour extended drawdown of storm inflow.

Step 2 - Sizing the Marsh Area Zones

The marsh area of an extended detention basin is comprised of three distinct zones. The surface area and storage volume allocated to each of the zones is very specific in an effort to provide maximum water quality benefit within the basin. The three zones are described as follows.

The *Deep Pool Zone* ranges in depth from 1.5' to 4', and may be comprised of the following three categories:

- sediment forebays

- micro pools
- deep water channels

A sediment forebay must be provided at any point in the basin that receives concentrated discharge from a pipe, open channel, or other means of stormwater conveyance. The inclusion of a sediment forebay in these locations assists maintenance efforts by isolating the bulk of sediment deposition in well-defined, easily accessible locations. The volume of storage provided at each forebay should range between 0.1” and 0.25” of runoff over the individual outfall’s new contributing impervious area, with the sum of all forebay volumes not less than 10% of the total extended detention volume.

A micro-pool should be provided near the basin outlet point (principal spillway). The inclusion of a deep pool near the basin outlet will reduce the likelihood of the water quality outlet becoming clogged by trash, debris, or floating plant matter.

Deep water channels may be employed to lengthen the flow path from pond inflow points to the principal spillway.

The sum of all forebay, micro-pool, and deep channel volumes should be no less than 40% of the computed water quality volume.

Low Marsh Zones are those regions of the marsh ranging in depth between 6 and 18”. The sum of all low marsh zones should be no less than 40% of the computed water quality volume.

High Marsh Zones are those regions of the marsh ranging in depth from 0 to 6”. The high marsh zone is capable of supporting the most diverse mix of vegetation. The sum of all high marsh zones should be no less than 20% of the computed water quality volume.

In addition to the marsh zone volume requirements, surface area guidelines exist. *At a minimum, the surface area of all marsh zones should equal 1% of the basin’s total contributing drainage area*. Table 3.2 shows the recommended surface area distribution among the three marsh zones.

Table 0.2 - Marsh Zone Surface Area Allocation

Zone	Percentage of Total Marsh Surface Area
Deep Pool	20
Low Marsh	40
High Marsh	40

When designing the marsh area of an enhanced detention basin, both surface area and volume guidelines must be considered. The following steps illustrate this process for the example project site.

Step 2B - Compute the Minimum Marsh Surface Area

The summation of all three marsh zone surface areas must not be less than 1% of the basin's total contributing drainage area. The minimum marsh surface area is therefore computed as:

$$24.8ac \times \frac{43{,}560\,ft^2}{ac} \times 0.01 = 10{,}803\,ft^2$$

Step 2C - Size the Deep Pool Zone

The deep pool zones must provide a minimum of 40% of the computed water quality volume, and comprise at least 20% of the marsh's total surface area. These minimum values are computed as follows:

$$V_{Min} = 0.40 \times 20{,}328\,ft^3 = 8{,}132\,ft^3$$

$$SA_{Min} = 0.20 \times 10{,}803\,ft^2 = 2{,}161\,ft^2$$

At this point, it is unknown which of these minimum values will govern the design. The proposed basin will have two inflow points and a micro-pool located near the principal spillway. At this point, we will assume each of these three deep water pools (two sediment forebays and the micro-pool) will average 4' in depth. Accounting for the side slopes of the deep pools, the effective depth is assumed to be 2'. The surface area required, at this effective depth, to provide the minimum volume of 8,132 ft^3 is therefore computed as:

$$SA = \frac{8{,}132\,ft^3}{2\,ft} = 4{,}066\,ft^2$$

This computed value is greater than the minimum surface area requirements previously established. Therefore, the *total deep water surface area is set at 4,066 ft^2*.

The total deep pool volume must be distributed across the two sediment forebays and the micro-pool. The following calculations demonstrate this volume allocation.

The total forebay volume should be calculated as 0.10 – 0.25" of runoff over the site's new impervious area, not to be less than 10% of the total water quality volume. With the water quality volume previously computed as ½" of runoff over the new impervious area, 0.10" over this same area will yield an acceptable forebay volume equaling 20% of the total water quality volume.

$$V_{Forebays} = \left(\frac{0.1in}{12\frac{in}{ft}} \right) \times 11.2acres \times \left(\frac{43{,}560\,ft^2}{ac} \right) = 4{,}066\,ft^3$$

At an *effective* depth of 2', the surface area allocated to the sediment forebays is calculated as:

$$SA_{Forebays} = \frac{4{,}066\,ft^3}{2\,ft} = 2{,}033\,ft^2$$

The total computed forebay volume and surface area will be distributed equally across the two required forebays (one at each inflow location).

The remaining deep pool volume must be obtained in the basin's micro-pool.

$$V_{Micropool} = 8{,}132\,ft^3 - Forebay\,Volume = 8{,}132ft^3 - 4{,}066\,ft^3 = 4{,}066\,ft^3$$

At an effective depth of 2', this volume is attained with a surface area computed as follows:

$$SA_{Micropool} = \frac{4{,}066\,ft^3}{2\,ft} = 2{,}033\,ft^2$$

The deep pool surface area and volume distribution is shown in Table 3.3.

Table 0.3 - Deep Pool Volume and Surface Area Allocation

Basin Location	**Volume (ft^3)**	**Surface Area (ft^2)**
Forebay 1	2,033	1,017
Forebay 2	2,033	1,017
Micropool	4,066	2,033
Total	***8,132***	***4,067***

Step 2D - Size the Low Marsh Area

The low marsh zone must provide a minimum of 40% of the computed water quality volume, and comprise at least 40% of the marsh's total surface area. These minimum values are computed as follows:

$$V_{Min} = 0.40 \times 20{,}328\,ft^3 = 8{,}132\,ft^3$$

$$SA_{Min} = 0.40 \times 10{,}803\,ft^2 = 4{,}322\,ft^2$$

At this point, it is unknown which of these minimum values will govern the design. The low marsh zone ranges in depth from 6" – 18". The surface area required, at an average depth of 12", to provide the minimum volume of 8,132 ft^3 is therefore computed as:

$$SA = \frac{8{,}132\, ft^3}{1\, ft} = 8{,}132\, ft^2$$

This computed value is greater than the minimum surface area requirements previously established. Therefore, the *total low marsh surface area is set at 8,132 ft^2*.

Step 2E - Size the High Marsh Area

The high marsh zone must provide a minimum of 20% of the computed water quality volume, and comprise at least 40% of the marsh's total surface area. These minimum values are computed as follows:

$$V_{Min} = 0.20 \times 20{,}328\, ft^3 = 4{,}066\, ft^3$$

$$SA_{Min} = 0.40 \times 10{,}803\, ft^2 = 4{,}322\, ft^2$$

At this point, it is unknown which of these minimum values will govern the design. The high marsh zone exhibits a ponding depth of 6". The surface area required, at a depth of 6", to provide the minimum volume of 4,066 ft^3 is therefore computed as:

$$SA = \frac{4{,}066\, ft^3}{0.5\, ft} = 8{,}132\, ft^2$$

This computed value is greater than the minimum surface area requirements previously established. Therefore, the *total high marsh surface area is set at 8,132 ft^2*.

Step 2F - Verify Marsh Zone Surface Area and Volume Allocations

The marsh zone calculations must now be evaluated to ensure that the previously determined minimum values are obtained. Table 3.4 illustrates this verification.

Table 0.4 - Marsh Surface Area and Volume Verification

Volume (ft^3)				
Deep Pool*	**Low Marsh**	**High Marsh**	**Total**	**Minimum Allowable**
8,132	8,132	4,066	20,330	20,328
Surface Area (ft^2)				
Deep Pool*	**Low Marsh**	**High Marsh**	**Total**	**Minimum Allowable**
4,067	8,132	8,132	20,331	10,803

* Includes sediment forebays and micro-pool

Step 3 - Construction of Storage Versus Elevation Data

Having determined the required surface area and storage volume for each of the three marsh zones, we turn to the next step of constructing a stage – storage relationship for the marsh-pond system. Each site is unique, both in terms of constraints and required storage volume. Because of this, the development of a proposed basin grading plan may be an iterative process. The stage – storage relationship should provide not only the required marsh volume, but also the 30-hour extended draw down volume, any required flood control storage volume(s), and the volume necessary to meet minimum freeboard requirements (see *Section 2 – Dry Extended Detention Basin*).

When a detention basin is to be enhanced, the ponding depth of the extended detention volume should not exceed 3'. Extended detention ponding depths greater than 3' and the frequent inundation of those areas are not conducive to the establishment of a dense, diverse mix of wetland vegetation. Typically, this restraint does not present a design problem, as the required surface area of the marsh will offset the limitation in ponding depth.

The required 30-hour draw down volume for this example is equal to the computed water quality volume (20,328 ft^3). This volume is "stacked" on top of the marsh, and must be attained at an elevation of no more than 3' above the marsh's permanent surface. This occurs at an approximate elevation of 2104' as shown in Table 3.5 and Figure 3.5.

Table 3.6 and Figure 3.5 present the stage – storage relationship for the computed marsh area and extended detention volumes.

Table 0.5 - Stage – Storage Relationship

Elevation	Incremental Volume (ft^3)	Total Volume (ft^3)
2100	0	0
2100.5	648	648
2101	648	1296
2101.5	864	2160
2102	864	3024
2102.5	1081	4105
2103	2301	6406
2103.5	5184	11590
2104	9250	20840
2104.5	10145	30985
2105	10160	41145

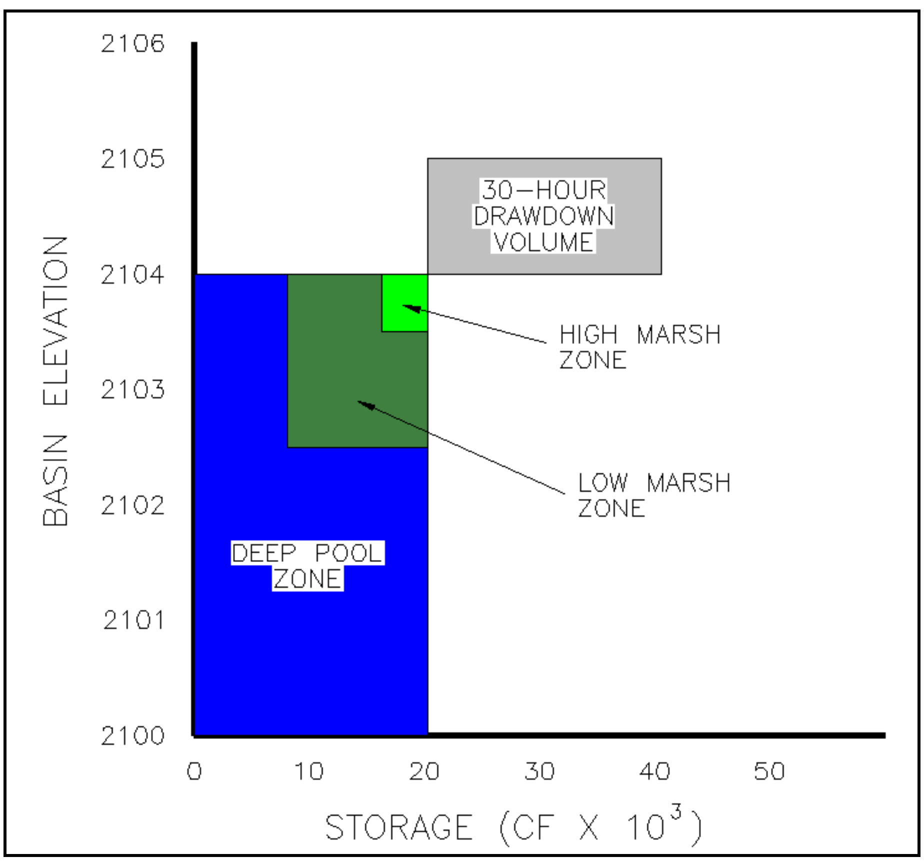


Figure 0.1 - Graphical Elevation – Storage Relationship

Upon development of the marsh and extended detention stage – storage relationships, the next step(s) are to design and evaluate the basin for mitigation of post-development inflows (both in terms of water quality detention and flood peak reduction). The reader is referred to *Section 2 – Dry Extended Detention Basin*, Steps 5 – 8 for detailed methodology on these topics.

Step 4 - Water Balance Calculation

To ensure that the basin's permanent marsh volume does not become dry during extended periods of low inflow, the designer must perform a water balance calculation. The approach considers a 45 day period with no significant precipitation and thus no significant surface runoff.

Table 3.6 presents potential evaporation rates for various locations in Virginia.

Table 0.6 - Potential Evaporation Rates (Inches)
Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, ET SEQ.)

Station	April	May	June	July	August	Sept.
Charlottesville	2.24	3.84	5.16	6.04	5.45	3.87
Danville	2.35	3.96	5.31	6.23	5.69	3.91
Farmville	2.34	3.81	5.13	6.00	5.41	3.71
Fredericksburg	2.11	3.80	5.23	6.11	5.46	3.83
Hot Springs	1.94	3.41	4.50	5.14	4.69	3.33
Lynchburg	2.21	3.72	4.99	5.85	5.31	3.70
Norfolk	2.20	3.80	5.37	6.34	5.79	4.14
Page County	1.68	3.06	4.09	4.71	4.26	3.05
Pennington Gap	2.14	3.59	4.72	5.45	4.97	3.60
Richmond	2.28	3.89	5.31	6.23	5.64	3.92
Roanoke	2.20	3.75	4.99	5.85	5.30	3.67
Staunton	2.00	3.52	4.77	5.52	4.95	3.47
Wash. National Airport	2.13	3.87	5.50	6.51	5.84	4.06
Williamsburg	2.27	3.86	5.23	6.14	5.61	3.97
Winchester	2.07	3.68	4.99	5.82	5.26	3.67
Wytheville	2.01	3.43	4.46	5.17	4.71	3.39

The greatest potential evaporation for the project site (Staunton) occurs during the months of July and August, 5.52" and 4.95" respectively. Therefore, the total evaporation over a 45 day period is estimated as follows:

Average evaporation per month = $\dfrac{5.52in + 4.95in}{2} = 5.24in$

Average evaporation per day = $\dfrac{5.24\dfrac{in}{month}}{31\dfrac{day}{month}} = 0.17\dfrac{in}{day}$

The evaporation loss over a 45-day period is calculated as follows.

$$45\,\text{days}\ X\ 0.17\frac{in}{day} = 7.65in = 0.64\,ft$$

The total surface area of the marsh is 20,331 ft^3. Therefore, the total volume of water lost to evaporation is computed as:

$$20{,}331\,ft^2 \times 0.64\,ft = 13{,}012\,ft^3$$

The volume of water lost to evaporation must be added to that lost to infiltration. As previously stated, the initial geotechnical tests revealed site soil infiltration rates to be 0.01 in/hr. The infiltration is assumed to occur over the entire marsh area, whose surface areas sum to 20,331 ft^2. The volume of water lost to infiltration is computed as:

$$20{,}331\,ft^2 \times 0.01\frac{in}{hr} \times \frac{1\,ft}{12in} 24\frac{hr}{day} \times 45days = 18{,}298\,ft^3$$

The total volume of water lost to evaporation and infiltration over the 45 day drought period is therefore computed as:

$$18{,}298\,ft^3 + 13{,}012\,ft^3 = 31{,}310\,ft^3$$

This value exceeds the total marsh volume of 20,328 ft^3, implying that a 45 day drought period will leave the marsh area in a completely dry state. Over time, it is quite likely that the infiltration rate of the basin soil will decrease considerably due to clogging of the soil pores. However, the aquatic and wetland plant species will likely not survive an extended period of drought that occurs prior to this clogging. Therefore, at this point in the design, it would be recommended to install a clay or synthetic basin liner as approved by the Materials Division. A typical infiltration rate for synthetic liner may be on the order of 3x10^{-7} in/sec. The calculation is repeated for this rate of infiltration.

$$20{,}331\,ft^2 \times 3x10^{-7}\frac{in}{\sec} \times \frac{1\,ft}{12in} \times 3{,}600\frac{\sec}{hr} \times 24\frac{hr}{day} \times 45days = 1{,}976\,ft^3$$

The recalculated volume of water lost to evaporation and infiltration over the 45 day drought period is therefore computed as:

$18{,}298\,ft^3 + 1{,}976\,ft^3 = 20{,}274\,ft^3$

While the extended drought period does impact the marsh area significantly, a minimal volume of water *is* retained in the marsh.

The volume of runoff necessary to replenish the depleted marsh volume is computed as follows:

Total contributing drainage area = 24.8 acres

Stored volume lost to evaporation and infiltration = 20,274 ft^3

$$\frac{20{,}274\,ft^3}{24.8ac \times \dfrac{43{,}560\,ft^2}{ac}} = 0.019 \text{ Watershed Feet} = 0.23 \text{ Watershed Inches}$$

A precipitation event yielding a total runoff of 0.23" or more across the contributing watershed will replenish the depleted marsh volume.

Step 5 - Landscaping

Generally, the non-marsh regions of an enhanced dry extended detention basin can be landscaped in the same manner as a non-enhanced basin (reference *Design Example One – Dry Extended Detention Basin)*. However, careful attention must be given to the types of vegetation selected for the basin marsh areas. For these regions, the vegetative species must be selected based on their inundation tolerance and the anticipated frequency and depth of inundation.

If appropriate vegetative species are selected, the entire marsh area should be colonized within three years. Because of this rapid colonization, only one-half of the total low and high marsh zone areas needs to be seeded initially. A total of five to seven different emergent species should be planted in the basin marsh areas. Both the high and low marsh areas should each be seeded with a *minimum* of two differing species.

The regions of varying depth within the basin are broadly categorized by zone as shown in Figure 3.6.

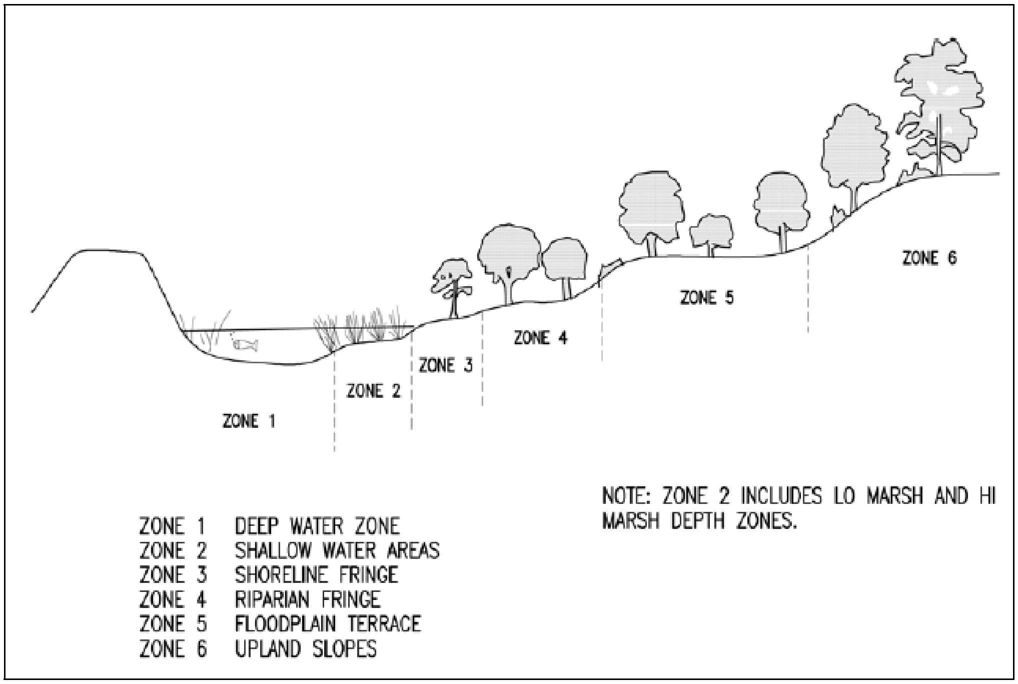


Figure 0.2 - Planting Zones for Stormwater BMPs
Virginia Stormwater Management Handbook (DCR/DEQ, 1999, ET SEQ.)

Suitable planting species for each of the zones identified in Figure 3.6 are recommended in Chapter 3-05 of the *Virginia Stormwater Management Handbook*, (DCR/DEQ, 1999, ET SEQ.). Ultimately, the choice of planting species should be largely based on the project site's physiographic zone classification. Additionally, the selection of plant species should match the native plant species as closely as possible. Surveying a project site's native vegetation will reveal which plants have adapted to the prevailing hydrology, climate, soil, and other geographically-determined factors. Figure 3.05-4 of the *Virginia Stormwater Management Handbook* provides guidance in plant selection based on project location.

Generally, stormwater management basins should be permanently seeded within 7 days of attaining final grade. This seeding should comply with Minimum Standard 3.32, Permanent Seeding, of the *Virginia Erosion and Sediment Control Handbook*, (DCR/DEQ, 1992). It must be noted, however, that permanent seeding is *prohibited* in Zones one through four of Figure 3.6. The use of conventional permanent seeding in these zones will result in the grasses competing with the requisite wetland emergent species.

When erosion of basin soil prior to the establishment of mature stand of wetland vegetation is a concern, Temporary Seeding (Minimum Standard 3.31) of the *Virginia Erosion and Sediment Control Handbook*, (DCR/DEQ, 1992) may be considered. However, the application rates specified should be reduced to as low as practically possible to minimize the threat of the Temporary Seeding species competing with the chosen emergent wetland species.

All chosen plant species should conform to the *American Standard for Nursery Stock*, current issue, and be suited for USDA Plant Hardiness Zones 6 or 7, see Figure 3.7.

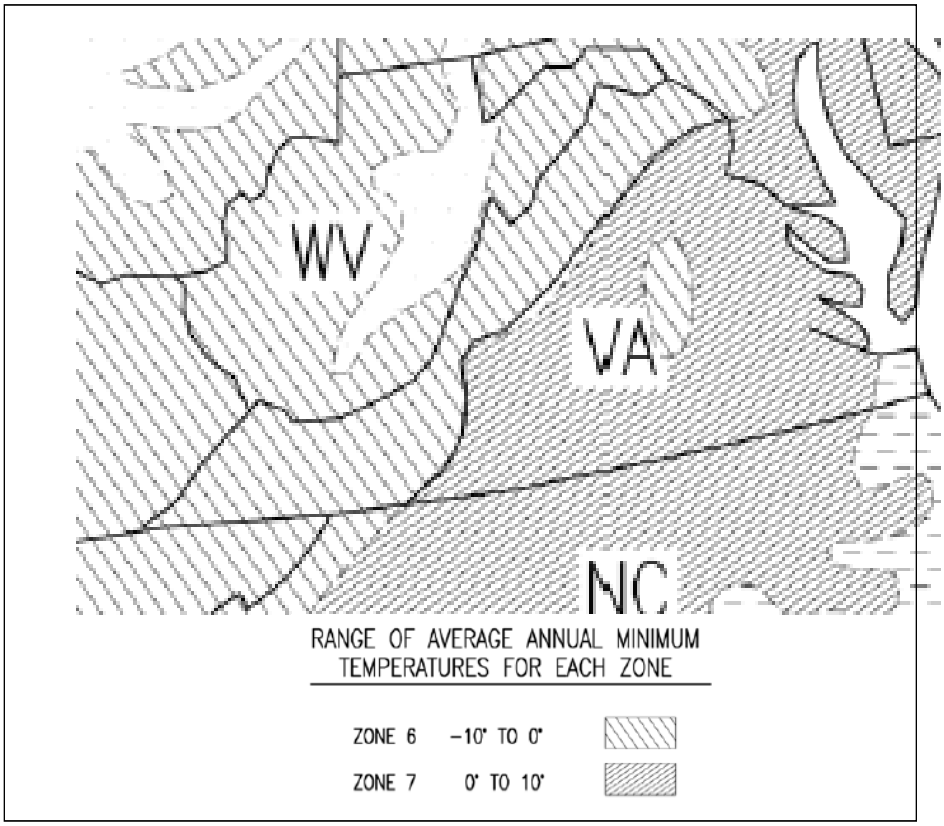


Figure 0.3. USDA Plant Hardiness Zones

Under no circumstances should trees or shrubs be planted on the basin embankment. The large root structure may compromise the structural integrity of the embankment.

4.1 Retention Basin - Overview of Practice

A retention basin (also called a “wet pond”), by definition, is a basin which retains a portion of its inflow in a permanent pool such that the basin is typically wet even during non-runoff producing periods. Generally, stormwater runoff is stored above the permanent pool, as necessary, to provide flood control and/or downstream channel protection. Retention basins are capable of providing downstream flood control, water quality improvement, channel erosion control, and the reduction of post-development runoff rates to pre-development levels. Retention basins have some of the highest pollutant removal efficiencies of any BMP available.

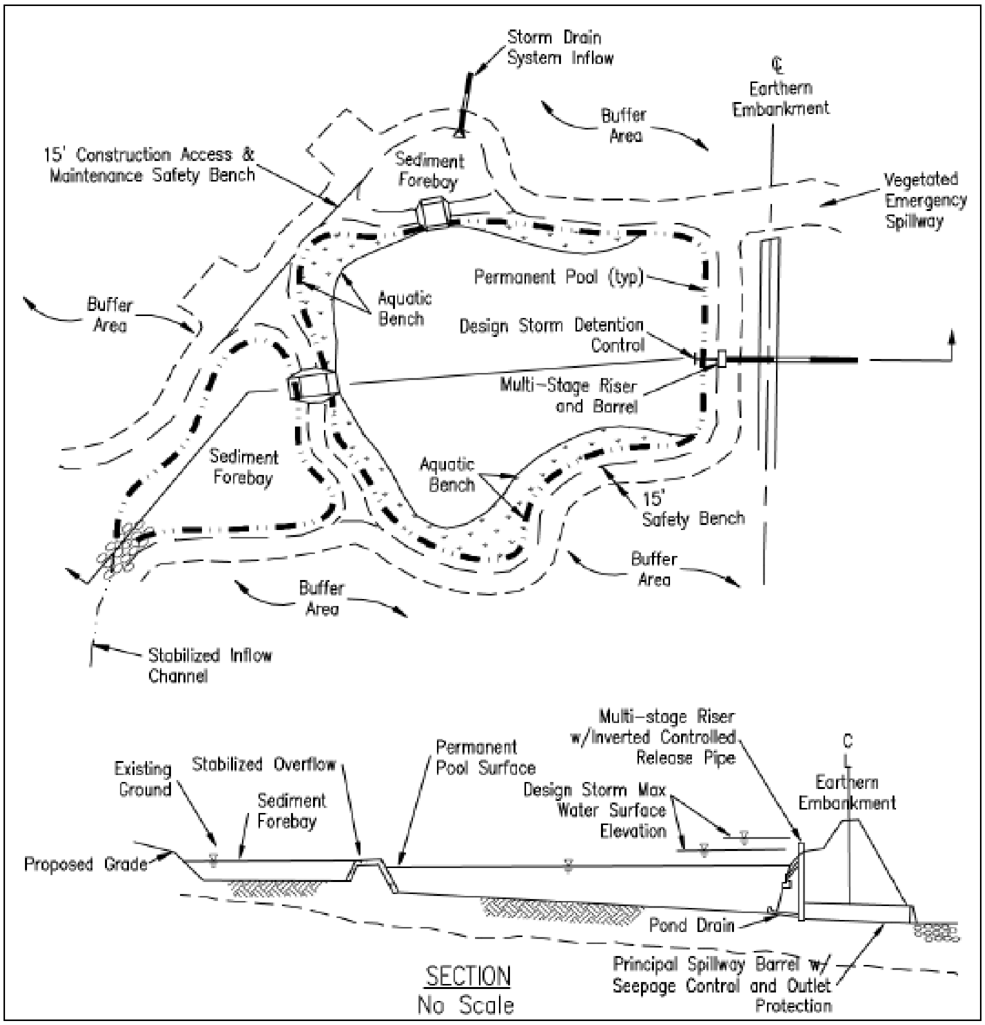


Figure 4.1.1 – DCR/DEQ Schematic Retention Basin Plan and Sectional View
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

Figure 4.1 presents the schematic layout of a retention basin presented in the *Virginia Stormwater Management Handbook* (DCR/DEQ, 1999, Et seq.).

4.2 Site Constraints and Siting of the Facility

In addition to new impervious cover, the engineer must consider a number of additional site constraints when the implementation of a retention basin is proposed. These constraints are discussed as follows.

4.2.1 Minimum Drainage Area

A retention basin should generally not be considered for contributing drainage areas of less than 10 acres. Critical concern is the presence of adequate baseflow to the pond. Should the pond become dry or stagnant, problems such as algae blooms and undesirable odors will arise. Regardless of drainage area, all proposed retention basins should be subjected to a low flow analysis to ensure that an adequate permanent pool volume is retained even during periods of dry weather when evaporation and/or infiltration are occurring at a high rate. The anticipated baseflow from a fixed drainage area can exhibit great variability, and insufficient baseflow may require consideration of alternate BMP measures.

The presence of a shallow groundwater table, which is common in the Tidewater region of the state, may allow for the implementation of a retention basin whose contributing drainage area is very small. These circumstances are site-specific, and the groundwater elevation must be monitored closely to establish the design elevation of the permanent pool.

4.2.2 Maximum Drainage Area

The maximum drainage area to retention basin is not explicitly restricted; however, the designer should consider that, generally, an area ranging between 1 and 3% of the total contributing drainage area is required for construction of the basin. Therefore, the total contributing drainage area to a retention basin is frequently limited to 10 square miles. (FHWA, 1996) It is noted that a retention basin serving 10 square miles will require a minimum of 128 acres in area. Such a facility would be considered “regional,” and is not typically encountered on linear development projects.

4.2.3 Separation Distances

Retention basins should be kept a minimum of 20’ from any permanent structure or property line, and a minimum of 100’ from any septic tank or drainfield.

4.2.4 Site Slopes

Generally, retention basins should not be constructed within 50’ of any slope steeper than 15%. When this is unavoidable, a geotechnical report is required to address the potential impact of the facility in the vicinity of such a slope. This report should be submitted to the Materials Division for evaluation.

4.2.5 Site Soils

The implementation of a retention basin can be successfully accomplished in the presence of a variety of soil types; however, when such a facility is proposed, *a subsurface analysis and permeability test is required*. The required subsurface analysis should investigate soil characteristics to a depth of no less than 3' below the proposed bottom of the basin. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils. When a retention basin is being considered for a site, water inflows (baseflow, surface runoff, and groundwater) must be greater than losses to evaporation and infiltration. Consequently, soils exhibiting high infiltration rates are not suited for the construction of a retention basin. Often, soils of moderately high permeability are capable of supporting dry extended detention facilities and even the permanent marsh areas of an enhanced dry extended detention facility; however, the hydraulic head (pressure) generated from a permanent pool may increase a soil's effective infiltration rate rendering similar soils unsuitable for a retention basin. A clay liner, geosynthetic membrane, or other material (as approved by the Materials Division) may be employed to combat excessively high infiltration rates. The basin embankment material must meet the specifications detailed later in this section and/or be approved by the Materials Division.

4.2.6 Rock

The presence of rock within the proposed construction envelope of a retention basin should be examined during the aforementioned subsurface investigation. When blasting of rock is necessary to obtain the desired basin volume, a liner should be used to eliminate unwanted losses through seams in the underlying rock.

4.2.7 Existing Utilities

Basins should not be constructed over existing utility rights-of-way or easements. When this situation is unavoidable, permission to impound water over these easements must be obtained from the utility owner *prior* to design of the basin. When it is proposed to relocate existing utility lines, the costs associated with their relocation should be included in the overall basin construction cost.

4.2.8 Karst

The presence of karst topography places even greater importance on the initial subsurface investigation. Implementation of retention basins in karst regions may greatly increase the design and construction cost of the facility, and must be evaluated early in the planning phases of a project. ***Construction of stormwater management facilities within a sinkhole is prohibited.*** When the construction of such a facility is planned along the periphery of a sinkhole, the facility design must comply with the guidelines found in Chapter 5 of this Manual and DEQ's Technical Bulletin #2 *"Hydrologic Modeling and Design in Karst."*

4.2.9 Wetlands

When the construction of a retention basin is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify the wetlands' boundaries, their protected status, and the feasibility of BMP implementation in their vicinity. In Virginia, the Department of Environmental Quality (DEQ) and the U.S. Army Corps of Engineers (USACOE) should be contacted when such a facility is proposed in the vicinity of known wetlands.

4.2.10 Upstream Sediment Considerations

Close examination should be given to the flow velocity at all basin inflow points. When entering flows exhibit erosive velocities, they have the potential to greatly increase the basin's maintenance requirements by depositing large amounts of sediment. Additionally, when the basin contributing drainage area is highly pervious, it has the potential to hinder basin performance through the deposition of excessive sediment. Sediment forebays should be located at all entrance points to the basin which receive concentrated runoff. A 20' wide vegetated buffer should be located around the entire periphery of the basin to further combat against excessive sediment deposition. The designer must consider this buffer early in the project planning stages, as it inherently increases the land area that is dedicated to the basin.

4.2.11 Downstream Considerations

Retention basins can significantly alter the characteristics of the watercourses to which they discharge. These impacts are most often recognized in terms of biological oxygen demand (BOD), dissolved oxygen (DO), and water temperature. These impacts may be quite detrimental to the receiving water body, particularly if the body of water is a designated cold water trout stream. Careful consideration must be given during the design process, particularly to the depth and configuration of the basin permanent pool, to minimize the impacts to downstream waters. When the proposed basin will discharge into a stream which supports a trout population, the designer should contact the Department of Game and Inland Fisheries (DGIF) to determine the feasibility of the basin and any additional measures which may be required should its design and construction proceed.

The designer must also be aware of other impounding facilities within the same watershed as the proposed basin. The presence of multiple basins in a single watershed may give rise to *peak synchronization* such that releases from individual basins coincide resulting in a cumulative flow rate beyond what downstream receiving channels are capable of accommodating. Basin discharge synchronization may also lead to an increased duration of high flow in downstream channels. Flow durations beyond what are historically observed in natural channels may lead to excessive erosion and degradation.

4.2.12 Floodplains

The construction of stormwater impounding facilities within floodplains is strongly discouraged. When this situation is deemed unavoidable, critical examination must be given to ensure that the proposed basin remains functioning *effectively* during the 10-year flood event. The structural integrity and safety of the basin must also be evaluated thoroughly under 100-year flood conditions as well as the basin's impact on the characteristics of the 100-year floodplain. When basin construction is proposed within a floodplain, construction and permitting must comply with all applicable regulations under FEMA's National Flood Insurance Program.

4.2.13 Basin Location

Unlike dry detention facilities, retention basins are often considered a desirable site amenity. Therefore, when properly designed, landscaped, and maintained, retention basins may be suitable for high visibility locations; however, when a retention basin is proposed in a high visibility location, ongoing maintenance of the facility is critical to its acceptance by neighboring landowners.

4.2.14 Implementation as a Regional Stormwater Management Facility

The costs associated with constructing and maintaining a retention basin are often prohibitive; however, as the area contributing runoff to a retention basin increases, the total cost per acre decreases. Therefore, when a retention basin is chosen as the stormwater BMP it should, when possible, be implemented as part of a regional approach to stormwater management. The concept of regional stormwater management is endorsed by VDOT provided that the requirements are met.

- Development and use of regional stormwater management facilities must be a joint undertaking by VDOT and the local governing body. The site must be part of a master stormwater management plan developed and/or approved by the local governing body and any agreements related to 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 SWM program that complies with the Virginia SWM 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.

- Where an existing or potential VDOT roadway embankment will serve as an impounding structure for a regional facility, the right of way line will normally be set at the inlet face of the main drainage structure. The local government would be responsible for the maintenance and liabilities outside of the right of way and the VDOT would accept the same responsibilities inside the right of way.

- The design of regional stormwater management facilities must address any mitigation needed to meet the water quality and quantity requirements of proposed or future roadway projects within the contributing watershed. Regional SWM facilities located upstream of a roadway project shall provide sufficient mitigation for any water quality and quantity impacts of run-off from the roadway project which may bypass the facility.

4.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing a retention basin. Many of these items are expanded upon later in this document within the context of a full design example.

4.3.1 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam.

> *"The foundation material under the dam and the material used for the embankment of the dam should be an AASHTO Type A-4 or finer and/or meet the approval of the Materials Division. If the native material is not adequate, the foundation of the dam is to be excavated and backfilled a minimum of 4' or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."*

The presence of a permanent pool requires that the dam of a retention basin be composed of homogenous material with seepage controls or zoned embankments.

During the initial subsurface investigation, additional borings should be made near the center of the proposed basin when:

- Excavation from the basin will be used to construct the embankment
- The likelihood of encountering rock during excavation is high
- A high or seasonally high water table, generally 2' or less below the ground surface, is suspected

4.3.2 Outfall Piping

The pipe culvert under or through the basin embankment shall be reinforced concrete equipped with rubber gaskets. Pipe: Specifications Section 232 (AASHTO M170), Gasket: Specification Section 212 (ASTM C443).

A concrete cradle shall be used under the pipe to prevent seepage through the dam. The cradle shall begin at the riser or inlet end of the pipe, and extend the pipe's full length.

4.3.3 Embankment

The top width of the embankment should be a minimum of 10' in width to provide ease of construction and maintenance.

To permit mowing and other maintenance, the embankment slopes should be no steeper than 3H:1V. When the basin is proposed in a highly populated area, more gradual side slopes should be considered.

4.3.4 Embankment Height

A retention basin embankment may be regulated under the Virginia Dam Safety Act, Article 2, Chapter 6, Title 10.1 (10.1-604 et seq.) of the Code of Virginia and Dam Safety Regulations established by the Virginia Soil and Water Conservation Board (VS&WCB). A retention basin embankment may be excluded from regulation if it meets any of the following criteria:

- is less than 6' in height
- has a capacity of less than 50 acre-ft and is less than 25' in height
- has a capacity of less than 15 acre-ft and is more than 25' in height
- will be owned or licensed by the Federal Government

When an embankment is not regulated by the Virginia Dam Regulations, it must still be evaluated for structural integrity when subjected to the 100-year flood event.

4.3.5 Permanent Pool Volume

The volume of the basin permanent pool greatly influences the anticipated pollutant removal performance of the basin. Table 4.1 presents target phosphorus removal efficiencies corresponding to varying permanent pool volumes, and the impervious percentage to which each volume is best applied.

Table 4.3.1 – DCR/DEQ Retention Basin Removal Efficiencies
(*Virginia Stormwater Management Handbook,* 1999, Et seq.)

Pool Volume (Relative to WQV)	Target Phosphorus Removal Efficiency	Impervious Cover
3 x WQV	40%	22-37%
4 x WQV	50%	38-66%
4 x WQV with Aquatic Bench	65%	67-100%

Presently, the Department of Environmental Quality (DEQ) gives no additional water quality credit for an extended detention volume located above the basin permanent pool. Consequently, the water quality benefit of a retention basin is expressed solely as a function of its permanent pool volume.

The basin volume required to provide flood control in the form of reduced runoff peaks for various return frequency storms of interest is termed *dry storage*. This volume is "stacked" on top of the permanent pool volume and is released from the pond, generally, within a few hours of the conclusion of the runoff producing event.

If the basin is to serve the function of downstream *channel protection*, an additional volume must be stacked on top of the permanent pool and released over a period of not less than 24 hours. This volume is computed as the volume of runoff generated from the basin contributing drainage area by the 1-year return frequency storm.

The total basin volume is thus comprised of the permanent pool volume, the flood control volume for the greatest return frequency storm of interest, required freeboard, and, when applicable, the computed channel protection volume.

4.3.6 Prevention of Short-Circuiting (Basin Geometry)

Short-circuiting occurs when flows entering the basin pass rapidly through the basin without displacing an equal volume of previously stored water. Short-circuiting of flow can greatly reduce the hydraulic residence time within the basin, thus negatively impacting the water quality benefit. While site conditions will ultimately dictate the geometric configuration of the basin, it is preferable to construct the basin such that the length-to-width ratio is 3:1 or greater, with the widest point observed at the outlet end. 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. When this minimum ratio is not possible, consideration should be given to baffles constructed of gabions, earthen berms, or other permeable materials.

In addition to increasing the basin length-to-width ratio, the likelihood of short-circuiting can be further reduced by designing meandering flow paths rather than straight line paths from stormwater entrance points to the basin principal spillway.

4.3.7 Ponded Depth

The depth of the basin permanent pool affects the planting species selected for the basin as well as the types of aquatic and wildlife species that will inhabit the basin and its surrounding areas. Additionally, the depth of the permanent pool has a significant impact on pollutant removal performance of the basin. Basins sized too shallow will not support a diverse population of aquatic species, while basins whose permanent pool is excessively deep will tend to stratify. This stratification can potentially create anaerobic conditions leading to the resuspension / resolubilization of captured pollutants. (DCR/DEQ, 1999, Et seq.). The majority of the permanent pool volume should range in depth from 2' to 6'. Approximately 15% of the permanent pool volume should be comprised of regions less than 18" in depth. These regions are easily obtained with the inclusion of an *aquatic bench.*
An aquatic bench provides not only improved pollutant removal efficiency in the basin, but also serves as an important safety feature (discussed later). Table 4.2 presents recommended surface area – pool depth relationships.

Table 4.3.2 – DCR/DEQ Surface Area – Permanent Pool Depth Relationships
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

Pool Depth (ft)	Surface Area (% of Total Surface Area)
0 - 1.5	15%
1.5 - 2	15%
2 - 6	70%

4.3.8 Aquatic Bench

An aquatic bench is a 10' to 15' wide area that slopes from a depth of 0" at the shoreline of the basin to a depth of approximately 18" in the basin permanent pool. The shallow depth of the aquatic bench supports a diverse mix of emergent and wetland plant species as well as providing ideal habitat to predatory insects that feed on mosquitoes and other nuisance insects. Table 4.1 shows a target phosphorus removal efficiency of 65% for a basin equipped with an aquatic bench, compared to 50% for a basin with an equal pool volume, but no bench. The ability of an aquatic bench to support a dense and diverse mix of vegetation will also make the shoreline of the basin less susceptible to the erosive action associated with fluctuating water levels. Figure 4.2 illustrates the general configuration of an aquatic bench.

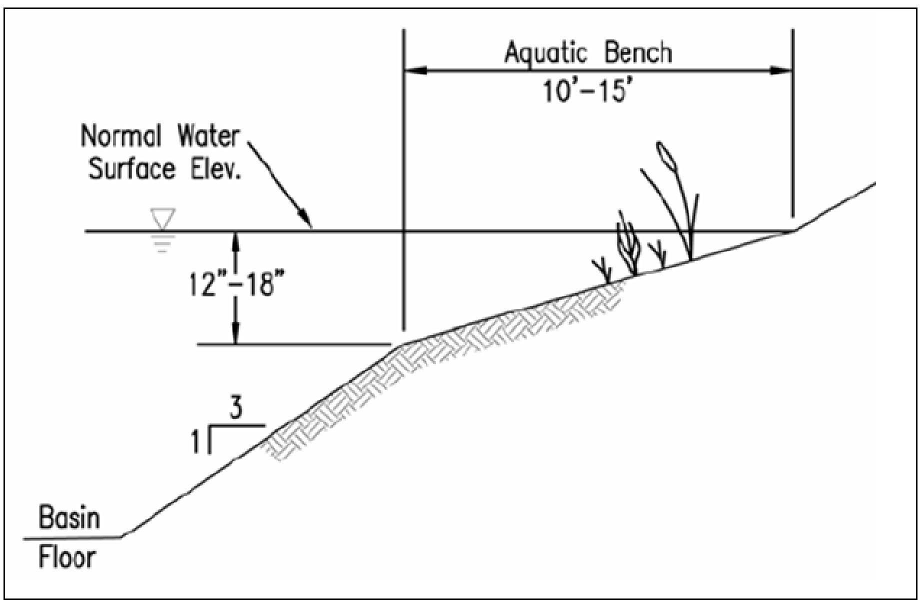


Figure 4.3.1 – DCR/DEQ Schematic Aquatic Bench Section
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

The inclusion of an aquatic bench adds a significant safety feature to the basin, as it provides spatial disconnection from the basin's peripheral slope and its submerged slope. Whenever the total surface area of the basin permanent pool exceeds 20,000 ft^2 an aquatic bench should be considered an essential safety feature.

4.3.9 Principal Spillway Design

The basin outlet should be designed in accordance with Minimum Standard 3.02 of the Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.). *The primary control structure (riser or weir) should be designed to operate in weir flow conditions for the full range of design flows*. This is to avoid vortex formation which can be highly destructive to the outlet structure. If this is not possible, and orifice flow regimes are anticipated, the outlet must be equipped with an anti-vortex device, consistent with that described in Minimum Standard 3.02 of the Virginia Stormwater Management Handbook.

4.3.10 Fencing

Fencing is typically *not required or recommended* on most VDOT detention facilities. However, exceptions do arise, and the fencing of a dry extended detention facility may be needed. Such situations include:

- Ponded depths greater than 3' and/or excessively steep embankment slopes
- The basin is situated in close proximity to schools or playgrounds, or other areas where children are expected to frequent
- It is recommended by the VDOT Field Inspection Review Team, the VDOT Residency Administrator, or a representative of the City or County who will take over maintenance of the facility

"No Trespassing" signs should be considered for inclusion on all detention facilities, whether fenced or unfenced.

4.3.11 Signage

"No Trespassing" signs should be considered for inclusion on all stormwater impoundment facilities, whether fenced or unfenced. Additionally, retention basins should be identified as potentially exhibiting the following hazards:

- Deep water
- Waterborne disease
- Vortex conditions (if applicable)

Signs should be easily viewed from all streets, sidewalks, and paths adjacent to the basin.

4.3.12 Sediment Forebays

Each basin inflow point should be equipped with a sediment forebay. The forebay volume should range between 0.1" and 0.25" over the individual outfall's new impervious area or 10% of the required WQ_V.

4.3.13 Discharge Flows

All basin outfalls must discharge into an adequate receiving channel per the most current Virginia Erosion and Sediment Control (ESC) laws and regulations. Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

4.4 Design Process

Many of the design elements in a retention basin are identical to those of a dry extended detention basin. These elements include estimation of flood control storage volumes, design of a multi-stage riser, storage indication (reservoir) routing, emergency spillway design, riser buoyancy calculations, and the design of sediment forebays. For those design items, the reader is referred to *Section 2 – Dry Extended Detention Basin*.

This section presents the elements of the design process as it pertains to retention basins serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered during linear development projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 4 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.) for expanded coverage on hydrologic methodology.

The following example basin design is founded on the development scenario described in *Section 3 – Dry Extended Detention Basin Enhanced*. This example project entailed the construction of a small interchange and new section of two lane divided highway in Staunton. The total project site, including right-of-way and all permanent easements, consists of 24.8 acres. Pre and post-development hydrologic characteristics are summarized below in Table 4.3. Initial geotechnical investigations reveal a soil infiltration rate of 0.01 in/hr with site soils classified as Hydrologic Soil Group C.

Table 4.4.1 - Hydrologic Characteristics of Example Project Site

	Pre-Development	Post-Development
Project Area (acres)	24.80	24.80
Land Cover	Unimproved Grass Cover	11.28 acres *new* impervious cover
Impervious Percentage	0	45

Step 1 - Determine Permanent Pool Volume of the Basin as a Function of the Project Site Water Quality Volume

The project site water quality volume is a function of the developed new impervious area. This basic water quality volume is computed as follows:

$$WQV = \frac{NIA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

NIA= New Impervious Area (square feet)

For a retention basin serving a contributing drainage area comprised of 45% impervious cover, the permanent pool volume should be a minimum of four times the computed water quality volume (reference Table 4.1).

The demonstration project site is comprised of a total drainage area of 24.80 acres. The total new impervious area within the project site is 11.28 acres. Therefore, the water quality volume is computed as follows:

$$WQV = \frac{11.28ac \times 43{,}560\frac{ft^2}{ac} \times \frac{1}{2}in}{12\frac{in}{ft}} = 20{,}473.2\,ft^3$$

The basin permanent pool volume is computed as:

$$4 \times 20{,}473.2\,ft^3 = 81{,}893\,ft^3$$

Step 2 - Allocate the Computed Permanent Pool Volume into Regions of Varying Depth

The greatest pollutant removal efficiency of a retention basin is achieved when the surface area of the permanent pool is allocated to the regions of varying depth as shown in Table 4.2; however, initially, the total surface area of the basin permanent pool is unknown. The following steps illustrate the design process for sizing each of the three depth zones.

Approximately 15% of the total surface area of the permanent pool should be dedicated to depths ranging between zero and 18". This depth zone may include or be comprised entirely of the aquatic bench, if one is proposed. Depths ranging between 18" and 24" should comprise an additional 15% of the total basin surface area. The remaining 70% of the basin surface area should be made up of deep water ranging in depth from 2' to 6'.

The total surface area of the basin is designated as *A*. Following this convention, the surface area of each depth zone can be expressed as follows:

$$A_1 = 0.15A$$
$$A_2 = 0.15A$$
$$A_3 = 0.70A$$

The average depth of zone A_1 ranges between 0 and 18". The 9" average depth can be employed as the zone's *effective depth* for purposes of volume calculations. Therefore, the total volume encompassed by the basin's shallowest pool zone is approximated as follows:

$$V_1 = 9in \times \frac{1ft}{12in} \times A_1 = (0.75ft)(0.15)(A)$$

Similarly, the effective depth of zone A_2 is computed as:

$$D_{e_2} = \frac{18in + 24in}{2} = 21in$$

The total volume encompassed by the basin's intermediate depth zone is approximated as follows:

$$V_2 = 21in \times \frac{1ft}{12in} \times A_2 = (1.75ft)(0.15)(A)$$

The deep water regions of the basin range in depth from 2' to 6'. Therefore the effective depth of zone A_3 is 4' and the volume is expressed as:

$$V_3 = 4ft \times A_3 = (4ft)(0.70)(A)$$

The sum of all incremental pool volumes must equal or exceed the previously established permanent pool volume of 4xWQV. Therefore, the basin surface area, *A*, is approximated as follows:

$$V = 81{,}893 ft^3$$
$$V = (0.75ft)(0.15)(A) + (1.75ft)(0.15)(A) + (4ft)(0.70)(A)$$

Rearranging and solving for surface area, *A*:

$$3.175A = 81{,}893 ft^3$$
$$A = 25{,}793 ft^2$$

Table 4.4 summarizes the minimum surface area and approximate volume of each depth zone.

Table 4.4.2 - Summary of Varying Depth Zones

Zone / Depth	Surface Area (ft^2)	Approximate Volume (ft^3)
Shallow (0 - 18")	3,869	2,902
Intermediate (18 - 24")	3,869	6,771
Deep (2 - 6')	18,055	72,220*
Total	***25,793***	***81,893***

*Includes sediment forebay volume(s)

It is noted that the permanent pool surface area of 25,793 ft^2 exceeds 20,000 ft^2. Therefore, the inclusion of an aquatic bench is required for purposes of safety.

Step 3 - Estimate Total Land Area of the Retention Basin

The total proposed surface area of the basin permanent pool is 25,793 ft^2. This represents 2.4% of the total basin drainage area of 24.8 acres. Typically, the total surface area of a retention basin permanent pool will range between 1 and 3% of the total drainage area (FHWA, 1996).

At this point, to determine basin feasibility, the designer must consider the land area required for construction of the basin. Factors to examine include land acquisition costs, availability of right-of-way, and site topography. In addition to the area required for the basin permanent pool, area must be provided for flood control storage, freeboard, and the required 20-foot vegetated buffer strip that must occupy the basin periphery.

Applying the Modified Rational method (presented in detail in *Section 2 – Dry Extended Detention Basin*) we estimate the volume required to provide peak runoff rate reduction for the 10-year return frequency storm:

Peak pre-development runoff, q_{10} = 23.8 cfs

Peak post-development runoff, Q_{10} = 43.2 cfs

Critical duration storm, T_d = 23.5 minutes

Estimated detention volume, V_{10} = 33,978 ft^3

In this example, we will consider a basin of rectangular orientation, with a 2.5:1 length-to-width ratio. The demonstrated methodology is applicable to basins of other geometries. However, the results are only estimates of the total land area required for the basin.

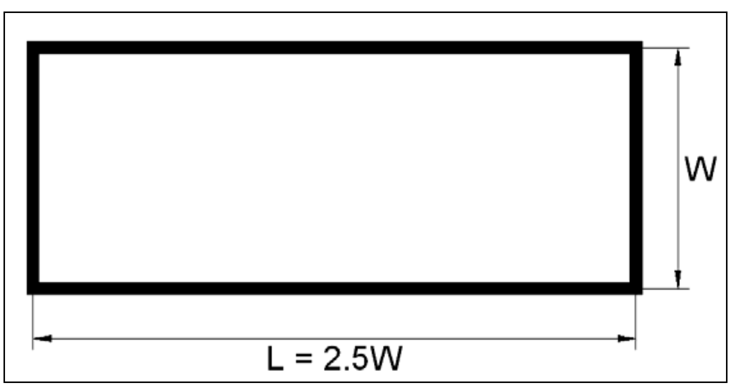


Figure 4.4.1 - Schematic Basin Configuration

The dimensions of the basin permanent pool can then be approximated by solving the following expression:

$$W \times 2.5W = 25{,}793\,ft^2$$
$$W = 101.6\,ft$$
$$L = 254\,ft$$

The volume of flood control storage provided above the permanent pool can be approximated by the following equation:

$$V = \left(\frac{A_1 + A_2}{2}\right)d$$

V = volume of flood control storage (ft^3)
A_1 = surface area of permanent pool (25,793 ft^2)
A_2 = surface area above permanent pool dedicated to flood control storage
d = incremental depth between A_1 and A_2

Surface area, A_2, can be expressed as a function of depth, d:

$$A_2 = [101.6 + (2)(d)(Z)] \times [254 + (2)(d)(Z)]$$

Z = basin side slopes (ZH:1V)

In this example, we will consider that the basin side slopes are 3H:1V. The updated A_2 expression then becomes:

$$A_2 = [101.6 + (2)(d)(3)] \times [254 + (2)(d)(3)]$$

A total flood control volume of 33,978 ft^3 must be provided above the surface of the permanent pool. At this point, the designer can construct a plot of storage versus depth by employing the previously developed expression for volume, *V*. This plot is shown in Figure 4.4.

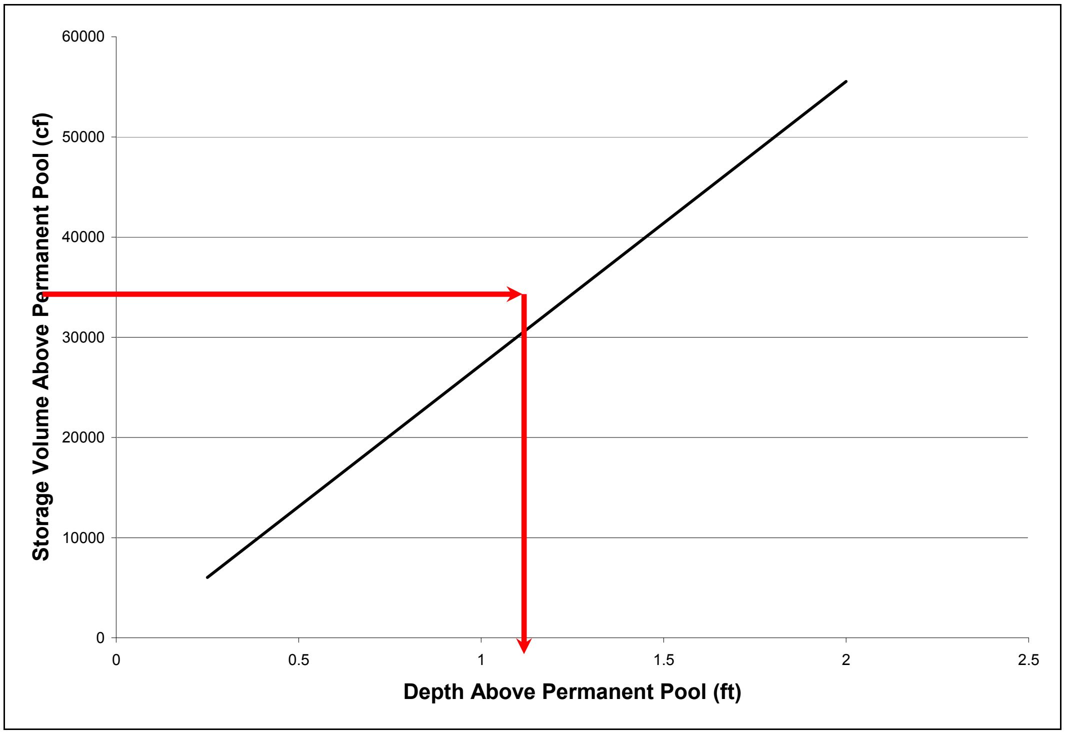


Figure 4.4.2 - Plot of Storage Volume Versus Depth Above Permanent Pool

The plot indicates that the flood control storage is provided at an approximate depth of 1.25' above the permanent pool. This estimate can be verified as follows:

$$A_2 = [101.6 + (2)(1.25)(3)] \times [254 + (2)(1.25)(3)] = 28{,}530\ ft^2$$

The total storage volume provided above the permanent pool is then computed as:

$$V = \left(\frac{25{,}793 + 28{,}530}{2}\right)1.25 = 33{,}952\ ft^3$$

The volume is very close to the required storage volume of 33,978 ft^3, and is deemed adequate for the total basin land area estimate.

Maintaining the 2.5:1 length-to-width ratio, we now compute the surface area of the basin as:

$$W \times 2.5W = 28{,}530\ ft^2$$
$$W = 106.8\ ft$$
$$L = 267\ ft$$

Next, the required freeboard must be considered. The required freeboard depths under 100-year conditions are as follows (per DCR/DEQ minimum standards):

- When equipped with an emergency spillway, the basin must provide a minimum of 1’ of freeboard from the maximum water surface elevation arising from the 100-year event and the lowest point in the embankment (excluding the emergency spillway itself).

- When no emergency spillway is provided, a minimum of 2’ of freeboard should be provided between the maximum water surface elevation produced by the 100-year runoff event and the lowest point in the embankment.

We will assume that the basin is to be equipped with an emergency spillway and that approximately 0.5’ of head is observed on the crest of the emergency spillway during conveyance of the 100-year event. At this point, these values are only estimates. The procedures detailed in *Section Two – Dry Extended Detention Basin* must be employed to determine the actual basin stage – storage relationship.

The freeboard depth (1’) and the head on the emergency spillway (0.5’) increase the basin length and width as follows:

$$W = 106.8\,ft + (2)(3)(1.5\,ft) = 115.8\,ft$$
$$L = 267\,ft + (2)(3)(1.5\,ft) = 276\,ft$$

Finally, we must consider the required minimum 20‘vegetated buffer located around the basin periphery. Adding this buffer width to the basin length and width results in the approximate basin surface dimensions shown in Table 4.5.

Table 4.4.3. Basin Surface Dimensions

Length	156’	
Width	316’	
Area	49,296 ft^2	1.13 ac

Step 4 - Development of Stage – Storage Relationship

Having determined the required surface area and storage volume for the basin permanent pool, flood storage volume, and freeboard we move on to the next step of constructing a stage – storage relationship. Each site is unique, both in terms of constraints and required storage volume. Because of this, the development of a proposed basin grading plan may be an iterative process. The stage storage volume relationship for the example basin is shown in Figures 4.5 and 4.6. The basin floor is assumed to be at elevation 2000’ MSL. Upon development of the basin stage – storage relationships, the next step(s) are to design and evaluate the basin for flood (peak rate) control. The reader is referred to *Section Two – Dry Extended Detention Basin*, Steps 6 – 8 for detailed methodology on these topics.

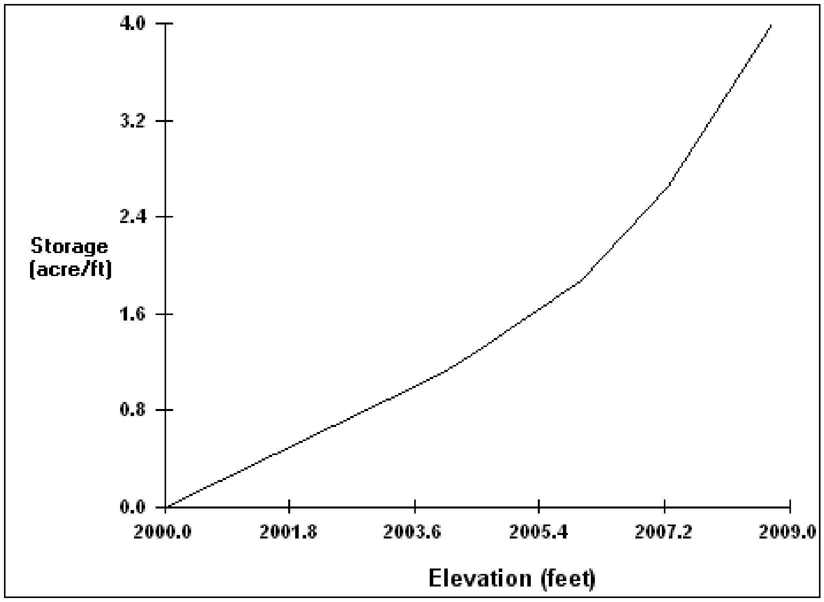


Figure 4.4.3 - Retention Basin Stage – Storage Relationship

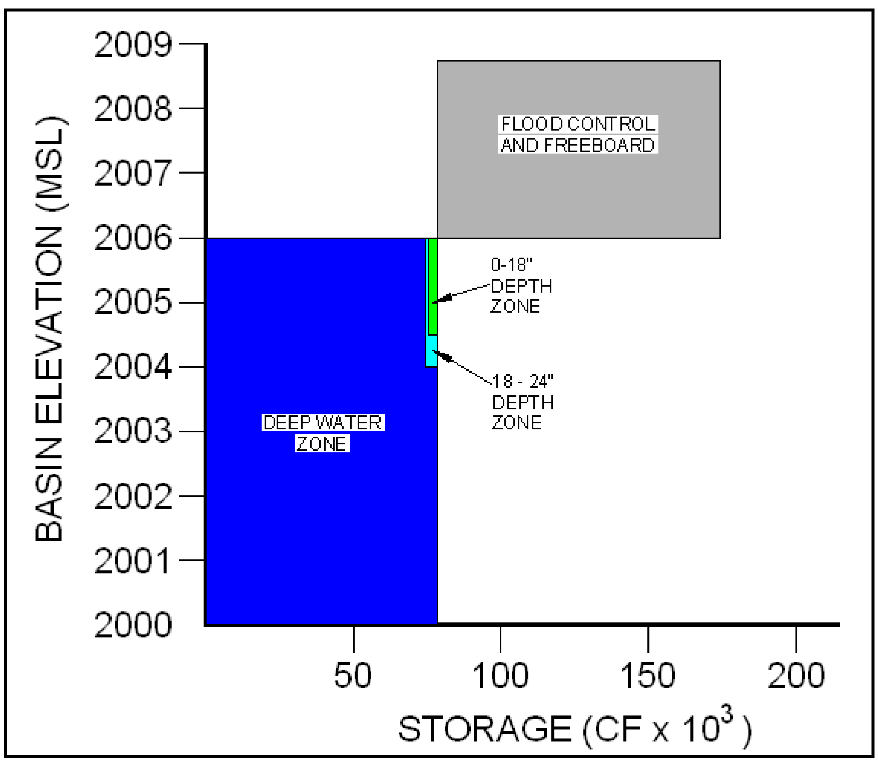


Figure 4.4.4 - Graphical Depiction of Varying Depth Zones – Permanent Pool and Flood Control Storage

Step 5 - Design of the Submerged Release Outlet

A retention basin must be equipped with a means by which baseflow can pass through the basin without accumulating and encroaching upon the volume of storage allocated to flood control. This conveyance is typically accomplished by a submerged, inverted pipe as shown in Figure 4.7.

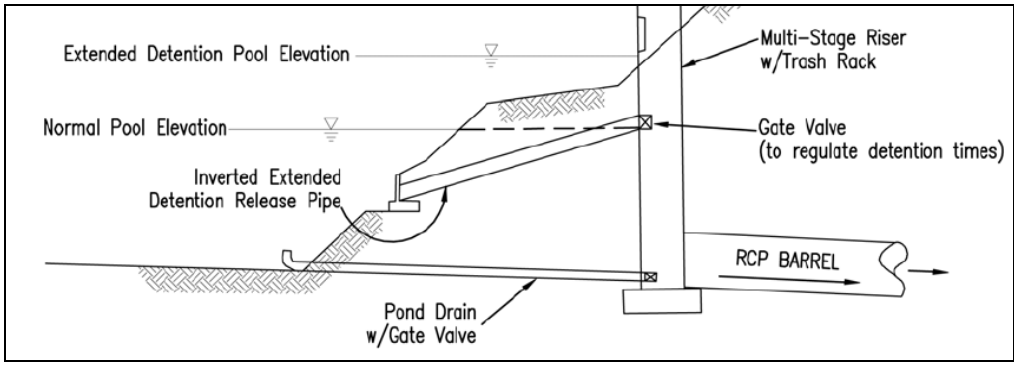


Figure 4.4.5 – DCR/DEQ Schematic Retention Basin Outlet Configuration
Virginia Stormwater Management Handbook, 1999, Et seq.)

Generally, the highest quality of water in a retention basin is found at or near the surface of the permanent pool. In addition to the low levels of dissolved oxygen found near the basin floor, there are also potentially high levels of pollutants which have accumulated through gravitational settling. Though the pollutant levels near the pool surface tend to be lower than at points of greater depth in the water column, the water temperature tends to be higher. This elevated temperature arises from both solar heating and the influence of heated stormwater inflow. The release of heated runoff to downstream receiving channels may be detrimental to fish and other aquatic species inhabiting those channels. Consequently, a release depth of approximately 18” is recommended. (*Virginia Stormwater Management Handbook,* (DCR/DEQ, 1999, Et seq.)

The first step in computing the required outlet size is to establish the maximum anticipated baseflow which must be conveyed through the basin once the permanent pool volume is present. This maximum baseflow arises during the month exhibiting the highest average precipitation. The Virginia State Climatology Office maintains an online database with monthly climate information from various stations across the state. This information can be obtained at:
http://climate.virginia.edu/online_data.htm#monthly

Examining this data for the Staunton station, we see that the month exhibiting the highest average precipitation total is September, with *3.91”*.

This precipitation total must now be converted into a runoff rate. This is accomplished by first employing the NRCS runoff depth equation.

The post-development site is comprised of a total of 24.8 acres, 11.2 acres of which is impervious and 13.6 acres of which is unimproved grass cover. Appendix 6H-3 and 6H-4 of the VDOT Drainage Manual contain runoff curve numbers for various land covers and Hydrologic Soil Groups.

The site Hydrologic Soil Group is *C*. Because the site pervious cover is grass in fair condition, the runoff curve number taken from Appendix 6H-3 is *79*. The curve number for the site impervious fraction is *98*.

Next, the 2-year 24-hour precipitation depth must be obtained in order to estimate the average runoff efficiency. This information can be obtained from the National Weather Service at:

http://hdsc.nws.noaa.gov/hdsc/pfds/orb/va_pfds.html

Examining this data for the Staunton station reveals the 2-year 24-hour precipitation depth, *P*, to be *2.86"*.

Next, the NRCS runoff depth equations are employed to determine the 2-year 24-hour runoff depth for the post-developed site:

Pervious Fraction

$$S = \frac{1000}{CN} - 10 = \frac{1000}{79} - 10 = 2.66$$

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} = \frac{(2.86 - (0.2)(2.66))^2}{(2.86 + (0.8)(2.66))} = 1.09 inches$$

Impervious Fraction

$$S = \frac{1000}{CN} - 10 = \frac{1000}{98} - 10 = 0.20$$

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} = \frac{(2.86 - (0.2)(0.20))^2}{(2.86 + (0.8)(0.20))} = 2.63 inches$$

The total depth of runoff over the entire developed site is then computed as:

$$\frac{(1.09 inches)(13.6 acres) + (2.63 inches)(11.2 acres)}{24.8 acres} = 1.79 inches$$

The Efficiency of Runoff, *E*, is computed as the ratio of runoff depth to the total depth of precipitation for the 2-year event:

$$E = \frac{1.79 in}{2.86 in} = 0.63$$

Employing this efficiency ratio, we can estimate the average runoff volume for the month of September as:

$$3.91 inches \times 0.63 \times \frac{1 ft}{12 in} \times 24.8 ac \times \frac{43{,}560 ft^2}{ac} = 221{,}756 ft^3$$

The average baseflow rate is then computed as:

$$\frac{221{,}756 ft^3}{30 days} \times \frac{1 day}{24 hour} \times \frac{1 hour}{3{,}600 \sec} = 0.09 cfs$$

The elevation at which the baseflow bypass outlet begins to discharge from the basin must be set equal to the basin elevation corresponding to the permanent pool volume. This ensures that the permanent pool volume is maintained in the basin at all times, while perennial baseflow is passed through the principal spillway and does not accumulate in the basin. Referencing Figures 4.5 and 4.6, we see that the permanent pool volume occurs at basin elevation 2006'. The crest of the baseflow bypass outlet is therefore set at 2006' and sized as follows:

We will initially try a 3" diameter orifice, and restrict the maximum head to that occurring just as the outlet becomes submerged. Employing the orifice equation:

$$Q = Ca\sqrt{2gh}$$

Q= discharge (cfs)
C= orifice coefficient (0.6)
a= orifice area (ft^2)
g= gravitational acceleration (32.2 ft/sec^2)
h= head (ft)

$$a = \pi r^2 = \pi \times \left(\frac{3in/2}{\frac{12in}{ft}} \right)^2 = 0.049 ft^2$$

The head is measured from the centerline of the orifice. The head when the orifice has just become submerged by a small increment, 0.01', is expressed as:

$$h = 1.5 inches \times \frac{1 ft}{12 in} + 0.01 ft = 0.135 ft$$

Discharge is now computed as:

$$Q = (0.6)(0.049)\sqrt{(2)(32.2)(0.135)} = 0.09 cfs$$

The selected 3" diameter orifice appears ideally suited for conveying the basin perennial baseflow.

Step 6 - Embankment Design

When a stormwater impounding facility exceeds 15' in height or, as is the case with a retention basin, holds a permanent pool of water, the earthen embankment must be comprised of homogenous material with seepage controls or zoned embankments. The following steps provide guidance in designing a zoned embankment.

The steps presented in this example *do not* apply to embankments whose height exceed 25' and exhibit a maximum storage capacity of 50 acre-ft or more. Such an embankment may be regulated under the Virginia Dam Safety Act, Article 2, Chapter 6, Title 10.1 (10.1-604 et seq.) of the Code of Virginia and Dam Safety Regulations established by the Virginia Soil and Water Conservation Board (VS&WCB). As previously stated, a retention basin embankment may be excluded from regulation if it meets any of the following criteria:

- is less than 6' in height
- has a capacity of less than 50 acre-ft and is less than 25' in height
- has a capacity of less than 15 acre-ft and is more than 25' in height
- will be owned or licensed by the Federal Government

The design and construction of an earthen embankment is a complex process, and is inherently site-specific. Such a design must consider all unique site constraints, the characteristics of both native and imported construction materials, and the downstream hazard potential should the embankment fail. It is the engineer's responsibility to evaluate all of these considerations, including the potential for significant property damage and/or loss of life in the event of embankment failure. The guidance presented in this example does not constitute a standard or specification, and is not intended to replace the need for a thorough site investigation whenever a stormwater impounding facility is proposed.

The *Virginia Stormwater Management Handbook*, (DCR/DEQ, 1999, Et seq.) defines a zoned embankment as containing a central impervious core, flanked by zones of more pervious material called shells. The pervious shells serve the function of enclosing, supporting, and protecting the impervious core. Often, the pervious shells are comprised of native site materials while the impervious core, comprised of material with very low permeability, is imported.

The first element in the design of an earthen embankment is that of a cutoff trench. The cutoff trench should be situated along the centerline of the embankment, or slightly upstream of the centerline. Along the width of the embankment, the trench should extend up the embankment abutments to a point coinciding with the 10-year water surface elevation.

When a zoned embankment is proposed, the cutoff trench material should be identical to that of the embankment core. The trench bottom width and depth should be no less than 4', and the trench slopes should be no steeper than 1H:1V. (*Virginia Stormwater Management Handbook* (DCR/DEQ, 1999, Et seq.). Figure 4.8 illustrates the *minimum* cutoff trench size configuration.

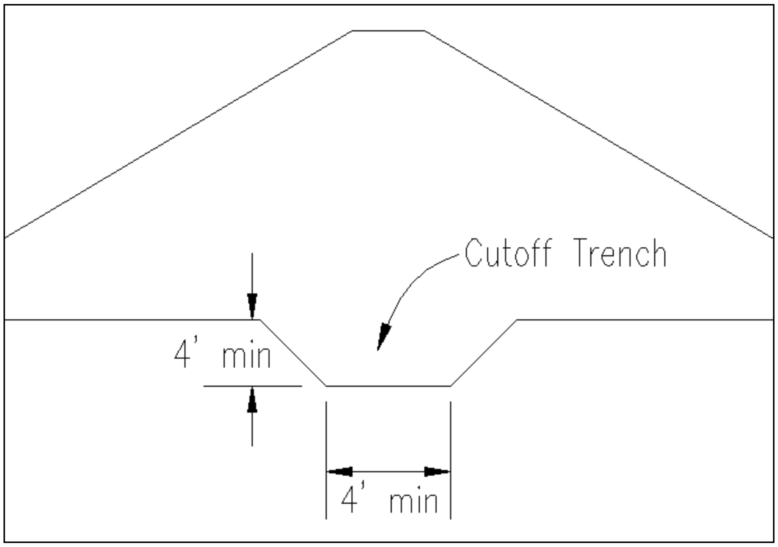


Figure 4.4.6 - Typical Cutoff Trench Configuration

It must be noted that the dimensions shown in Figure 4.8 are absolute minimum values. Typically, as the ponded depth (and resulting hydraulic head) in a basin increase the bottom width of the trench should also increase. This increase in trench width may be reduced if the depth of the trench is also increased. The U.S. Bureau of Reclamation publication *Design of Small Dams* (revised 1977) gives the following relationship between head in the basin, trench width, and trench depth:

$$w = h - d$$

w = bottom width of cutoff trench
h = reservoir head above ground surface
d = depth of cutoff trench excavation below ground surface

The example basin permanent pool occurs at a basin depth of 6' (reference Figure 4.6). Fixing the cutoff trench depth as 4' and employing the trench width equation:

$$w = 6\,ft - 4\,ft = 2\,ft < \text{Minimum } 4\text{ ft}$$

Retention basins whose primary function is water quality improvement and flood control should typically exhibit permanent pool depths of less than 8'. Consequently, the minimum cutoff trench width and depth dimensions of 4' are generally adequate. However, when a proposed basin pool depth increases beyond the typical range, consideration should be given to increasing the dimensions of the embankment cutoff trench.

The next consideration is sizing the zones of the embankment. When a cutoff trench is provided, as required for a retention basin, sizing of the embankment zones should adhere to the guidelines illustrated in Figure 4.9.

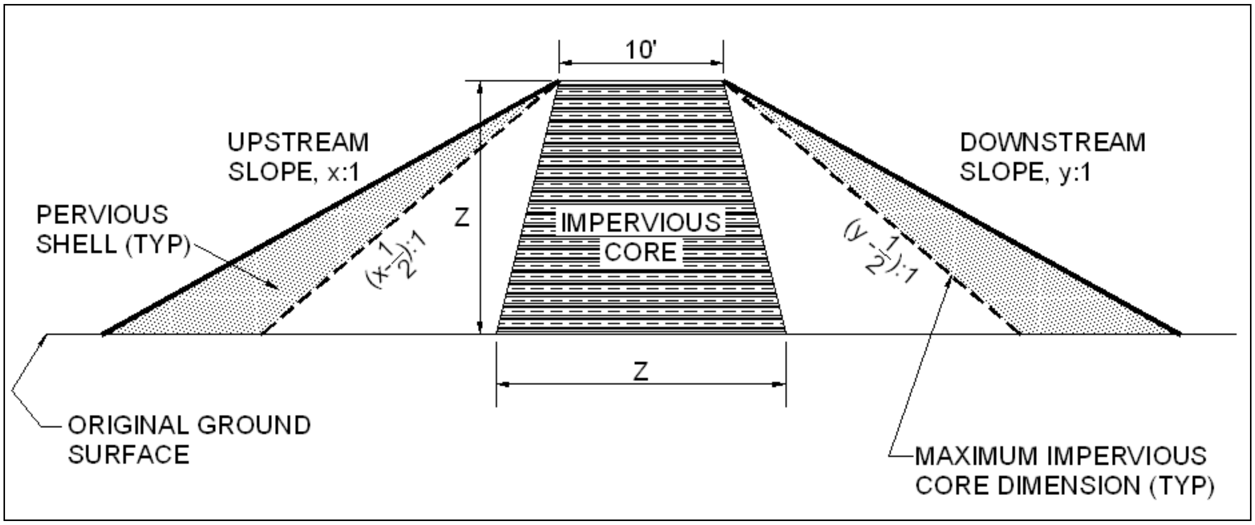


Figure 4.4.7 - Minimum and Maximum Size of Embankment Core
(U.S. Bureau of Reclamation, 1977)

As illustrated in Figure 4.9, the bottom width of the impervious core should, at a minimum, equal the total embankment height. This ensures that the core width at any basin elevation exceeds the height of embankment remaining above that elevation. Consequently, for all basin elevations, the hydraulic gradient through the core is less than unity and seepage potential is reduced. The maximum size of the impervious core is a function of the embankment's upstream and downstream external slopes. Should the impervious core be sized larger than these guidelines, the stabilization function of the pervious shell would be largely ineffective and, from a stabilization standpoint, the embankment would behave similar to a homogeneous type. (U.S. Bureau of Reclamation, 1977)

In the example problem, the proposed basin height is 9' (reference Figures 4.5 and 4.6), which is less than the embankment top width of 10'. Constructing the core bottom width equal to the embankment height would result in a negative slope for the sides of the impervious core. Such a configuration is impractical from a construction standpoint. The maximum side slope of the impervious core is a function of the embankment's external slopes, previously established as 3:1. Generally, the construction of the impervious core will require material to be imported to the site. It is both costly and unnecessary to size the core to its maximum dimensions (unless native site soils meet the classification for core material). In the example basin, we will consider impervious core side slopes of 1:1. This configuration is illustrated in Figure 4.10.

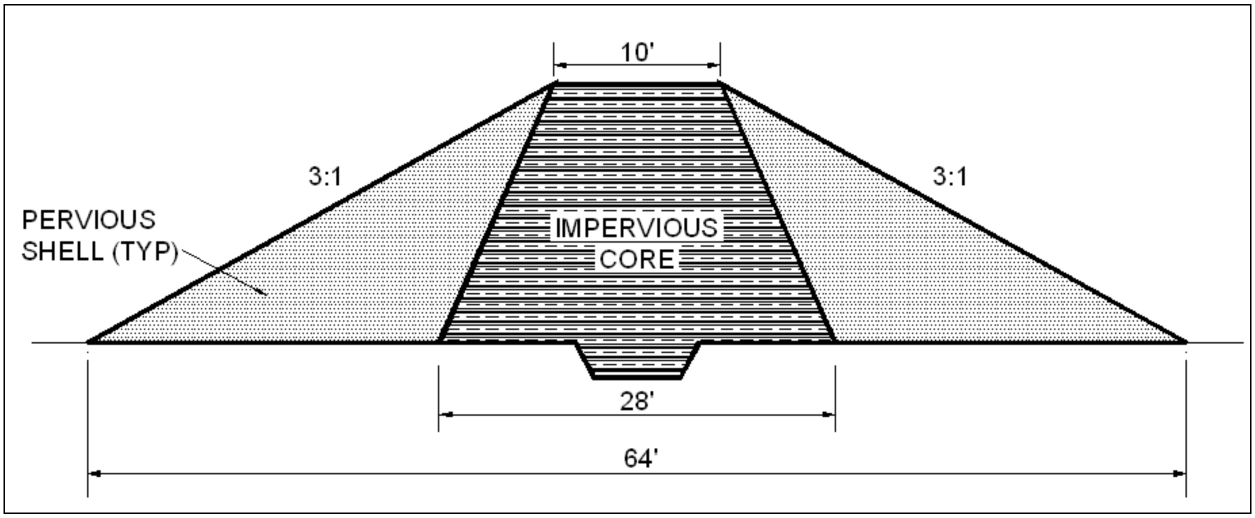


Figure 4.4.8 - Example Basin Embankment Dimensions

Selection of core and pervious flanking material should conform to the Unified Soil Classifications shown in Table 4.6.

Table 4.4.4 - Suitable Embankment Material
(U.S. Bureau of Reclamation, 1977)

Zone	Core Material Classification
Impervious Core	GC, SC, CL*
Pervious Shell	Rockfill, GW, GP, SW, SP

*Some materials approved by the U.S. Bureau of Reclamation have been omitted, and those shown are only those approved by the Virginia Department of Environmental Quality

When the classification of adjacent zone materials differs significantly, such as a clay impervious core adjoining a rockfill pervious shell, a transition zone is strongly recommended. The transition zone helps to prevent the fines of the core material from piping into the voids of the more pervious material. Additionally, on the embankment's upstream face, should voids or cracks appear in the core, the transition material can often effectively "plug" the voids, thus minimizing seepage. To facilitate ease of construction, the U.S. Bureau of Reclamation recommends that transition zones range between 8' and 12' in width; however, the effectiveness of a transition zone only a few feet wide can be significant. Transition zones are not required between impervious material and sand-gravel zones or between sand-gravel zones and rockfill.

Step 7 - Water Balance Calculation

To ensure that the basin's permanent pool does not become dry during extended periods of low or absent inflow, the designer must perform a water balance calculation. Note that this water balance evaluation differs from the baseflow calculation made previously. Two approaches are described in the following section.

Step 7A - 45-Day Drought Condition

The first approach considers the extreme condition of a 45-day drought period with no precipitation and thus no significant surface runoff.

Table 4.7 presents potential evaporation rates for various locations in Virginia.

Table 4.4.5 - Potential Evaporation Rates (Inches)
Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.)

Station	April	May	June	July	August	Sept.
Charlottesville	2.24	3.84	5.16	6.04	5.45	3.87
Danville	2.35	3.96	5.31	6.23	5.69	3.91
Farmville	2.34	3.81	5.13	6.00	5.41	3.71
Fredericksburg	2.11	3.80	5.23	6.11	5.46	3.83
Hot Springs	1.94	3.41	4.50	5.14	4.69	3.33
Lynchburg	2.21	3.72	4.99	5.85	5.31	3.70
Norfolk	2.20	3.80	5.37	6.34	5.79	4.14
Page County	1.68	3.06	4.09	4.71	4.26	3.05
Pennington Gap	2.14	3.59	4.72	5.45	4.97	3.60
Richmond	2.28	3.89	5.31	6.23	5.64	3.92
Roanoke	2.20	3.75	4.99	5.85	5.30	3.67
Staunton	2.00	3.52	4.77	5.52	4.95	3.47
Wash. National Airport	2.13	3.87	5.50	6.51	5.84	4.06
Williamsburg	2.27	3.86	5.23	6.14	5.61	3.97
Winchester	2.07	3.68	4.99	5.82	5.26	3.67
Wytheville	2.01	3.43	4.46	5.17	4.71	3.39

The greatest potential evaporation for Staunton occurs during the months of July and August, 5.52" and 4.95" respectively. Therefore, the total evaporation over a 45-day period is estimated as follows:

Average evaporation per month = $\dfrac{5.52in + 4.95in}{2} = 5.24in$

Average evaporation per day = $\dfrac{5.24\dfrac{in}{month}}{31\dfrac{day}{month}} = 0.17\dfrac{in}{day}$

The evaporation loss over a 45-day period is calculated as follows:

$$45\,\text{days}\ X\ 0.17\frac{in}{day} = 7.65in = 0.64\,ft$$

The total surface area of the permanent pool is 25,793 ft^2. Therefore, the total volume of water lost to evaporation is estimated as:

$$25{,}793\,ft^2 \times 0.64\,ft = 16{,}508\,ft^3$$

The volume of water lost to evaporation must be added to that lost to infiltration. As previously stated, the initial geotechnical tests revealed site soil infiltration rates to be 0.01 in/hr. The infiltration is assumed to occur over the entire permanent pool, whose surface area is 25,793 ft^2. The volume of water lost to infiltration is estimated as:

$$25{,}793\,ft^2 \times 0.01\frac{in}{hr} \times \frac{1\,ft}{12in} \times 24\frac{hr}{day} \times 45days = 23{,}214\,ft^3$$

The total volume of water lost to evaporation and infiltration over the 45-day drought period is therefore computed as:

$$16{,}508\,ft^3 + 23{,}214\,ft^3 = 39{,}722\,ft^3$$

The total volume of the basin permanent pool is 1.88 acre-ft (81,893 ft^3). The estimated evaporation and infiltration loss over a 45-day drought period is slightly less than half of the total permanent pool volume. While the extended drought period does impact the basin pool significantly, a volume of more than twice the project site water quality volume does remain in the basin, and is thus considered adequate against drought.

The volume of runoff necessary to replenish the pool volume is computed as follows:

Total contributing drainage area = 24.8 acres

Stored volume lost to evaporation and infiltration = 39,722 ft^3

$$\frac{39{,}722\,ft^3}{24.8ac \times \frac{43{,}560\,ft^2}{ac}} = .0368 \text{ watershed - feet} = 0.44 \text{ watershed - inches}$$

A precipitation event yielding a total runoff of 0.44" or more across the contributing watershed will replenish the depleted marsh volume.

Step 7B - Period of Greatest Evaporation (in Average Year)

The second water balance calculation examines impacts on the basin permanent pool during the one-month period of greatest evaporation. This calculation reflects an *anticipated* pool drawdown during the summer months of an average year. In contrast, the first calculation method reflects an extreme *infrequent* drought event.

From Table 4.7, the greatest monthly evaporation total for the project site is 5.52" in July. The Virginia State Climatology Office reports an average July rainfall for the Staunton station as 3.78" (reference Step 5 for link to data).

Applying the previously computed runoff efficiency ratio for the basin watershed, the average July inflow to the basin is computed as:

$$3.78inches \times 0.63 \times \frac{1ft}{12in} \times 24.8ac \times \frac{43{,}560\,ft^2}{ac} = 214{,}383\,ft^3$$

Evaporation losses are computed as the product of total monthly evaporation and the surface area of the permanent pool:

$$5.52inches \times \frac{1ft}{12in} \times 25{,}793\,ft^2 = 11{,}865\,ft^3$$

Infiltration losses over the entire month of July are estimated as:

$$25{,}793\,ft^2 \times 0.01\frac{in}{hr} \times \frac{1ft}{12in} \times 24\frac{hr}{day} \times 31days = 15{,}992\,ft^3$$

The water balance expression and total monthly loss/gains are computed as follows:

Monthly loss/gain = Inflow – Evaporation – Infiltration

$$= 214{,}383\,ft^3 - 11{,}865\,ft^3 - 15{,}992\,ft^3 = 186{,}526\,ft^3$$

The monthly climate data and site land cover characteristics indicate that the basin will not experience drawdown during the average period of highest evaporation.

Step 8 - Landscaping

Generally, the non-inundated (dry storage) regions of a retention basin can be landscaped in the same manner as a dry basin (reference *Section Two – Dry Extended Detention Basin)*; however, careful attention must be given to the types of vegetation selected for the basin pool and aquatic bench areas. For these regions, the vegetative species must be selected based on their inundation tolerance and the anticipated frequency and depth of inundation.

The regions of varying depth within the basin are broadly categorized by zone as shown in Figure 4.11. Note the basin aquatic bench would be encompassed by Zone 2.

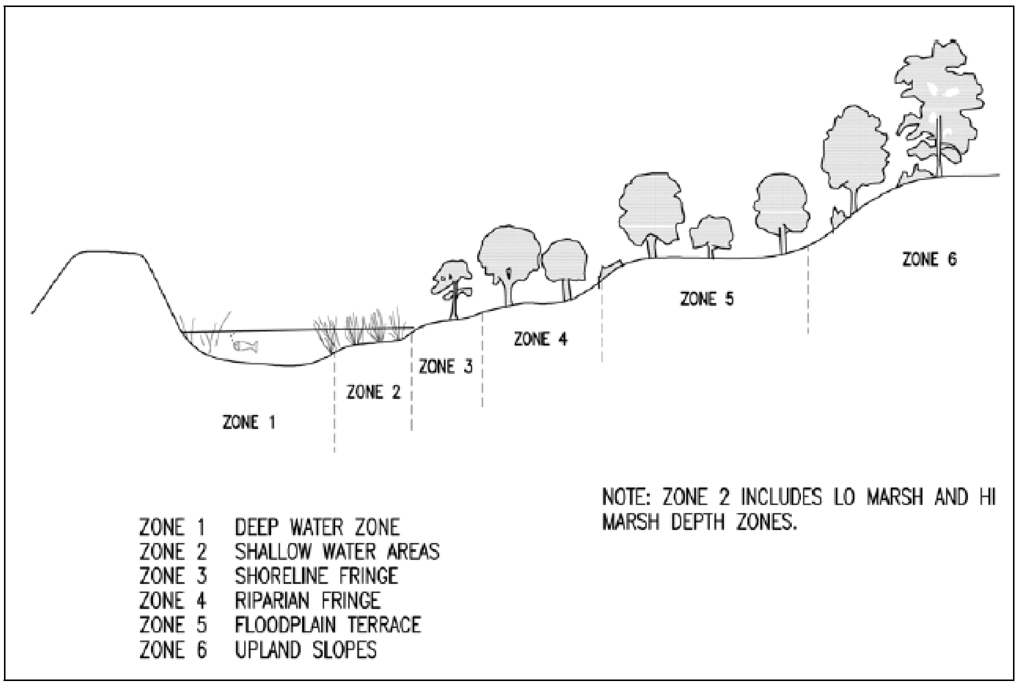


Figure 4.4.9 - Planting Zones for Stormwater BMPs
Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.)

Suitable planting species for each of the zones identified in Figure 4.11 are recommended in Chapter 3-05 of the *Virginia Stormwater Management Handbook*, (DCR/DEQ, 1999, Et seq.). Ultimately, the choice of planting species should be largely based on the project site's physiographic zone classification. Additionally, the selection of plant species should match the native plant species as closely as possible. Surveying a project site's native vegetation will reveal which plants have adapted to the prevailing hydrology, climate, soil, and other geographically-determined factors. Figure 3.05-4 of the *Virginia Stormwater Management Handbook* provides guidance in plant selection based on project location.

Generally, stormwater management basins should be permanently seeded within 7 days of attaining final grade. This seeding should comply with Minimum Standard 3.32, Permanent Seeding, of the Virginia Erosion and Sediment Control Handbook, (DCR/DEQ, 1992, Et seq.). It must be noted that permanent seeding is *prohibited* in Zones one through four of Figure 4.11. The use of conventional permanent seeding in these zones will result in the grasses competing with the requisite wetland emergent species.

When erosion of basin soil prior to the establishment of mature stand of wetland vegetation is a concern, temporary seeding (Minimum Standard 3.31) of the Virginia Erosion and Sediment Control Handbook, (DCR/DEQ, 1992, Et seq.) may be considered. However, the application rates specified should be reduced to as low as practically possible to minimize the threat of the temporary seeding species competing with the chosen emergent wetland species.

All chosen plant species should conform to the American Standard for Nursery Stock, current issue, and be suited for USDA Plant Hardiness Zones 6 or 7, see Figure 4.12.

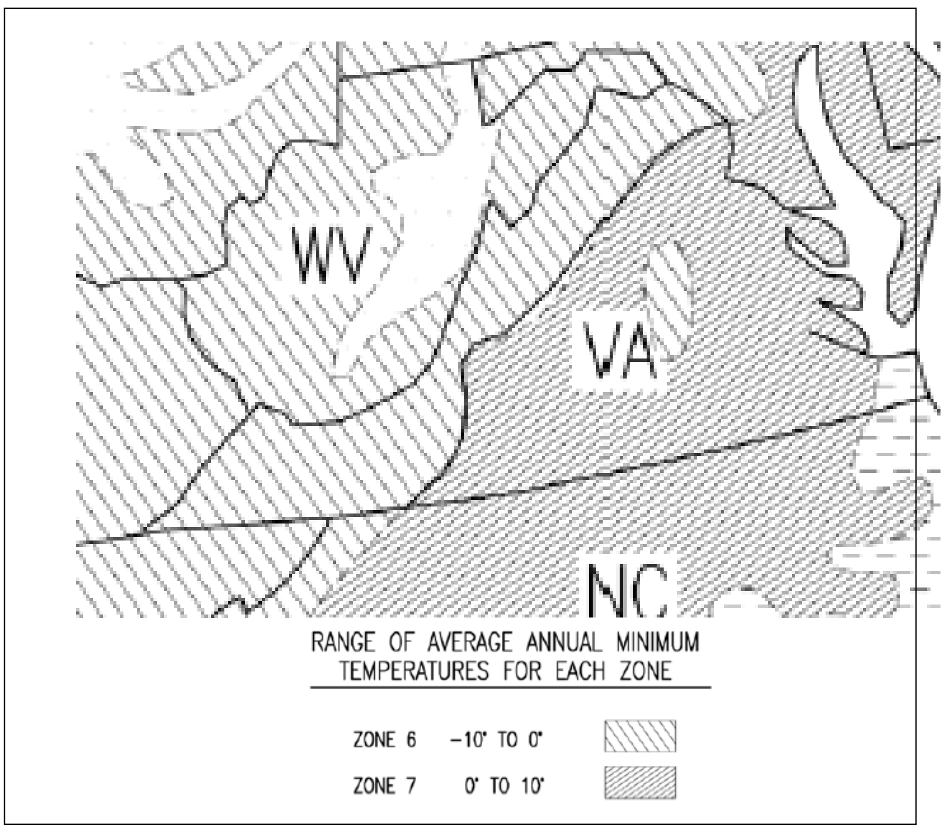


Figure 4.4.10 - USDA Plant Hardiness Zones

Under no circumstances should trees or shrubs be planted on the basin embankment. The large root structure may compromise the structural integrity of the embankment.

5.1 Overview of Practice

Constructed stormwater wetlands fall into a structural BMP category having the capacity to improve the quality of stormwater runoff in much the same manner as retention and enhanced extended detention basins. Like these impounding facilities, stormwater wetlands are seeded with a diverse mix of aquatic and emergent vegetation, which plays an integral role in the pollutant removal efficiency of the practice. Wetland BMPs improve the quality of runoff by physical, chemical, and biological means. The physical treatment of runoff occurs as a result of decreased flow velocities in the wetland, thus leading to evaporation, sedimentation, adsorption, and/or filtration. Chemical treatment arises in the form of chelation (bonding of heavy metal ions), precipitation, and chemical adsorption. The biological treatment processes occurring in wetlands include decomposition, plant uptake and removal of nutrients, and biological transformation and degradation. (FHWA, 1996)

Constructed stormwater wetlands should not be confused with naturally occurring wetlands. When proper pre-treatment measures are implemented, naturally occurring wetlands are *sometimes* capable of receiving runoff from development projects; however, constructed wetlands serve the primary function of receiving stormwater runoff, and generally exhibit less biodiversity than naturally occurring wetlands both in terms of plant and animal life (Yu, 2004). Similarly, constructed wetlands differ from *created wetlands*, which are intended to replace and mimic naturally occurring wetlands for mitigation purposes.

Constructed stormwater wetlands should, generally, *not be used for flood control or downstream channel control*. When a BMP is employed as a quantity control practice, there is an inherent expectation of rapidly fluctuating water levels in the practice following runoff producing events. Rapid fluctuations in water level subject emergent wetland and upland vegetation to enormous stress, and many wetland species cannot survive such conditions. In addition to producing large surges of stormwater runoff, land use conversion resulting in a loss of pervious cover will often result in a decrease of perennial baseflow from a watershed. The decrease or absence of such baseflow is problematic for the establishment of a diverse and healthy mix of wetland vegetation.

Figures 5.1, 5.2, and 5.3 present various schematic views of constructed stormwater wetlands.

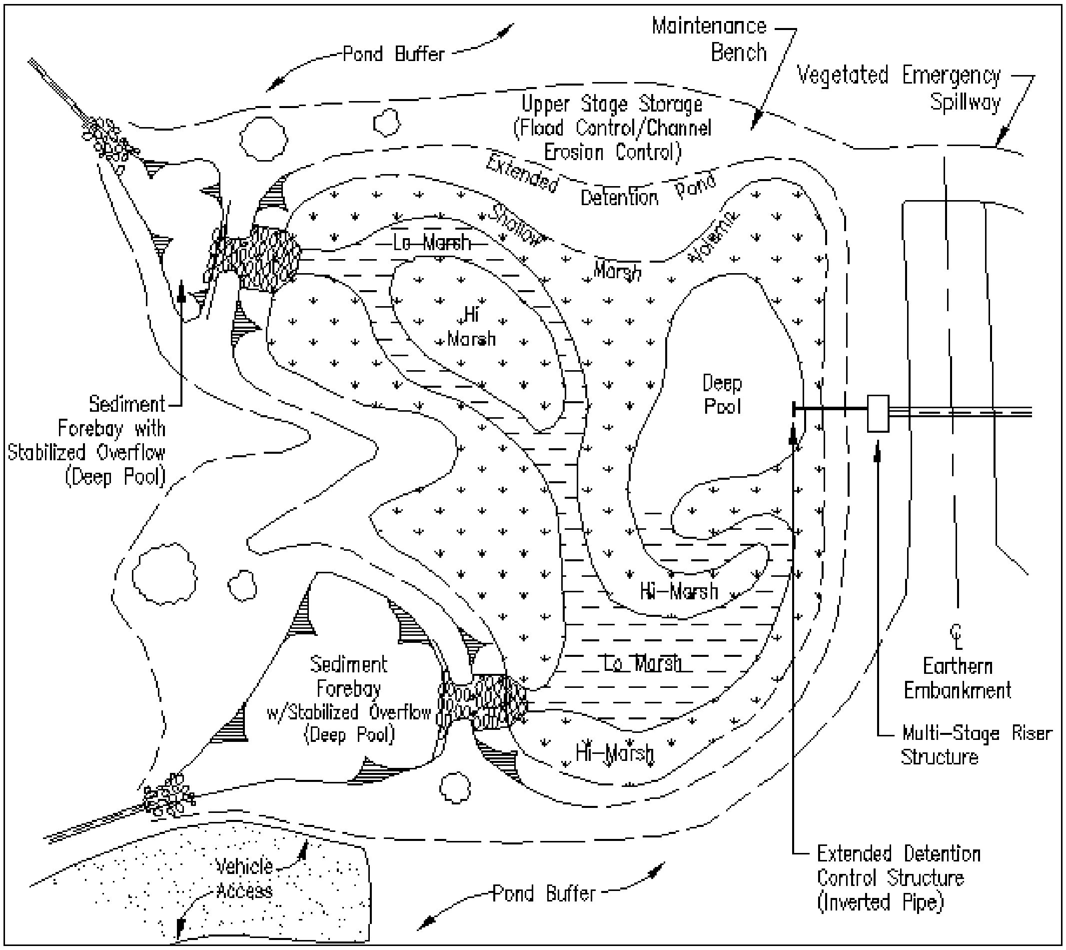


Figure 5.1.1 - Constructed Stormwater Wetlands (Plan View)
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

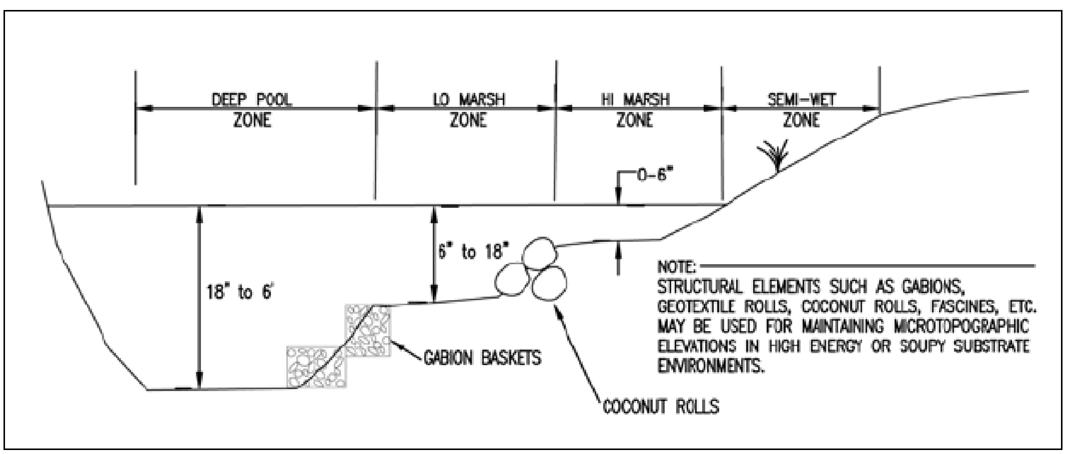


Figure 5.1.2 – DCR/DEQ Varying Wetland Depth Zones (Profile)
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

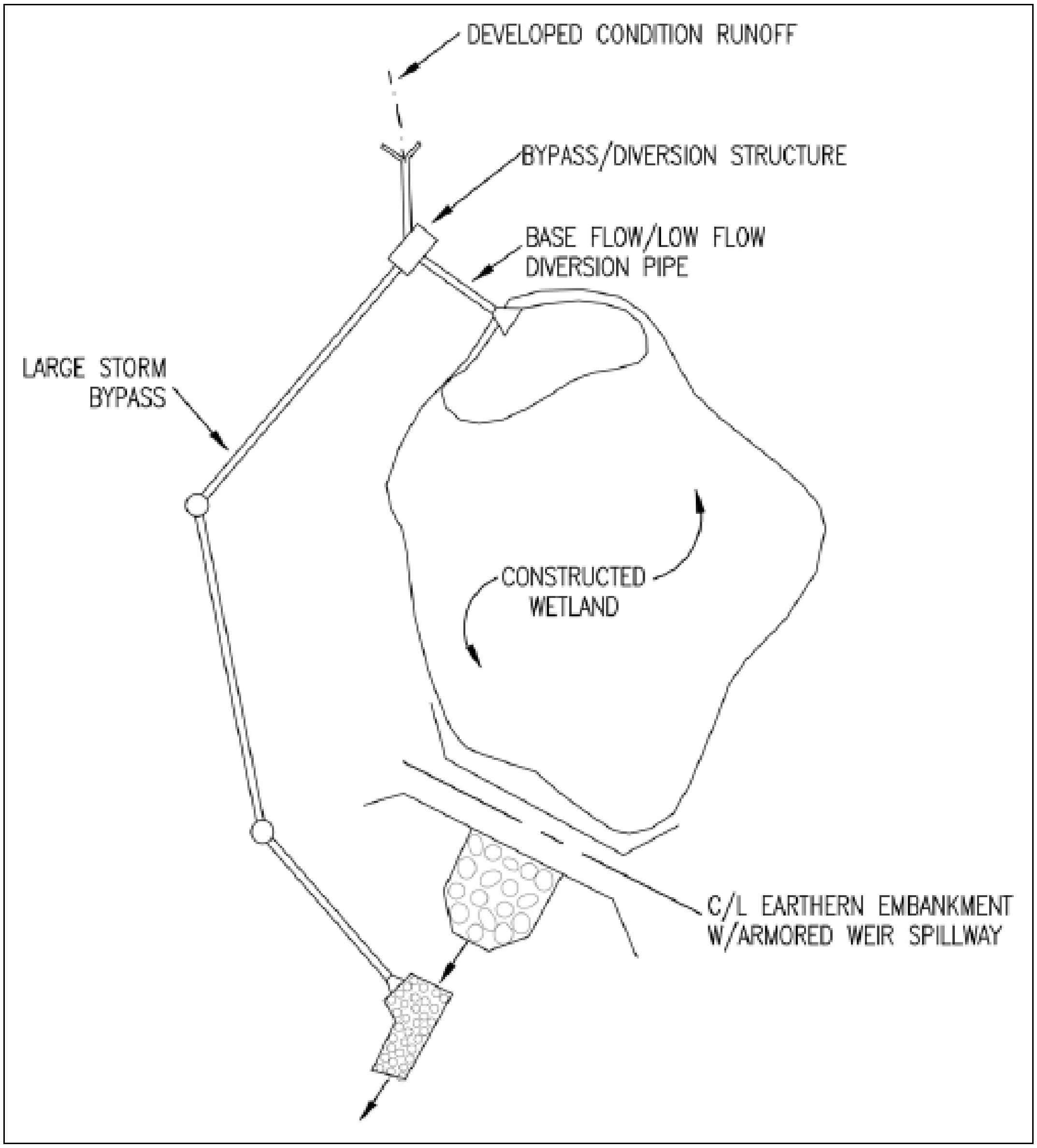


Figure 5.1.3 – DCR/DEQ Offline Wetland Configuration
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

As evidenced in Figure 5.1, the wetland is comprised of three distinct zones – "low marsh," "high marsh," and "deep pool." These varying-depth zones introduce *microtopography* to the basin floor. Detailed surface area and depth requirements of the various marsh zones are discussed later in this section.

5.2 Site Constraints and Siting of the Facility

The engineer must consider a number of site constraints in addition to site impervious area when the implementation of constructed stormwater wetlands is proposed.

5.2.1 Minimum Drainage Area

Constructed stormwater wetlands should generally not be considered when contributing drainage area is less than 10 acres. Of critical concern is the presence of adequate baseflow to the facility. Many species of wetland vegetation cannot survive extreme drought conditions. Additionally, insufficient baseflow and the subsequent stagnation of wetland marsh areas can lead to the emergence of undesirable odors from the wetland. Regardless of drainage area, all proposed wetlands should be subjected to a low flow analysis to ensure that an adequate marsh volume is retained even during periods of dry weather when evaporation and/or infiltration are occurring at a high rate. The anticipated baseflow from a fixed drainage area can exhibit great variability, and insufficient baseflow may require consideration of alternate BMP measures. When infiltration losses from the wetland are excessive, a clay liner or geosynthetic membrane may be considered. Such a liner should meet the approval and specifications of the Materials Division.

The presence of a shallow groundwater table, as common in the Tidewater region of the state, may allow for the implementation of a constructed wetland whose contributing drainage area is very small. These circumstances are site-specific, and the groundwater elevation must be monitored closely to establish the design elevation of the permanent pool.

5.2.2 Maximum Drainage Area

The maximum drainage area to a constructed stormwater wetland is not explicitly restricted. However, the designer must consider that, due to the needs of aquatic plant species, storage volume in the form of excessive pool depth (vertical storage) is typically not possible. Therefore, the land area required for constructed wetland may be two to three times the site area required of alternative BMPs. (MWCOG, 1992) The *minimum* surface area of the wetland marsh area is 2% of the contributing drainage area.

5.2.3 Separation Distances

Constructed stormwater wetlands should be located a minimum of 20’ from any permanent structure or property line, and a minimum of 100’ from any septic tank or drainfield.

5.2.4 Site Slopes

Stormwater wetlands should, generally, not be constructed within 50' of any slope steeper than 10%. When this is unavoidable, or when the facility is located at the toe of a slope greater than 10%, a geotechnical report should be performed to address the potential impact of the facility in the vicinity of such a slope.

5.2.5 Site Soils

The implementation of constructed stormwater wetlands can be successfully accomplished in the presence of a variety of soil types. However, when such a facility is proposed, *a subsurface analysis and permeability test is required*. The required subsurface analysis should investigate soil characteristics to a depth of no less than 3' below the proposed bottom of the wetland. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils. To ensure the long-term success of a constructed wetland, it is essential that water inflows (baseflow, surface runoff, and groundwater) be greater than losses to evaporation and infiltration. This requires the designer to calculate a monthly water budget. Due to excessive infiltration losses, soils exhibiting high infiltration rates are not suited for the construction of stormwater wetlands. Often, soils of moderate permeability (on the order of $1x10^{-6}$ cm/sec) are capable of supporting the shallow marsh areas of a stormwater wetland. However, the hydraulic head (pressure) generated from deeper regions, such as the wetland micro-pool, may increase the effective infiltration rate rendering similar soils unsuitable for wetland construction. Mechanical compaction of existing subsoils, a clay liner, geosynthetic membrane, or other material (as approved by the Materials Division) may be employed to combat excessively high infiltration rates. The wetland embankment material must meet the specifications detailed later in this section and/or be approved by the Materials Division.

5.2.6 Rock

The presence of rock within the proposed construction envelope of a stormwater wetland should be examined during the aforementioned subsurface investigation. When blasting of rock is necessary to obtain the desired storage volume, a liner (of material approved by the Materials Division) should be used to eliminate unwanted losses through seams in the underlying rock.

5.2.7 Existing Utilities

Generally, wetlands should not be constructed over existing utility rights-of-way or easements. When this situation is unavoidable, permission to impound water over these easements must be obtained from the utility owner *prior* to design of the basin. When it is proposed to relocate existing utility lines, the costs associated with their relocation should be included in the overall basin construction cost.

5.2.8 Karst

The presence of Karst topography places even greater importance on the subsurface investigation. Construction of stormwater wetlands in Karst regions may greatly impact the design and cost of the facility, and must be evaluated early in the planning phases of a project. *Construction of stormwater management facilities within a sinkhole is prohibited.* When the construction of such facilities is planned along the periphery of a sinkhole, the facility design must comply with the guidelines found in Chapter 5 of this Manual and DEQ's Technical Bulletin #2 *"Hydrologic Modeling and Design in Karst."*

5.2.9 Existing Wetlands

When the construction of stormwater wetlands is planned in the vicinity of naturally occurring wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify existing wetland boundaries, their protected status, and the feasibility of BMP construction in their vicinity. In Virginia, the Department of Environmental Quality (DEQ) and the U.S. Army Corps of Engineers (USACOE) should be contacted when such a facility is proposed in the vicinity of known wetlands.

5.2.10 Upstream Sediment Considerations

Close examination should be given to the flow velocity at all points discharging concentrated runoff to the wetland. When entering flows exhibit erosive velocities, they have the potential to greatly increase maintenance requirements by depositing large amounts of sediment within the wetland. Regardless of entering flow velocities, a highly disturbed contributing drainage area can hinder the wetland pollutant removal performance through the deposition of excessive sediment. Constructed wetlands are extremely vulnerable to sediment loading, as excessive sediment loading has the potential to greatly alter the microtopography of the marsh floor. The negative impacts associated with excessive sediment loading reinforce the need for sediment forebays as discussed in Section 5.3.

5.2.3 Location

When properly designed, landscaped, and maintained, constructed wetlands may be suitable for high visibility locations. However, when a constructed wetland is proposed in a high visibility location, ongoing maintenance of the facility is critical to its acceptance by neighboring landowners. Additionally, early in the project planning stages, careful attention should be given to the general characteristics of neighboring land uses. The landscape of a constructed wetland exhibits natural and sometimes rapid growth and vegetative colonization. This may be undesirable in the vicinity of an otherwise manicured landscape. The designer must also be aware of the significant land area requirements of a constructed stormwater wetland.

5.2.4 Hydrology

To achieve the pollutant removal efficiencies expressed in Table 1.1, the marsh area of a constructed wetland must support aquatic and emergent plant species. While a quantified volumetric flow rate is not explicitly required, the wetland's contributing watershed should supply enough runoff to ensure that the marsh pools of varying depth are maintained as intended.

5.3 General Design Guidelines

The following presents a collection of issues to be considered when designing a constructed stormwater wetland.

5.3.1 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam.

> *"The foundation material under the dam and the material used for the embankment of the dam should be an AASHTO Type A-4 or finer and/or meet the approval of the Materials Division. If the native material is not adequate, the foundation of the dam is to be excavated and backfilled a minimum of 4' or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."*

If the basin embankment height exceeds 15', or if the basin includes a permanent pool (excluding the shallow marsh area), the design of the dam should employ a homogenous embankment with seepage controls or zoned embankments.

During the initial subsurface investigation, additional borings should be made near the center of the proposed basin when:

- Excavation from the basin will be used to construct the embankment
- The likelihood of encountering rock during excavation is high
- A high or seasonally high water table, generally 2' or less, is suspected

5.3.2 Embankment Geometry

The top width of the embankment should be a minimum of 10' in width to provide ease of construction and maintenance. Positive drainage should be provided along the embankment top.

The embankment slopes should be no steeper than 3H:1V to permit mowing and other maintenance.

5.3.3 Embankment Height

An embankment may be regulated under the Virginia Dam Safety Act, Article 2, Chapter 6, Title 10.1 (10.1-604 et seq.) of the Code of Virginia and Dam Safety Regulations established by the Virginia Soil and Water Conservation Board (VS&WCB). A detention basin embankment may be excluded from regulation if it meets any of the following criteria:

- is less than 6' in height
- has a capacity of less than 50 acre-ft and is less than 25' in height
- has a capacity of less than 15 acre-ft and is more than 25' in height
- will be owned or licensed by the Federal Government

When an embankment is not regulated by the Virginia Dam Regulations, it must still be evaluated for structural integrity when subjected to the 100-year flood event.

5.3.4 Principal Spillway Design

When a riser outlet is employed, it should be designed in accordance with Minimum Standard 3.02 of the Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.). *The primary control structure (riser or weir) should be designed to operate in weir flow conditions for the full range of design flows*. If this is not possible, and orifice flow regimes are anticipated, the outlet must be equipped with an anti-vortex device, consistent with that described in Minimum Standard 3.02.

The primary outlet of a constructed stormwater wetland should be a weir if at all possible. Weirs can be configured to convey large volumetric flow rates with relatively low head. Minimization of ponding depth in a wetland helps to avoid unnecessarily stressing the sensitive vegetative species.

5.3.5 Outfall Piping

The pipe culvert under or through the embankment shall be reinforced concrete equipped with rubber gaskets. Pipe: Specifications Section 232 (AASHTO M170), Gasket: Specification Section 212 (ASTM C443).

A concrete cradle shall be used under the pipe to prevent seepage through the dam. The cradle shall begin at the riser or inlet end of the pipe, and extend the pipe's full length.

5.3.6 Prevention of Short-Circuiting (Wetland Geometry)

Short-circuiting occurs when entering flows pass rapidly through the wetland without achieving effective hydraulic residence times. Short-circuiting of flow negatively impacts the observed water quality benefit of the wetland. While site conditions will ultimately dictate the geometric configuration of a constructed wetland, it is preferable to construct the facility such that the *dry* length-to-width ratio is 2:1 or greater, and the *wet* length-to-width ratio is at least 1:1.

The dry length-to-width ratio is computed by dividing the dry weather flow path length (from entrance point to primary outlet) by the wetland's average width. The wet length-to-width ratio is calculated by dividing the straight line distance (from entrance point to primary outlet) by the wetlands average width.

The dry weather length-to-width ratio is easily increased through the creative use of microtopography, such as situating high marsh berms perpendicular to straight line flow paths. This reduces the likelihood of short-circuiting by creating meandering flow paths rather than straight line paths from stormwater entrance points to the principal spillway.

5.3.7 Volume

The pollutant removal efficiency of a constructed stormwater wetland (expressed in Table 1.1) is based on a permanent pool/marsh volume of twice the computed water quality volume ($2xWQ_V$) from the contributing drainage area.

5.3.8 Surface Area

The surface area of the wetland permanent marsh should, *at a minimum*, be 2% of the area contributing runoff to the wetland. A permanent pool surface area of 3% (or greater) of the wetland's contributing drainage area is optimal.

5.3.9 Ponded Depth

The depth of the wetland marsh affects the planting species selected for the wetland as well as the types of aquatic and wildlife species that will inhabit the wetland and its surrounding areas. Additionally, the depth allocation of the permanent pool has a significant impact on the pollutant removal performance of the wetland. Table 5.1 presents the recommended surface area and volume allocation for the various permanent pool depth zones. The characteristics of each zone are discussed later in the context of a design example.

Table 5.2.1 – DCR/DEQ Recommended Allocation of Surface Area and Treatment Volume for Various Depth Zones
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

Depth Zone	Surface Area (% of Total Surface Area)	Treatment Volume (% of Total Treatment Volume)
Deep Water (1.5 – 6' deep)	10	20
Low Marsh (0.5 – 1.5' deep)	40	*
High Marsh (0 – 0.5' deep)	50	*

* The combined marsh areas should sum to approximately 80% of the total treatment volume. If the surface area criteria conflict with volume allocations, the surface area allocations are considered more critical to an effective design. (DCR/DEQ, 1999, Et seq.)

5.3.10 Maximum Flood Control Ponded Depth

The use of constructed stormwater wetlands for flood control is strongly discouraged. Offline configurations, such as that shown in Figure 5.3, can provide effective water quality improvement while not subjecting the wetland to the extreme water fluctuations typically associated with flood control facilities. When a proposed wetland will be subjected to storm inflows beyond the water quality volume, it is critical to restrict the vertical ponding depth to as shallow as practically possible. Outlet structures must be sized to pass the 10-year return frequency storm with a maximum ponded depth of 2' above the wetland marsh pool. (DCR/DEQ, 1999, Et seq.)

5.3.11 Fencing

Fencing is typically *not required or recommended* on most VDOT detention facilities. However, exceptions do arise, and the fencing of a dry extended detention facility may be needed. Such situations include:

- Ponded depths greater than 3' and/or excessively steep embankment slopes
- The basin is situated in close proximity to schools or playgrounds, or other areas where children are expected to frequent
- It is recommended by the VDOT Field Inspection Review Team, the VDOT Residency Administrator, or a representative of the City or County who will take over maintenance of the facility

"No Trespassing" signs should be considered for inclusion on all detention facilities, whether fenced or unfenced.

5.3.12 Sediment Forebays

Each stormwater inflow point should be equipped with a sediment forebay. Individual forebay volumes should range between 0.1" and 0.25" over the individual outfall's contributing impervious area, with the sum of all forebay volumes not less than 10% of the total WQ_V. When properly constructed, the forebay volumes can be considered a portion of the deep pool zone volume requirement.

5.3.13 Discharge Flows

All concentrated basin outfalls must discharge into an adequate receiving channel per the most current Virginia Erosion and Sediment Control (ESC) laws and regulations. Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

5.4 Design Process

This section presents the steps in the design process as it pertains to constructed stormwater wetlands serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered during linear development projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 4 of the *Virginia Stormwater Management Handbook* (DCR/DEQ, 1999, Et seq.) for expanded hydrologic methodology.

The following design example is founded on the development scenario described in *Section Two – Dry Extended Detention Basin*. The project entails the construction of a section of two lane divided highway situated in Montgomery County. The total project site, including right-of-way and all permanent easements, consists of 17.4 acres. Pre and post-development hydrologic characteristics are summarized below in Tables 5.2 and 5.3. Peak rates of runoff for both pre and post-development conditions were computed by the Rational Method and the regional NOAA NW-14 factors recommended in the VDOT Drainage Manual. Initial geotechnical investigations reveal a soil infiltration rate of 0.02 in/hr.

Table 5.4.1 - Hydrologic Characteristics of Example Project Site

	Pre-Development	Post-Development
Project Area (acres)	17.4	17.4
Land Cover	Unimproved Grass Cover	4.8 acres *new* impervious cover
Rational Runoff Coefficient	0.30	0.50*
Time of Concentration (min)	45	10

*Represents a weighted runoff coefficient reflecting undisturbed site area and new impervious cover

Table 5.4.2 - Peak Rates of Runoff (cfs)

	Pre-Development	Post-Development
2-Year Return Frequency	7.97	15.7
10-Year Return Frequency	11.37	21.0

Step 1 - Compute the Required Water Quality Volume

The project site water quality volume is a function of the developed new impervious area. This basic water quality volume is computed as follows:

$$WQV = \frac{NIA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

NIA= New Impervious Area (square feet)

The demonstration project site has a total drainage area of 17.4 acres. The total new impervious area within the project site is 4.75 acres. Therefore, the water quality volume is computed as follows:

$$WQV = \frac{4.8ac \times 43{,}560 \frac{ft^2}{ac} \times \frac{1}{2} in}{12 \frac{in}{ft}} = 8{,}712\, ft^3$$

The permanent marsh area of the wetlands will be sized to provide twice this volume (17,424 ft^3).

Step 2 - Sizing the Marsh Area Zones

The marsh area of a constructed wetlands is comprised of four distinct zones. The surface area and storage volume allocated to each of the zones is very specific in an effort to provide maximum water quality benefit within the wetlands. The four zones are described as follows.

The *Deep Pool Zone* ranges in depth from 1.5' to 6', and may be comprised of the following three categories:

- sediment forebays
- micro pools
- deep water channels

A sediment forebay must be provided at any point in the wetland that receives concentrated discharge from a pipe, open channel, or other means of stormwater conveyance. The inclusion of a sediment forebay in these locations assists maintenance efforts by isolating the bulk of sediment deposition in well-defined, easily accessible locations. The volume of storage provided at each forebay should range between 0.1" and 0.25" of runoff over the individual inlet's new contributing impervious area, with the sum of all forebay volumes not less than 10% of the total water quality volume.

Micro-pools provide open water areas which promote plant and wildlife diversity. When the wetland is equipped with a riser structure, a micro-pool should be provided near the riser. When a baseflow conveyance pipe is provided, it should be constructed on a negative slope that extends to an approximate depth of 18" below the normal surface of the micro-pool.

Deep water channels may be employed to lengthen the flow path from pond inflow points to the principal spillway.

The sum of all forebay, micro-pool, and deep channel volumes should be 10% of the marsh surface area and provide approximately 20% of the water quality volume (reference Table 5.1).

Low Marsh Zones are those regions of the marsh ranging in depth between 6" and 18". The sum of all low marsh zones should equal 40% of the total marsh surface area.

High Marsh Zones are those regions of the marsh ranging in depth from 0 to 6". The high marsh zone is capable of supporting the most diverse mix of vegetation. The sum of all high marsh zones should comprise 50% of the total marsh surface area.

Semi-Wet Zones are those regions of the marsh that are situated above the permanent marsh pool. During non runoff-producing periods, the semi-wet zone is generally dry. This zone becomes inundated during runoff-producing events.

When designing the marsh area of a constructed stormwater wetlands, both surface area and volume guidelines must be considered. The following steps illustrate this process for the example project site. As indicated earlier, the example site is a section of two lane divided highway in Montgomery County.

Step 2B - Compute the Minimum Marsh Surface Area

The summation of all "wet" marsh zone surface areas must not be less than 2% of the wetland's total contributing drainage area. The minimum marsh surface area is therefore computed as:

$$17.4ac \times \frac{43{,}560\, ft^2}{ac} \times 0.02 = 15{,}159\, ft^2$$

This minimum area must be distributed across the three "wet" marsh zones as shown in Table 5.1. The total volume provided by this distribution should yield the computed treatment volume of 17,424 ft^3. If the surface area criteria conflict with storage volume requirements, the surface area allocations are considered more critical to an effective wetland design. (DCR/DEQ, 1999, Et seq.) Consequently, it is considered essential to attain the surface area distributions shown in Table 5.1. The following steps illustrate a procedure for meeting the surface area allocation targets while also achieving the desired water quality volume.

Step 2C - Size the Zones of Varying Depth

50% of the total surface area of the marsh should be dedicated to the *high marsh zone* (depths ranging between zero and 6”). The *low marsh zone* (depths ranging between 6 and 18”) should comprise an additional 40% of the total marsh surface area. The remaining 10% of the marsh surface area should be made up of the *deep water zone* (ranging in depth from 1.5 to 6’).

The total surface area of the marsh is designated as *A*. Following this convention, the surface area of each depth zone can be expressed as follows:

$$A_1 = 0.50A$$
$$A_2 = 0.40A$$
$$A_3 = 0.10A$$

Because of its shallow depth, the side slopes of the high marsh zone can be considered negligible, and the *effective* depth of the zone is assumed to be the maximum depth of 0.5’. This effective depth can be employed for purposes of volume calculations. Therefore, the total volume encompassed by the marsh’s shallowest pool zone is approximated as follows:

$$V_1 = 0.5\,ft \times A_1 = (0.5\,ft)(0.50)(A)$$

The effective depth of the low marsh zone is computed as its average depth:

$$D_e = \frac{6in + 18in}{2} = 12in = 1\,ft$$

With the total volume encompassed by the low marsh zone approximated as follows:

$$V_2 = 1\,ft \times A_2 = (1\,ft)(0.40)(A)$$

For this example, the deep water zone of the marsh (sediment forebays and micro pool) will be designed at an average depth of 4’. Therefore, the effective depth is 2’ and the volume is expressed as:

$$V_3 = 2\,ft \times A_3 = (2\,ft)(0.10)(A)$$

The sum of all incremental marsh volumes should equal or exceed 0.40 acre-ft. Therefore, the basin surface area, *A*, is approximated as follows:

$$V = 17{,}424\,ft^3$$
$$V = (0.5\,ft)(0.50)(A) + (1\,ft)(0.40)(A) + (2\,ft)(0.1)(A)$$

Rearranging and solving for surface area, *A*:

$$0.85A = 17{,}424\ ft^3$$
$$A = 20{,}499\ ft^2$$

This value exceeds the minimum allowable surface area of 15,159 ft^2 and is therefore acceptable. The computed surface area is 2.7% of the wetland contributing drainage area of 17.4 acres.

Tables 5.4 and 5.5 summarize the surface area and approximate volume of each marsh depth zone.

Table 5.4.3 - Surface Area Summary of Varying Depth Zones

Zone / Depth	Surface Area (ft^2)	Percentage of Total Surface Area (%)
High Marsh (0 - 6")	10,250	50
Low Marsh (6 - 18")	8,199	40
Deep (0 - 4')	2,050	10
Total	***20,499***	***100***

Table 5.4.4 - Volume Summary of Varying Depth Zones

Zone / Depth	Approximate Volume (ft^3)	Percentage of Total Treatment Volume (%)
High Marsh (0 - 6")	5,125	30
Low Marsh (6 - 18")	8,199	47
Deep (0 - 4')	4,100*	23
Total	***17,424***	***100***

*Includes sediment forebay and micro pool volumes

It is noted that the treatment volume provided in the deep water zone is 23% of the total treatment volume. This slightly exceeds the target of 20%. However, as previously stated, attainment of surface area allocation targets is of greater importance than volume distribution.

The computed deep pool surface area must be distributed among two sediment forebays and the outlet micro-pool. Obtained from *Section Two – Extended Dry Detention Basin*, Table 5.6 presents the respective storage volume of each sediment forebay.

Table 5.4.5 - Deep Pool Volume Allocation

Basin Location	Volume (ft^3)
Forebay 1	817
Forebay 2	908

The total forebay volume is 1,725 ft^3. The remaining deep pool volume (2,375 ft^3) is allocated to the micro-pool located at the wetland outlet.

Step 3 - Construct Elevation – Storage Relationship

Having determined the required surface area and storage volume for each of the three "wet" marsh zones, the next step is to construct a stage – storage relationship. This step is required in order to perform final flood routing for selected storms, thereby testing the final grading plan and outlet structure design for adequacy. The reader is referred to Step 6 of *Section Two – Dry Extended Detention Basin* for detailed flood routing procedure. Each site is unique, both in terms of constraints and required storage volume. Because of this, the development of a proposed grading plan may be an iterative process. The reader is referred to *Section Four – Retention Basin* for detailed embankment design procedures.

Table 5.7 presents the stage – storage relationship for the computed marsh area. The wetland floor elevation is assumed to be 2000' MSL.

Table 5.4.6 - Stage – Storage Relationship

Elevation	Incremental Volume (ft^3)	Total Volume (ft^3)
2100	0	0
2100.5	512.5	512.5
2101	512.5	1025
2101.5	512.5	1537.5
2102	512.5	2050
2102.5	512.5	2,562.5
2103	3245.5	5,808
2103.5	3245.5	9,053.5
2104	8,370.5	17,424

Step 4 - Evaluate Impact of the 10-Year Runoff Producing Event

The use of constructed stormwater wetlands for flood control is strongly discouraged. Offline configurations, such as that shown in Figure 5.3, can provide effective water quality improvement while not subjecting the wetland to the extreme water fluctuations typically associated with a flood control facility. When a proposed wetland will be subjected to storm inflows beyond the water quality volume, it is critical to restrict the vertical ponding depth to as shallow as practically possible. Outlet structures must be sized to pass up to the 10-year return frequency storm with a maximum ponded depth of 2' above the surface of the wetland marsh. (DCR/DEQ, 1999, Et seq.) The following steps illustrate a procedure for ensuring that the 10-year return frequency storm is routed through the example wetland facility without inducing a ponded depth of more than 2' above the marsh surface. The reader is referred to *Section Two – Dry Extended Detention Basin* for detailed routing and principal spillway design steps.

This design example will employ a riser consistent with the SWM-1 structure detailed in the *Virginia Department of Transportation Road and Bridge Standards,* (VDOT, 2016). A detail of this type of inlet top is shown in Figure 5.4.

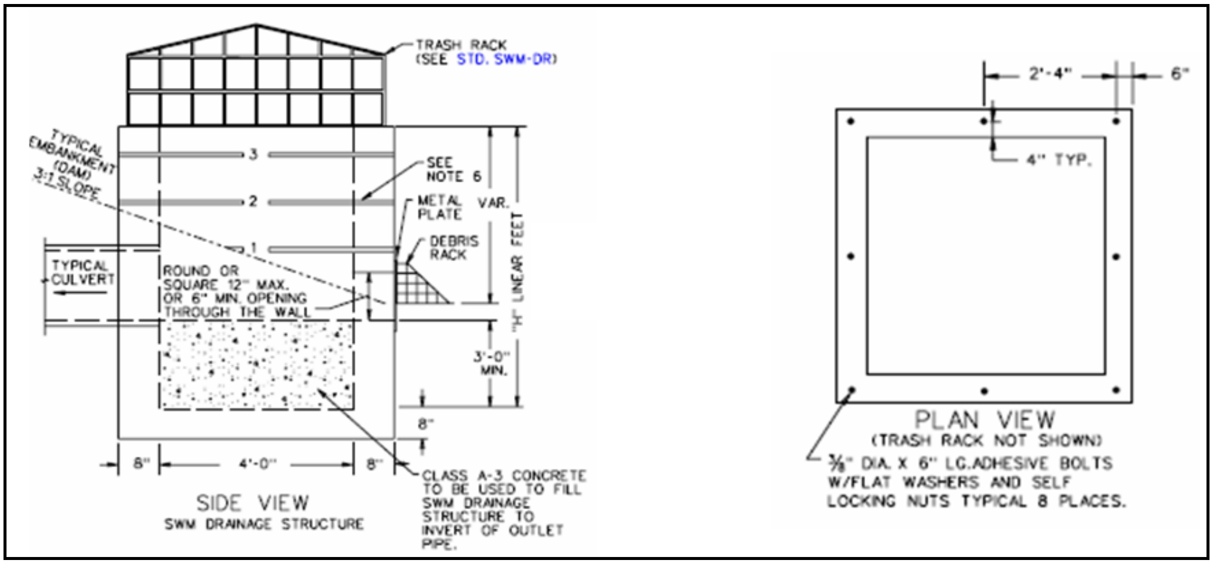


Figure 5.4.1 - VDOT SWM-1 Plan and Section
VDOT Road and Bridge Standards

Obtained from *Section Two – Extended Dry Detention Basin* the effective weir length and flow area of the SWM-1 grate top is:

Effective flow perimeter (weir length): 16’

Effective flow area: 16 ft^2

The crest of the grate will be set at an elevation just above the surface of the wetland permanent pool – 2004.1’. This will minimize the depth of ponding observed during runoff producing events.

The next step is to estimate the volume of storage provided above the permanent marsh in the wetland *semi-dry zone*.

In this example, we will consider a wetland of rectangular orientation, with a 2.5:1 length-to-width ratio. The demonstrated methodology can be adapted to wetlands exhibiting different geometry.

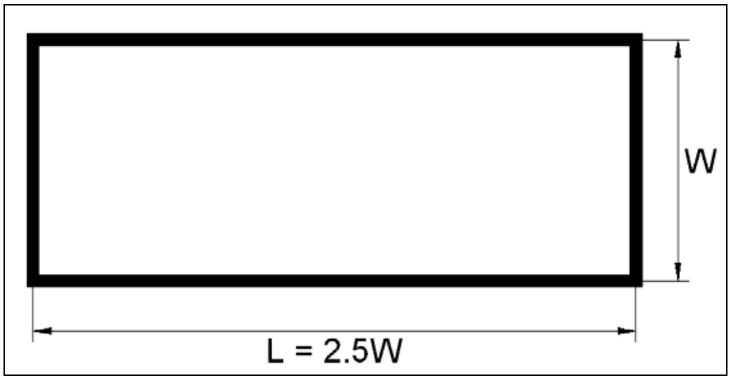


Figure 5.4.2 - Schematic Wetland Orientation

The dimensions of the basin permanent pool can be approximated by solving the following expression:

$$W \times 2.5W = 20{,}499\,ft^3$$
$$W = 90.6\,ft$$
$$L = 226.5\,ft$$

Considering side slopes of 4H:1V, at a depth of 2' above the permanent pool the wetland area is computed as:

$$W = 90.6 + (2)(4)(2) = 106.6\,ft$$
$$L = 226.5 + 16 + (2)(4)(2) = 242.5\,ft$$
$$A = (106.6\,ft)(242.5\,ft) = 25{,}851\,ft^2$$

The storage volume provided between the surface of the permanent marsh and a depth of 2' above the marsh is computed by the trapezoidal rule as follows:

$$V = \left[\frac{20{,}499\,ft^2 + 25{,}851\,ft^2}{2}\right] \times 2\,ft = 46{,}350\,ft^3$$

Using the procedures described at length in *Section Two – Dry Extended Detention Basin*, we can develop elevation – discharge and elevation – storage relationships. The permanent marsh pool is assumed to be present in the basin at the onset of the 10-year runoff producing event. Therefore, only storage above the marsh surface elevation is considered. The discharge – elevation relationship is for a VDOT SWM-1 riser structure as shown in Figure 5.4. This relationship is shown in Table 5.8 and Figure 5.6.

Table 5.4.7 - Stage – Discharge Relationship

Wetland Water Elevation (ft)	Basin Outflow (cfs)
2104.00	0.00
2104.50	12.55
2105.00	42.35
2105.50	82.16
2106.00	106.19

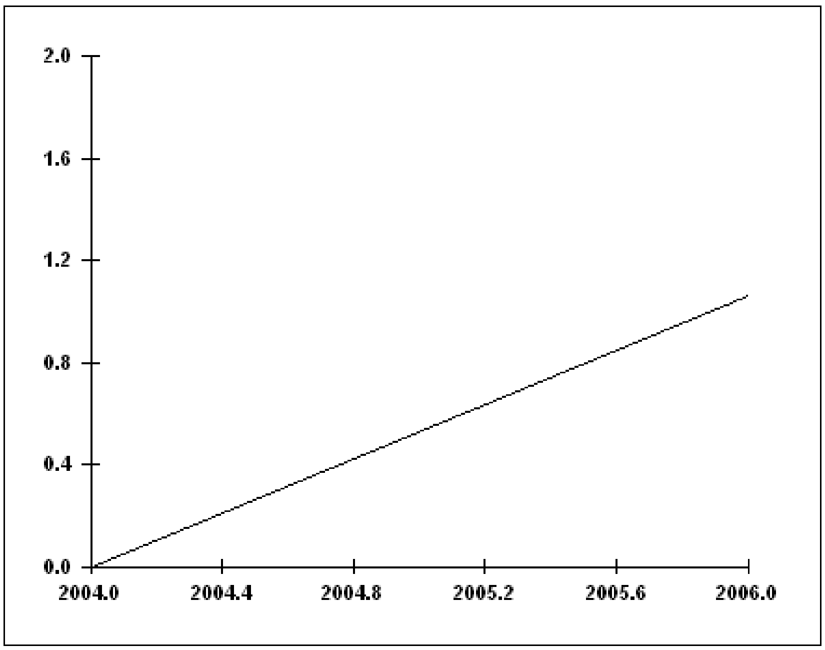
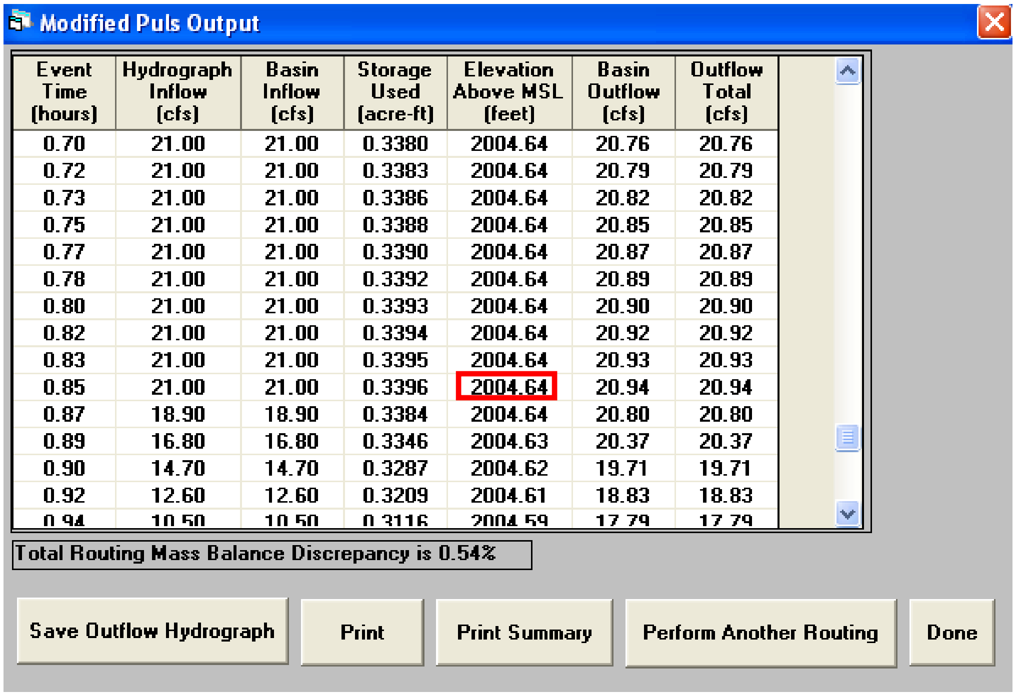


Figure 5.4.3 - Stage – Storage Relationship

Next, we utilize the 10-year return frequency Modified Rational hydrograph from *Section Two – Dry Extended Detention Basin* and route it through the wetland. While this Modified Rational hydrograph does not exhibit the maximum volumetric runoff *rate* from the project site, it does reflect the storm event which generates the greatest *volume* of required storage. It is this event which yields the greatest ponding depth in the wetland, and therefore it must be evaluated. The results of this routing are shown in Figure 5.7.



Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
0.70	21.00	21.00	0.3380	2004.64	20.76	20.76
0.72	21.00	21.00	0.3383	2004.64	20.79	20.79
0.73	21.00	21.00	0.3386	2004.64	20.82	20.82
0.75	21.00	21.00	0.3388	2004.64	20.85	20.85
0.77	21.00	21.00	0.3390	2004.64	20.87	20.87
0.78	21.00	21.00	0.3392	2004.64	20.89	20.89
0.80	21.00	21.00	0.3393	2004.64	20.90	20.90
0.82	21.00	21.00	0.3394	2004.64	20.92	20.92
0.83	21.00	21.00	0.3395	2004.64	20.93	20.93
0.85	21.00	21.00	0.3396	2004.64	20.94	20.94
0.87	18.90	18.90	0.3384	2004.64	20.80	20.80
0.89	16.80	16.80	0.3346	2004.63	20.37	20.37
0.90	14.70	14.70	0.3287	2004.62	19.71	19.71
0.92	12.60	12.60	0.3209	2004.61	18.83	18.83
0.94	10.50	10.50	0.3116	2004.59	17.79	17.79

Figure 5.4.4 - Routing of 10-Year Modified Rational Hydrograph Through Wetland

Figure 5.7 shows the maximum water surface in the wetland as 2004.64'. Therefore, the 10-year runoff producing event is conveyed through the wetland with a maximum depth of 0.64' above the surface of the wetland marsh. This value is less than the 2.0' allowable, and therefore is acceptable.

Step 5 - Design of the Submerged Release Outlet

Generally, a constructed wetland facility must be equipped with a means by which baseflow can pass through the wetland without continually accumulating. This conveyance is typically accomplished by a submerged, inverted pipe (see detail in *Section Four – Retention Basin*. The submerged outlet pipe should extend into the outlet micro-pool to a depth of approximately 18" in order to reduce the likelihood of clogging by debris and floating plant matter.

The first step in computing the required outlet size is to establish the maximum anticipated baseflow which must be conveyed through the wetland once the permanent marsh/pool volume is present. This maximum baseflow arises during the month exhibiting the highest average precipitation. The Virginia State Climatology Office maintains an online database with monthly climate information from various stations across the state. This information can be obtained at:
http://climate.virginia.edu/online_data.htm#monthly

Examining this data for the Montgomery County (Blacksburg) station reveals the month exhibiting the highest average precipitation total as May, with *4.00"*.

This precipitation total must now be converted into a runoff rate. This is accomplished by employing the NRCS/SCS runoff depth equation.

The post-development site is comprised of a total of 17.4 acres, 4.75 acres of which is impervious and 12.65 acres of which is unimproved grass cover. Appendix 6H-3 and 6H-4 of the VDOT Drainage Manual contain runoff curve numbers for various land covers and Hydrologic Soil Groups.

The site's Hydrologic Soil Group is *B*. Estimating the site's pervious cover as grass in fair condition, the runoff curve number taken from Appendix 6H-3 is *69*. The curve number for the site's impervious fraction is *98*.

Next, the 2-year 24-hour precipitation depth must be obtained. This information can be obtained from the National Weather Service at:
http://hdsc.nws.noaa.gov/hdsc/pfds/orb/va_pfds.html

Examining this data for the Blacksburg station reveals the 2-year 24-hour precipitation depth, *P*, to be *2.76"*.

Next, the SCS runoff depth equations are employed to determine the 2-year 24-hour runoff depth for the post-developed site:

Pervious Fraction

$$S = \frac{1000}{CN} - 10 = \frac{1000}{69} - 10 = 4.49$$

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} = \frac{(2.76 - (0.2)(4.49))^2}{(2.76 + (0.8)(4.49))} = 0.55 inches$$

Impervious Fraction

$$S = \frac{1000}{CN} - 10 = \frac{1000}{98} - 10 = 0.20$$

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} = \frac{(2.76 - (0.2)(0.20))^2}{(2.76 + (0.8)(0.20))} = 2.53 inches$$

The total depth of runoff over the entire developed site is then computed as:

$$\frac{(0.55 inches)(12.65 acres) + (2.53 inches)(4.75 acres)}{17.4 acres} = 1.09 inches$$

The Efficiency of Runoff, *E*, is computed as the ratio of runoff depth to the total depth of precipitation for the 2-year event:

$$E = \frac{1.09in}{2.76in} = 0.39$$

Employing this efficiency ratio, the estimated average runoff volume for the month of May is computed as:

$$4.00inches \times 0.39 \times \frac{1ft}{12in} \times 17.4ac \times \frac{43{,}560\,ft^2}{ac} = 98{,}533\,ft^3$$

The baseflow rate is then computed as:

$$\frac{98{,}533\,ft^3}{31days} \times \frac{1day}{24hour} \times \frac{1hour}{3{,}600\sec} = 0.04cfs$$

The elevation at which the baseflow bypass outlet begins to discharge from the wetland must be set equal to the elevation corresponding to the surface of the wetland marsh. This ensures that the permanent pool volume is maintained in the wetland at all times, while perennial baseflow is passed through the principal spillway and does not accumulate. Referencing Figure 5.4, we see that the permanent pool volume occurs at elevation 2004'. The crest of the baseflow bypass outlet is therefore set at 2004' and sized as follows:

We will initially try a 3" diameter orifice, and restrict the maximum head to that occurring just as the outlet becomes submerged. Employing the orifice equation:

$$Q = Ca\sqrt{2gh}$$

Q= discharge (cfs)
C= orifice coefficient (0.6)
a= orifice area (ft^2)
g= gravitational acceleration (32.2 ft/sec^2)
h= head (ft)

$$a = \pi r^2 = \pi \times \left(\frac{3in/2}{\frac{12in}{ft}} \right)^2 = 0.049\,ft^2$$

The head is measured from the centerline of the orifice. The head when the orifice has just become submerged by a small increment, 0.01', is expressed as:

$$h = 1.5inches \times \frac{1ft}{12in} + 0.01ft = 0.135\,ft$$

Discharge is now computed as:

$$Q = (0.6)(0.049)\sqrt{(2)(32.2)(0.135)} = 0.09cfs$$

The selected 3” diameter orifice will easily convey the perennial baseflow (0.04 cfs) entering the wetland. A smaller diameter orifice would meet the required hydraulic function. However, a smaller orifice would be susceptible to clogging by debris and floating/suspended plant matter and is therefore not recommended.

Step 6 - Water Balance Calculation

To ensure that the wetland permanent marsh does not become dry during extended periods of low or absent inflow, the designer must perform a water balance calculation. Two approaches are described in the following section.

Step 6A - 45-Day Drought Condition

The first approach considers the extreme condition of a 45-day drought period with no precipitation and thus no significant surface runoff.

Table 5.9 presents potential evaporation rates for various locations in Virginia.

Table 5.4.8 - Potential Evaporation Rates (Inches)

Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.)

Station	April	May	June	July	August	Sept.
Charlottesville	2.24	3.84	5.16	6.04	5.45	3.87
Danville	2.35	3.96	5.31	6.23	5.69	3.91
Farmville	2.34	3.81	5.13	6.00	5.41	3.71
Fredericksburg	2.11	3.80	5.23	6.11	5.46	3.83
Hot Springs	1.94	3.41	4.50	5.14	4.69	3.33
Lynchburg	2.21	3.72	4.99	5.85	5.31	3.70
Norfolk	2.20	3.80	5.37	6.34	5.79	4.14
Page County	1.68	3.06	4.09	4.71	4.26	3.05
Pennington Gap	2.14	3.59	4.72	5.45	4.97	3.60
Richmond	2.28	3.89	5.31	6.23	5.64	3.92
Roanoke	2.20	3.75	4.99	5.85	5.30	3.67
Staunton	2.00	3.52	4.77	5.52	4.95	3.47
Wash. National Airport	2.13	3.87	5.50	6.51	5.84	4.06
Williamsburg	2.27	3.86	5.23	6.14	5.61	3.97
Winchester	2.07	3.68	4.99	5.82	5.26	3.67
Wytheville	2.01	3.43	4.46	5.17	4.71	3.39

The greatest potential evaporation for the station nearest the project site (Roanoke) occurs during the months of July and August, 5.85" and 5.30" respectively. Therefore, the total evaporation over a 45-day period is estimated as follows:

Average evaporation per month = $\frac{5.85in + 5.30in}{2} = 5.58in$

Average evaporation per day = $\frac{5.58\frac{in}{month}}{31\frac{day}{month}} = 0.18\frac{in}{day}$

The evaporation loss over a 45-day period is calculated as follows.

$45\,\text{days}\ X\ 0.18\frac{in}{day} = 8.1in = 0.68\,ft$

The total surface area of the marsh is 20,499 ft^2. Therefore, the total volume of water potentially lost to evaporation is estimated as:

$$20{,}499\,ft^2 \times 0.68\,ft = 13{,}939\,ft^3$$

The volume of water lost to evaporation must be added to that lost to infiltration. As previously stated, the initial geotechnical tests revealed site soil infiltration rates to be 0.02 in/hr. The infiltration is assumed to occur over the entire marsh, whose surface area is 15,160 ft^2. The volume of water lost to infiltration is estimated as:

$$20{,}499\,ft^2 \times 0.02\frac{in}{hr} \times \frac{1\,ft}{12in} \times 24\frac{hr}{day} \times 45 days = 36{,}898\,ft^3$$

The total volume of water lost to evaporation and infiltration over the 45-day drought period is therefore computed as:

$$13{,}939\,ft^3 + 36{,}898\,ft^3 = 50{,}837\,ft^3$$

This value exceeds the total marsh volume of 17,424 ft^3, implying that a 45-day drought period will leave the marsh area in a completely dry state. Over time, it is quite likely that the infiltration rate of the basin soil will decrease considerably due to clogging of the soil pores. However, the aquatic and wetland plant species will likely not survive an extended period of drought that occurs prior to this clogging. Therefore, at this point in the design, it would be recommended to install a clay or synthetic basin liner as approved by the Materials Division. A typical infiltration rate for synthetic liner may be on the order of 3x10^{-7} in/sec. The calculation is repeated for this rate of infiltration.

$$20{,}499\,ft^2 \times 3x10^{-7}\,\frac{in}{\sec} \times \frac{1\,ft}{12in} \times 3{,}600\frac{\sec}{hr} \times 24\frac{hr}{day} \times 45 days = 1{,}993\,ft^3$$

The recalculated volume of water lost to evaporation and infiltration over the 45-day drought period is therefore computed as:

$$13{,}939\,ft^3 + 1{,}993\,ft^3 = 15{,}932\,ft^3$$

While the extended drought period does impact the marsh area significantly, a minimal volume of water *is* retained in the marsh.

The volume of runoff necessary to replenish the depleted marsh volume is computed as follows:

Total contributing drainage area = 17.4 acres

Stored volume lost to evaporation and infiltration = 15,932 ft^3

$$\frac{15{,}932\,ft^3}{17.4ac \times \frac{43{,}560\,ft^2}{ac}} = 0.02\,Watershed\text{ - }Feet = 0.24\,Watershed\text{ - }Inches$$

A precipitation event yielding a total runoff of 0.24" or more across the contributing watershed will replenish the depleted marsh volume.

Step 6B - Period of Greatest Evaporation (in Average Year)

The second water balance calculation examines impacts on the marsh during the one-month period of greatest evaporation during an average year. This calculation reflects an *anticipated* marsh drawdown during the summer months. In contrast, the first calculation method reflects an extreme *infrequent* drought event.

From Table 5.9, the greatest monthly evaporation total for the station nearest the project site is 5.85" in July. The Virginia State Climatology Office reports an average July rainfall for the Blacksburg station as 3.99" (reference Step 5 for link to data).

Applying the previously computed runoff efficiency ratio for the basin watershed, the average July inflow to the basin is computed as:

$$3.99inches \times 0.39 \times \frac{1ft}{12in} \times 17.4ac \times \frac{43{,}560\,ft^2}{ac} = 98{,}286\,ft^3$$

Evaporation losses are computed as the product of total monthly evaporation and the surface area of the permanent pool:

$$5.85inches \times \frac{1ft}{12in} \times 20{,}499\,ft^2 = 9{,}993\,ft^3$$

Infiltration losses (with synthetic liner) over the entire month of July are estimated as:

$$20{,}499\,ft^2 \times 3x10^{-7}\,\frac{in}{\sec} \times \frac{1ft}{12in} \times 3{,}600\frac{\sec}{hr} \times 24\frac{hr}{day} \times 31days = 1{,}373\,ft^3$$

The water balance expression and total monthly loss/gains are computed as follows:

Monthly loss/gain = Inflow – Evaporation – Infiltration
$$= 98{,}286\,ft^3 - 9{,}993\,ft^3 - 1{,}373\,ft^3 = 86{,}920\,ft^3$$

The monthly climate data and site land cover characteristics indicate that the wetland marsh will not experience drawdown during the average period of highest evaporation.

Step 7 - Landscaping

Generally, the non-marsh regions of constructed stormwater wetlands (i.e. the *semi wet* zone) can be landscaped in much the same manner as a typical stormwater impounding facility. However, careful attention must be given to the types of vegetation selected for the wetland marsh areas. For these regions, the vegetative species must be selected based on their inundation tolerance and the anticipated frequency and depth of inundation.

If appropriate vegetative species are selected, the entire marsh area should be colonized within three years. Because of this rapid colonization, only one-half of the total low and high marsh zone areas need to be seeded initially. A total of five to seven different emergent species should be planted in the wetland marsh areas. Both the high and low marsh areas should each be seeded with a *minimum* of two differing species.

The regions of varying depth within the wetland are broadly categorized by zone as shown in Figure 5.8.

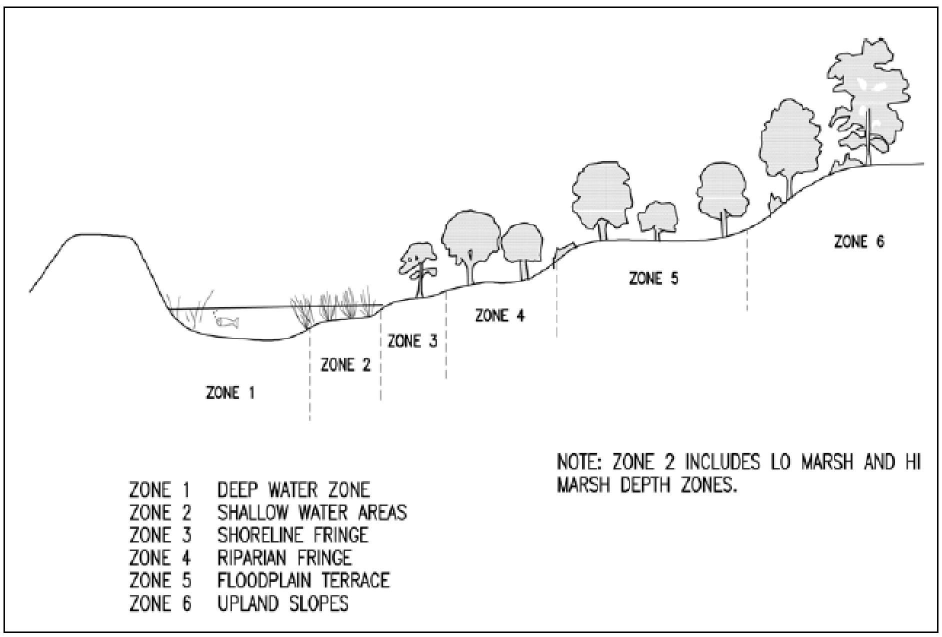


Figure 5.4.5 - Planting Zones for Stormwater BMPs
Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.)

Suitable planting species for each of the zones identified in Figure 5.9 are recommended in Chapter 3-05 of the *Virginia Stormwater Management Handbook*, (DCR/DEQ, 1999, Et seq.). Ultimately, the choice of planting species should be largely based on the project site's physiographic zone classification. Additionally, the selection of plant species should match the native plant species as closely as possible. Surveying a project site's native vegetation will reveal which plants have adapted to the prevailing hydrology, climate, soil, and other geographically-determined factors. Figure 3.05-4 of the *Virginia Stormwater Management Handbook* provides guidance in plant selection based on project location.

Generally, stormwater management facilities should be permanently seeded within 7 days of attaining final grade. This seeding should comply with Minimum Standard 3.32, Permanent Seeding, of the *Virginia Erosion and Sediment Control Handbook*, (DCR/DEQ, 1992, Et seq.). It must be noted, however, that permanent seeding is *prohibited* in Zones one through four of Figure 5.9. The use of conventional permanent seeding in these zones will result in the grasses competing with the requisite wetland emergent species.

When erosion of basin soil prior to the establishment of mature stand of wetland vegetation is a concern, Temporary Seeding (Minimum Standard 3.31) of the *Virginia Erosion and Sediment Control Handbook*, (DCR/DEQ, 1992, Et seq.) may be considered. However, the application rates specified should be reduced to as low as practically possible to minimize the threat of the Temporary Seeding species competing with the chosen emergent wetland species.

All chosen plant species should conform to the *American Standard for Nursery Stock*, current issue, and be suited for USDA Plant Hardiness Zones 6 or 7, see Figure 5.9.

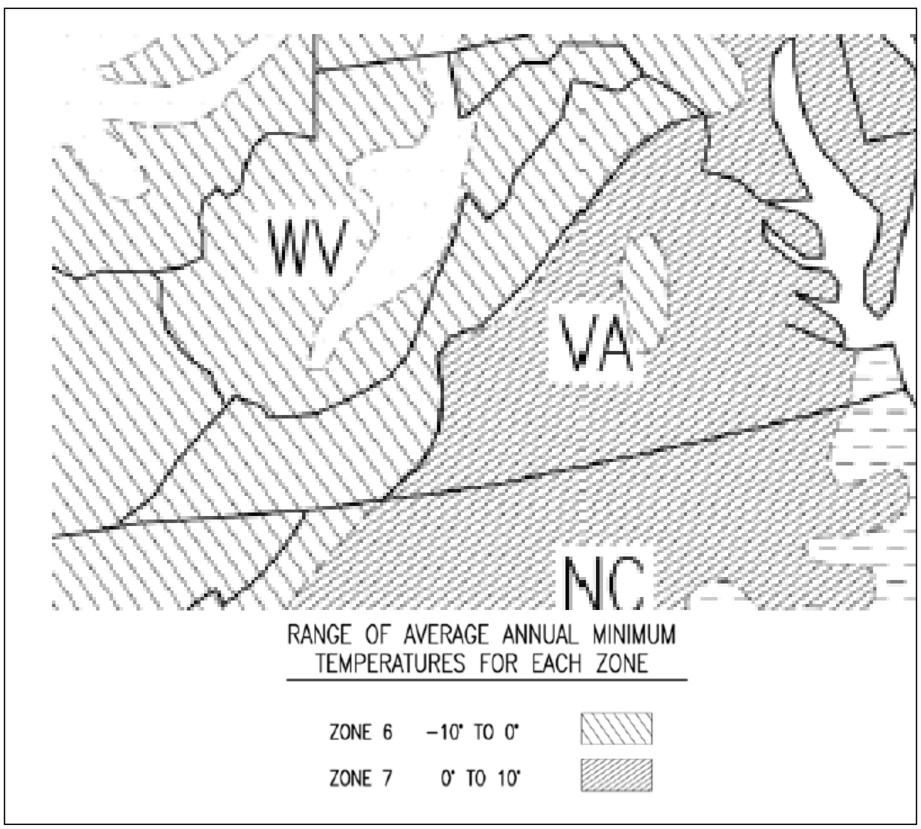


Figure 5.4.6 - USDA Plant Hardiness Zones

If the wetland is equipped with an impounding embankment, *under no circumstances should trees or shrubs be planted on the basin embankment*. The large root structure may compromise the structural integrity of the embankment.

6.1 Vegetated Water Quality Swale - Overview of Practice

Vegetated swales are broadly described as surface depressions which collect and convey stormwater runoff from roadways, driveways, rooftops, and other impervious surfaces. However, when applied as a *Best Management Practice*, an engineered grassed swale functions beyond simple collection and conveyance, seeking to also improve the quality of stormwater runoff through sedimentation and filtration. The inherent linear orientation of a vegetated swale makes it an attractive option for treatment and conveyance of highway runoff.

Vegetated swales function by minimizing flow velocity and inducing ponding behind strategically placed check dams. While infiltration of some runoff associated with ponding can attenuate peak runoff rates, this attenuation can be considered minimal at best. *Vegetated swales are water quality improvement practices, and cannot be considered effective flood control strategies.*

The Virginia Stormwater Management Handbook, (DCR/DEQ, 1999) identifies two categories of vegetated conveyance BMPs – “Grassed Swales” and “Water Quality Swales” (Minimum Standard 3.13). Grassed swales, also termed “dry swales,” function by slowing the velocity of runoff and inducing ponding behind strategically placed check dams. The swale’s controlled velocity permits filtration of runoff pollutants by the dense vegetation lining the channel. Ponding increases the hydraulic residence time within the swale, thus providing an increased opportunity for the gravitational settling of pollutants. Water quality swales, or *wet swales*, can be conceptualized as a linear wetland. Their underlying soils, in contrast to dry swales, are comprised of a very specific mixture in order to permit controlled infiltration as well as the growth of wetland vegetation. The rigid underlying soil characteristics of a wet swale will typically require native site soils to either be amended or excavated completely and replaced with imported material. While wet water quality swales are considered capable of achieving phosphorus removal beyond that of dry swales, they are best suited for contributing drainage areas whose impervious cover ranges from 16 – 37%. When a project site’s new impervious cover enters that range, there will be a need for flood control in the form of mitigation of post-developed runoff rates to those of pre-developed levels. The inability of a wet water quality swale to also provide peak attenuation will generally render it cost prohibitive, with BMPs capable of providing both water quality improvement and peak mitigation preferred. Therefore, as evidenced in Table 1.1, the VDOT BMP selection table only considers the grassed, or dry, variation of a water quality swale.

6.2 Site Constraints and Siting of the Facility

In addition to the contributing drainage area's new impervious cover, a number of site constraints must be considered when the implementation of a grassed swale is proposed. These constraints are discussed as follows.

6.2.1 Minimum Drainage Area

The minimum drainage area contributing to a vegetated swale is not restricted. Vegetated swales are particularly well suited to small drainage areas.

6.2.2 Maximum Drainage Area

The water quality improvement function of a vegetated swale is predicated on its ability to maintain minimal flow velocities within the channel. Therefore, within the confines of feasible cross-sectional areas, such channels cannot simultaneously be designed to convey large flow rates and/or volumes. The channel cross-section geometry, roughness, longitudinal slope, and design discharge will ultimately dictate flow velocity within the channel. The design discharge is a function of the contributing drainage area, and therefore the area must be limited such that desired velocities are maintained. In addition to meeting velocity restrictions (discussed later), the swale must be designed to convey the 10-year flow with a minimum of 6" of freeboard.

6.2.3 Site Slopes

Sites on which a vegetated swale is proposed should exhibit relatively flat topography. The maximum permissible slope of a grassed swale is *6%*. Alternative BMPs should be considered when site topography is such that this maximum slope is exceeded. Grassed swales function best when their slope is a flat as practically possible.

6.2.4 Site Soils

The implementation of a grassed swale can be successfully accomplished in the presence of a variety of soil types exhibiting at least moderate permeability. However, when such a practice is proposed, *a permeability test is strongly recommended*. This data should be provided to the Materials Division early in the project planning stages to determine if a grassed swale is feasible on native site soils. Because ponding is induced within the swale, site soils should permit the emptying of the swale through infiltration. The inability of native site soils to completely drain a swale within a period of less than 72 hours can introduce undesirable marshy conditions and mosquito habitat. The minimum soil infiltration rate considered for construction of a grassed swale is *0.27 in/hr*. Soils underlying a vegetated grass should be USDA *ML, SM, or SC*. Sites exhibiting sandy soils should conform to *ASTM C-33*, VDOT fine aggregate grading *A* or *B*, or as otherwise approved by the Materials Division.

6.2.5 Depth to Water Table

Grassed swales inevitably infiltrate detained runoff into the subsurface. The infiltrated runoff may potentially carry a significant pollutant load. Therefore, *grassed swales should not be used on sites exhibiting a seasonally-high water table of less than 2' below the proposed swale bottom.*

6.2.6 Existing Utilities

When possible, swales should not cross existing utility rights-of-way or easements. When this situation is unavoidable, permission to construct the swale over these easements must be obtained from the utility owner *prior* to design of the swale. When it is proposed to relocate existing utility lines, the costs associated with their relocation should be included in the overall project construction cost.

6.2.7 Wetlands

When the construction of a grassed swale is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify the wetland boundaries, their protected status, and the feasibility of BMP implementation in their vicinity. The presence of existing wetlands may reveal native soils capable of accommodating a wet water quality swale at the site.

6.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing a vegetated swale for improvement of water quality.

6.3.1 Swale Geometry

Because the fundamental goal of a grassed swale is to improve the *quality* of runoff, it is essential to avoid any concentration of the flow within the channel. In addition to presenting problems of constructability, parabolic and triangular channels will concentrate low flows, and thus are undesirable. Similarly, rectangular channels should be avoided because of the inherent instability of their side slopes. Therefore, to satisfy both the issues of constructability and that of desired flow regime, only *trapezoidal cross section* channels are considered. Channel side slopes should be no steeper than 3H:1V.

6.3.2 Bottom Width

Channel bottom widths of less than 2' are essentially non-constructible, and should not be considered. Conversely, bottom widths greater than 6' will tend to concentrate small flow events thereby reducing the pollutant removal ability of the swale. With a range of 2 to 6' established as acceptable, the precise channel bottom width becomes largely a function desired flow depth. This topic is discussed later in this section in the context of an example swale design.

6.3.3 Channel Depth

The swale should be designed such that the water quality volume flows at a depth approximately equal to the grass height. For most applications this will be 4". The overall depth should permit conveyance of the 10-year runoff event while providing a minimum of 6" of freeboard. Additionally, channel depth should be such that the check dam height does not exceed one half of the total channel depth.

6.3.4 Longitudinal Slope

The generally accepted minimum constructible slope is 0.75%. The slope of a grassed swale should be as flat as practically possible for the given site topography. The site-specific allowable longitudinal slope will ultimately be governed by the desired flow depth and velocity. In general, however, this maximum slope should not exceed 6%.

6.3.5 Flow Velocity

The flow velocity should be as low as practically possible in order to achieve maximum pollutant removal. Additionally, the swale must be designed such that larger runoff events do not result in re-suspension of previously deposited sediments. The following design velocities should be met:

Table 6.3.1 - Permissible Flow Velocities

Design Flow	Permissible Velocity (fps)
2-year	4
10-year	7

Source: Virginia Stormwater Management Handbook, (DCR/DEQ, 1999)

6.3.6 Shear Stress

In addition to considering the velocity in the channel, the shear stress exhibited by the flow must also be examined. Table 5.2 presents permissible shear stresses for five different classes of vegetative linings. These classes are further described later in the context of a design example.

Table 6.3.2 - Permissible Shear Stresses

Lining Category	Lining Type	Permissible Shear Stress, τ_p lb/ft^2	kg/m^2
Vegetative	Class A	3.70	18.06
	Class B	2.10	10.25
	Class C	1.00	4.88
	Class D	0.60	2.93
	Class E	0.35	1.71

Source: FHWA/Chen and Cotton (1988)

6.3.7 Swale Length

The length of a grassed swale is not restricted, but rather must be sized together with the channel cross-sectional area and check dam height to provide the desired water quality storage volume.

6.3.8 Discharge Flows

When a grassed swale empties into an existing swale or other surface conveyance system, the receiving channel must be evaluated for adequacy as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR/DEQ, 1992). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

6.4 Design Process

This section presents the design process applicable to grassed swales serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered during linear development projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 4 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999) for expanded hydrologic methodology.

The following swale design will provide the technology-based water quality requirements arising from the construction of approximately 1,800 LF of secondary subdivision roadway in the City of Hampton. Topography is such that runoff from the road is collected in VDOT CG-6 curb and gutter and conveyed to curb inlets in a sump near the mid station of the road. The runoff is then discharged into the proposed swale. The total project site, including right-of-way and all permanent easements, consists of 5.27 acres. Pre and post-development hydrologic characteristics are summarized below in Tables 5.3 and 5.4. The project site exhibits topography typical of the coastal region of Virginia, with slopes less than 2%. Site constraints limit the swale length to 275'.

Table 6.4.1 - Hydrologic Characteristics of Example Project Site

	Pre-Development	Post-Development
Project Area (acres)	5.27	5.27
Land Cover	Unimproved Grass Cover	1.03 acres *new* impervious cover
Impervious Percentage	0	19.5

Table 6.4.2 - Peak Roadway Runoff

		York County – 10-Year				
Acreage	**Rational C**	**A Constant**	**B Constant**	**t_c (min)**	**i_{10} (iph)**	**Q_{10} (cfs)**
1.03	0.9	186.78	21.22	8	6.39	5.9

York County – 2-Year				
A Constant	**B Constant**	**t_c (min)**	**i_2 (iph)**	**Q_2 (cfs)**
122.93	16.72	8	4.97	4.6

Step 1 - Compute the Required Water Quality Volume

The project site's water quality volume is a function of the developed new impervious area. This basic water quality volume is computed as follows:

$$WQ_V = \frac{NIA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

NIA= New Impervious Area (ft^2)

The project site in this example is comprised of a total drainage area of 5.27 acres. The total new impervious area within the site is 1.03 acres (19.5% of the total site area). Therefore, the water quality volume for this site is computed as follows:

$$WQ_V = \frac{1.03ac \times \frac{1}{2} in \times \frac{43{,}560\, ft^3}{ac}}{12 \frac{in}{ft}} = 1{,}870\, ft^3$$

A vegetated swale must be sized to provide ponding for the computed water quality volume. This ponding occurs behind check dams (height and longitudinal spacing discussed later).

Step 2 - Determine the Cross-Sectional Dimensions of the Channel

Ponding in the swale will occur behind check dams 18" in height. Because the cross-sectional size and configuration of the channel remain constant throughout its length, the total volume of water detained throughout the swale can be estimated by the *average end area method.* This volume calculation simply averages the wet cross-sectional area at the upstream and downstream ends of the channel and computes the stored volume as the product of this average area and the channel length. This approach assumes that the available ponding depth at the downstream end of the channel is equal in depth to the check dam height. The depth of water at the most remote upstream point in the channel is assumed to be zero. For a trapezoidal channel with 3:1 side slopes and 18" (1.5') check dams, the downstream wet cross-sectional area is computed as:

$$A = (w_b)(1.5) + (2)\left(\frac{1}{2}\right)(1.5)(3)(1.5)$$

With: w_b= channel base width (ft)

Because the ponded upstream depth is zero, the effective cross sectional area of the swale is one half this value, expressed as:

$$A_{avg} = \frac{(w_b)(1.5) + (2)\left(\frac{1}{2}\right)(1.5)(3)(1.5)}{2}$$

The design is continued for a total channel length of 275', longitudinal slope of 2%, and side slopes of 3:1. The required average cross-sectional area of the channel is computed by dividing the required water quality volume by the channel length.

$$A_{avg} = \frac{1{,}870\, ft^3}{275\, ft} = 6.80\, ft^2$$

Rearranging the earlier channel cross-sectional area expression in terms of base width, w_b:

$$w_b = \frac{2A_{avg} - (1.5)(3)(1.5)}{1.5}$$

The required channel base width is then computed as:

$$w_b = \frac{(2)(6.80) - (1.5)(3)(1.5)}{1.5} = 4.56\, ft$$

To address any underestimation in storage volume arising from the average end computation, the base width of the channel is increased to *5'*.

Step 3 - Determine the Depth of the Channel

The ten-year flood peak, Q_{10}, is selected as the design discharge for establishing the conveyance properties of the channel, while providing a minimum 6" of freeboard. The presence of check dams in the swale introduces difficulty in modeling flow through the channel. Two approaches are presented in this example for determining the required channel depth. The first approach conceptualizes the swale as linear detention facility, with storage-indication routing employed to establish the maximum water surface elevation under 10-year runoff producing conditions. This approach yields accurate results, yet is computationally intensive. The second approach simply ignores the presence of check dams and computes the normal depth in the channel under 10-year flow conditions. This computed normal depth is added to the check dam height and the required 6" freeboard. While computationally simpler, the second approach tends to oversize the channel because it does not consider that a significant portion of the 10-year runoff volume is detained behind the check dams and, thus not contributing to computed *flow* depth.

Step 3A - Channel Depth – Method 1

Because water is ponded in the swale behind 18” check dams, the swale behaves much like a detention facility, with flow through the swale occurring as weir flow over the check dams. Thus a reasonable approach to determining the required swale depth is to perform storage indication routing. This approach yields the maximum water surface elevation under 10-year inflow conditions. Adding 6” of freeboard to this depth provides the minimum swale depth.

The first step is to establish a stage – storage relationship for the swale. Storage volumes are computed based on channel geometry, with all variables as defined:

$$V = \left[\frac{(w_b)(d) + (2)\left(\frac{1}{2}\right)(d)(Z)(d)}{2} \right] \times L$$

V = ponded volume (ft^3)
w_b = channel base width (ft)
d = ponded depth (ft)
Z = channel side slope (ZH:1V)
L = channel length (ft)

Employing the previously established channel parameters, the ponded volume can be computed solely as a function of ponded depth:

$$V = \left[\frac{(5)(d) + (2)\left(\frac{1}{2}\right)(d)(3)(d)}{2} \right] \times 275$$

This calculation is employed for various incremental depths. The results are shown in Table 5.5 below, assuming a downstream bottom channel elevation of 300’ mean sea level (MSL). Note that the approximate water quality volume is provided at a depth of 1.5’, equaling the check dam height.

Table 6.4.3 - Swale Stage – Storage Relationship

Elevation	Volume (ft^3)
300	0
300.5	447
301	1,100
301.5	1,959
302	3,025
302.5	4,297
303	5,775
303.5	7,459
304	9,350
304.5	11,447
305	13,750
305.5	16,259
306	18,975

Next, the stage – discharge relationship is constructed. The channel check dams function as broad-crested weirs. At a depth of 18", the weir length is calculated as follows, with parameters as previously defined:

$$L = w_b + (2)(d)(z)$$
$$= 5\,ft + (2)(1.5\,ft)(3) = 14\,ft$$

Discharge over a broad-crested weird is a function of the head acting on the weir crest. The weir equation is as follows, and used to establish the stage – discharge relationship shown in Table 5.6. Note there is no flow occurring below the check dam crest elevation.

$$Q = C_W L h^{1.5}$$

Q = Weir discharge (cfs)
C_W = Weir coefficient (3.0)
L = Weir length (14')
h = hydraulic head acting on weir crest (ft)

Table 6.4.4 - Swale Stage – Discharge Relationship

Elevation	Discharge (cfs)
301.5	0
302	15
302.5	42
303	77
303.5	119
304	166
304.5	218
305	275
305.5	336
306	401

Next, using the stage – storage data, stage – discharge data, and the 10-year return frequency post-development runoff hydrograph, storage-indication routing is performed to determine the actual water surface elevation observed in the swale during this event. Figure 6.1, below, illustrates the 10-year post-development runoff hydrograph developed using the NOAA NW-14 regional rainfall I-D-F parameters recommended in the VDOT Drainage Manual.

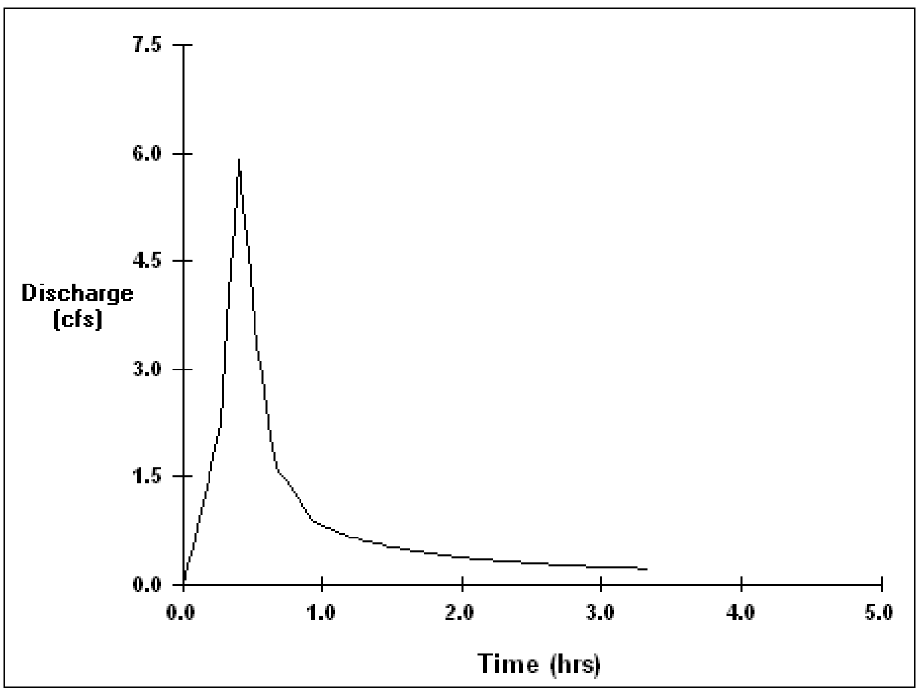


Figure 6.4.1 - 10-Year Post-Development Flow Entering Swale

Figure 6.2 on the following page illustrates the results of the storage-indication routing operation.

Modified Puls Output

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
0.23	1.96	1.96	0.0184	300.78	0.000	0.000
0.27	2.26	2.26	0.0242	300.97	0.000	0.000
0.30	3.20	3.20	0.0317	301.17	0.000	0.000
0.33	4.13	4.13	0.0418	301.42	0.000	0.000
0.37	5.06	5.06	0.0515	301.63	2.17	2.17
0.40	5.99	5.99	0.0567	301.74	5.11	5.11
0.43	5.34	5.34	0.0575	301.76	5.64	5.64
0.47	4.69	4.69	0.0566	301.74	5.05	5.05
0.50	4.03	4.03	0.0556	301.72	4.40	4.40
0.53	3.38	3.38	0.0546	301.70	3.74	3.74
0.57	2.95	2.95	0.0536	301.68	3.27	3.27
0.60	2.52	2.52	0.0528	301.66	2.83	2.83
0.63	2.09	2.09	0.0519	301.64	2.40	2.40
0.67	1.67	1.67	0.0511	301.63	1.97	1.97
0.70	1.58	1.58	0.0505	301.61	1.68	1.68

Figure 6.4.2 - Routing of 10-Year Flow Through Swale

The routing reveals a maximum flow depth of 1.76', equal to 0.26' (3.12") over the check dams. Therefore, the minimum swale depth is computed as the sum of the computed water depth and the required freeboard:

$$1.76\,ft + 0.5\,ft = 2.26\,ft = 27.12in$$

Step 3B - Channel Depth – Method 2

An alternative approach for determining the necessary swale depth is to compute the normal flow depth observed during the 10-year runoff producing event, under the assumption that there is water stored behind each check dam at the onset of the 10-year runoff event. This depth is then added to the check dam height and the required freeboard depth to determine the minimum swale depth. This is a conservative approach, as it does not consider that a significant portion of the 10-year runoff volume is detained behind the check dams and, thus not contributing to computed *flow* depth.

The computed 10-year post-development runoff exhibits a peak discharge of 5.9 cfs. The first step is to compute the flow depth (normal depth) of the 5.9 cfs discharge in the proposed channel. This task is accomplished by employing both the continuity and Manning's equations.

In order to apply Manning's Equation, the roughness coefficient of the channel must first be established. This coefficient can be estimated *initially* and then adjusted as needed to satisfy flow velocity and hydraulic radius requirements. It is an iterative process since these hydraulic parameters depend, in turn, on the Manning's n value.

The first step in computing the Manning roughness coefficient is to estimate the retardance class of the vegetation lining the channel. The channel retardance factor is based on the type of vegetative lining, and can be found in Table 5.7.

For this example, the proposed swale will be seeded with Kentucky bluegrass and maintained at a height of approximately 6”. This vegetative cover falls in retardance class C.

The next step is to select an initial value of Manning’s *n* and then estimate the product of the flow velocity and hydraulic radius (VR_h) in the channel, using the following SCS graph.

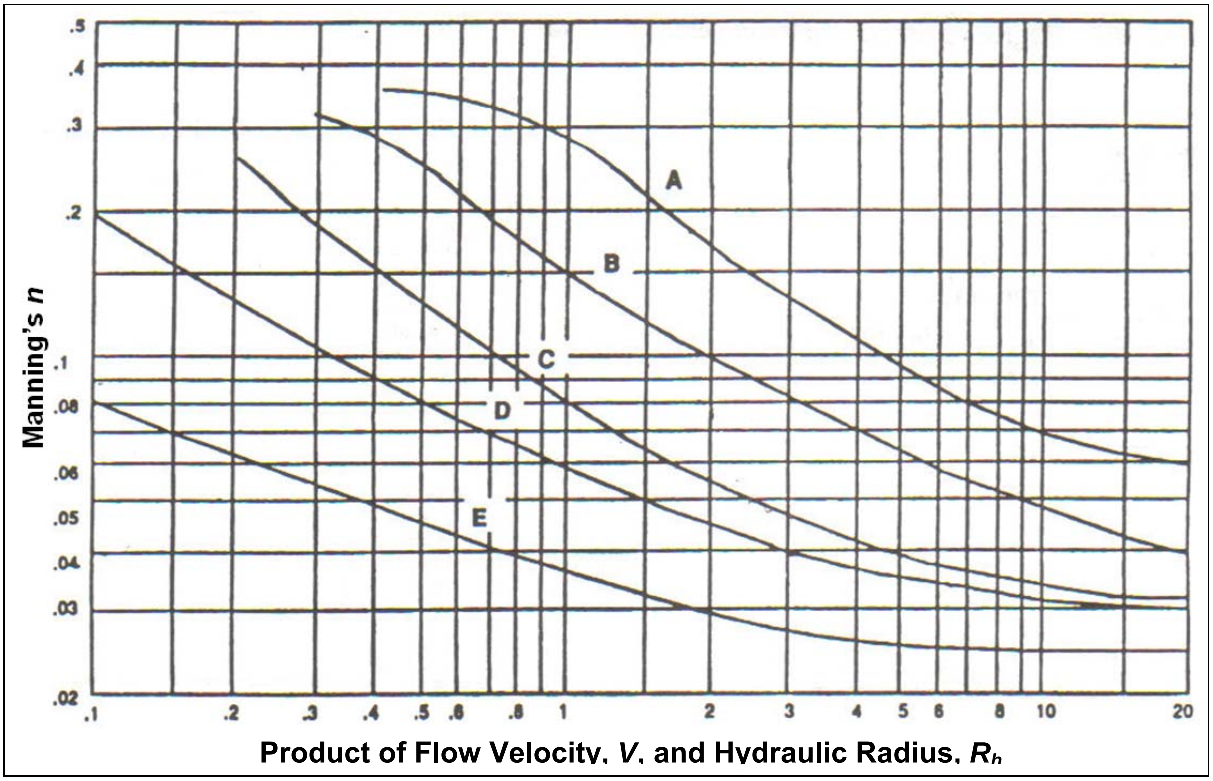


Figure 6.4.3 - Relationship of Manning’s n to VR_h

Sources: U.S. Department of Transportation. Federal Highway Administration. Evaluation and Management of Highway Runoff Water Quality. Washington, D.C., 1996. Presents part of SCS Tech. Paper 61, 1954.

USDA, Soil Conservation Service, Technical Paper 61, Handbook of Channel Design for Soil and Water Conservation, 1954.

Table 6.4.5 - Classes of Retardance by Vegetation Type and Height

Retardance Class	Cover	Condition
A	Weeping Lovegrass	Excellent stand, tall (average 30in [76cm])
	Yellow bluestem *Ischaemum*	Excellent stand, tall (average 36” [91cm])
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12” [30cm])
	Native grass mixture	Good stand, unmowed
	(little bluestem, bluestem, blue gamma,	
	and other long and short midwest grasses)	
	Weeping Lovegrass	Good stand, (average 24in [61cm])
	Lespedeza sericea	Good stand, not woody, tall (average 19in [48cm])
	Alfalfa	Good stand, uncut (average 11” [28cm])
	Weeping Lovegrass	Good stand, unmowed (average 13in [28cm])
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 11” [28cm])
C	Crabgrass	Fair stand, uncut (10-48” [25-120cm])
	Bermuda grass	Good stand, mowed (average 6” [15cm])
	Common lespedeza	Good stand, uncut (average 11” [28cm])
	Grass-legume mixture -- summer	Good stand, uncut (6-8” [15-20cm])
	(orchard grass, redtop, Italian ryegrass,	
	and common lespedeza)	
	Centipedegrass	Very dense cover (average 6” [15cm])
	Kentucky bluegrass	Good stand, headed (6-12” [15-30cm])
D	Bermuda grass	Good stand, cut 2.5” height (6cm)
	Common lespedeza	Excellent stand, uncut (average 4.5” [11cm])
	Buffalo grass	Good stand, uncut (3-6” [8-15cm])
	Grass-legume mixture -- fall	Good stand, uncut (4-5” [10-13cm])
	(orchard grass, redtop, Italian ryegrass,	
	and common lespedeza)	
	Lespedeza sericea	After cutting to 2” in height (5cm)
		Very good stand before cutting
E	Bermuda grass	Good stand, cut to 1.5” in height (4cm)
	Bermuda grass	Burned stubble

Source: Adapted from Mays (2005), and FHWA (1996).

Employing an initial *trial* Manning's roughness coefficient of 0.10, Figure 5.3 yields an estimated value of VR_h as 0.73 ft^2/s. Next, the actual value of VR_h corresponding to a roughness coefficient of 0.10 is computed. The actual VR_h value is determined using the Manning's equation as follows:

$$VR_h = \frac{1.49}{n} R_h^{\ 1.67} S^{\ 0.5}$$

The following flow parameters are considered for this example:

Channel base width	5ft
Channel side slopes	3H:1V
Channel longitudinal slope	2.00%
Manning's Roughness Coefficient	0.10
Design Discharge	5.9 cfs

Employing VTPSUHM to solve the Manning's equation for these parameters yields the following results:

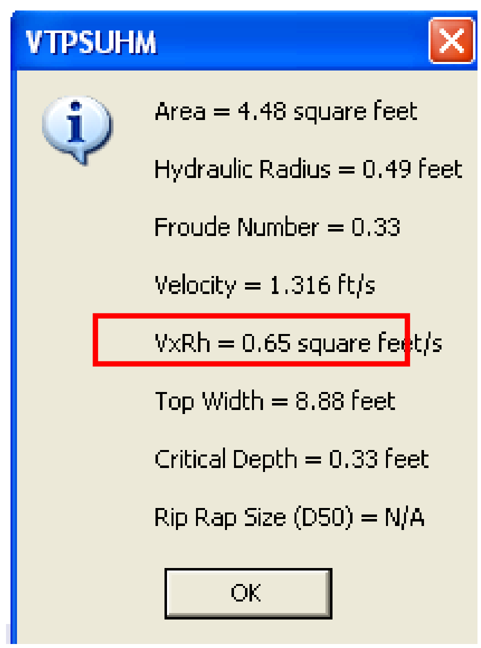


Figure 6.4.4 - Results of Initial Manning's Roughness of 0.10

The product of the flow velocity and hydraulic radius is found to be 0.65 ft^2/s. This value is now used to determine a new Manning's roughness value from Figure 5.3. Entering Figure 5.3 with a VR_h value of 0.65 ft^2/s and a vegetative retardance class of C yields a roughness coefficient of 0.12.

Employing the new roughness coefficient with all previously defined flow and channel size parameters yields the following results:

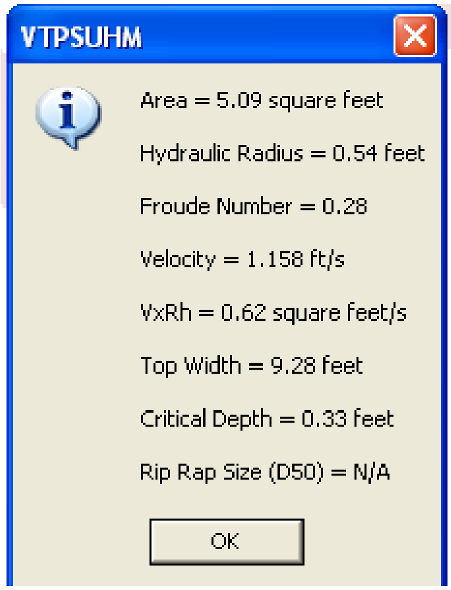


Figure 6.4.5 - Results of Second Manning's Roughness of 0.12 (Q_{10})

The new product of the flow velocity and hydraulic radius is found to be 0.62 ft^2/s. This value is less than 5% different than the estimated value of 0.65 ft^2/s, and thus is acceptable. Had the results yielded a discrepancy of greater than 5%, subsequent iterations would have been carried out until convergence was observed.

With an acceptable Manning's roughness coefficient established, the next step is to compute the required channel depth. Employing the aforementioned flow parameters, we now compute the 10-year flow depth (normal depth) in the channel by Manning's equation. The VTPSUHM results of this calculation are shown as follows.

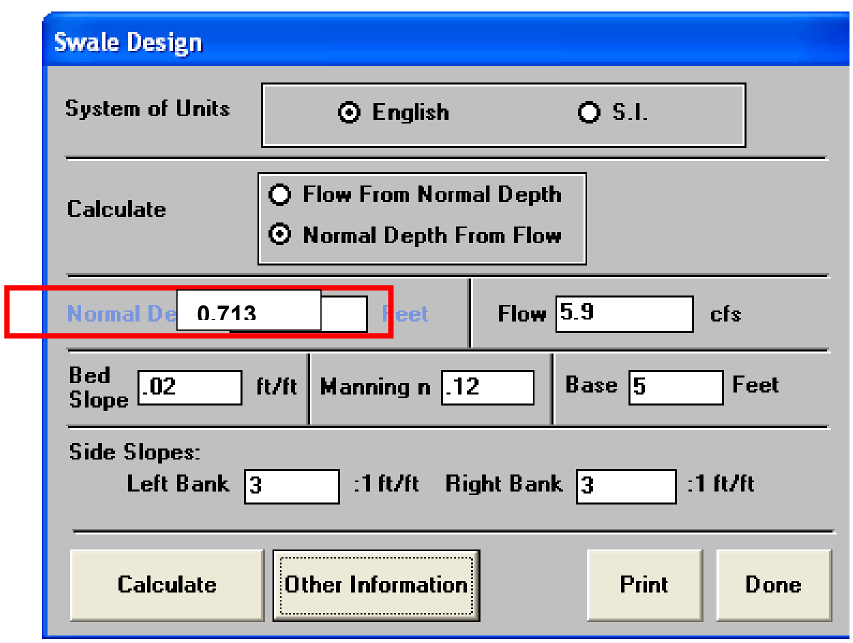


Figure 6.4.6 - Results of Normal Depth Calculation (Q_{10})

The output exhibits a 0.713’ flow depth (normal depth) for the 10-year return frequency discharge.

Examining the VTPSUHM output (Figure 5.5) on the previous page reveals that the flow velocity of 1.16 fps is less than the maximum allowable velocity of 7 fps for the 10-year return frequency flow.

The minimum depth of the channel can now be computed by summing the segmental depths, based on the conservative assumption that there is an 18” ponded depth in the swale prior to the arrival of the 10-year storm hydrograph. The Q_{10} normal depth will then be added to the ponded depth under this assumption.

$$d_{min} = d_{Ponded} + d_{10\text{ yr. storm}} + d_{Freeboard}$$

$$d_{min} = 1.5ft + 0.71ft + 0.5ft = 2.71ft = 32.5”$$

This approach yields a required channel depth predictably greater than that found by storage indication routing.

The next step is to evaluate the 2-year flow conditions for compliance with the maximum permissible flow velocity of 4 fps. Employing VTPSUHM to perform the Manning’s equation calculation:

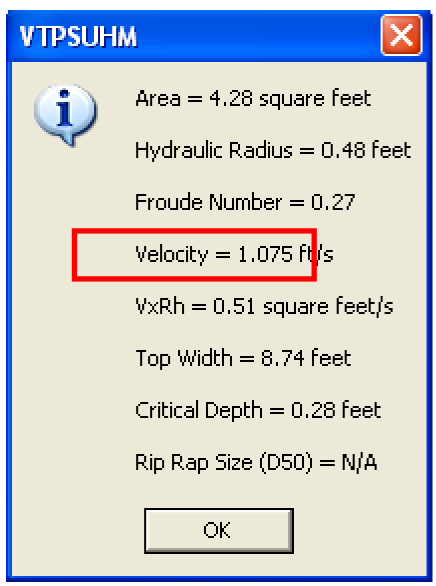


Figure 6.4.7 - Flow Parameters (Q_2)

The output reveals that the flow velocity of 1.08 fps is less than the allowable velocity of 4 fps for the 2-year return frequency discharge. Additionally, it should be noted that the Froude number of 0.27 indicates a sub critical flow regime. Designs for which the Froude number approaches unity should be avoided.

Step 3C - Channel Depth – Method 3

A third alternative for computing the required channel depth was developed by Dr. Osman Akan, Associate Dean of Engineering and Professor of Civil Engineering at Old Dominion University. First reported in 2001 by Akan and Hager in the ASCE *Journal of Hydraulic Engineering*, this method employs charts developed from a dimensionless form of the Manning equation. Application of these charts permits a direct solution of channel depth and width. The results obtained by this method are, generally, comparable to the previously described Method 2 normal depth calculation. However, for side slopes milder than 2:1, the Akan direct solution approach may overdesign the swale size by approximately 5%. Readers interested in applying the Akan direct solution method are referred to:

Akan, A. O. (2006). Open Channel Hydraulics. Elsevier/Butterworth-Heinemann, Burlington, MA, ISBN-13:978-0-7506-6857-6 and ISBN-10: 0-7506-6857-1

Table 5.8 summarizes the computed channel depth for the three design approaches.

Table 6.4.6 - Summary of Computed Channel Depth

Design Method	Computed Swale Depth (ft)
1 - Hydrograph Routing	2.26
2 - Normal Depth Calculation	2.71
3 - Akan-Hager Direct Solution Method	2.72*

*Computed value provided by Akan (personal communication).

It should be noted at this point that, (adhering to previously established design guidelines) the channel check dam height should not exceed one half of the total channel depth. The check dams employed in this design were assumed to be 18” in height. Therefore, the minimum channel depth that should be considered is 3’. Per the calculations presented in Step 3, a channel depth of 3’ yields a conservative design which provides more than the minimum 6” of a freeboard under 10-year inflow conditions. The check dam height could be reduced, but doing so would necessarily require an increased channel cross-sectional area to provide storage for the computed water quality volume. Increased channel area results in a need for greater right-of-way acquisition, and this is generally undesirable. A channel depth of 3’ is therefore adopted.

Step 4 - Ensure Allowable Levels of Shear Stress

The final step in verifying the adequacy of the proposed design is a check to ensure that the shear stress exhibited by the flow does not exceed the allowable values previously presented (Table 5.2).

The average shear stress associated with the flow is given by the following equation:

$$\tau_{Design} = \gamma R S_0$$

γ = specific weight of water ($62.4^{lb}/_{cf}$)
R = design hydraulic radius for the 10-year event (ft)
S_0 = channel longitudinal slope (ft/ft)

We note parenthetically that due to non-unifrom velocity distribution in the cross section, the maximum shear stress developed on the bed and sides of most trapezoidal channels of practical interest will be approximately 1.0 and 0.75 times the average shear, respectively. (Chow,1959).

The output from the 10-year flow reveals a hydraulic radius of 0.54'. Employing the previously presented equation, shear stress on the channel is found as follows:

$$\tau_{Design} = (62.4\frac{lb}{ft^3})(0.54\,ft)(.020\frac{ft}{ft}) = 0.67\frac{lb}{ft^2}$$

For a vegetative lining with a Class C retardance factor, the permissible shear stress is 1 lb/sf. Thus, the proposed design is acceptable.

Step 5 - Investigation of Alternative Swale Designs

Best Hydraulic Section

In the design *of non-erodible st*ormwater conveyance channels, the concept of the *best hydraulic section* is often employed. The best hydraulic section is the channel configuration for which wetted perimeter is minimized for a fixed cross-sectional area and desired discharge. In other words, the hydraulic radius is maximized. The best hydraulic section exhibits side slopes of 0.58:1. These excessively steep side slopes lend themselves well to concrete or other manmade systems, but are usually impractical for vegetated swales.

For the swale of interest in this design (base width of 5' and side slopes of 3:1), computing the swale depth by the best hydraulic section methodology yields a value of 15.4'. While potentially useful as a starting design point, best hydraulic section methodology will usually require significant modification to section properties to accommodate local site conditions. Design of an erodible channel, such as the vegetated water quality swale, should be carried out according to allowable shear stress principles, as shown in the above example.

Vegetated Swale Without Check Dams

Another design possibility is to construct the swale with no check dams. The primary purpose of the check dams is to level the grade, decrease erosion, and increase the contact time for the flow as it passes through the vegetative cover. Without check dams the length of equivalent swale must increase.

For many sites, this alternative will not be feasible because of the excessive length required to achieve an acceptable hydraulic residence time for the flow entering the channel. This length calculation is shown as follows:

$L = V\, T_r\, (60s/min)$

L = Required swale length (ft)
V = Flow velocity for the 10-year return event (ft/s)
T_r = Hydraulic residence time in minutes (9minutes minimum, FHWA, 1996)

Previous calculations show a flow velocity of 1.2 ft/s for the 10-year return event. For the example presented here, the required swale length is calculated as:

L = (1.2 ft/s)(9min)(60 s/min) = 648'

When vegetated swales employ check dams, ponding results in easy attainment of the 9 minute hydraulic residence time. Consequently, swale length can be reduced greatly, as illustrated in the initial design where the length was 275'. BMP swales without check dams are intended to serve only as a single treatment step in a series of multiple BMPs. In the absence of check dams, infiltration of runoff in the swale is negligible.

Step 6 - Check Dam Design

Check dam materials and construction techniques shall conform to those described in Minimum Standard 3.13 of the Virginia Stormwater Management Manual (DCR/DEQ, 1999). All check dams shall be equipped with toe protection as described in Minimum Standard 3.13. When the check dam material is riprap or gabion baskets, the check dams shall be underlain by a filter fabric approved by the Materials Division.

Check dams shall be placed longitudinally in the channel such that the dam height and the channel slope combine to provide the desired water quality volume. After establishing the swale dimensions as previously outlined, the total number of check dams required is computed as follows:

$$L_d = \frac{H}{S}$$

L_d = longitudinal distance behind each check dam (ft)
H = depth of ponding behind check dam (ft)
S = channel longitudinal slope (ft/ft)

$$L_d = \frac{(18'')(\frac{1\,ft}{12''})}{0.02} = 75\,ft$$

The total number of check dams is then computed by dividing the overall swale length by L_d:

$$\#Dams = \frac{275\,ft}{75\,ft} = 3.67 \qquad \text{Use four check dams}$$

In addition to providing a minimum of 6" of freeboard during 10-year flow conditions, the check dams should be equipped with a notch to ensure that the 2-year flow does not contact the check dam abutments. At the check dam height of 18", the channel width is 14'. Providing 6" of abutment freeboard on each end, the 2-year flow notch can be evaluated as a broad-crested weir of length 13'. The required depth of the notch can then be determined by the weir equation as follows.

$$Q = C_W L h^{1.5}$$

Rearranging the equation to solve for head:

$$h = \left[\frac{Q}{C_W L}\right]^{\frac{2}{3}}$$

The peak 2-year discharge is 4.6 cfs, and the flow depth, h, is computed as:

$$h = \left[\frac{4.6}{(3.0)(13.0)}\right]^{\frac{2}{3}} = 0.24\,ft = 2.9in$$

Therefore, a notch 2.9" or greater in depth will ensure that the 2-year flow is conveyed through the channel without contacting the check dam abutments.

Step 7 - Selection of Vegetation

The chosen vegetative channel lining must be water-tolerant, erosion–resistant and be suited to site-specific climate, soils, and topography. Selection of vegetation should conform to Standard and Specification 3.32 of the *Virginia Erosion and Sediment Control Handbook* (DCR/DEQ, 1992). The use of fertilization should be minimized as it contradicts the water quality improvement function of the swale.

The example channel is shown in profile and cross-section in Figures 5.8 and 5.9 respectively.

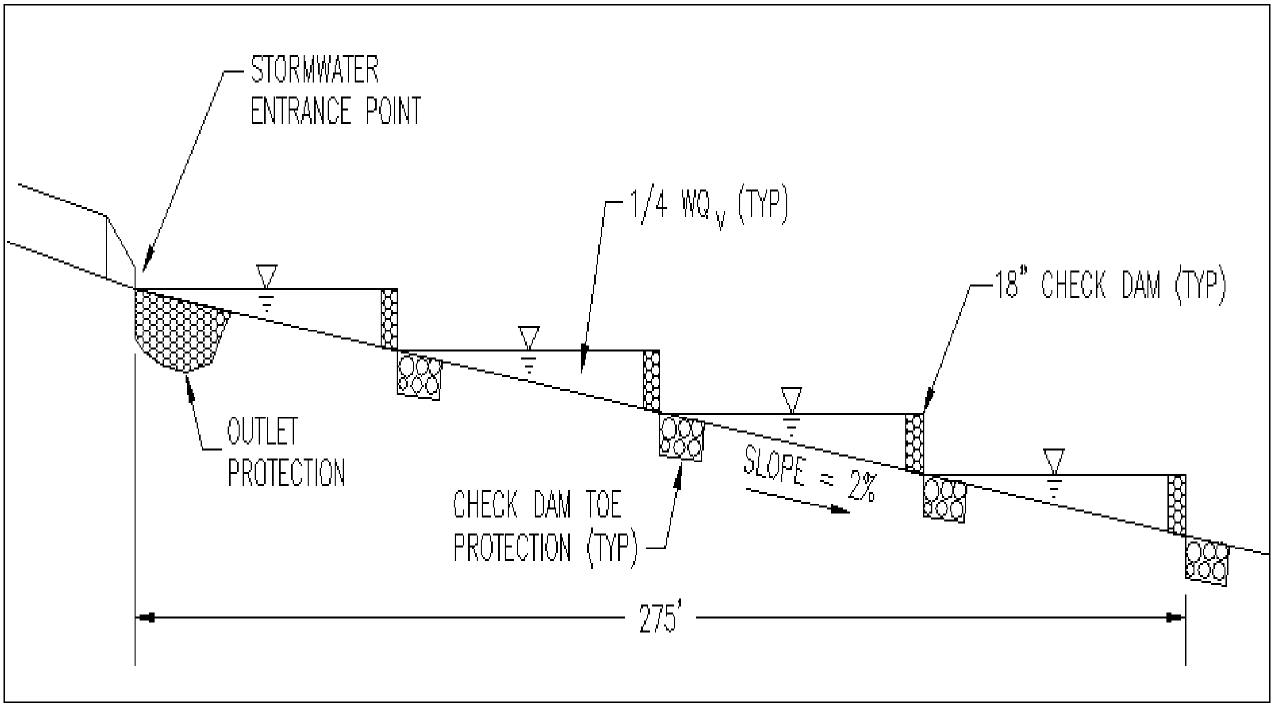


Figure 6.4.8 - Profile of Example Swale
Not to Scale

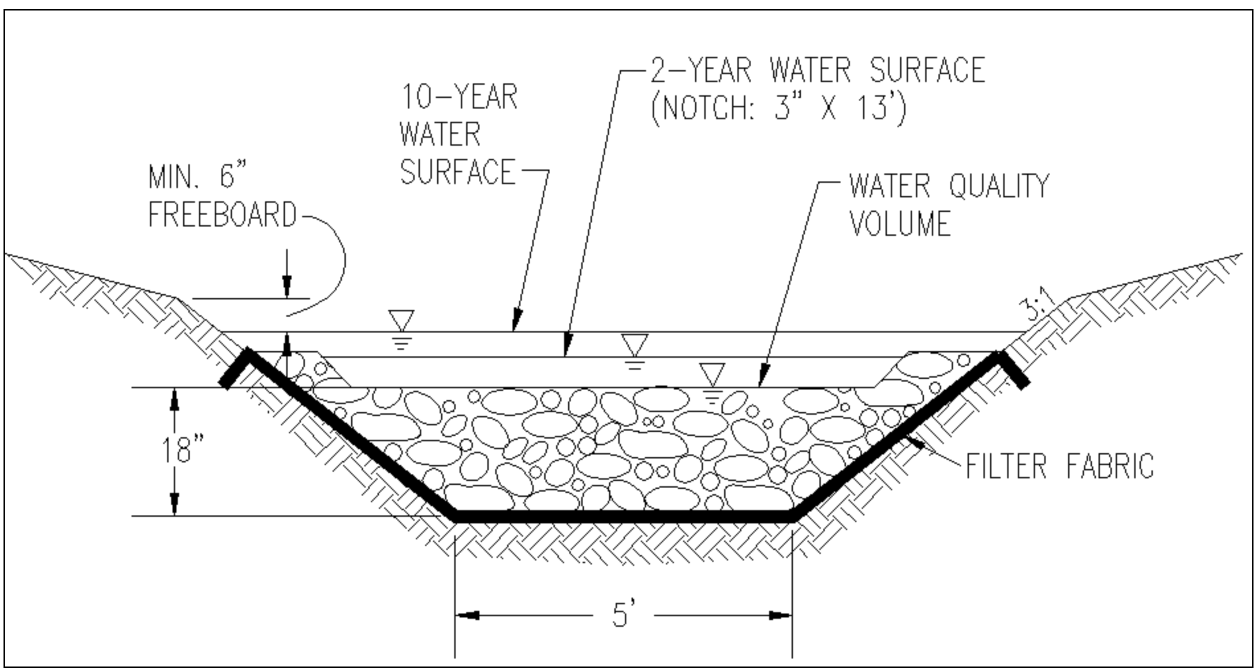


Figure 6.4.9 - Cross-Sectional View of Example Swale
Not to Scale

7.1 Vegetated Filter Strip - Overview of Practice

A vegetated filter strip is a densely vegetated strip of land, similar to a grassed swale, but engineered to accept runoff from upstream development only as *overland sheet flow* (Yu, 2004). The type of vegetation selected may range from native species, to grass meadow, to forest. In addition to serving as a primary water quality improvement practice, vegetated filters strips function extremely well as pre-treatment measures for other BMPs whose function may be compromised if sediment loading is excessive.

Vegetated filter strips are water quality improvement practices, and cannot be considered effective flood control strategies.

7.2 Site Constraints and Siting of the Facility

A number of site constraints must be considered in addition to the contributing drainage area's new impervious cover when the implementation of a vegetated filter strip is proposed. These constraints are discussed as follows.

7.2.1 Minimum Drainage Area

The minimum drainage area contributing to a vegetated filter strip is not restricted. Vegetated filter strips are particularly well suited to small drainage areas.

7.2.2 Maximum Drainage Area

The water quality improvement function of a vegetated filter strip is predicated on its ability to maintain sheet flow across the strip. When flow on the strip becomes concentrated, forming channels, the hydraulic residence time on the strip is reduced to ineffective levels. As contributing drainage area increases, so does the difficulty in ensuring that the volume of runoff generated from the area can remain as sheet flow across the strip. *The contributing area to a filter strip should never exceed five acres.* Regardless of the strip's contributing drainage area, flow entering onto the strip must never be concentrated. If sheet flow cannot be maintained upstream of the filter strip, a level spreader should be employed to convert concentrated flows back to sheet flow prior to their entrance onto the strip.

7.2.3 Site Slopes

Sites upon which a vegetated filter strip is proposed should exhibit relatively flat topography. Alternative BMPs should be considered when site topography is such that slopes exceed 5%.

7.2.4 Site Soils

The implementation of a vegetated filter strip is restricted to those soils having an infiltration rate of at least 0.52 in/hr. *A permeability test is required* for this BMP. This data should be provided to the Materials Division early in the project planning stages to determine if a vegetated filter strip is feasible on native site soils. In addition to infiltration rate restrictions, the soil must be capable of sustaining a dense stand of vegetation with minimal fertilization.

7.2.5 Depth to Water Table

The presence of a shallow water table in the vicinity of a proposed filter strip may hinder the infiltration function of the strip. The lowest elevation of the filter strip should be a minimum of 2' above the local seasonally high water table.

7.2.6 Existing Utilities

Filter strips often can be constructed over existing easements, provided permission to construct the strip over these easements is obtained from the utility owner *prior* to design of the strip.

7.2.7 Wetlands

When the construction of a vegetated filter strip is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation in their vicinity.

7.3 General Design Guidelines

The following presents a collection of broad design issues to be considered when designing a vegetated swale for improvement of water quality.

7.3.1 Length

Ultimately, the required length of a filter strip (in the direction of flow) is a function of the target hydraulic residence time for flows entering onto the strip. A 9 minute hydraulic residence time is recommended with five minutes being the absolute minimum for water quality improvement (FHWA, 1996). Generally, for strips exhibiting a longitudinal slope of less than 2%, the *minimum* strip length that should be considered is 25'. For any 1% increase in slope, the filter length should increase by at least 4'. These values, however, are only estimates and computational procedures (discussed later in this Section) must be used to ensure target hydraulic residence times are met. Optimal filter strip lengths will range from 80 to 100'. Flow over pervious surfaces tends to become concentrated when the flow path exceeds 150' (CWP, 1996). Therefore, strips of excessive length are discouraged.

7.3.2 Width

Ideally, the width of the filter strip (perpendicular to the flow direction) should, if at all possible, be equal to the width of the area contributing runoff to the strip. When this is not possible, a level spreader may be used to distribute flow evenly onto the strip. The *minimum* width of the filter strip should be the greater of the two values:

0.2 x Filter Length

or

8'

7.3.3 Slope

The filter strip slope should be as flat as practically possible while still providing positive drainage across the strip. Excessive ponding of runoff is undesirable as this will lead to saturation of the strip's underlying soil, resulting in difficulty maintaining a dense stand of vegetation on the strip. The slope of a vegetated filter strip is not restricted to any specific maximum value. However, as the strips slope is increased the flow velocity on the strip increases. The increase in velocity will necessarily require lengthening of the strip to attain an effective hydraulic residence time. As filter strip length increases so does the likelihood of the flow becoming concentrated. Filter strips function best on slopes of 5% or less (Yu, 2004). Table 7.1 presents maximum recommended filter strip slopes as a function of Hydrologic Soil Group and vegetative cover.

Table 0.1 - Recommended Maximum Filter Strip Slopes

Filter Strip Soil Type	Hydrologic Soil Group	Maximum Filter Strip Slope (Percent)	
		Turf Grass, Native Grasses, and Meadows	Planted and Indigenous Woods
Sand	A	7	5
Sandy Loam	B	8	7
Loam, Silt Loam	B	8	8
Sandy Clay Loam	C	8	8
Clay Loam, Silty Clay, Clay	D	8	8

Source: Pennsylvania Department of Environmental Protection. *Stormwater Best Management Practices Manual*. 2006.

7.3.4 Pervious Berm

When soil infiltration rates, site groundwater depths, and/or slopes do not adhere to the guidelines previously described, the filter strip may be equipped with a berm at its downstream end. Such a berm will effectively force ponding on the surface of the strip, thus increasing the hydraulic residence time of the entering flows. The berm should be constructed of moderately permeable soils as approved by the Materials Division. Generally acceptable soils are ASTM *ML, SM,* or *SC* or soils meeting USDA sandy loam or loamy sand texture with a minimum of 10 – 25% clay. The berm must be equipped with an armored overflow section to permit safe passage of large flows which would otherwise overtop the berm. The maximum depth of ponding behind the berm should not exceed 1'. The use of a berm should only be considered as a last resort, as the forced ponding of runoff on the strip will hinder the establishment of a dense stand of vegetation.

7.3.5 Discharge Flows

When a grassed swale empties into an existing swale or other surface conveyance system, the receiving channel must be evaluated for adequacy as defined by Regulation MS-19 in the *Virginia Erosion and Sediment Control Handbook*, (DCR/DEQ, 1992). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

7.4 Design Process

This section presents the steps in the design process applicable to vegetated filter strips serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered during linear development projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 4 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999) for expanded hydrologic methodology.

The following filter strip design will provide the technology-based water quality requirements arising from a linear development scenario similar to that described in *Section Six – Vegetated Swale*. The new scenario entails the construction of approximately 1,300 LF of secondary subdivision roadway in the City of Hampton. Topography is such that runoff from the road is collected in roadside ditches and conveyed to a low point near the mid station of the road. The concentrated runoff is discharged into a level spreader from which it then enters onto the proposed filter strip as overland sheet flow. The total project site, including right-of-way and all permanent easements, consists of 4.6 acres. Pre and post-development land cover characteristics and peak rates of runoff are summarized below in Tables 7.2 and 7.3. The project site exhibits topography typical of the coastal region of Virginia, with slopes generally less than 2%. Site soils are categorized as a sandy loam (Hydrologic Soil Group B).

Table 0.1 - Land Cover Characteristics of Example Project Site

	Pre-Development	Post-Development
Project Area (acres)	4.6	4.6
Land Cover	Unimproved Grass Cover	0.75 acres *new* impervious cover
Impervious Percentage	0	16.3

Table 0.2 - Peak 10-Year Runoff from Example Project Site

		York County - 10 Year				
Acreage	**Rational C**	**A Constant**	**B Constant**	**t_c (min)**	**i_{10} (iph)**	**Q_{10} (cfs)**
0.75	0.9	186.78	21.22	8	6.39	4.3

Step 1 - Compute the Required Water Quality Volume

The project site water quality volume is a function of the developed new impervious area. This basic water quality volume is computed as follows:

$$WQ_V = \frac{NIA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

NIA = New Impervious Area (ft^2)

The project site is comprised of a total drainage area of 4.6 acres. With new impervious area within the project site of 0.75 acres, the water quality volume is computed as:

$$WQ_V = \frac{0.75ac \times \frac{1}{2} in \times \frac{43{,}560\, ft^2}{ac}}{12 \frac{in}{ft}} = 1{,}361\, ft^3$$

The vegetated filter strip should be sized to provide a minimum hydraulic residence time of five minutes for the computed water quality volume.

Step 2 - Estimate the Required Strip Length

The next step is to estimate the strip's required length. Making an initial estimate of the required length will assist in evaluating the feasibility of the practice for the given site conditions. The following nomographs, Figures 7.1 – 7.5 (obtained from the Pennsylvania Department of Environmental Protection *Stormwater Best Management Practices Manual*, 2006), provide a means by which to estimate the required filter strip length as a function of the underlying Hydrologic Soil Group (HSG), strip slope, and type of vegetative cover. As stated previously, the proposed strip's underlying soil is a sandy loam of HSG *B*. At this point in the design, the vegetative cover is assumed to be native grasses. Figure 7.2 reflects the site-specific conditions.

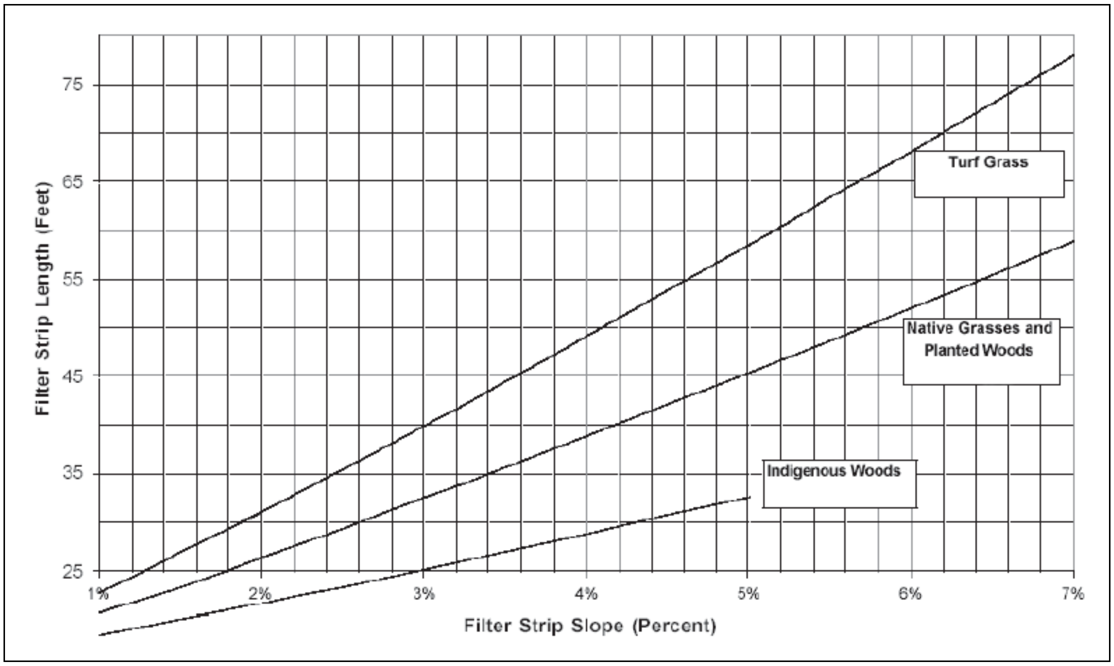


Figure 0.1 - Filter Strip Length – Sand, HSG A (PADEP, 2006)

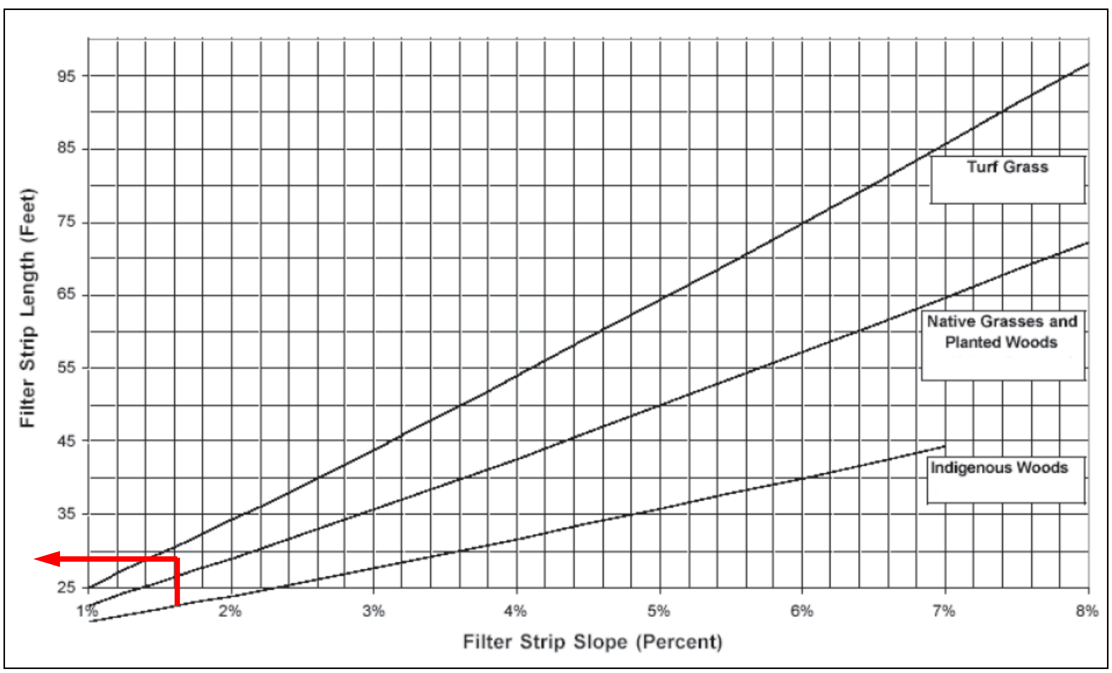


Figure 0.2 - Filter Strip Length – Sandy Loam, HSG B (PADEP, 2006)

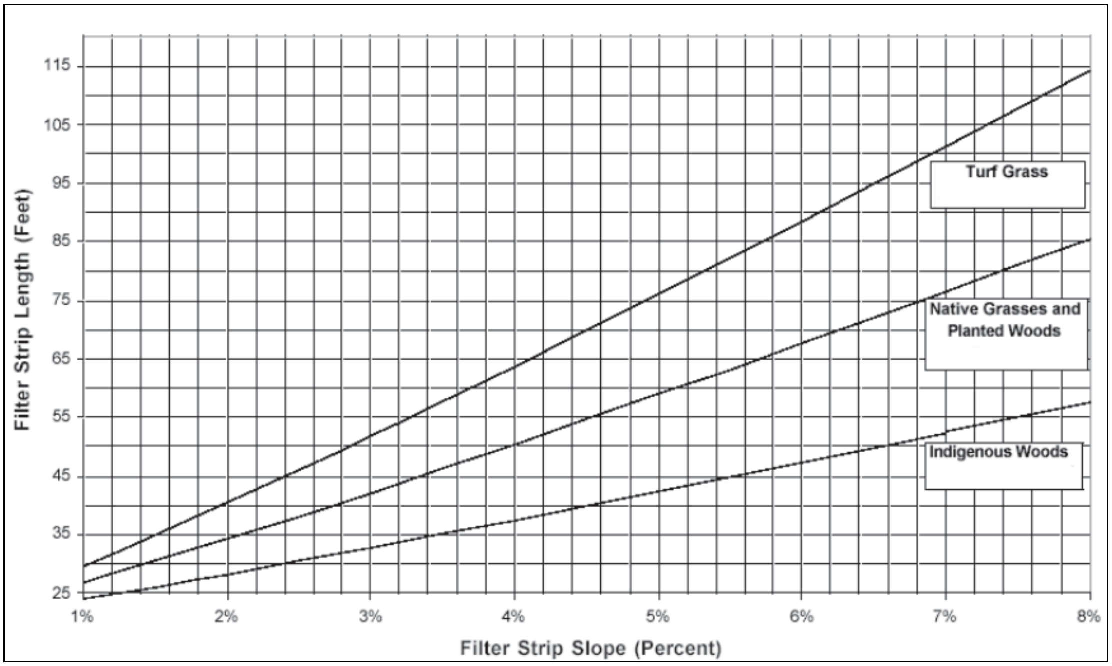


Figure 0.3 - Filter Strip Length – Loam / Silt Loam, HSG B (PADEP, 2006)

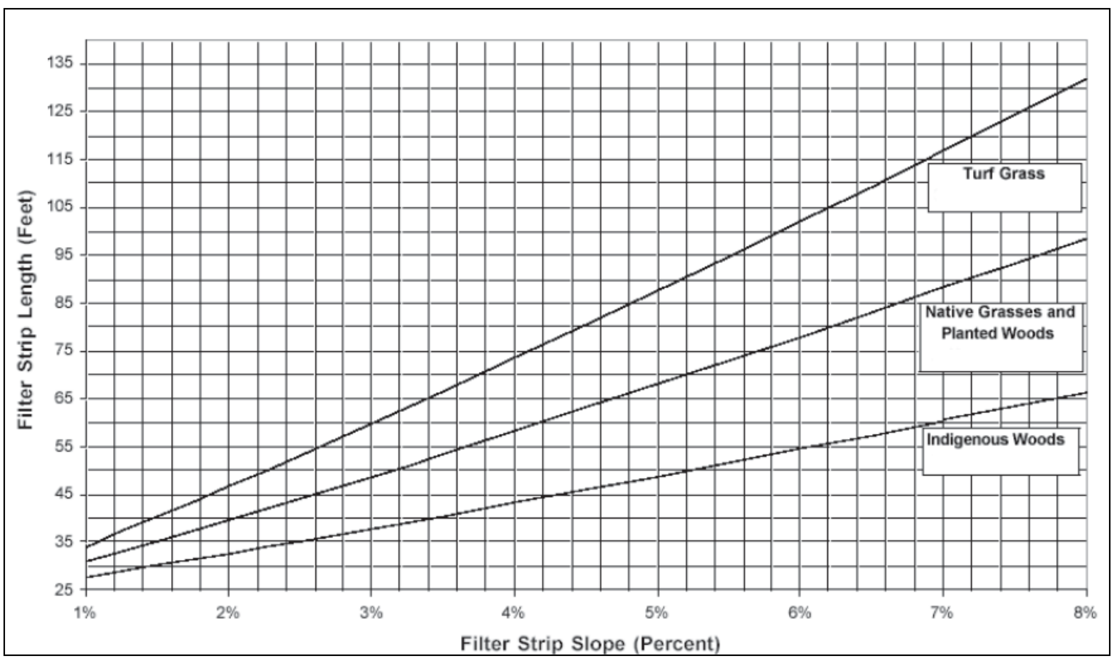


Figure 0.4 - Filter Strip Length – Sandy Clay Loam, HSG C (PADEP, 2006)

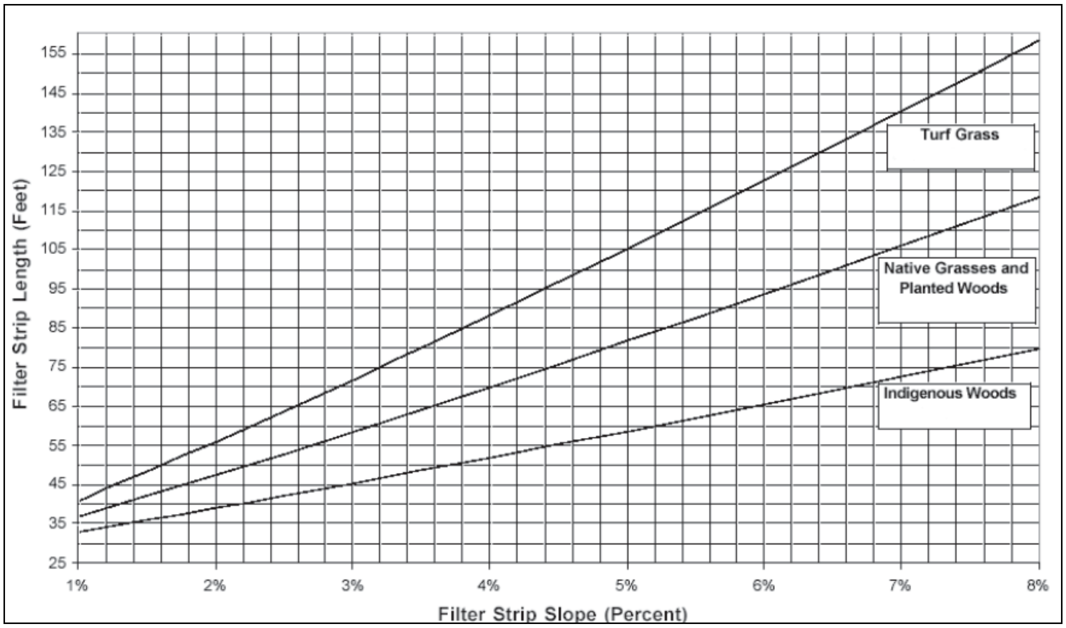


Figure 0.5 - Filter Strip Length – Clay Loam / Silty Clay / Clay, HSG D (PADEP, 2006)

Figure 7.2 provides an estimated filter strip length of 29'. It should be noted that this is a short strip, whose estimated length is largely a function of the relatively high permeability rates exhibited by sandy loams categorized as HSG *B*. While the filter strip may be able to infiltrate a large portion of its received runoff under ideal conditions, conservative design practice will size the strip to provide effective hydraulic residence times even when antecedent moisture conditions are such that the underlying soils are in a near-saturated condition. This sizing procedure is discussed in the next steps. The estimated strip length of 29' is the absolute minimum length that should be considered for this example.

Step 3 - Estimate the Peak Rate of Runoff Corresponding to the Water Quality Volume

A detailed filter strip design requires that the design discharge onto the strip be known. The length of the strip can then be sized to accommodate this discharge while providing the desired hydraulic residence time. The site's water quality volume was computed previously as 1,361 ft^3. The peak volumetric rate of discharge which generates this runoff volume can be estimated by examining the basic Rational Method hydrograph shape shown in Figure 7.6.

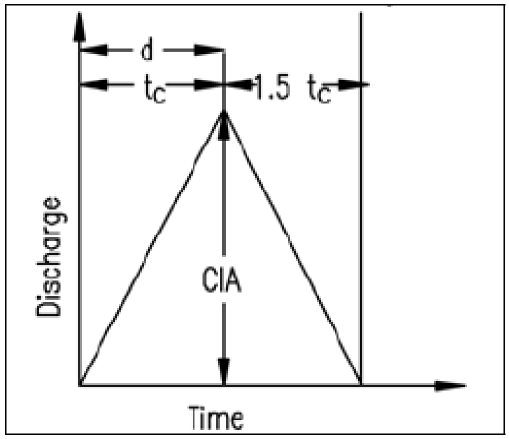


Figure 0.6. Basic Rational Hydrograph Shape
Virginia Stormwater Management Handbook, (DCR/DEQ, 1999)

The time of concentration is known to be 8 minutes. Therefore, the “base” of the triangular shaped hydrograph is 20 minutes (1,200 seconds). The total area under the hydrograph is the water quality volume (1,361 ft^3). Therefore, employing the area relationship of a triangle, the lone unknown, Q, is computed as follows:

$$A = \left(\frac{1}{2}\right) \times b \times h$$

$$h = \frac{2A}{b}$$

$$Q = \frac{(2)(1{,}361\,ft^3)}{1{,}200s} = 2.3cfs$$

The water quality volume from the new 0.75 acre impervious development generates an estimated peak discharge of 2.3 cfs. This value is now used to size the strip.

Step 4 - Compute the Strip Length (Flow Direction)

Runoff will enter onto the strip from a level spreader. The size of the level spreader is a function of the 10-year flow from the contributing drainage area. The required level spreader dimensions are shown in Table 7.4.

Table 0.3 - Minimum Level Spreader Dimensions
Virginia Erosion and Sediment Control Handbook (DCR/DEQ, 1992)

Q10 (cfs)	Depth (ft)	Width of Lower Side Slope of Spreader (ft)	Length (ft)
0-10	0.5	6	10
20-10	0.6	6	20

The 10-year peak rate of runoff from the roadway is 4.3 cfs. Therefore, the minimum level spreader "lip" length that will discharge runoff onto the strip is 10'.

In order to assure that the minimum five minute hydraulic residence time is achieved, the length of the strip (in the direction of flow) must be sized as a function of the anticipated flow velocity on the strip.

Flow velocity is computed by the Manning's equation. A Manning roughness coefficient of 0.20 is typically used in grass filter strip flow calculations. If the filter strip is mowed infrequently, a roughness coefficient of 0.24 may be used. (FHWA, 1996, pg 325; also, Horner, 1993). This Manning roughness coefficient is derived from employing the anticipated flow velocity and flow depth on the filter strip. Manning's n values for various categories of vegetative ground covers are presented in Table 7.5.

Table 0.4 - Recommended Manning's *n* Values for Overland Flow

Surface	Recommended Value	Range of Values
Range (natural)	0.13	0.01-0.32
Range (clipped)	0.08	0.02-0.24
Grass (bluegrass sod)	0.45	0.39-0.63
Short Grass Prairie	0.15	0.10-0.20
Dense Grass	0.24	0.17-0.30
Bermuda Grass	0.41	0.30-0.48

Source: Mays, Larry W. Water Resources Engineering. John Wiley & Sons, Inc. New York, NY, 2001.

By the principal of continuity, flow on the strip can be expressed as:

$$Q = V \times W \times h$$

Q = volumetric flow rate (cfs)
V = average flow velocity on the strip (fps)
W= strip width (ft)
h= flow depth on the strip (ft)

For shallow overland flow, the anticipated flow depth is assumed equal to the hydraulic radius. Expressing flow in terms of the Manning's equation, the previous expression becomes:

$$Q = \frac{1.49}{n} \times h^{\frac{2}{3}} \times S^{\frac{1}{2}} \times (W \times h)$$

n = Manning roughness coefficient
S= filter strip slope (ft/ft)
other terms as previously defined

This equation can then be rearranged to isolate the desired unknown, *h.*

$$\frac{Q}{W} = \frac{1.49}{n} \times h^{\frac{5}{3}} \times S^{\frac{1}{2}}$$

At this stage in the design, the filter strip width is unknown. Therefore, an assumption must be made and its adequacy later verified. We will assume a filter strip width of 25'. Then, solving for h:

$$\frac{2.3}{25} = \frac{1.49}{0.20} \times h^{\frac{5}{3}} \times 0.02^{\frac{1}{2}}$$

h = 0.23'

Employing the previously established parameters, flow velocity on the strip is computed as follows:

$$V = \frac{1.49}{n} R_h^{\frac{2}{3}} S^{\frac{1}{2}}$$

V = velocity (fps)
n = Manning roughness coefficient
R_h= hydraulic radius (ft, equal to flow depth for shallow overland flow)
S= filter strip slope (ft/ft)

$$V = \frac{1.49}{0.20}(0.23)^{\frac{2}{3}}(0.02)^{\frac{1}{2}} = 0.395\frac{ft}{s}$$

Next, the filter strip length can be computed as a function of this flow velocity and the target hydraulic residence time. First, the minimum residence time of five minutes is considered:

$$L = t \times V$$

L = filter strip length (ft)
t = target hydraulic residence time (sec)
V= flow velocity (fps)

$$L = 5\min \times \frac{60\sec}{\min} \times 0.395\frac{ft}{\sec} = 119\,ft$$

It is again noted that this approach does not consider that a portion of the water quality volume will infiltrate into the strip's subsoil. Additionally, the accumulation of flow depth and subsequent decrease in velocity is not considered. Therefore, the computed length of 119' reflects a conservative design which can reasonably be assumed to provide a hydraulic residence time in excess of the minimum value of five minutes.

Step 5 - Verify Adequacy of the Assumed Strip Width (Perpendicular to Flow Direction)

The *minimum* width of the filter strip should be the greater of the two values:

0.2 x Filter Length

or

8'

Therefore, the minimum strip width is computed as follows:

$$0.2 \times 119\,ft = 23.8\,ft$$

The assumed strip width of 25' is therefore adequate.

Ideally, the filter strip width will equal the width of the contributing drainage area. When a level spreader is used, as in this example, the lip of the spreader must extend to within *a minimum* of 10' of the filter strip on each end (*Virginia Stormwater Management Handbook, (DCR/DEQ, 1999)*. The proposed level spreader lip is 10' in length. Therefore the spreader extends to within 7.5' of the filter edges (see calculation below):

$$\frac{25\,ft - 10\,ft}{2} = 7.5\,ft$$

If this value was found to exceed 10' the level spreader length would need to be increased.

Step 6 - Selection of Vegetation

Filter strips must be constructed of dense, soil-binding deep rooted water-resistant plants. If a grass filter strip is to be employed, a dense turf is necessary to achieve desirable pollutant removal percentages while avoiding erosion. If turf grass is used, the height shall be maintained between 2" and 4". The specific species of vegetation should be appropriate for the climatic conditions and expected maintenance.

Filter strips should be planted with a minimum of two of the following vegetation types, per the *Virginia Stormwater Management Handbook (DCR/DEQ, 1999)*:

- deep-rooted grasses, ground covers, or vines
- deciduous and evergreen shrubs
- under-and over-story trees

The choice of planting species should be largely based on the project site's physiographic zone classification. Additionally, the selection of plant species should match the native plant species as closely as possible. Surveying a project site's native vegetation will reveal which plants have adapted to the prevailing hydrology, climate, soil, and other geographically-determined factors. Figure 3.05-4 of the *Virginia Stormwater Management Handbook* provides guidance in plant selection based on project location. All chosen plant species should conform to the *American Standard for Nursery Stock*, current issue, and be suited for USDA Plant Hardiness Zones 6 or 7, see Figure 7.7 on the following page.

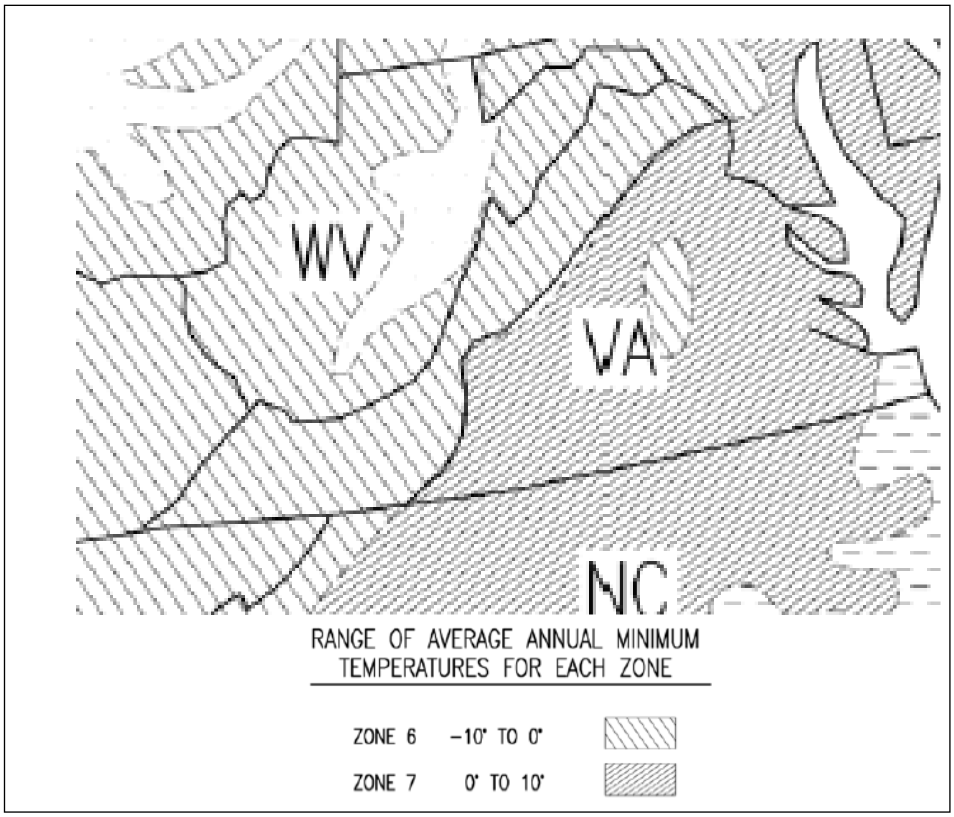


Figure 0.7 - USDA Plant Hardiness Zones

The presences of trees, shrubs, and other woody vegetation can further increase the water quality performance of vegetated filter strips. In addition to intercepting a portion of stormwater before it even reaches the ground, trees and shrubs increase the infiltration and retention present in the filter strip. However, when trees are incorporated into the filter strip design, one must be aware that the overall density of vegetation is decreased. Consequently, while filter strips with trees and other woody vegetation can demonstrate higher pollutant removal efficiencies than their strictly grass counterparts, they require that the filter strip be longer in length to account for the reduced vegetation density. Additionally, tree and shrub trunks have the potential to support the development of gullies and channels in the strip. To offset this phenomenon, filter strips equipped with trees and shrubs should be designed with flatter slopes than those employing only grass.

8.1 Infiltration Trench - Overview of Practice

Infiltration trenches are shallow trenches equipped with an underground reservoir comprised of coarse stone aggregate. The void space created by the aggregate provides storage for surface runoff that has been diverted into the trench. This runoff then infiltrates into the surrounding soil, through the bottom and sides of the trench.

Infiltration trenches act primarily as water quality BMPs; however, when equipped with underground piping, the temporary storage volume of the trench may be increased to a volume that provides peak runoff rate reduction for the one and two year return frequency storms. Peak rate control of the 10-year and greater storm events is typically beyond the capacity of an infiltration practice.

8.2 Site Constraints and Siting of the Facility

The designer must consider a number of site constraints in addition to the contributing drainage area's new impervious cover when an infiltration trench is proposed. These constraints are discussed as follows.

8.2.1 Minimum Drainage Area

The minimum drainage area contributing to an infiltration trench is not restricted. Infiltration trenches are particularly well suited to small drainage areas.

8.2.2 Maximum Drainage Area

The maximum drainage area to a single infiltration trench should be restricted to no more than five acres. Multiple trenches may be employed to receive runoff from larger drainage areas; however, when considering required trench maintenance, the implementation of multiple infiltration trenches is often undesirable.

8.2.3 Site Slopes

Infiltration trenches are suitable for installation on sites exhibiting slopes generally less than 20%. Infiltration trenches should be located a minimum of 50' away from any slope steeper than 15%. When site slopes exceed 20%, alternative BMP measures should be considered.

8.2.4 Site Soils

The soil infiltration rate is a critical design element of an infiltration trench. When such a facility is proposed, *a subsurface analysis and permeability test is required*. The required subsurface analysis should investigate soil characteristics to a depth of no less than 3' below the proposed bottom of the stone trench. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils.

The soil's infiltration rate should be measured when the soil is in a saturated condition. Soil infiltration rates which are deemed acceptable for infiltration trenches range between *0.52 and 8.27 in/hr* (DCR/DEQ, 1999, Et Seq.). Infiltration rates falling within this range are typically exhibited by soils categorized as loam, sandy loam, and loamy sand.

Soils exhibiting a clay content of greater than 30% are unacceptable for infiltration facilities. Similarly, soils exhibiting extremely high infiltration rates, such as sand, should also be avoided. Table 8.1 presents typical infiltration rates observed for a variety of soil types. This table is provided as a reference only, and does not replace the need for a detailed site soil survey.

Table 0.1 – DCR/DEQ Hydrologic Soil Properties Classified by Soil Texture (*Virginia Stormwater Management Handbook, 1999*)

Texture Class	Effective Water Capacity (C_w) (inch per inch)	Minimum Infiltration Rate (f) (inch per hour)	Hydrologic Soil Grouping
Sand	0.35	8.27	A
Loamy Sand	0.31	2.41	A
Sandy Loam	0.25	1.02	B
Loam	0.19	0.52	B
Silt Loam	0.17	0.27	C
Sandy Clay Loam	0.14	0.17	C
Clay Loam	0.14	0.09	D
Silty Clay Loam	0.11	0.06	D
Sandy Clay	0.09	0.05	D
Silty Clay	0.09	0.04	D
Clay	0.08	0.02	D

8.2.5 Depth to Water Table

Infiltration trenches should not be installed on sites with a high groundwater table. Inadequate separation between the trench bottom and the surface of the water table may result in contamination of the water table. This potential contamination arises from the inability of the soil surrounding the trench to filter pollutants prior to their entrance into the water table. Additionally, a high water table can flood an infiltration trench and render it inoperable during periods of high precipitation and/or runoff. A separation distance of no less than 2’ is required between the bottom of an infiltration trench and the surface of the *seasonally* high water table. Unique site conditions may arise which require an even greater separation distance. The separation distance provided should allow the trench to empty completely within a maximum of 48 hours following a runoff producing event.

8.2.6 Separation Distances

Infiltration trenches should be located at least 20’ down-slope and at least 100’ up-slope from building foundations. Infiltration trenches should not be located within 100’ of any water supply well. Local health officials should be consulted when the implementation of an infiltration trench is proposed within the vicinity of a septic drainfield.

8.2.7 Bedrock

A minimum of 2’ of separation is required between the bottom of an infiltration trench and bedrock, with 4’ or greater recommended.

8.2.8 Placement on Fill Material

Infiltration trenches should not be constructed on or nearby fill sections due to the possibility of creating an unstable subgrade. Fill areas are vulnerable to slope failure along the interface of the in-situ and fill material. The likelihood of this type of failure is increased when the fill material is frequently saturated, as anticipated when an infiltration BMP is proposed.

8.2.9 Karst

The concentration of runoff into an infiltration trench may result in the formation of flow channels. Such channels may lead to collapse in karst areas, and therefore the implementation of infiltration trenches in known karst areas should be avoided.

8.2.10 Existing Utilities

Infiltration trenches can often be constructed over existing easements, provided permission to construct the strip over these easements is obtained from the utility owner *prior* to design of the strip.

8.2.11 Wetlands

When the construction of an infiltration trench is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation in their vicinity.

8.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing an infiltration trench for improvement of water quality.

8.3.1 Design Infiltration Rate

To provide a factor of safety, and to account for the decline in performance as the facility ages, the soil infiltration rate upon which a trench design is founded should be one-half the infiltration rate obtained from the geotechnical analysis.

8.3.2 Maximum Storage Time

Infiltration trenches should be designed to empty within 48 hours following a runoff producing event.

8.3.3 Trench Sizing

Generally, the trench's total depth ranges from 2' to 10'. The surface area of the trench is that area which, when multiplied by the trench depth and the aggregate porosity, provides the computed treatment volume. Trench widths greater than 8' require large excavation equipment rather than smaller trenching equipment. When treatment volumes require a width greater than 8', an infiltration basin or other BMP should be considered.

8.3.4 Runoff Pretreatment

Infiltration trenches *must* be preceded by a pretreatment facility. Roadways and parking lots often produce runoff with high levels of sediment, grease, and oil. These pollutants can potentially clog the pore space in the trench, thus rendering its infiltration and pollutant removal performance ineffective. Suitable pretreatment practices include vegetated buffer strips, sediment forebays, and proprietary water quality inlets.

All infiltration trenches that receive surface runoff as sheet flow should be equipped with a vegetated buffer strip at least 20' wide (see *Section Seven – Vegetated Filter Strip)*.

8.3.5 Aggregate Material

The infiltration trench material should be comprised of clean aggregate with a maximum diameter of 3.5" and a minimum diameter of 1.5". Aggregate meeting this specification should be VDOT No. 1 Open-graded Coarse Aggregate or its equivalent as recommended by the Materials Division.

An 8" deep sand layer must be installed at the bottom of the trench. This material should be VDOT Fine Aggregate, Grading A or B, or equivalent as approved by the Materials Division.

8.3.6 Observation Well

An observation well is recommended at an interval of every 50' along the entire trench length. Observation wells provide a means by which dewatering times can be observed to ensure that the trench is emptying within the maximum allowable time of 48 hours. Generally, the observation well is constructed of 4" or 6" perforated PVC pipe, configured as shown in Figure 8.1

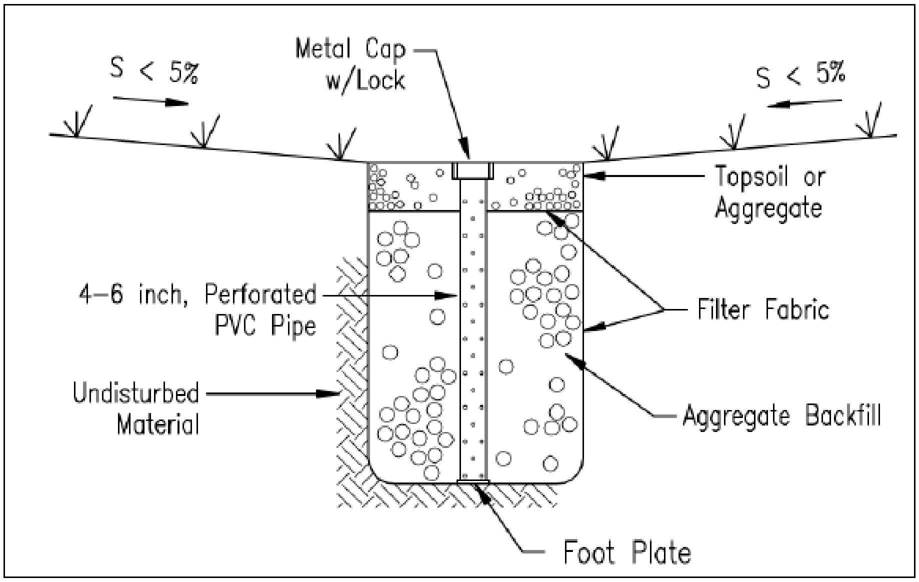


Figure 0.1 – DCR/DEQ Infiltration Trench Observation Well Configuration
(*Virginia Stormwater Management Handbook, 1999, Et seq.)*

8.3.7 Filter Fabric

The trench aggregate material should be surrounded with filter fabric as shown in Figure 8.2. The filter fabric should be a material approved by the Materials Division. Filter fabric should *not* be placed on the trench bottom. When the trench is constructed as a "surface trench" with no soil overlay, a separate piece of filter fabric should be used as the top layer. This enables replacement of the upper filter fabric upon its eventual clogging.

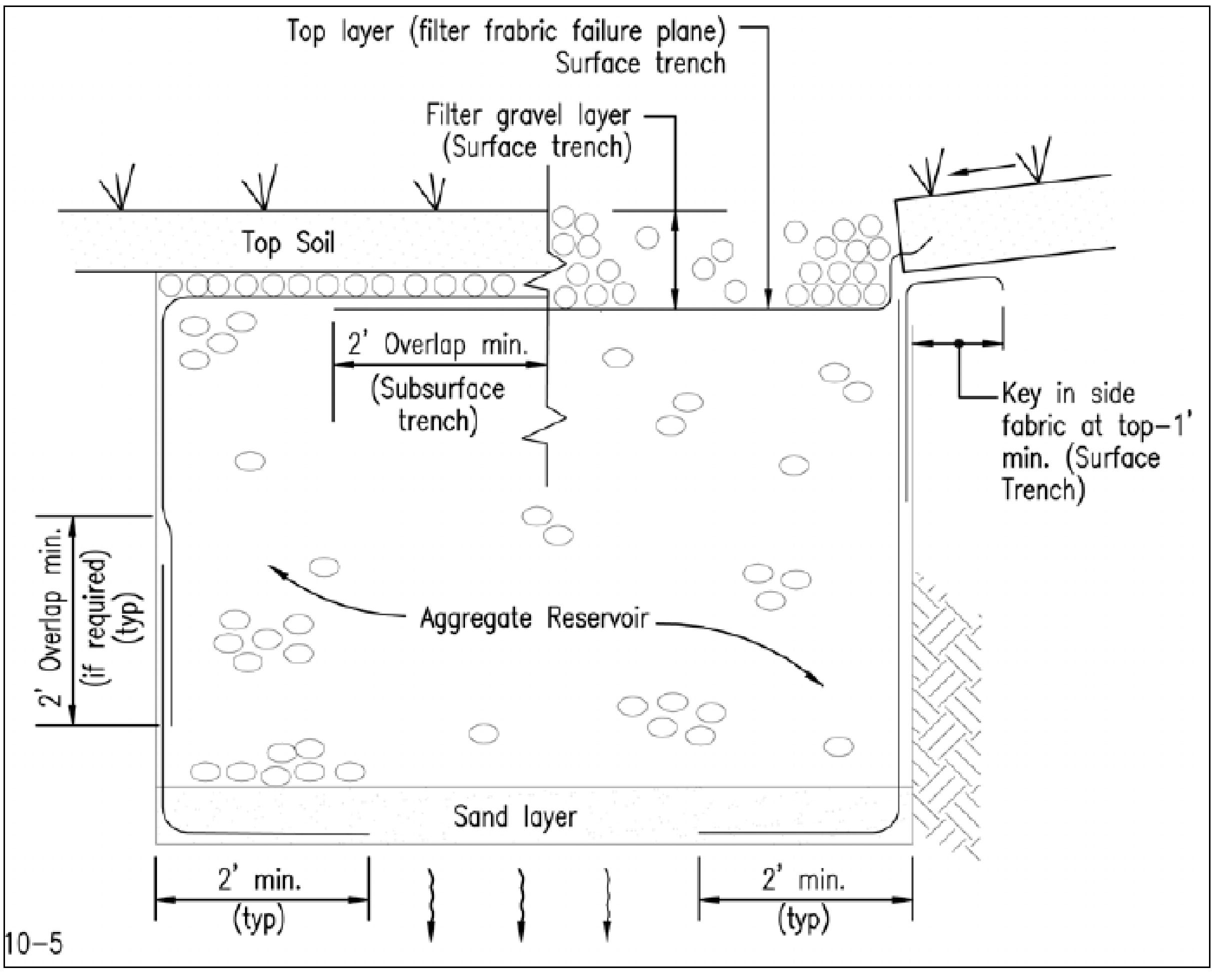


Figure 0.2 – DCR/DEQ Infiltration Trench Filter Fabric Installation
(*Virginia Stormwater Management Handbook, 1999, Et seq.)*

8.4 Design Process

This section presents the design process applicable to infiltration trenches serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered during linear development projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 4 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et Seq.) for expanded hydrologic methodology.

The infiltration trench design will meet the technology-based water quality requirements arising from the construction of approximately 2,000 LF of roadway in Halifax County. Topography is such that runoff from the road is collected in VDOT CG-6 curb and gutter and conveyed to curb inlets along the road. The runoff is then discharged into sediment forebays from which it then enters onto the surface of the proposed trench, which is located in the median of the divided roadway. The total project site, including right-of-way and all permanent easements, consists of 6.2 acres. Pre and post-development hydrologic characteristics are summarized below in Table 8.2. Approximately 300 LF is available for construction of the trench. Geotechnical investigations reveal the site's saturated soil infiltration rate to be 2.3 in/hr. The project site does not exhibit a high or seasonally high groundwater table.

Table 8.4.1 - Hydrologic Characteristics of Example Project Site

	Pre-Development	**Post-Development**
Project Area (acres)	6.2	6.2
Land Cover	Unimproved Grass Cover	3.4 acres *new* impervious cover
Impervious Percentage	0	54.8

Step 1 - Compute the Required Water Quality Volume

The project site's water quality volume is a function of the developed new impervious area. This basic water quality volume is computed as follows:

$$WQV = \frac{NIA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

NIA= New Impervious Area (ft^2)

The project site in this example has a total drainage area of 6.2 acres. The total new impervious area within the site is 3.4 acres. Therefore, the water quality volume is computed as follows:

$$WQV = \frac{3.4ac \times \frac{1}{2}in \times \frac{43{,}560\,ft^2}{ac}}{12\frac{in}{ft}} = 6{,}171\,ft^3$$

The new impervious cover within the project site is less than 67% of the total project site. Therefore, in accordance with Table 1.1, the infiltration trench will be sized to treat the computed water quality volume of 6,171 ft^3.

Step 2 - Compute the Design Infiltration Rate

Per DEQ guidelines, the design infiltration rate, f_d, is computed as one-half the infiltration rate obtained from the required geotechnical analysis. For the given site conditions, the infiltration rate is computed as:

$$f_d = 0.5f = (0.5)\left(2.3\frac{in}{hr}\right) = 1.15\frac{in}{hr}$$

Step 3 - Compute the Maximum Allowable Trench Depth

The trench must be designed such that it is completely empty within a maximum of 48 hours following a runoff producing event. To ensure compliance with this requirement, we will compute the maximum allowable trench depth by the following equation:

$$d_{max} = \frac{f_d \times T_{max}}{V_r}$$

d_{max} = maximum allowable trench depth (ft)
f_d = design infiltration rate (in/hr)
T_{max} = maximum allowable drain time (48 hours)
V_r = void ratio of the stone trench (0.40 for VDOT No. 1 Coarse-graded Aggregate)

The maximum allowable trench depth is therefore computed as:

$$d_{max} = \frac{\left(1.15\frac{in}{hr}\right)\left(\frac{1ft}{12in}\right)(48hrs)}{0.40} = 11.5\,ft$$

Step 4 - Compute the Minimum Allowable Trench Bottom Area

Employing the principles of Darcy's Law, and assuming one-dimensional flow through the bottom of the trench, we can compute the minimum allowable surface area of the trench by the following equation:

$$SA_{\min} = \frac{WQV}{(f_d)(T_{\max})}$$

SA_{min} = minimum trench bottom surface area (ft^2)
WQV = treatment volume (ft^3)
f_d = design infiltration rate (in/hr)
T_{max} = maximum allowable drain time (48 hours)

The minimum allowable trench surface area is computed as follows:

$$SA_{\min} = \frac{6{,}171\,ft^3}{\left(1.15\,\frac{in}{hr}\right)\left(\frac{1\,ft}{12in}\right)(48hr)} = 1{,}342\,ft^2$$

Step 5 - Size the Trench Based on Site-Specific Parameters

The example trench is to be located in the median of a divided highway. Per the problem statement, approximately 300 LF are available for construction of the trench. This entire length will be utilized in an effort to minimize the trench depth.

The maximum desirable trench width is 8'. Employing this maximum width with the available 300' length results in a trench bottom surface area computed as follows:

$$SA = (300\,ft)(8\,ft) = 2{,}400\,ft^2$$

This value is greater than the minimum value (computed previously as 1,342 ft^2), and is therefore considered acceptable.

Next, the trench depth must be computed. The volume of storage provided in the void space of the trench aggregate must provide the computed treatment volume. Therefore, the minimum trench depth is computed by the following equation, with variables as previously defined.

$$d = \frac{WQ_V}{(V_r)(SA)}$$

The trench depth is then computed as:

$$d = \frac{6{,}171\,ft^3}{(0.4)(2{,}400\,ft^2)} = 6.43\,ft$$

The computed trench depth is less than the maximum value (computed previously as 11.5’), and is therefore considered acceptable.

A summary of the trench parameters are provided in Table 8.3.

Table 8.4.2 - Summary of Trench Dimensions

Length	300’
Width	8’
Depth	6.5’
Storage Volume	6,240 ft^3

Step 6 - Alternative Trench Sizing Procedure

The addition of a large perforated pipe(s) within the trench can greatly increase the trench storage capacity. This increased storage capacity can be used to reduce the overall dimensions of the trench, or, keeping the trench size fixed, provide a greater overall infiltration volume. The following steps illustrate the procedure for decreasing the trench depth by providing perforated corrugated metal pipes within the trench. The demonstrated methodology can also be adapted to resize the trench length and/or depth.

In this example, we will consider placement of two 36” perforated corrugated metal pipes within the trench. Assuming the pipes extend the full length of the trench, we can compute the total volume provided by the pipes as follows:

$$V_{Pipe} = L \times \pi \times r^2$$

$$V_{Pipe} = \left[300 \times \pi \times 1.5^2\right] \times 2Pipes = 4{,}242\,ft^3$$

The volume provided by the stone aggregate to be replaced by the pipes is computed as:

$$V_{Stone} = 4{,}242\,ft^3 \times 0.4 = 1{,}696.8\,ft^3$$

Therefore, the net “gain” in storage volume by replacing the aggregate with the pipes is computed as:

$$V_{Net} = 4{,}242\,ft^3 - 1{,}696.8\,ft^3 = 2{,}545.2\,ft^3$$

The reduction in trench depth can then be computed as a function of the net gain in storage volume and the trench's length and width:

$$D_{\text{Re}\,duction} = \frac{2{,}545.2\,ft^3}{300\,ft \times 8\,ft \times 0.4} = 2.65\,ft$$

The new trench depth is computed as:

$$D = 6.43\,ft - 2.65\,ft = 3.8\,ft$$

The overall volume provided by the re-sized trench is then computed as:

$$V_{Trench} = 300\,ft \times 8\,ft \times 3.8\,ft \times 0.4 = 3{,}648\,ft^3$$

This volume is then added to the net gain in volume provided by the two 36" diameter pipes:

$$V_{Total} = 3{,}648\,ft^3 + 2{,}545\,ft^3 = 6{,}193\,ft^3$$

A schematic illustration of the re-sized trench is shown in Figure 8.3.

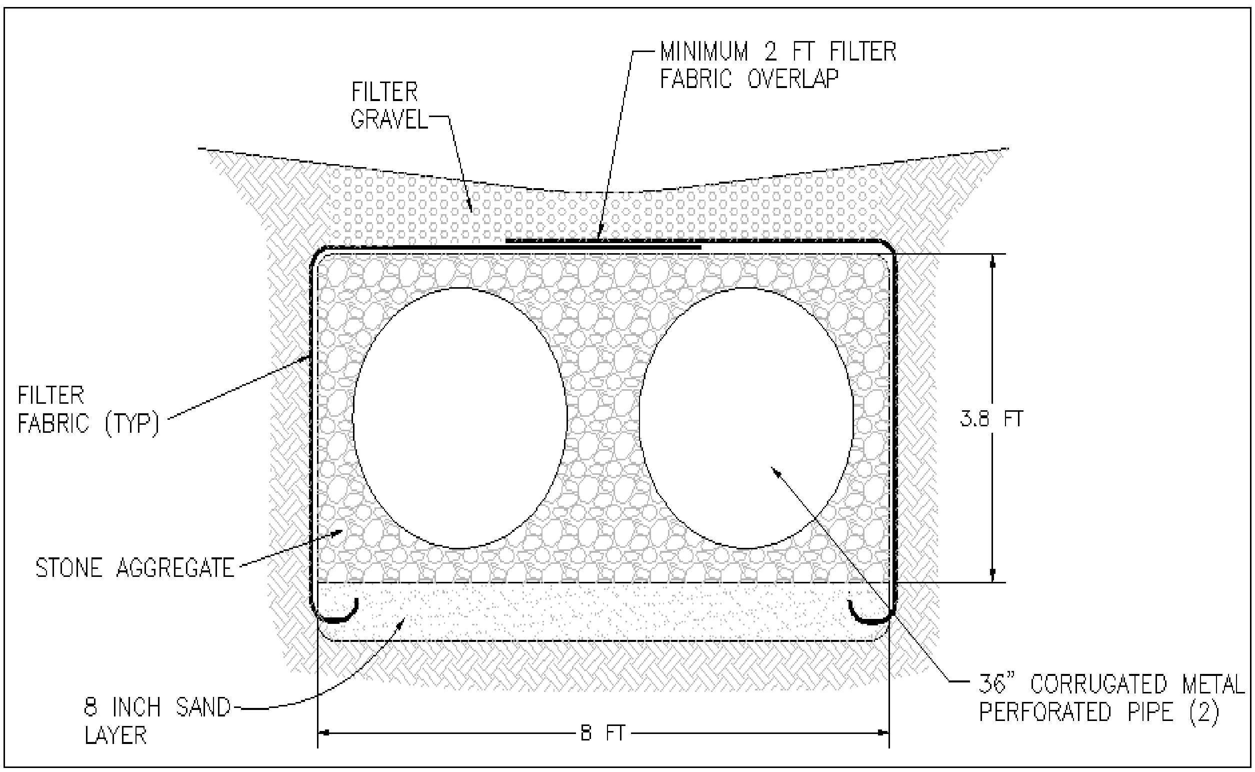


Figure 8.4.1 - Infiltration Trench Equipped with Perforated Pipes

Step 7 - Provide Provision for Overflow

Infiltration trenches serve primarily as water quality BMPs. Typically, it is impractical to size the trench to accommodate a volume of runoff beyond that which must be captured for water quality purposes. Therefore, provisions must be provided for runoff conveyance when the capacity of the trench is exceeded. Because of the small drainage area served by an infiltration trench, an emergency spillway is typically not required; however, a non-erosive channel or storm sewer system must be located at the downstream end of the trench. The channel or sewer should carry excess flows to an adequate receiving channel as defined by Regulation MS-19 in the *Virginia Erosion and Sediment Control Handbook*, (DCR/DEQ, 1992, Et seq.). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

When a storm sewer or other conduit is used to convey excess runoff, the invert must be located at an elevation that is not below the surface of the infiltration trench's aggregate storage volume. *Only the volume of storage provided below the invert of the bypass pipe can be considered infiltration (treatment) volume.* A typical bypass configuration is shown below in Figure 8.4.

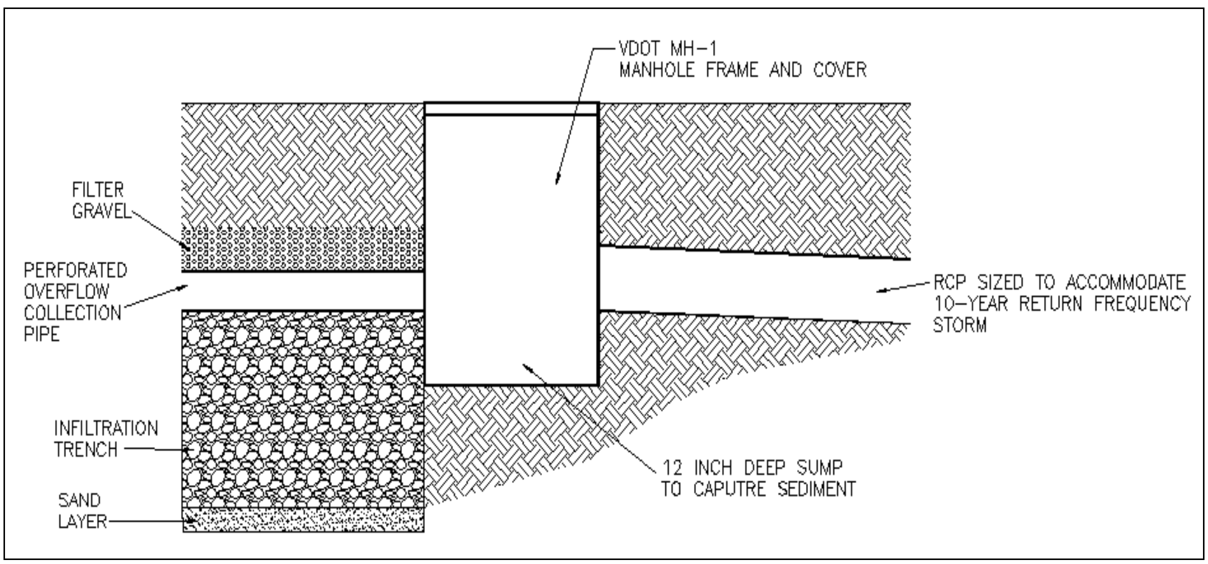


Figure 8.4.2 - Infiltration Trench Section Equipped with RCP Overflow Pipe

Step 8 - Landscaping

Trenches that are not designed to function as a surface trench (as shown in Figure 8.2) must exhibit a dense vegetative cover before any stormwater runoff is directed to the facility. Careful attention must be given to the types of vegetation selected for the trench surface. The vegetative species must be selected based on their inundation tolerance and the anticipated frequency and depth of inundation. The designer is referred to the *Virginia Erosion and Sediment Control Handbook* (DCR/DEQ, 1992, Et seq.) for recommendations of specific vegetative species based on the facility's geographic location. Generally, low-growing stoloniferous grasses are good candidates for infiltration facilities as they permit long intervals between mowing, thus minimizing the frequency of traffic on the surface of the facility.

Maintenance of the facility's vegetative cover is essential to the long-term performance of the facility. A dense vegetative stand enhances infiltration, minimizes surface erosion, and deters invasive and detrimental vegetative species. Any bare spots on the surface of the facility should be re-seeded immediately.

The use of fertilizers should be minimized and avoided completely if practically possible. Excessive use of fertilizers on highly permeable soil may lead to groundwater contamination. Reference the *Virginia Erosion and Sediment Control Handbook* (DCR/DEQ, 1992, Et seq.) for recommendations on appropriate fertilizer types and minimum effective application rates.

9.1 Infiltration Basin - Overview of Practice

Infiltration basins are impounding facilities which temporarily store surface runoff and infiltrate a designated portion of it into the soil strata.

Unlike infiltration trenches, infiltration basins may also serve as peak mitigation facilities. This is accomplished by providing “dry” storage above the designated infiltration volume. This dry, flood control volume is then released through a multi-stage riser and barrel system. Conceptually, an infiltration basin can be viewed as an extended dry detention basin whose water quality volume is infiltrated into the soil strata rather than released through a small orifice over a 30-hour period.

As shown in Table 1.1, the water quality volume of an infiltration trench can vary, and the anticipated pollutant removal performance of the trench varies as a function of this volume.

9.2 Site Constraints and Siting of the Facility

The designer must consider a number of site constraints in addition to the contributing drainage area's new impervious cover when an infiltration basin is proposed. These constraints are discussed as follows.

9.2.1 Minimum Drainage Area

The minimum drainage area contributing to an infiltration trench is not restricted. However, when contributing drainage areas are particularly small, infiltration trenches will often provide a more cost-effective option.

9.2.2 Maximum Drainage Area

The drainage area contributing runoff to an infiltration basin should be restricted to no more than 50 acres.

9.2.3 Site Slopes

Infiltration basins are suitable for installation on sites exhibiting slopes generally less than 20%. Infiltration basins should be located a minimum of 50' away from any slope steeper than 15%. When site slopes exceed 20%, alternative BMP measures should be considered. The floor slope of an infiltration basin should be as flat as practically possible in order to maximize the area upon which effective infiltration can occur.

9.2.4 Site Soils

When an infiltration basin is proposed the soil infiltration rate is of critical design importance. *A subsurface analysis and permeability test is required*. The required subsurface analysis should investigate soil characteristics to a depth of no less than 3' below the proposed bottom of the basin. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils.

The soil's design infiltration rate should be measured when the soil is in a saturated condition. Soil infiltration rates which are deemed acceptable for infiltration trenches range between *0.52 and 8.27 in/hr* (DCR/DEQ, 1999, Et Seq.). Infiltration rates falling within this range are typically exhibited by soils categorized as loam, sandy loam, and loamy sand.

Soils exhibiting a clay content of greater than 30% are unacceptable for infiltration facilities. Similarly, soils exhibiting extremely high infiltration rates, such as sand, should also be avoided. Table 9.1 presents typical infiltration rates observed for a variety of soil types. This table is provided as a reference only, and does not replace the need for a detailed site soil survey.

Table 9.2.1DCR/DEQ Hydrologic Soil Properties Classified by Soil Texture
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

Texture Class	Effective Water Capacity (C_w) (inch per inch)	Minimum Infiltration Rate (f) (inch per hour)	Hydrologic Soil Grouping
Sand	0.35	8.27	A
Loamy Sand	0.31	2.41	A
Sandy Loam	0.25	1.02	B
Loam	0.19	0.52	B
Silt Loam	0.17	0.27	C
Sandy Clay Loam	0.14	0.17	C
Clay Loam	0.14	0.09	D
Silty Clay Loam	0.11	0.06	D
Sandy Clay	0.09	0.05	D
Silty Clay	0.09	0.04	D
Clay	0.08	0.02	D

9.2.5 Depth to Water Table

Infiltration basins should not be installed on sites with a high groundwater table. Inadequate separation between the basin bottom and the surface of the water table may result in contamination of the water table. This potential contamination arises from the inability of the soil surrounding the trench to filter pollutants prior to their entrance into the water table. Additionally, a high water table may flood an infiltration basin during periods of high precipitation and/or runoff. A minimum separation distance of no less than 2’ is required between the bottom of an infiltration basin and the surface of the *seasonally* high water table, with four or more feet of separation preferred. Unique site conditions may arise which require an even greater separation distance. The separation distance provided should allow the basin to empty completely within a maximum of 48 hours following a runoff producing event.

9.2.6 Separation Distances

Infiltration basins should be located at least 20’ down-slope and at least 100’ up-slope from building foundations. Infiltration basins should not be located within 100’ of any water supply well. Local health officials should be consulted when the implementation of an infiltration basin is proposed within the vicinity of a septic drainfield.

9.2.7 Bedrock

A minimum of 2’ of separation is required between the bottom of an infiltration basin and bedrock, with 4’ or greater recommended.

9.2.8 Placement on Fill Material

Infiltration basins should not be constructed on or nearby fill sections due to the possibility of creating an unstable subgrade. Fill areas are vulnerable to slope failure along the interface of the in-situ and fill material. The likelihood of this type of failure is increased when the fill material is frequently saturated, as anticipated when an infiltration BMP is proposed. Additionally, construction traffic and compaction activities will generally result in fill material exhibiting an infiltration rate below that which is desirable for an infiltration facility.

9.2.9 Karst

The concentration of runoff into an infiltration facility may result in the formation of flow channels. Such channels may lead to collapse in karst areas, and therefore the implementation of infiltration basins in known karst areas should be avoided.

9.2.10 Basin Location

When possible, infiltration basins should be placed in low visibility areas. When such a basin must be situated in a high profile area, care must be given to ensure that the facility empties completely within a 48 hour maximum. The location of an infiltration basin in a high visibility area places a great emphasis on the facility's ongoing maintenance.

9.2.11 Existing Utilities

Infiltration basins should not be constructed over existing utility rights-of-way or easements. When this situation is unavoidable, permission to impound water over these easements must be obtained from the utility owner *prior* to design of the basin. When it is proposed to relocate existing utility lines, the costs associated with their relocation should be included in the overall basin construction cost.

9.2.12 Wetlands

When the construction of an infiltration basin is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation in their vicinity.

9.2.13 Floodplains

The construction of infiltration basins within floodplains is strongly discouraged. When this situation is deemed unavoidable, critical examination must be given to ensure that the proposed basin remains functioning *effectively* during the 10-year flood event. The structural integrity and safety of the basin must also be evaluated thoroughly under 100-year flood conditions as well as the basin's impact on the characteristics of the 100-year floodplain. When basin construction is proposed within a floodplain, construction and permitting must comply with all applicable regulations under FEMA's National Flood Insurance Program.

9.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing an infiltration basin for improvement of water quality.

9.3.1 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam.

"The foundation material under the dam and the material used for the embankment of the dam should be an AASHTO Type A-4 or finer and/or meet the approval of the Materials Division. If the native material is not adequate, the foundation of the dam is to be excavated and backfilled a minimum of 4' or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."

If the basin embankment height exceeds 15', or if the basin includes a permanent pool, the design of the dam should employ a homogenous embankment with seepage controls or zoned embankments, or similar design in accordance with the Virginia SWM Handbook and recommendations of the VDOT Materials Division.

During the initial subsurface investigation, additional borings should be made near the center of the proposed basin when:

- Excavation from the basin will be used to construct the embankment
- There is a potential of encountering rock during excavation
- A high or seasonally high water table, generally 2' or less, is suspected

9.3.2 Outfall Piping

If the basin is equipped with a riser structure and outlet barrel, the pipe culvert under or through the basin embankment shall be reinforced concrete equipped with rubber gaskets. Pipe: Specifications Section 232 (AASHTO M170), Gasket: Specification Section 212 (ASTM C443).

A concrete cradle shall be used under the pipe to prevent seepage through the embankment. The cradle shall begin at the riser or inlet end of the pipe, and extend the pipe's full length.

9.3.3 Principal Spillway Design

The basin outlet should be designed in accordance with Minimum Standard 3.02 of the Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et Seq.). *The primary control structure (riser or weir) should be designed to operate in weir flow conditions for the full range of design flows*. If this is not possible, and orifice flow regimes are anticipated, the outlet must be equipped with an anti-vortex device, consistent with that described in Minimum Standard 3.02.

The principal spillway should be equipped with a low flow orifice to permit draining of the facility in the event the infiltration surface becomes clogged and runoff cannot be infiltrated. This low flow orifice should remain plugged as long as the facility is infiltrating runoff at the rate for which it was designed.

9.3.4 Embankment

The top width of the embankment should be a minimum of 10' in width to provide ease of construction and maintenance. Positive drainage should be provided along the embankment top.

The embankment slopes should be no steeper than 3H:1V to permit mowing and other maintenance.

9.3.5 Embankment Height

A basin embankment may be regulated under the Virginia Dam Safety Act, Article 2, Chapter 6, Title 10.1 (10.1-604 Et seq.) of the Code of Virginia and Dam Safety Regulations established by the Virginia Soil and Water Conservation Board (VS&WCB). An infiltration basin embankment may be excluded from regulation if it meets any of the following criteria:

- is less than 6' in height
- has a capacity of less than 50 acre-ft and is less than 25' in height
- has a capacity of less than 15 acre-ft and is more than 25' in height
- will be owned or licensed by the Federal Government

When an embankment is not regulated by the Virginia Dam Regulations, it must still be evaluated for structural integrity when subjected to the 100-year flood event.

9.3.6 Fencing

Fencing is typically *not required or recommended* on most VDOT detention facilities. However, exceptions do arise, and the fencing of a dry extended detention facility may be needed. Such situations include:

- Ponded depths greater than 3' and/or excessively steep embankment slopes
- The basin is situated in close proximity to schools or playgrounds, or other areas where children are expected to frequent
- It is recommended by the VDOT Field Inspection Review Team, the VDOT Residency Administrator, or a representative of the City or County who will take over maintenance of the facility

"No Trespassing" signs should be considered for inclusion on all detention facilities, whether fenced or unfenced.

9.3.7 Design Infiltration Rate

To provide a factor of safety, and to account for the decline in performance as the facility ages, the soil infiltration rate upon which a basin design is founded should be one-half the infiltration rate obtained from the geotechnical analysis (DCR/DEQ, 1999, Et Seq.).

9.3.8 Maximum Storage Time

Infiltration basins should be designed to empty completely within 48 hours following a runoff producing event.

9.3.9 Runoff Pretreatment

Infiltration basins should be preceded by a pretreatment facility. Roadways and parking lots may produce runoff with high levels of sediment, grease, and oil. These pollutants can potentially clog the pore space in the basin floor, thus reducing its infiltration and pollutant removal performance. Suitable pretreatment practices include vegetated buffer strips, sediment forebays, and proprietary water quality inlets. At a minimum, each basin inflow point should be equipped with a sediment forebay. Individual forebay volumes should range between 0.1 and 0.25" over the outfall's contributing new impervious area with the sum of all forebay volumes not less than 10% of the total WQV.

All infiltration basins that receive surface runoff as sheet flow should be equipped with a vegetated buffer strip at least 20' wide.

9.3.10 Discharge Flows

All basin outfalls must discharge into an adequate receiving channel as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR/DEQ, 1992, Et seq.). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

9.4 Design Process

Many of the design elements in an infiltration basin are identical to those of a dry extended detention basin. These elements include estimation of flood control storage volumes, design of a multi-stage riser, storage indication (reservoir) routing, emergency spillway design, riser buoyancy calculations, and the design of sediment forebays. For those design items, the reader is referred to *Section 2 – Dry Extended Detention Basin*.

This section presents the design steps exclusive to infiltration basins serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered during linear development projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 4 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et Seq.) for expanded hydrologic methodology.

The following design example entails the construction of a small interchange and new section of two lane divided highway in Williamsburg. The total project site, including right-of-way and all permanent easements, consists of 24.8 acres. Pre and post-development hydrologic characteristics are summarized below in Table 9.2. Initial geotechnical investigations reveal a soil infiltration rate of 1.84 in/hr with site soils classified as Hydrologic Soil Group B.

Table 9.4.1 - Hydrologic Characteristics of Example Project Site

	Pre-Development	**Post-Development**
Project Area (acres)	24.8	24.8
Land Cover	Unimproved Grass Cover	11.2 acres *new* impervious cover
Impervious Percentage	0	45

Step 1 - Compute the Required Water Quality Volume

The project water quality volume is a function of the developed new impervious area, and is computed as follows:

$$WQV = \frac{NIA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

NIA= New Impervious Area (ft^2)

The project site in this example is comprised of a total drainage area of 24.8 acres. The total new impervious area within the site is 11.2 acres. Therefore, the water quality volume is computed as follows:

$$WQV = \frac{11.2ac \times \frac{1}{2} in \times \frac{43{,}560\, ft^2}{ac}}{12 \frac{in}{ft}} = 20{,}328\, ft^3$$

The new impervious cover within the project site is less than 67% of the total project site. Therefore, the infiltration basin will be sized to treat the computed water quality volume of 20,328 ft^3.

Step 2 - Compute the Design Infiltration Rate

The design infiltration rate, f_d, is computed as one-half the infiltration rate obtained from the required geotechnical analysis. For the given site conditions, the design infiltration rate is computed as:

$$f_d = 0.5 f = (0.5)\left(1.84 \frac{in}{hr}\right) = 0.92 \frac{in}{hr}$$

Step 3 - Compute the Maximum Ponded Depth of Infiltration Volume

The basin must be designed such that it is completely empty within a maximum of 48 hours following a runoff producing event. To ensure compliance with this requirement, the maximum ponding depth for the infiltration (treatment) volume is computed by the following equation:

$$d_{max} = f_d \times T_{max}$$

d_{max} = maximum allowable basin depth (ft)
f_d = design infiltration rate (in/hr)
T_{max} = maximum allowable drain time (48 hours)

The maximum allowable ponding depth is therefore computed as:

$$d_{max} = \left(0.92 \frac{in}{hr}\right)\left(\frac{1\, ft}{12 in}\right)(48hr) = 3.68\, ft$$

Step 4 - Compute the Minimum Allowable Basin Surface Area

Employing Darcy's Law, and assuming one-dimensional flow through the bottom of the basin, we can compute the minimum allowable surface area of the basin floor by the following equation:

$$SA_{\min} = \frac{WQV}{(f_d)(T_{\max})}$$

SA_{min} = minimum basin bottom surface area (ft^2)
WQV = treatment volume (ft^3)
f_d = design infiltration rate (in/hr)
T_{max} = maximum allowable drain time (48 hours)

The minimum allowable basin floor area is computed as follows:

$$SA_{\min} = \frac{20{,}328\, ft^3}{\left(0.92 \frac{in}{hr}\right)\left(\frac{1\, ft}{12 in}\right)(48 hr)} = 5{,}524\, ft^2$$

Step 5 - Size the Basin Based on Site-Specific Parameters

In order to reduce the amount of required right-of-way acquisition, the surface area of a structural BMP is minimized during the design process. However, minimization of surface area may require a BMP depth that is either impractical or, in the case of an infiltration facility, violates design parameters. The following design approach attempts to minimize the surface area of the basin while meeting restrictions on ponding depth.

The minimum allowable basin floor area was previously computed as 5,524 ft^2. This is the minimum basin area that, when considering a factor of safety, will ensure that the basin empties within a maximum of 48 hours. In practice, the actual configuration of an infiltration basin will be dictated largely by topography and other site-specific constraints. The final design may require multiple iterations to provide the required treatment volume. In this design, we will consider a basin of rectangular orientation, with a 2.5:1 length to width ratio. A schematic illustration of this basin configuration is shown in Figure 9.2.

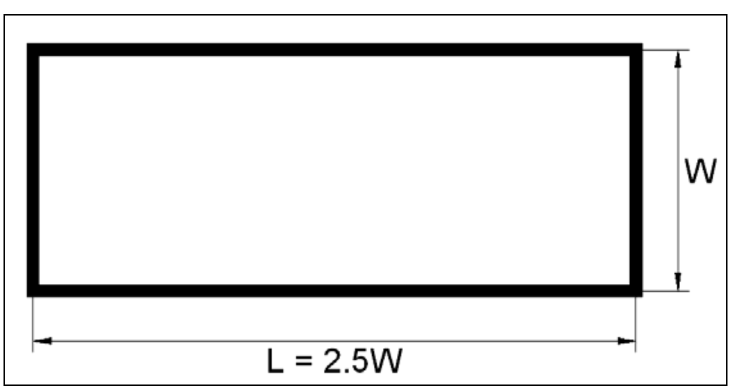


Figure 9.4.1 - Schematic Basin Orientation

The dimensions of the basin floor can then be approximated by solving the following expression:

$$W \times 2.5W = 5{,}524\ ft^2$$
$$W = 47.0\ ft$$
$$L = 117.5\ ft$$

The volume above the basin floor that is allocated to infiltration can be approximated by the following equation:

$$V = \left(\frac{A_1 + A_2}{2}\right)d$$

V = infiltration (treatment) volume (ft^3)
A_1 = surface area of basin floor (5,524 ft^2)
A_2 = surface area above the basin floor allocated to infiltration
d = incremental depth between A_1 and A_2

Based on a trapezoidal approximation, the surface area, A_2, can be expressed as a function of depth, d:

$$A_2 = [47.0 + (2)(d)(Z)] \times [117.5 + (2)(d)(Z)]$$

Z = basin side slopes (ZH:1V)

In this example, we will consider that the basin side slopes are 3H:1V. The updated A_2 expression then becomes:

$$A_2 = [47.0 + (2)(d)(3)] \times [117.5 + (2)(d)(3)]$$

A total infiltration volume of 20,328 ft^3 must be provided above the surface of the basin floor. At this point, the designer can construct a plot of storage versus depth by employing the above equation for A_2 in the previous expression for volume, *V*. This plot is shown in Figure 9.2.

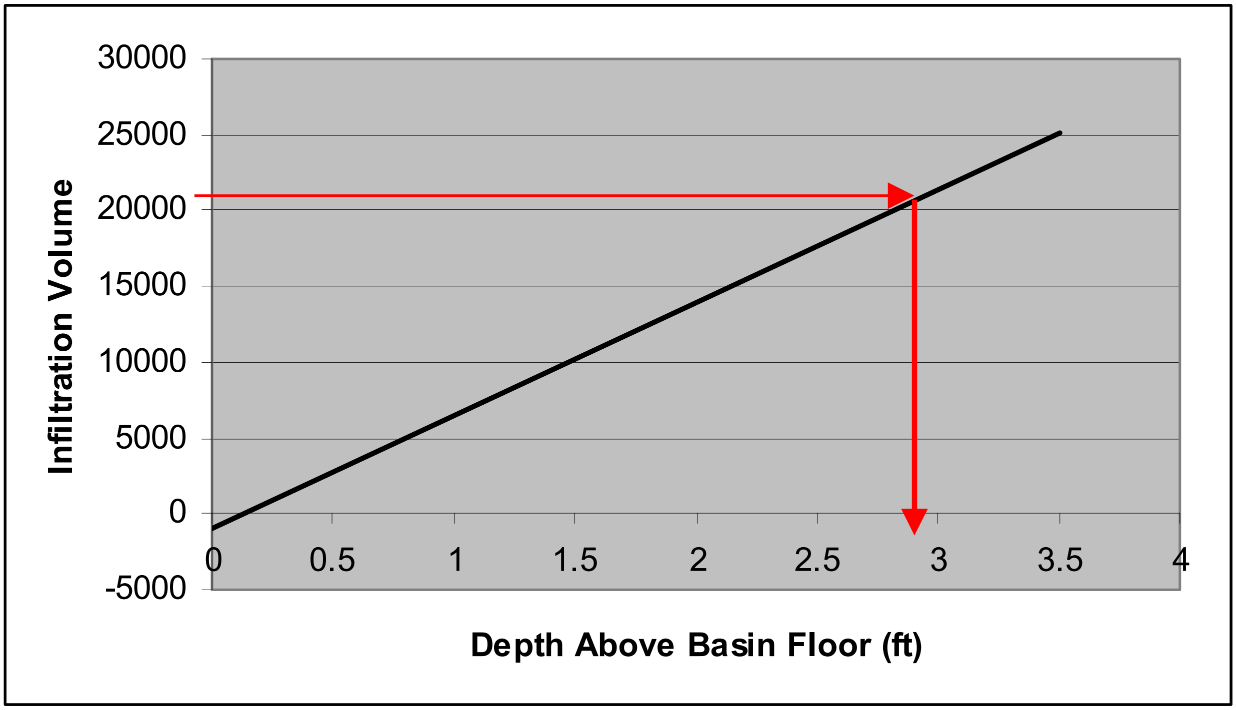


Figure 9.4.2 - Plot of Infiltration Volume Versus Depth Above Basin Floor

The plot indicates that the infiltration volume of 20,328 ft^3 is provided at an approximate depth of 2.8' above the basin floor. This estimate can be verified as follows:

$$A_2 = [47.0 + (2)(2.8)(3)] \times [117.5 + (2)(2.8)(3)] = 8{,}568\, ft^2$$

The total storage volume provided above the permanent pool is then computed as:

$$V = \left(\frac{5{,}524 + 8{,}568}{2}\right) 2.8 = 19{,}729\, ft^3$$

The volume is less than the required storage volume of 20,328 ft^3, and therefore must be increased. The calculation is repeated for a ponded infiltration depth of 2.9'.

$$A_2 = [47.0 + (2)(2.9)(3)] \times [117.5 + (2)(2.9)(3)] = 8{,}688\, ft^2$$

The total storage volume provided above the permanent pool is then computed as:

$$V = \left(\frac{5{,}524 + 8{,}688}{2}\right) 2.9 = 20{,}607\, ft^3$$

The infiltration volume provided at a ponded depth of 2.9' exceeds (slightly) the minimum treatment volume of 20,328 ft^3 and is therefore acceptable. Additionally, the infiltration volume is provided at a depth that is less than the maximum allowable depth of 3.68'. Therefore, it can be anticipated that the basin will empty completely within the maximum allowable time of 48 hours.

At this point, the remaining design process largely mimics that of a Dry Extended Detention facility. Flood control storage can be provided in the facility beginning at 2.9' above the basin floor (the upper limit of the infiltration volume). The remaining design elements include estimation of flood control storage volumes, design of a multi-stage riser, storage indication (reservoir) routing, emergency spillway design, riser buoyancy calculations, and the design of sediment forebays. For those design items, the reader is referred to *Section 2 – Dry Extended Detention Basin*.

Step 6 - Landscaping

Infiltration basins must exhibit a dense vegetative cover before any stormwater runoff is directed to the facility. Careful attention must be given to the types of vegetation selected for the basin floor and embankment. The vegetative species must be selected based on their inundation tolerance and the anticipated frequency and depth of inundation. The designer is referred to the Virginia Erosion and Sediment Control Handbook (DCR/DEQ, 1992, Et seq.) for recommendations of specific vegetative species based on the facility's geographic location. Generally, low-growing stoloniferous grasses are good candidates for infiltration facilities as they permit long intervals between mowing, thus minimizing the frequency of traffic on the surface of the facility.

Maintenance of the facility's vegetative cover is essential to the long-term performance of the facility. A dense vegetative stand enhances infiltration, minimizes surface erosion, and deters invasive and detrimental vegetative species. Any bare spots on the surface of the facility should be re-seeded immediately.

The use of fertilizers should be minimized and avoided completely if practically possible. Excessive use of fertilizers on highly permeable soil may lead to groundwater contamination. Reference the Virginia Erosion and Sediment Control Handbook (DCR/DEQ, 1992, Et seq.) for recommendations on appropriate fertilizer types and minimum effective application rates.

10.1 Porous Pavement - Overview of Practice

Porous pavement is a pervious traffic-bearing surface placed over a stone reservoir which is, in turn, underlain by highly permeable soil. The void space created by the stone reservoir provides storage for surface runoff generated on or diverted onto the porous surface. This runoff then infiltrates into the surrounding soil, through the bottom and sides of the stone reservoir. Porous pavement may substitute for conventional pavement on parking areas and areas with light traffic. Porous pavement is generally not suited for areas with high traffic volumes.

Porous pavement acts primarily as a water quality BMP. However, much like an infiltration trench (*Section 8 – Infiltration Trench*), when equipped with underground piping, the temporary storage volume of the reservoir may be increased to provide peak runoff reduction for the one and two year return frequency storms. Peak rate control of the 10-year and greater storm events is considered to be beyond the ability of the practice.

Studies have shown that particulates tend to settle to the bottom of a porous pavement system's stone reservoir while other pollutants often adsorb to the aggregate material. Consequently, the pollutant removal efficiency of a porous pavement system may not be as high as that of other types of infiltration practices. Per DCR/DEQ recommendations, a porous pavement facility is considered to have apollutant removal efficiency comparable to that of an extended dry detention facility (*Section 2 – Dry Extended Detention Basin*).

10.2 Site Constraints and Siting of the Facility

The implementation of a porous pavement system requires the designer to consider many of the same site constraints as with an infiltration basin or trench. These constraints are discussed as follows.

10.2.1 Drainage Area

Porous pavement systems are generally not cost-effective for sites smaller than 0.25 acres in area. According to the FHWA (1996), the contributing drainage area to a porous pavement infiltration bed should be limited to a maximum of 10 acres in order to reduce the potential for excessive sediment loading. A primary cause of infiltration bed failure is clogging by sediment. The porous pavement system should not be located where runoff from adjacent areas introduces excessive sediment to the system. Additionally, for drainage areas of 10 acres and greater the cost effectiveness of porous paving systems is considered marginal compared to that of other BMPs.

10.2.2 Site Slopes

Unlike other infiltration-based BMPs, which can be installed on slopes of up to 20%, porous pavement should not be installed when the traffic bearing surface of the system exceeds 3% in slope. Site topography should also permit the construction of a stone reservoir bed that is essentially level along its bottom surface. Porous pavement systems and their associated infiltration beds should be located a minimum of 50' away from any slope steeper than 15%. When site slopes do not permit the construction of a level infiltration bed, alternative BMP measures should be considered.

10.2.3 Site Soils

The underlying soil infiltration rate is of critical importance in the design of a porous pavement system. *A subsurface analysis and permeability test is required* when such a facility is planned. The required subsurface analysis should include soil characteristics to a depth of no less than 3' below the proposed bottom of the stone reservoir. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils.

The soil infiltration rate should be measured when the soil is in a saturated condition. Soil infiltration rates which are deemed acceptable for porous pavement systems range between *0.52 and 8.27 in/hr*. Soils with infiltration rates in this range are typically categorized as loam, sandy loam, and loamy sand.

Soils exhibiting a clay content of greater than 30% are unacceptable for infiltration facilities. Similarly, soils exhibiting extremely high infiltration rates, such as sand, should be avoided. Table 10.1 presents typical infiltration rates observed for a variety of soil types. This table is provided as a reference only, and does not replace the need for a detailed site soil survey.

Table 10.2.1 – DCR/DEQ Hydrologic Soil Properties Classified by Soil Texture (*Virginia Stormwater Management Handbook*, 1999, Et seq.)

Texture Class	Effective Water Capacity (C_w) (inch per inch)	Minimum Infiltration Rate (*f*) (inch per hour)	Hydrologic Soil Grouping
Sand	0.35	8.27	A
Loamy Sand	0.31	2.41	A
Sandy Loam	0.25	1.02	B
Loam	0.19	0.52	B
Silt Loam	0.17	0.27	C
Sandy Clay Loam	0.14	0.17	C
Clay Loam	0.14	0.09	D
Silty Clay Loam	0.11	0.06	D
Sandy Clay	0.09	0.05	D
Silty Clay	0.09	0.04	D
Clay	0.08	0.02	D

10.2.4 Depth to Water Table

Porous pavement systems should not be installed on sites with a high groundwater table. Inadequate separation between the reservoir bottom and the surface of the water table may result in contamination of the water table. This potential contamination arises from the inability of the soil underlying the reservoir to filter pollutants prior to their entrance into the water table. Additionally, a high water table may flood the stone reservoir and render it inoperable during periods of high precipitation and/or runoff. A separation distance of no less than 4’ is required between the bottom of the stone reservoir and the surface of the *seasonally* high water table. Unique site conditions may arise which require an even greater separation distance. The separation distance provided should allow the reservoir to empty completely within a maximum of 48 hours following a runoff producing event.

10.2.5 Separation Distances

Porous pavement systems should be located at least 20’ down-slope and at least 100’ up-slope from building foundations. Porous pavement systems should not be located within 100’ of any water supply well. Local health officials should be consulted when the implementation of such a facility is proposed within the vicinity of a septic drainfield.

10.2.6 Bedrock

A minimum of 4’ of separation is required between the bottom of a porous pavement’s stone reservoir and bedrock, with 4’ or greater recommended.

10.2.7 Placement on Fill Material

Porous pavement systems should not be constructed on fill sections due to the possibility of creating an unstable subgrade. Fill areas are vulnerable to slope failure along the interface of the in-situ and fill material. The likelihood of this type of failure is increased when the fill material is frequently saturated, as anticipated when an infiltration BMP is proposed.

10.2.8 Implementation in Cold Weather Climates

Porous pavement systems can be implemented in cold weather climates, provided that the reservoir layer extends to a depth beyond the frost line. During winter months, abrasives such as grit and/or sand and deicing chemicals *must not be used* on porous pavement. Plowing must be performed carefully, and as infrequently as possible.

10.2.9 Karst

The concentration of runoff into a stone reservoir may lead to collapse in karst areas, and therefore the implementation of porous pavement in known karst areas should be avoided.

10.2.10 Existing Utilities

Porous pavement systems may be constructed over existing easements, provided permission to construct the infiltration bed over these easements is obtained from the utility owner *prior* to design of the facility.

10.2.11 Wetlands

When the construction of a porous pavement system is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation. In Virginia, the Department of Environmental Quality and the U.S. Army Corps of Engineers should be contacted when such a facility is planned in the vicinity of wetlands.

10.3 General Design Guidelines

The following section presents a collection of design issues to be considered when designing a porous pavement *system* for improvement of water quality. The design steps discussed in this report are those exclusive to the water quality improvement function of a porous pavement system. Design of the porous pavement surface layer is beyond the scope of this report, and is a function of the anticipated traffic intensity, the California Bearing Ratio (CBR) of the site soils, the susceptibility of site soils to frost heave, and numerous other factors. The design of the porous surface layer should be performed by a qualified professional familiar with all VDOT standards and specifications governing asphalt design.

10.3.1 System Storage Capacity

Porous pavement systems can be designed as *full*, *partial*, or *water quality exfiltration* systems. Full exfiltration systems retain and infiltrate 100% of captured runoff. When the reservoir underlying the porous surface is full, runoff bypasses the system completely and is handled by a conventional stormwater capture and conveyance system. (FHWA, 1996)

Partial exfiltration systems are equipped with a bypass piping system. The bypass system routes runoff in excess of what can be infiltrated to a downstream conveyance system. Two types of bypass pipe configurations are shown in Figure 10.1 and Figure 10.2. The first configuration locates the perforated bypass pipe at the bottom of the aggregate reservoir layer. This configuration requires that the outlet manhole be equipped with a concrete weir such that water only discharges through the bypass system when the aggregate layer is in a saturated state. An alternative configuration locates the bypass pipe at the surface of the aggregate reservoir layer. This configuration is similar to the bypass configuration for an infiltration trench (see *Design Example Seven – Infiltration Trench*). When the bypass pipe is not located at the reservoir bottom, the pipe should have perforations on the underside only, else the bypass pipe shall be perforated as necessary to permit flow to freely enter the bypass system.

Water quality exfiltration systems function as partial exfiltration systems, but are designed only to hold and infiltrate the computed water quality volume.

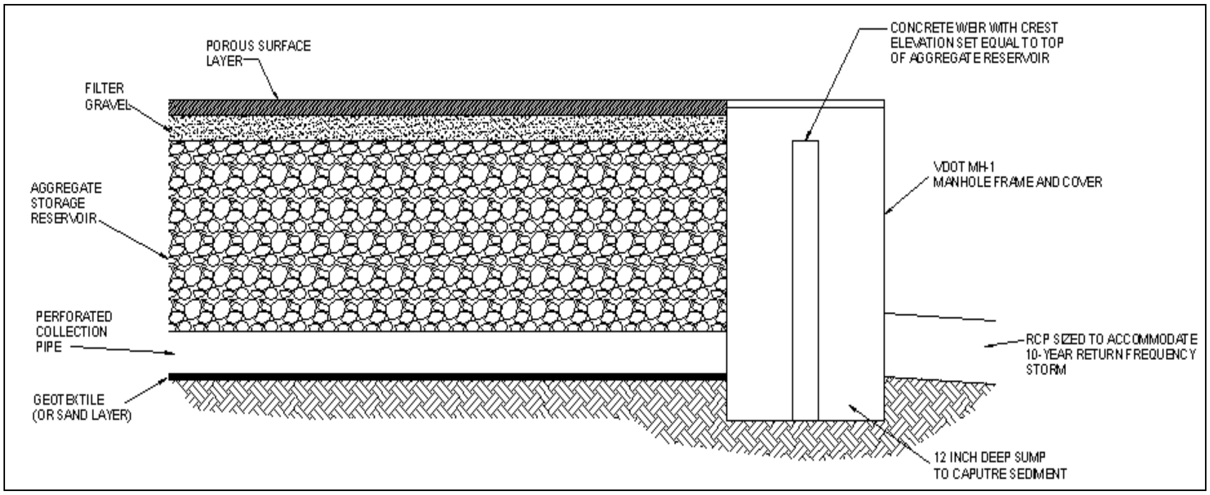


Figure 10.3.1 - Common Bypass Pipe Configuration

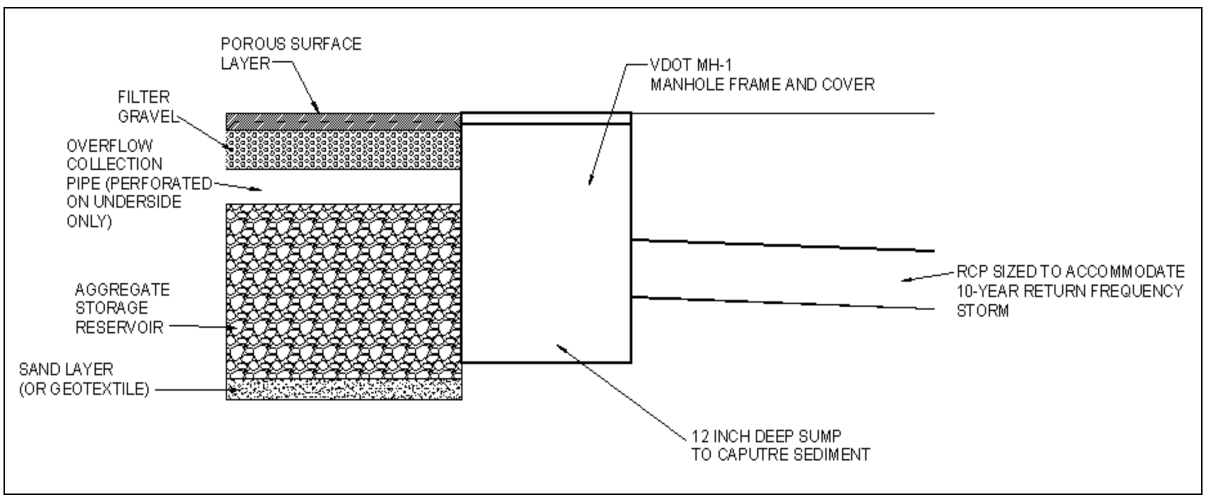


Figure 10.3.2 - Alternative Bypass Pipe Configuration

10.3.2 Design Infiltration Rate

To provide a factor of safety, and to account for the decline in performance as the facility ages, the design infiltration rate used to size a porous pavement system should be one-half the infiltration rate obtained from the geotechnical analysis (DCR/DEQ, 1999, Et seq.).

10.3.3 Maximum Storage Time

The stone reservoir of a porous pavement system should be designed to empty within 48 hours following a runoff producing event.

10.3.4 Stone Reservoir Sizing

The reservoir's aggregate depth should extend to a depth of at least that of the local frost line as specified by the Virginia Uniform Statewide Building Code. The surface area of the reservoir is that area which, when multiplied by the trench depth and the aggregate porosity, provides the computed treatment volume.

10.3.5 Aggregate Material

The porous pavement's reservoir layer should be overlain by a 2" thick filter layer comprised of VDOT Open-graded Course Aggregate #57. The reservoir should be comprised of 1" – 2" diameter clean aggregate (VDOT open-graded course aggregate No. 3). The reservoir layer should be underlain by an 8" layer of sand *or* filter fabric as approved by the Materials Division. This configuration is illustrated in Figure 10.3.

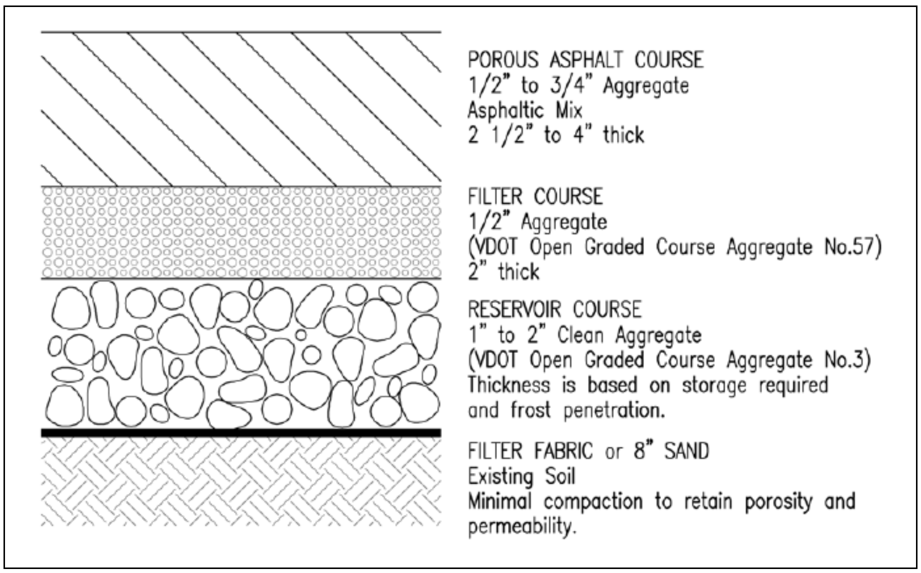


Figure 10.3.3 – DCR/DEQ General Configuration of Porous Pavement Section
(*Virginia Stormwater Management Handbook, 1999, Et seq.)*

10.3.6 Filter Fabric

When the reservoir aggregate material is not underlain by a layer of sand, it must be underlain with filter fabric as shown in Figure 10.3. The filter fabric should be comprised of material approved by the VDOT Materials Division in accordance with all applicable DCR/DEQ requirements.

10.3.7 Provision for Surface Clogging

Porous pavement systems must have a backup method for water to enter the infiltration bed in the event that the porous surface fails or is altered. In parking lots without curbing, this can be accomplished by constructing an unpaved 2' wide stone drain along the downstream edge of the parking lot. The stone drain is then connected directly to the infiltration bed. When curbing is present, sump inlets with sediment traps can be installed in low-lying areas, and then connected directly to the infiltration bed.

10.4 Design Process

This section presents an example of the design process applicable to porous pavement systems serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT *facilities* projects. The design steps discussed in this report are those exclusive to the water quality improvement function of a porous pavement system. Design of the porous pavement surface layer is beyond the scope of this report, and is a function of the anticipated traffic intensity, the California Bearing Ratio (CBR) of the site soils, the susceptibility of site soils to frost heave, and numerous other factors. The design of the porous surface layer should be performed by a qualified professional familiar with all VDOT standards and specifications governing asphalt design.

The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 4 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.) for expanded hydrologic methodology.

The porous pavement design will provide the technology-based water quality requirements arising from the parking lot of a VDOT-maintained interstate rest area facility located near Charlottesville. The total parking lot area consists of 4.8 acres, with no offsite drainage entering the parking facility. The total project site, including right-of-way and all permanent easements, consists of 6.2 acres. Geotechnical investigations reveal the site's saturated soil infiltration rate to be 2.7 in/hr. The project site does not exhibit a high or seasonally high groundwater table. Table 10.2 presents the 10-year hydrologic characteristics of the parking facility.

Table 10.4.1 - Peak Parking Lot Runoff Characteristics

		Albemarle County - 10 Year				
Acreage	Rational C	A Constant	B Constant	t_c (min)	i_{10} (iph)	Q_{10} (cfs)
4.8	0.9	161.6	18.73	5	6.81	29.4

Step 1 - Compute the Required Water Quality Volume

The project site's water quality volume is calculated as ½" over the developed new impervious area. In this example, the total parking lot area will be considered new impervious cover:

$$WQV = \frac{NIA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

NIA= New Impervious Area (ft^2)

The project site in this example is comprised of a total drainage area of 4.8 acres. Therefore, the basic water quality volume is computed as follows:

$$WQV = \frac{4.8ac \times \frac{1}{2}in \times \frac{43{,}560\,ft^2}{ac}}{12\frac{in}{ft}} = 8{,}712\,ft^3$$

The parking lot area (4.8 acres) comprises 77% of the total project site area (6.2 acres). Therefore, adhering to the requirements for infiltration practices detailed in Table 1.1, we will set the design water quality volume as twice the basic water quality volume:

$$WQV_{Design} = 2 \times 8{,}712\,ft^3 = 17{,}424\,ft^3$$

Step 2 - Compute the Design Infiltration Rate

The design infiltration rate, f_d, is computed as one-half the infiltration rate obtained from the required geotechnical analysis. For the given site conditions, the infiltration rate is computed as:

$$f_d = 0.5f = (0.5)\left(2.7\frac{in}{hr}\right) = 1.35\frac{in}{hr}$$

Step 3 - Compute the Maximum Allowable Reservoir Depth

The aggregate reservoir must be designed such that it is completely empty within a maximum of 48 hours following a runoff producing event. To ensure compliance with this requirement, the maximum allowable trench depth is computed by the following equation:

$$d_{\max} = \frac{f_d \times T_{\max}}{V_r}$$

d_{max} = maximum allowable reservoir depth (ft)
f_d = design infiltration rate (in/hr)
T_{max} = maximum allowable drain time (48 hours)
V_r = void ratio of the stone trench (0.40 for VDOT Coarse-graded Aggregate)

The maximum allowable trench depth is therefore computed as:

$$d_{\max} = \frac{\left(1.35\frac{in}{hr}\right)\left(\frac{1\,ft}{12in}\right)(48hrs)}{0.40} = 13.5\,ft$$

Step 3b - Determine the Minimum Allowable Reservoir Depth

The bottom of the aggregate reservoir layer must be located below the frost line as specified by the Virginia Uniform Statewide Building Code. The frost line depth for the City of Charlottesville is 18”. Therefore, the bottom of the aggregate layer must extend to a depth of not less than 18” below the finished surface of the pavement.

Step 4 - Compute the Required Reservoir Surface Area

The maximum loading ratio, defined as total drainage area to infiltration area is generally restricted to 6:1. The total parking lot area is 4.8 acres, therefore the minimum surface area of stone infiltration reservoir is computed as:

$$A_{\min} = \frac{4.8ac \times \dfrac{43{,}560\, ft^2}{ac}}{6} = 34{,}848\, ft^2$$

The surface area of the stone reservoir, along with its depth must provide storage for the computed water quality volume. Employing the minimum reservoir surface area, we compute the depth of the stone reservoir as:

$$d = \frac{WQV_{Design}}{(V_r)(A_{\min})} = \frac{17{,}424\, ft^3}{(0.40)(34{,}848\, ft^2)} = 1.25\, ft$$

The computed depth is less than the minimum allowable reservoir depth as stipulated by the local frost line depth (18” for the City of Charlottesville). Therefore, the reservoir depth is set at 18”.

Table 10.4.2 - Summary of Stone Reservoir Dimensions

Surface Area	34,848 ft^2
Depth	1.5’
Storage Volume*	20,909 ft^3

*Volume Based on Aggregate Porosity of 0.4

Step 5 - Provision for Overflow / Bypass

Because the design configuration presented in this example is a partial exfiltration system intended only to retain and infiltrate the water quality volume, provisions must be made for runoff events producing volumes in excess of this amount.

The overflow/bypass system will function as a conventional storm sewer system upon saturation of the stone reservoir layer. Therefore, the bypass system should be designed to carry a peak 10-year flow rate of 29.4 cfs (reference Table 10.2). The bypass system/storm sewer must discharge into an adequate receiving channel as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR/DEQ, 1992, Et seq.). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

The bypass system may be constructed as shown in either Figure 10.1 or 10.2. In this example, the bypass will be designed as a PVC pipe placed on a 1.5% slope along the entire downstream edge of the stone reservoir. The pipe shall be perforated on its underside only. The bottom of the pipe will be placed at an elevation equal to the top surface of the stone reservoir layer (as shown in Figure 10.2). Therefore, flow will only enter the bypass system upon saturation of the stone reservoir layer Sizing of the underdrain pipe is accomplished by use of the Manning equation shown below:

$$Q = \frac{1.49}{n} \cdot AR_h^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$$

A typical Manning's *n* value for PVC pipe is 0.009 (Mays, 2001). For a fixed discharge, *Q*, the minimum required diameter, *D*, of a circular pipe flowing full can be computed by the following equation:

$$D = \left[\frac{(Q)(n)}{0.463} \times \frac{1}{\sqrt{s}} \right]^{3/8}$$

D= Minimum Pipe Diameter (ft)
Q= Pipe Discharge (cfs)
n= Manning's Roughness Coefficient
s= Pipe slope (ft/ft)

The minimum pipe diameter required to convey the facility's 10-year runoff is therefore computed as:

$$D = \left[\frac{(29.4)(0.009)}{0.463} \times \frac{1}{\sqrt{0.015}} \right]^{3/8} = 1.78 - ft = 21.4 - inches$$

The underdrain pipe shall be 24" in diameter.

The 24” perforated PVC underdrain shall connect to a conventional stormwater conveyance system and carry runoff volumes in excess of the water quality volume to an adequate receiving channel.

A cross section of this porous section is presented in Figure 10.4.

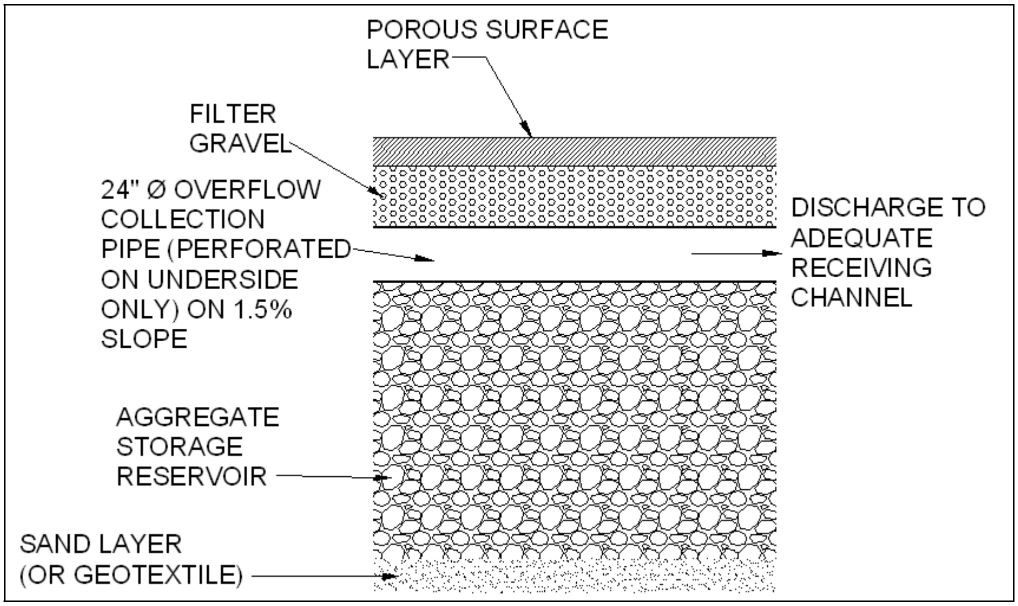


Figure 10.4.1 - Profile Along Downstream Edge of Stone Reservoir

Table 10.4.3 - Summary of Porous Pavement Section

Course	Thickness (in)	Comments
Porous Surface	2.5-4	Permeability > 8 in/hr
Top Filter Course	1-2	0.5” diameter gravel
Underdrain Piping	24	Perforation on bottom side only
Stone Reservoir	17	Cleanly washed - 40% void space
Bottom Filter Course	2	0.5” diameter gravel
Filter Fabric*	N/A	MIRIFI #14 or equivalent
Undisturbed Soil	N/A	Min. Permeability 0.5 in/hr

* The filter fabric should be comprised of material approved by the VDOT Materials Division in accordance with all applicable DCR/DEQ requirements.

11.1 Bioretention - Overview of Practice

Bioretention practices form a class of BMP whose primary function is to improve the quality of stormwater runoff by means of adsorption, filtration, volitization, ion exchange, and microbial decomposition. However, some runoff rate and volume reduction is observed through the infiltration of runoff. In the most general sense, a bioretention BMP can be thought of as a modified infiltration area comprised of a *specific* mix of trees, plants, and shrubs intended to mimic the ecosystem of an upland (non-wetland) forest floor. There are two categories of bioretention BMP: *basins* and *filters.*

Bioretention *basins* are planting areas constructed as shallow basins in which stormwater inflow is treated by filtration through the surface plant material, biological and chemical reactions within the soil and basin vegetation, and the eventual infiltration into the underlying soil media. Bioretention *filters* function much the same as bioretention basins, but are used in locations where full infiltration is not feasible due to inadequate soil permeability or the proximity to wells, drainfields, or structural foundations. Bioretention filters are equipped with a connection to a local storm sewer system such that water enters the storm sewer after it has filtered through the bioretention cell. Figures 11.1 and 11.2 present the general configuration of a bioretention basin and filter. The designer is also referred to Figures 3.11-2 – 3.11-5 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq., Et seq.) for location and conceptual layout suggestions for bioretention facilities.

Yu (2004) states that bioretention units can be applied in treating stormwater runoff from VDOT facilities such as weigh stations, park-and-ride facilities, and welcome stations. Other possible application scenarios include rooftop runoff and runoff from short stretches of roadway. Because of their use of specific vegetative plantings and landscaping techniques, bioretention BMPs can provide significant aesthetic benefit to a developed site.

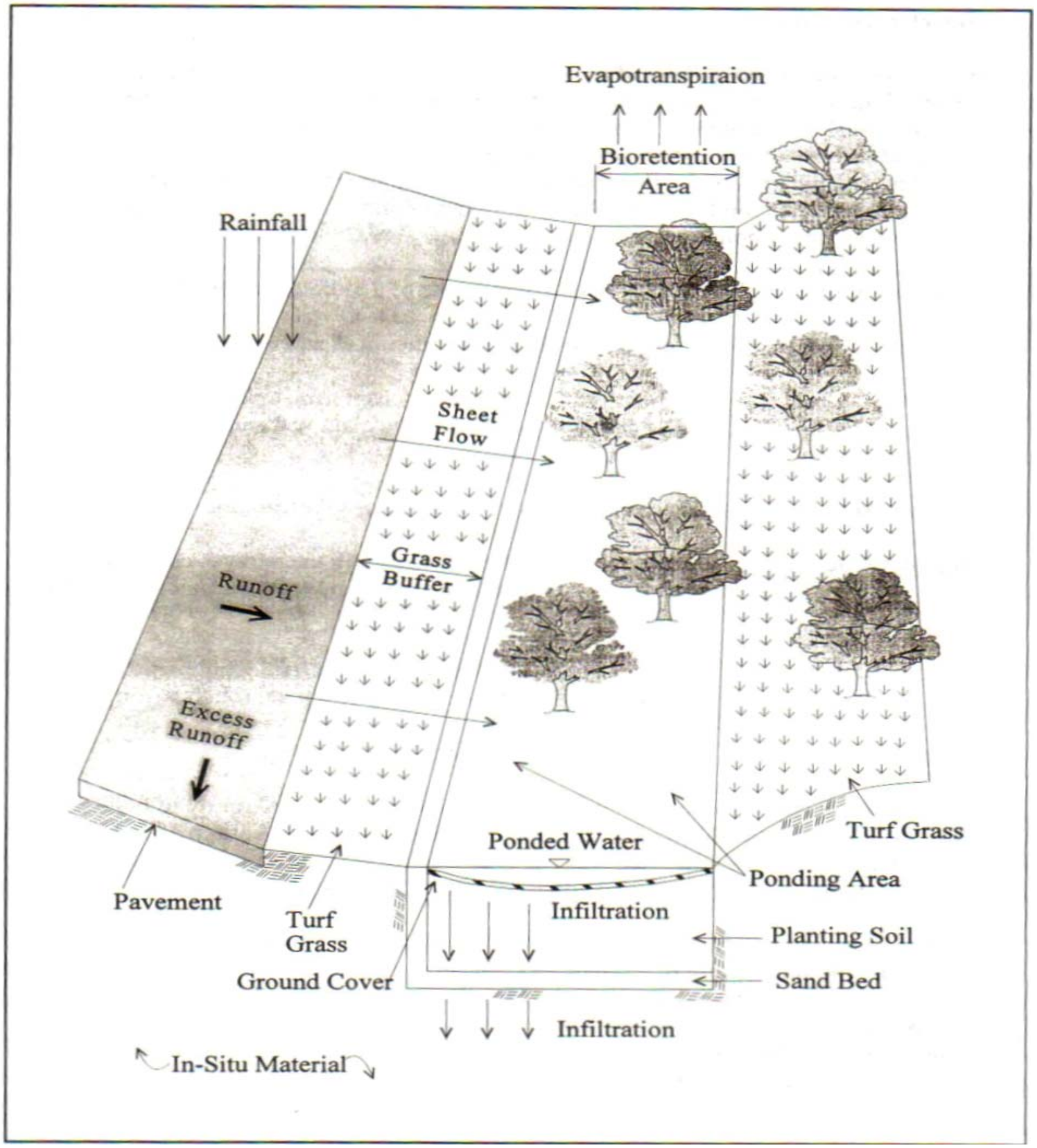


Figure 11.1.1 – DCR/DEQ Schematic Bioretention Basin
(Virginia Stormwater Management Handbook, 1999, Et seq.)

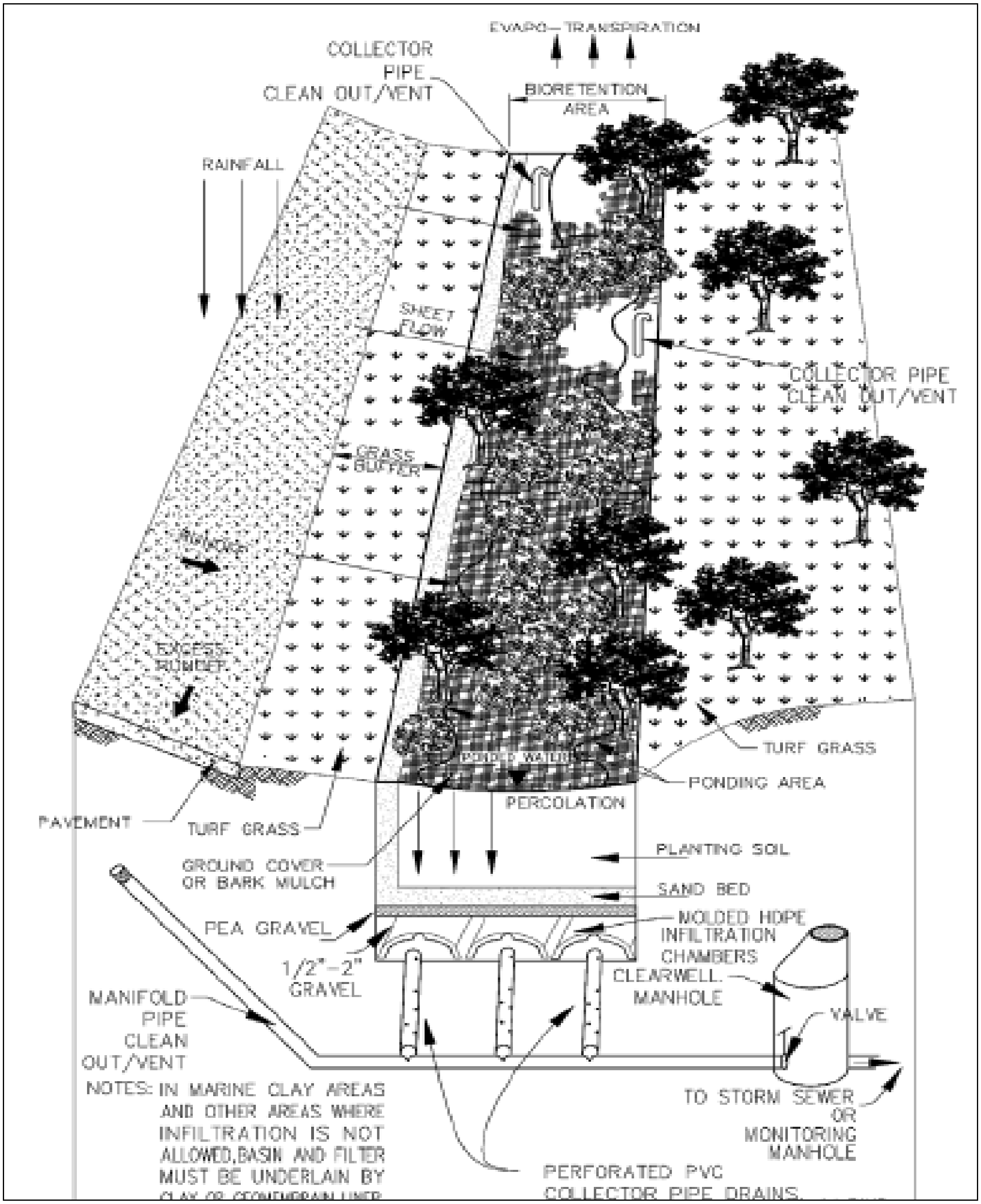


Figure 11.1.2 – DCR/DEQ Schematic Bioretention Filter
(*Virginia Stormwater Management Handbook*, 1999, *Et seq.)*

11.2 Site Constraints and Siting of the Facility

When a bioretention facility is proposed the designer must consider a number of site constraints in addition to the contributing drainage area's new impervious cover. These constraints are discussed as follows.

11.2.1 Minimum Drainage Area

The minimum drainage area contributing runoff to a bioretention cell is not restricted. However, the cost associated with constructing and maintaining a bioretention facility typically limits its use to drainage areas of at least 0.25 acres. Bioretention basins and filters are particularly well suited to small drainage areas.

11.2.2 Maximum Drainage Area

The maximum drainage area to a single bioretention facility should be restricted to no more than one acre.

11.2.3 Site Slopes

Bioretention facilities are suitable for installation on sites exhibiting average slopes less than 20%. Bioretention practices should be located a minimum of 50' away from any slope steeper than 15%. When average site slopes exceed 20%, alternative BMP measures should be considered.

11.2.4 Site Soils

This section refers to the native site soils underlying a bioretention facility. The planting soil mix of a bioretention facility is governed by specific guidelines discussed later in this Section and also in the *Virginia Stormwater Management Handbook* (DCR/DEQ, 1999, Et seq.).

Soil infiltration rate is a critical design element in a bioretention basin. When such a facility is proposed, *a subsurface analysis and permeability test is required*. The required subsurface analysis should investigate soil characteristics to a depth of no less than 3' below the proposed bottom of the engineered media. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils.

The soil infiltration rate should be measured when the soil is in a saturated condition. Soil infiltration rates which are deemed acceptable for bioretention facilities range between *0.52 and 8.27 in/hr*. Infiltration rates falling within this range are typically exhibited by soils categorized as loam, sandy loam, and loamy sand.

Soils exhibiting a clay content of greater than 30% are unacceptable for bioretention facilities. Similarly, soils exhibiting extremely high infiltration rates, such as some types of sand, should also be avoided. Table 11.1 presents typical infiltration rates observed for a variety of soil types. This table is provided as a reference only, and does not replace the need for a detailed site soil survey.

Table 0.1 - Hydrologic Soil Properties Classified by Soil Texture

Texture Class	Effective Water Capacity (C_w) (inch per inch)	Minimum Infiltration Rate (f) (inch per hour)	Hydrologic Soil Grouping
Sand	0.35	8.27	A
Loamy Sand	0.31	2.41	A
Sandy Loam	0.25	1.02	B
Loam	0.19	0.52	B
Silt Loam	0.17	0.27	C
Sandy Clay Loam	0.14	0.17	C
Clay Loam	0.14	0.09	D
Silty Clay Loam	0.11	0.06	D
Sandy Clay	0.09	0.05	D
Silty Clay	0.09	0.04	D
Clay	0.08	0.02	D

Source: Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.)

11.2.5 Depth to Water Table

Bioretention basins should not be installed on sites with a high groundwater table. Inadequate separation between the BMP bottom and the surface of the water table may result in contamination of the water table. This potential contamination arises from the inability of the soil underlying the BMP to filter pollutants prior to their entrance into the water table. Additionally, a high water table can flood the bioretention cell and render it inoperable during periods of high precipitation and/or runoff. A separation distance of no less than 2’ is required between the bottom of a bioretention basin and the surface of the *seasonally* high water table. Unique site conditions may arise which require an even greater separation distance. Bioretention filters (Figure 11.2) may be considered for use on sites where a high groundwater table prohibits the use of a bioretention basin.

11.2.6 Separation Distances

Bioretention basins should be located at least 20’ down-slope and at least 100’ up-slope from building foundations. Bioretention basins should not be located within 100’ of any water supply well. Local health officials should be consulted when the implementation of a bioretention basin is proposed within the vicinity of a septic drainfield. Generally, bioretention filters should be considered over bioretention basins for implementation in the vicinity of water supply wells, septic drainfields, and structural foundations.

This is because bioretention filters provide conveyance of runoff by the local storm sewer upon percolation through the filter media, whereas bioretention basins infiltrate runoff to the surrounding subsoil.

11.2.7 Bedrock

A minimum of 2' of separation is required between the bottom of a bioretention basin and bedrock, with 4' or greater recommended.

11.2.8 Placement on Fill Material

Bioretention basins should not be constructed on or nearby fill sections due to the possibility of creating an unstable subgrade. Fill areas are vulnerable to slope failure along the interface of the in-situ and fill material. The likelihood of this type of failure is increased when the fill material is frequently saturated, as anticipated when a bioretention basin.

11.2.9 Karst

The concentration of runoff into a bioretention basin may result in the formation of flow channels. Such channels may lead to collapse in karst areas, and therefore the implementation of bioretention basins in known karst areas should be avoided.

11.2.10 Existing Utilities

Bioretention facilities can often be constructed over existing easements, provided permission to construct the strip over these easements is obtained from the utility owner *prior* to design of the strip.

11.2.11 Wetlands

When the construction of a bioretention facility is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation in their vicinity.

11.2.12 Perennial and Chlorinated Flows

Bioretention facilities must not be subjected to continuous or very frequent flows. Such conditions will lead to anaerobic conditions which support the export of previously captured pollutants from the facility. Additionally, bioretention facilities must not be subjected to chlorinated flows, such as those from swimming pools or saunas. The presence of elevated chlorine levels can kill the desirable bacteria responsible for the majority of nitrogen uptake in the facility.

11.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing a bioretention facility for improvement of water quality.

11.3.1 Facility Location

When the proposed bioretention facility is to receive runoff in the form of sheet flow, the overall grading of the site must direct all runoff to the facility prior to its leaving the site or entering a downstream conveyance system. Consequently, the proposed location of a bioretention facility must be established early in the project design phase and remain an integral component of the site design throughout.

11.3.2 Basin Size

The minimum floor area of a bioretention facility is a function of the water quality volume (WQV) to be treated from the facility's contributing drainage area. Table 11.2 shows the minimum bioretention floor areas as a function of WQV.

Table 0.1 - Minimum Bioretention Floor Area

Bioretention Floor Area	WQV
2.5% of Contributing Impervious Area	0.5" Over Impervious Area
4.0% of Contributing Impervious Area	1.0" Over Impervious Area

Source: Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.)

The minimum size for any bioretention facility should be 10' wide (perpendicular to incoming sheet flow direction) and 15' long.

11.3.3 Basin Depth

The depth of the facility's planting soil (reference Figure 11.1) should be approximately 30", *or* the diameter of the largest plant root ball plus 4".

11.3.4 Surface Ponding Depth

The depth of ponding on the facility surface should be restricted to no more than 6" to preclude the development of anaerobic conditions within the planting soil.

11.3.5 Design Infiltration Rate

To provide a factor of safety, and to account for the decline in performance as the facility ages, the soil infiltration rate upon which a bioretention basin design is founded should be one-half the infiltration rate obtained from the geotechnical analysis.

11.3.6 Runoff Pretreatment

Bioretention facilities *must* be preceded upstream by some form of runoff pretreatment. Roadways and parking lots often produce runoff with high levels of sediment, grease, and oil. These pollutants can potentially clog the pore space in the facility, thus greatly reducing its pollutant removal performance. The selection of runoff pretreatment is primarily a function of the type of flow entering the facility, as disused below.

Runoff entering a bioretention basin or filter as *sheet flow* may be treated by a grass filter strip. The purpose of the grass buffer strip/energy dissipation area is to reduce the erosive capabilities of runoff prior to its entrance into the bioretention area. The recommended length of the grass buffer strip is a function of the land cover of the contributing drainage area and its slope. Under no circumstance should the grass buffer strip be less than 10'. The following table provides guidance in sizing the grass buffer strip leading to the bioretention area:

Table 0.2 - Design Parameters for Grass Buffer Pretreatment

Parameter	Impervious Parking Lots				Residential Lawns				Notes
Maximum Inflow Approach Length (feet)	35		75		75		150		
Filter Strip Slope	≤ 2%	≥2%	≤2%	≥2%	≤2%	≥2%	≤2%	≥2%	Maximum = 6%
Filter Strip Minimum Length	10'	15'	20'	25'	10'	12'	15'	18'	

Source: Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.)

Flow may enter the bioretention facility in a concentrated flow regime. In such cases, a common pretreatment method is to pass the incoming flow through a grass-lined channel equipped with a pea gravel diaphragm prior to its entrance into the bioretention area. The recommended length of the grass swale is a function of the land cover of the contributing drainage area and its slope. When used as pre-treatment for bioretention facilities, grass swales should be at least 20' in length. The following table provides guidance in sizing the grass swale leading to the bioretention area:

Table 0.3 - Design Parameters for Grass Swale Pretreatment

Parameter	≤ 33% Impervious		Between 34% and 66% Impervious		≥ 67% Impervious		Notes
Slope	≤2%	≥2%	≤2%	≥2%	≤2%	≥2%	Maximum slope = 4%
Grassed channel minimum length (feet)	25	40	30	45	35	50	Assumes a 2' wide bottom width

Source: Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.)

11.3.7 Offline Configurations

Whenever possible, bioretention facilities should be placed off-line so that flow is diverted onto it. This permits the facility to fill with only the desired treatment volume and bypass any remaining flow to the storm drainage system. Because offline bioretention BMPs are sized to accommodate only the designated water quality volume, a flow-splitter or diversion weir must be designed to restrict inflows to the bioretention area. The flow-splitter or diversion weir must be designed to admit a designated *volume* of runoff into the basin rather than to simply regulate the flow *rate* into the basin. The diversion structure may be prefabricated, or cast in place during construction. A schematic illustration of the flow-splitting weir is shown as follows:

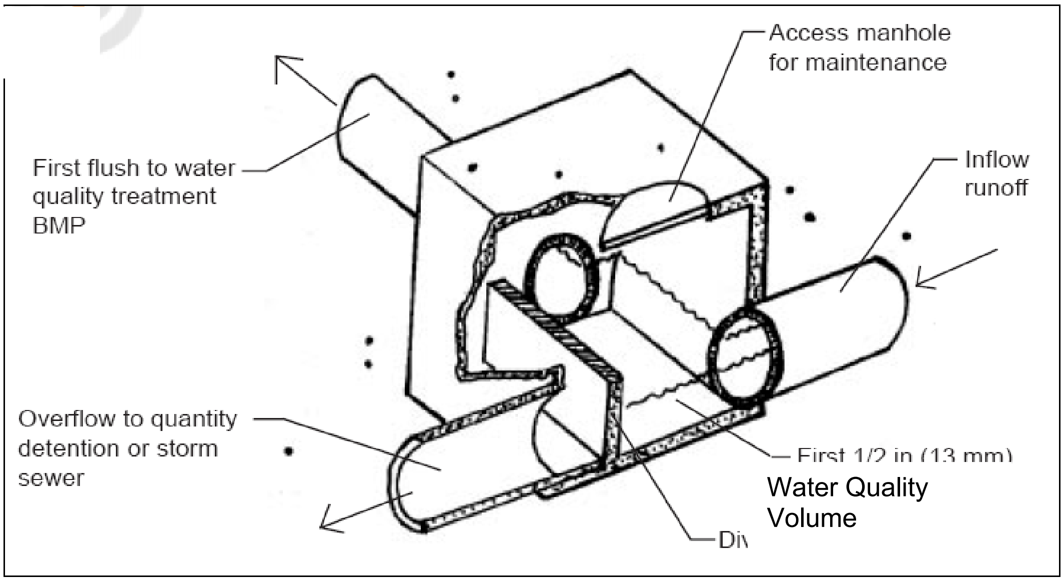


Figure 0.1 - Flow-splitting Diversion Weir (Bell, Warren, 1993)

Typically, the construction of the diversion weir will place its crest elevation equal to the maximum allowable ponding depth in the bioretention area (6” for bioretention basins and 12” for bioretention filters). Flow over the diversion weir will occur when runoff volumes exceed the computed water quality volume. These overflows then enter the stormwater conveyance channel. This configuration results in minimal mixing of the held water quality volume with flows from large runoff producing events in excess of this volume. A modified design referred to as a *dual pond system* is characterized by a diversion weir which directs the computed water quality volume into the bioretention area, while conveying excess volumes downstream to a peak mitigation detention pond.

11.3.8 Overflow/Bypass Structure

When a bioretention facility is constructed online, or the maximum volume of flow entering the facility is not otherwise restricted, an overflow structure *must* be provided. This structure provides bypass for excess runoff when the bioretention subsurface and surface capacity is met. Common overflow structures include domed risers, grate or slot inlets, and weir structures. Budget, site aesthetics, and maintenance will govern the selection of the overflow structure. The sizing of the overflow structure must consider the flow rate for the design storm of interest, typically the 10-year runoff producing event. The crest or discharge elevation of the overflow structure should be set an elevation of 6” above the mulch layer of the bioretention bed. When designed as a bioretention *filter*, and equipped with an underdrain system, the crest of the overflow may be set at an elevation as much as 1’ above the mulch layer of the facility. Typical domed riser overflow structures are shown in Figure 11.4.

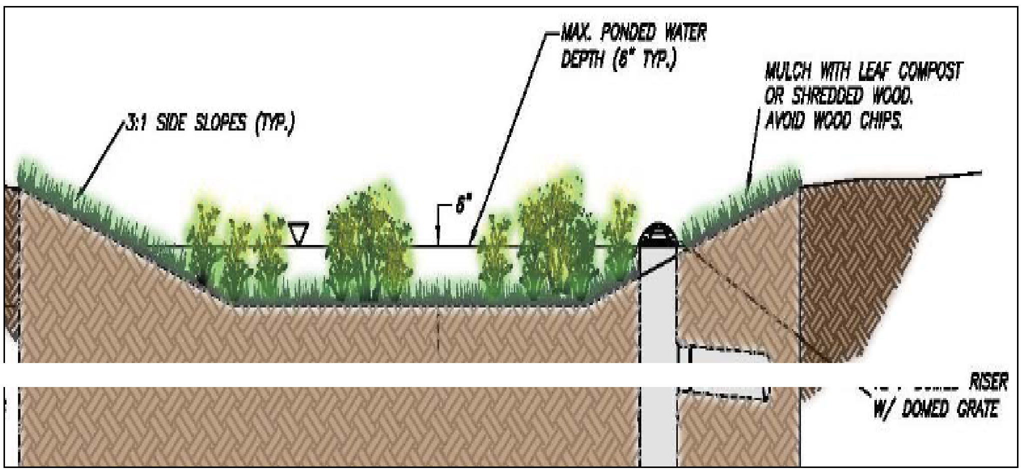


Figure 0.2 - Typical Domed Riser Bypass Structure Configuration (PADEP, 2006)

11.3.9 Planting Considerations

The ultimate goal in the selection and location of vegetation within a bioretention facility is to, as closely as possible, mimic an upland (non-wetland) terrestrial forest ecosystem. This type of planting scheme is based on a natively-occurring forest’s ability to effectively cycle and assimilate nutrients, metals, and other pollutants through the plant species, underlying soil, and also the system’s organic matter. Of additional concern in the selection of vegetative planting species is aesthetics. Bioretention BMPS can often be incorporated into the stormwater management plans of high profile areas, providing a desirable site amenity in the form of landscaping. The design of bioretention facilities requires a working knowledge of indigenous horticultural practices, and it is recommended that a landscape architect or other qualified professional participate in the design process.

The *Virginia Stormwater Management Handbook* (DCR/DEQ, 1999, Et seq.) provides a list of species suitable for inclusion in a bioretention facility. These species can be found in *Tables 3.11-7A – 3.11-7C* of the handbook. Species included have been deemed suitable based on their ability to tolerate pollutant loading, soil moisture fluctuations, and frequent inundation. Species not included in these tables *should not be selected* because they are not capable of surviving the conditions anticipated in a bioretention facility and/or they do not provide a desired level of pollutant uptake.

A minimum of three different species of trees and three different species of shrubs should be selected for *each* individual bioretention facility. Such diversity in species selection assists in reducing monoculture mortality concerns as well as providing a constant and predictable level of evapotranspiration and pollutant uptake. The ratio of shrubs to trees should range between 2:1 and 3:1.

A general guideline for determining the number of individual plantings required for a given bioretention area is 1,000 individual stems per planted acre. Table 11.5 provides average, maximum, and minimum planting guidelines as well as spacing recommendations.

Table 0.4 - Recommended Tree and Shrub Spacing

	Tree Spacing (feet)	Shrub Spacing (feet)	Total Density (stems/acre)
Maximum	19	12	400
Average	12	8	1000
Minimum	11	7	1250

Source: Virginia Stormwater Management Handbook, (DCR/DEQ, 1999, Et seq.)

The *Virginia Stormwater Management Handbook* (DCR/DEQ, 1999, Et seq.) provides a full discussion on the desirable planting soil and mulch layer characteristics of a bioretention facility in Minimum Standard 3.11. The planting soil of a bioretention facility should exhibit a pH ranging between 5.5 and 6.5 and a clay content of no greater than 5%.

11.4 Design Process

This section presents the design process applicable to bioretention facilities serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT *facilities* projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 4 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.) for details on hydrologic methodology.

The bioretention basin design will meet the technology-based water quality requirements arising from construction of a *Park-and-Ride* facility located in York County. Site grading is such that runoff from the facility's parking lot is directed onto the bioretention area through a curb cut along the parking lot's downstream edge. This example is an *online* configuration, and therefore the facility must be equipped with a bypass for flows exceeding the storage capacity of the bioretention cell.

The total project site, including right-of-way and all permanent easements, consists of 1.34 acres. Pre and post-development land cover and hydrologic characteristics are summarized below in Tables 11.6 and 11.7. Geotechnical investigations reveal the saturated soil infiltration rate to be 1.8 in/hr. The project site does not exhibit a high or seasonally high groundwater table.

Table 11.4.1 - Hydrologic Characteristics of Example Project Site

	Pre-Development	**Post-Development**
Project Area (acres)	1.34	1.34
Land Cover	Unimproved Grass Cover	0.83 acres *new* impervious cover
Impervious Percentage	0	62

Table 11.4.2. Peak Parking Lot Runoff

		York County - 10 Year Rainfall Constants				
Acreage	**Rational C**	**A**	**B**	**t_c (min)**	**i_{10} (iph)**	**Q_{10} (cfs)**
0.83	0.9	186.78	21.22	8	6.39	4.8

Step 1 - Compute the Required Water Quality Volume

The project site's water quality volume is calculated as ½" over the developed new impervious area. This *basic* water quality volume is computed as follows:

$$WQV = \frac{NIA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

NIA= new impervious area (ac.)

The project site in this example has a total drainage area of 1.34 acres. The total new impervious area within the site is 0.83 acres. Therefore, the water quality volume is computed as follows:

$$WQV = \frac{0.83ac \times \frac{1}{2} in \times \frac{43{,}560\, ft^2}{ac}}{12 \frac{in}{ft}} = 1{,}506\, ft^3$$

Step 2 - Compute the Minimum Basin Floor Area

The minimum allowable bioretention surface area is a function of the site's water quality volume. The water quality volume in this example was based on ½" of runoff from the site's new impervious cover. Therefore, referencing Table 11.2, the minimum floor area of the facility is 2.5% of the contributing impervious cover, computed as follows:

$$Area = 0.83ac \times \frac{43{,}560\, ft^2}{ac} \times 0.025 = 904\, ft^2$$

The minimum dimensions of a bioretention facility should be 10' wide (perpendicular to the incoming flow direction) and 15' long. The actual length to width ratio of the facility as well as its overall geometric configuration is determined by various site constraints such as topography and available area. In this example, we will employ a length to width ratio of 1.5:1. Therefore, the approximate dimensions of the facility are computed as follows:

$$L = 1.5W$$
$$L \times W = 904\, ft^2$$
$$1.5W \times W = 904\, ft^2$$
$$W = 24.5\, ft$$
$$L = 37\, ft$$

For bioretention areas with a preliminary computed length of greater than 20', the actual design length should be twice that which ensures dispersal of incoming sheet flow. The following steps illustrate the process for evaluating whether or not the preliminary computed length must be increased to meet this requirement.

The bioretention area will be preceded upstream by pretreatment in the form of a grass filter strip. Runoff will leave the proposed parking lot through a curb cut, and then discharge onto the filter strip after passing over a level spreader. The size of the level spreader is a function of the 10-year flow from the contributing drainage area. The required level spreader dimensions are shown in Table 11.8.

Table 11.4.3 - Minimum Level Spreader Dimensions

Q10 (cfs)	Depth (ft)	Width of Lower Side Slope of Spreader (ft)	Length (ft)
0-10	0.5	6	10
20-10	0.6	6	20

Source: Virginia Erosion and Sediment Control Handbook (DCR/DEQ, 1992)

The 10-year peak rate of runoff from the roadway is 4.8 cfs (see Table 11.7). Therefore, the minimum level spreader "lip" length that will discharge runoff onto the strip is 10'. The chosen bioretention length of 37' is more than twice the level spreader length of 10' discharging sheet flow onto the grass filter strip, and is therefore acceptable.

Step 3 - Specify Bioretention Depth

The depth of the facility's planting soil should be approximately 30", *or* the diameter of the largest plant root ball plus 4". Site grading and placement of the facility's overflow structure must ensure a maximum surface ponding depth of 6".

Step 4 - Design Overflow Structure

An overflow structure must be provided for large runoff producing events to bypass excess runoff when the bioretention surface and subsurface storage capacity is exceeded. The crest/outflow of the bypass system should be set at an elevation 6" above the surface of the bioretention floor. This will ensure discharge through the bypass system only when the design parameters of the bioretention area have been exceeded. Common overflow structures include domed risers, grate or slot inlets, and weir structures. The overflow/bypass system will function as a conventional storm sewer system when the facility's planting soil is saturated and a ponding depth of 6" is observed on the surface of the facility. Therefore, the bypass system should be designed to carry a peak 10-year flow rate of 4.8 cfs (reference Table 11.7). The bypass system must discharge into an adequate receiving channel as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR/DEQ, 1992). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

Sizing of the bypass pipe is accomplished by use of the Manning equation shown below:

$$Q = \frac{1.49}{n} \cdot AR_h^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$$

A typical Manning's *n* value for reinforced concrete pipe is 0.013. For a fixed discharge, *Q*, the minimum required diameter, *D*, of a circular pipe flowing full can be computed by the following equation:

$$D = \left[\frac{2.16(Q)(n)}{S^{1/2}}\right]^{0.375}$$

D= minimum pipe diameter (ft)
Q= pipe discharge (cfs)
n= Manning's roughness coefficient
S= pipe slope (ft/ft)

Assuming a slope of 1.5% on the overflow pipe, we compute the minimum pipe diameter required to convey the facility's 10-year runoff as:

$$D = \left[\frac{(2.16)(4.8)(0.013)}{0.015^{1/2}}\right]^{0.375} = 1.04\,ft = 12.5 inches$$

The bypass pipe shall be 15" in diameter.

The 15" bypass pipe shall connect to a conventional stormwater conveyance system and/or carry runoff volumes in excess of the water quality volume to an adequate receiving channel.

Step 5 - Specify Number of Vegetative Plantings

A typical bioretention facility should be planted with approximately 1,000 stems per acre. This vegetation should be comprised of both shrubs and trees, with a shrub to tree ratio ranging between 2:1 and 3:1. A minimum of three different species of trees and three different species of shrubs should be specified, with specific plant species determined from Tables 3.11-7A – 3.11-7C of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.).

Employing a 2.5:1 shrub to tree ratio, the number of shrubs and trees for the proposed bioretention area is determined as follows:

Total bioretention area: $24.5\,ft \times 37\,ft \times \frac{1ac}{43{,}560\,ft^2} = 0.02ac$

Total number of stems: $0.02ac \times 1{,}000\frac{stems}{ac} = 20$

Total number of shrubs (s): s = 2.5 x # trees

Total number of trees (t): $2.5t + t = 20 \Rightarrow t = 5.7$

The bioretention area should be planted with 6 trees, 2 each from three different species. Additionally, a total of 15 shrubs should be planted, 5 each from three different species.

Step 6 - Provide for Runoff Pretreatment

Runoff entering the proposed bioretention cell will pass through an upstream grass filter strip serving the purpose of pretreating the incoming runoff. Sizing of this filter strip is based on Table 11.3. The slope of the filter strip will be approximately 1.5% and the maximum flow path across the impervious parking lot is 75’. Obtained from Table 11.3, these parameters require a filter strip length of 20’.

Alternative Design – Bioretention Filter

Bioretention filters provide water quality improvement in essentially the same manner as bioretention basins, but are used in locations where full infiltration is not feasible either due to inadequate soil permeability or the proximity to wells, drainfields, or structural foundations. Bioretention filters are equipped with a connection to the site’s storm sewer system such that water enters the storm sewer after it has filtered through the bioretention cell (see Figure 11.2). The same sizing and design parameters apply to bioretention filters as apply to bioretention basins, with the exception of maximum surface ponding depth. Because runoff filters through a bioretention filter more quickly than through a bioretention basin, the maximum surface ponding depth may be increased to 12”.

When a bioretention filter is chosen due to the proximity of the facility to wells, structural foundations, or septic drainfields, *the entire basin must be underlain by a synthetic liner* as approved by the Materials Division. When the selection of a bioretention filter arises due to inadequately low percolation rates of the site’s native soils, the synthetic membrane may be omitted.

11.5 Stormwater Sand Filters - Overview of Practice

Stormwater sand filters are practices employed when the runoff from a site is expected to contain very high pollutant levels. These sand filters function by first pre-treating and temporarily storing runoff to remove the bulk of the large particle sediment, then percolating the runoff through the filter's sand media. As runoff filters through the sand media, water quality is improved through physical, chemical, and biological mechanisms. Various types of stormwater sand filters exist, and their application can be tailored to meet individual site needs. The most common types of stormwater sand filters are the Washington D.C. underground vault sand filter, the Delaware sand filter, and the Austin surface sand filter.

Stormwater sand filters act primarily as water quality BMPs; however, the water quality volume entering the filter is detained and released at a rate potentially capable of providing downstream channel erosion control. Peak rate control of the 10-year and greater storm events is typically beyond the capacity of a stormwater filtering system, and may require the use of a separate structural peak rate reduction facility.

Stormwater sand filters are commonly used in urbanized settings where entering runoff is generated from areas whose imperviousness ranges from 67 – 100%. The primary cause of failure in stormwater filtering systems is the clogging of the sand media through excessive sediment loading. The filters described in this document should not be used on sites having an impervious cover of less than 65%.

The *Virginia Stormwater Management Handbook*, (DCR/DEQ, 1999, Et seq., Et seq.) identifies three types of stormwater stand filters appropriate for use in the state. These are the Washington D.C. Underground Vault Sand Filter, the Delaware Sand Filter, and the Austin Surface Sand Filter. Each filter type is described briefly in the following section.

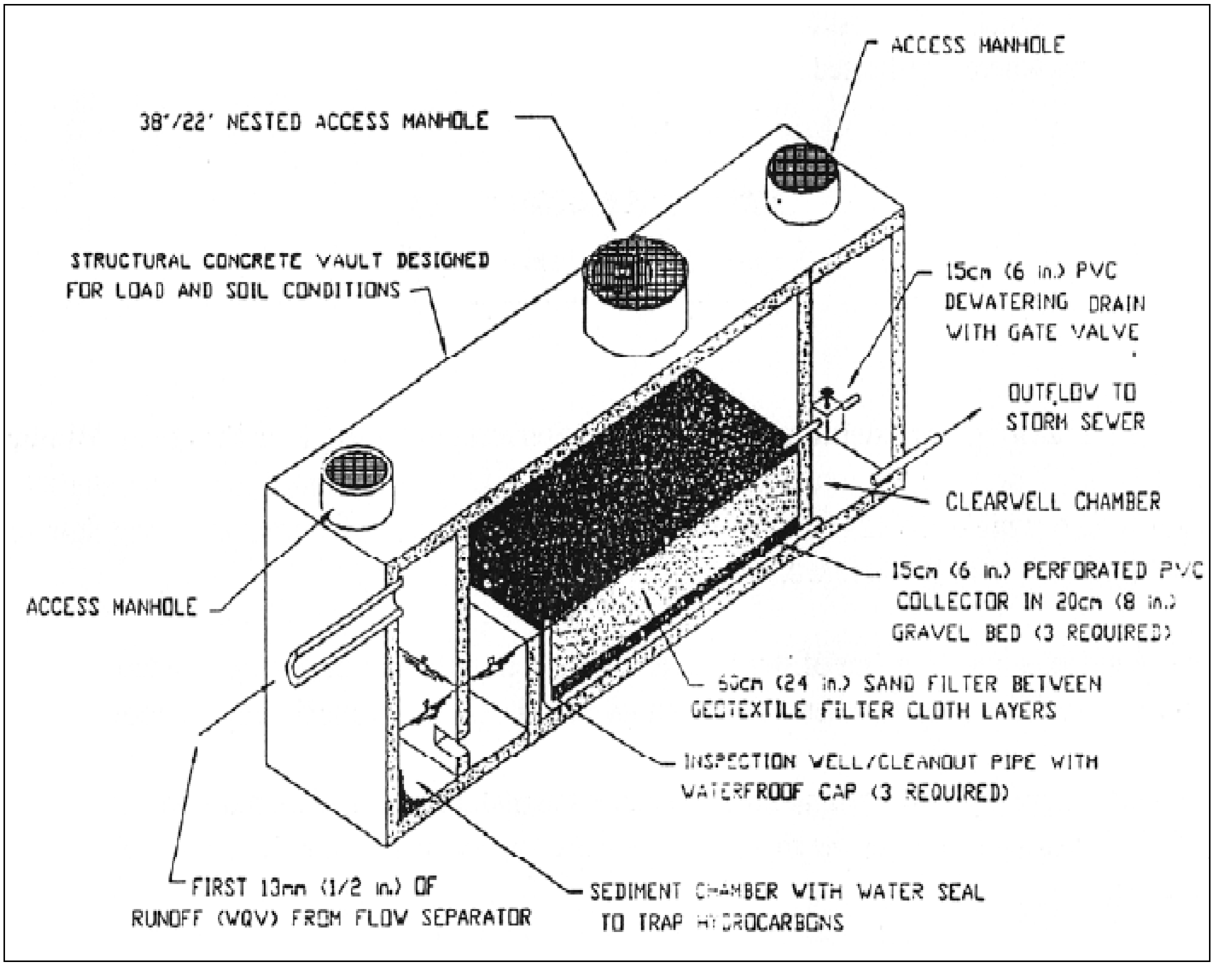


Figure 11.5.1 – DCR/DEQ Washington D.C. Underground Vault Sand Filter
(*Virginia Stormwater Management Handbook*, 1999, Et seq.)

The Washington D.C. underground vault sand filter shown in Figure 12.1 can be either precast or cast in place and is composed of three chambers. The first chamber is a 3' deep "plunge pool" which absorbs energy and pre-treats runoff by trapping sediment and floating organic matter. The first chamber is hydraulically connected to the second chamber containing the sand filter media. Finally, the third chamber serves as a collection point for filtered runoff, where it is then directed to the downstream storm sewer. This type of filter is typically constructed *offline*, with only the site water quality volume directed to the structure.

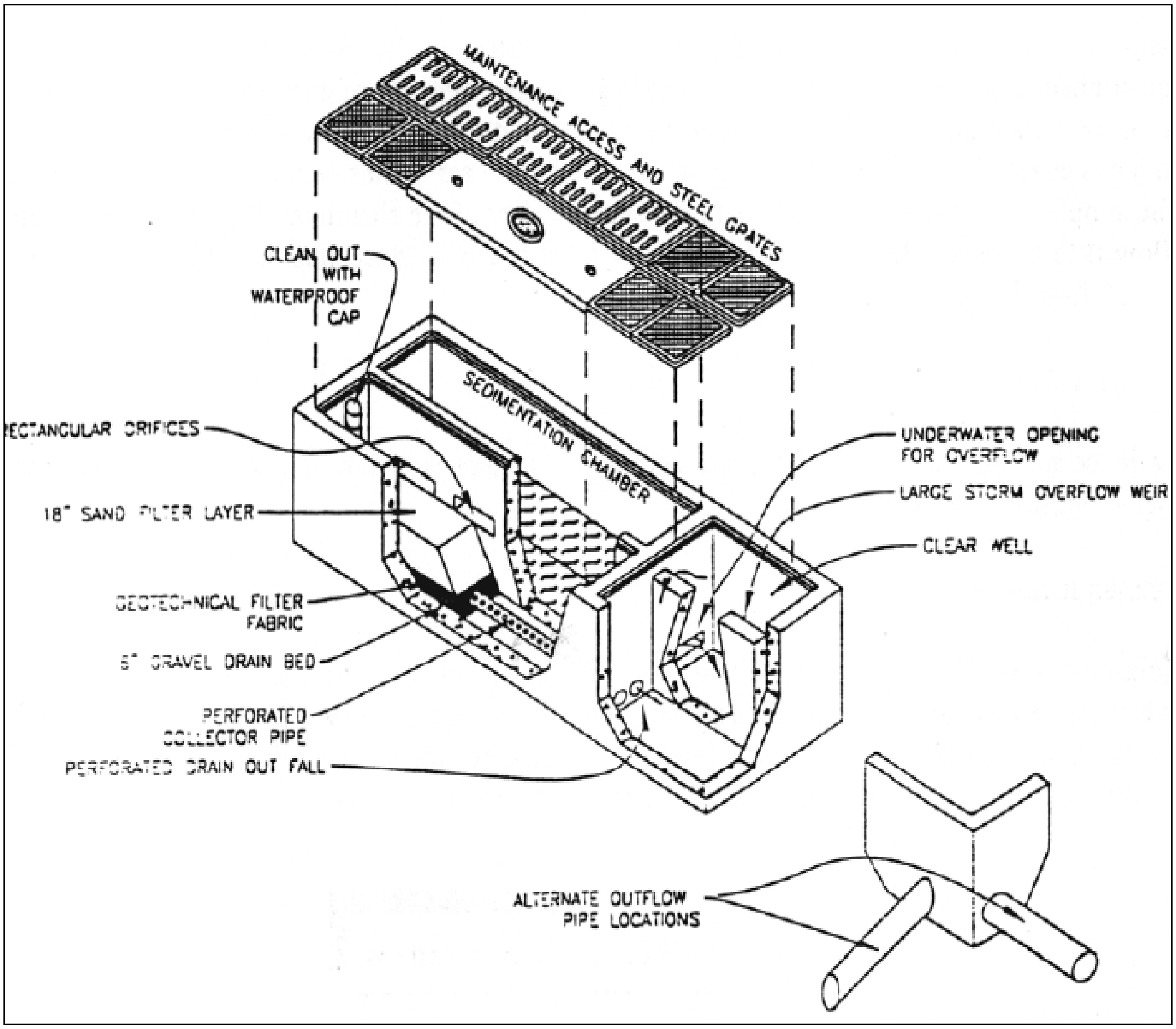


Figure 11.5.2 – DCR/DEQ Delaware Sand Filter
(*Virginia Stormwater Management Handbook, 1999, Et seq.)*

The Delaware sand filter shown in Figure 12.2 was originally conceived as an *online* facility (unlike the Washington D.C. sand filter), processing all runoff leaving its contributing drainage shed up to the point that overflow is reached. When applied on VDOT projects, the Delaware sand filter should be equipped with a flow-splitting device such that only the site water quality volume is treated by the filter. The Delaware sand filter is characterized by two parallel chambers, one serving as pre-treatment sedimentation chamber and the other holding the sand filter media. The pre-treatment chamber holds a permanent pool analogous to that of a septic tank. Flow entering the pre-treatment chamber causes the water level in the chamber to rise and eventually spill into the filter chamber where full treatment occurs. Upon filtering through the sand media, treated runoff is collected in the clearwell located at the lower end of the structure. From there, the treated runoff is directed to the receiving storm sewer.

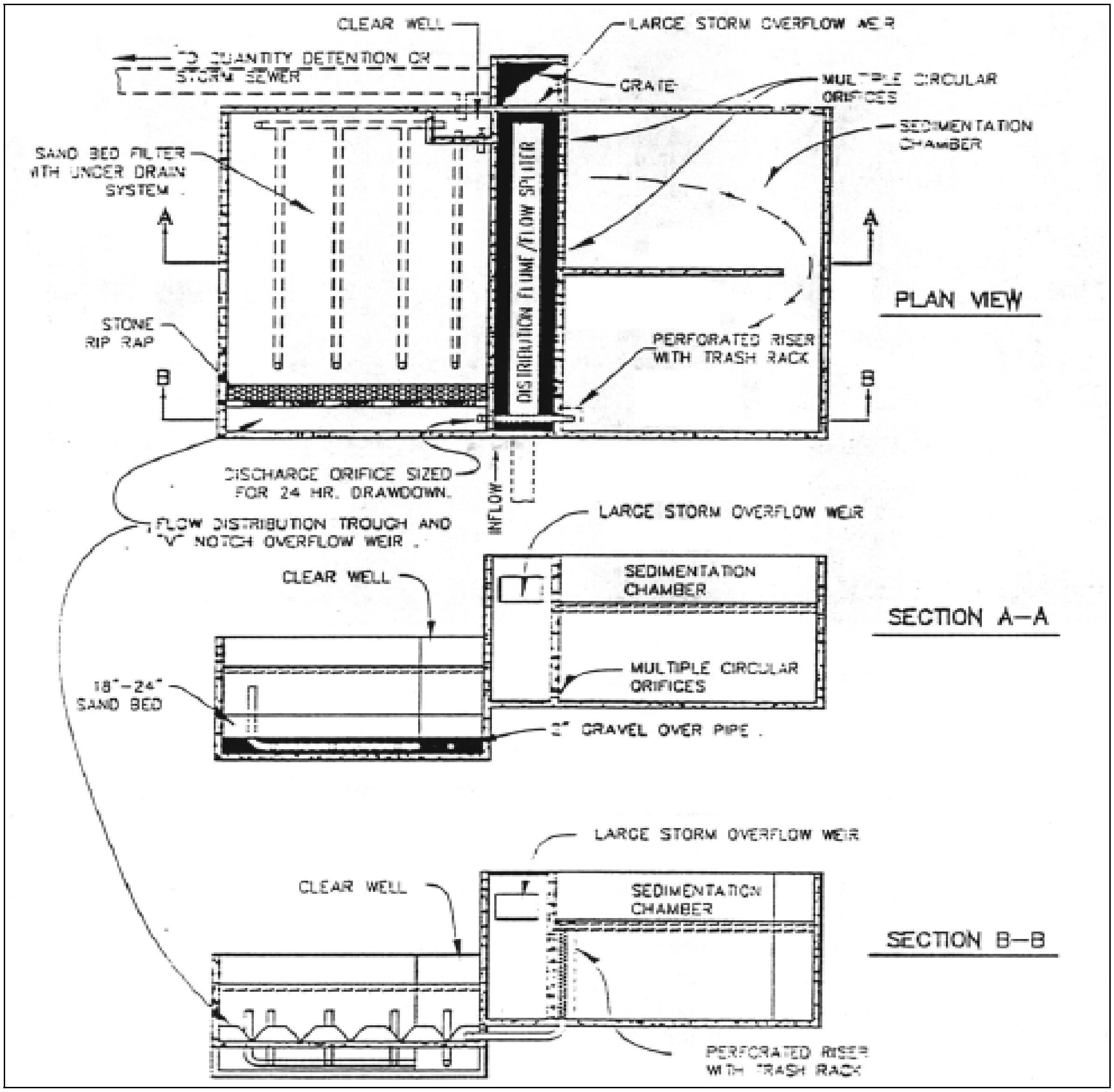


Figure 11.5.3 – DCR/DEQ Austin Surface Sand Filter
(*Virginia Stormwater Management Handbook, 1999, Et seq.)*

The Austin surface sand filter, as shown in Figure 12.3, is composed of an open basin characterized by a pre-treatment sedimentation basin that is often large enough to hold the entire water quality volume from the contributing drainage shed. This volume is then released into the sand bed filtration chamber over a period of 24 hours. Alternative designs employ a much smaller sedimentation chamber, and compensate for the increased clogging potential by increasing the surface area of the filtration chamber. Typically, both chambers of the Austin filter are constructed of concrete; however, when soil conditions and/or the application of a geomembrane liner permit, the pre-treatment sedimentation chamber may be constructed into the ground.

12.1 Site Constraints and Siting of the Filter

The designer must consider a number of site constraints in addition to the contributing drainage area's new impervious cover when a stormwater sand filter is proposed. These constraints are discussed as follows.

12.1.1 Minimum Drainage Area

The minimum drainage area contributing to an intermittent stormwater sand filter is not restricted. These types of filters are best suited to small drainage areas.

12.1.2 Maximum Drainage Area

The maximum drainage area to a single stormwater sand filter varies by filter type. Table 12.1 shows the impervious acreage which may be directed to a single filter, as a function of filter type.

Table 12.1.1 - Appropriate Drainage Area by Filter Type

Filter Type	Appropriate Drainage Shed (Impervious Acres)
D.C. Underground Vault	0.25 – 1.25
Delaware	1.25 Maximum
Austin Surface	Greater than 1.25

Austin surface sand filters have been applied on sites with drainage areas as large as 30 acres; however on sites greater than 10 acres, despite a reduction in cost per volume of runoff treated arising from the economy of scale, the cost-effectiveness of an Austin sand filter is often poor when compared to alternative BMP options.

12.1.3 Elevation of Site Infrastructure

Whenever possible, stormwater filtering systems should be designed to operate exclusively by gravity flow. This requires close examination of the difference in elevation between the filter's discharge point (manhole, pipe, or receiving channel) and the storm sewer discharging runoff into the filter. This difference in elevation dictates the hydraulic head available on the filter while still remaining in a state of gravity flow. When the filter's clearwell discharge point is below the elevation of the downstream receiving point, an effluent pump is a viable alternative; however, this option requires routine scheduled maintenance by trained crews knowledgeable in the maintenance of such mechanical equipment.

12.1.4 Depth to Water Table and/or Bedrock

The liner or concrete shell of a sand filter should generally be located 2’ to 4’ above the site seasonally high water table. The presence of a high water table can flood the filter during construction. Additionally, placing a sand filter within the groundwater table may give rise to infiltration, thus flooding the filter and rendering it inoperable during periods of inflow. When it is deemed feasible and desirable to employ an intermittent sand filter on a site exhibiting a shallow groundwater table, the effects of infiltration and flotation must be accounted for. The liner or concrete shell of the filter must be waterproofed in accordance with the methods and materials specified by the Materials Division. Additionally, buoyancy calculations must be performed and additional weight provided within the filter as necessary to prevent floatation.

12.1.5 Existing Utilities

Sand filters may be constructed over existing easements, provided permission to construct the facility over these easements is obtained from the utility owner *prior* to design.

12.1.6 Wetlands

When the construction of a sand filter is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation in their vicinity.

12.1.7 Upstream Sediment Loading

The primary cause of filter failure is premature clogging arising from the presence of excessive sediment in the runoff directed to the filter. Therefore, runoff directed to stormwater filters should originate primarily from small impervious watersheds. In most applications, runoff flows through an open air “pretreatment” chamber prior to entering the filter chamber. This process allows large particles and debris to settle out. The filters described in this document should not be used on sites exhibiting an impervious cover of less than 65%.

12.1.8 Aesthetic Considerations

Stormwater sand filters provide an attractive BMP option on high profile sites where visually obtrusive BMPs such as extended dry detention facilities and other basins are undesirable. Typically, sand filtration BMPs are visually unobtrusive and may be located on sites where aesthetic considerations and/or the preservation of open space is deemed a priority.

12.1.9 Control of Surface Debris

Sand filters constructed as underground vaults often receive “Confined Space” designation under Occupational Safety and Health Administration (OSHA) regulations. Consequently, maintenance operations involving personnel entering the vault may become quite costly. In an effort to reduce the frequency of this type of maintenance operation, prevention of trash and other debris from entering the filter should be prioritized. This is accomplished through the use of trash racks and flow-splitting devices on offline facilities.

12.1.10 Hydrocarbon Loading

Sand filters are capable of receiving hydrocarbon-laden runoff; however, the facility owner must realize that such loading conditions will inevitably lead to rapid clogging of the filter media. When the presence of hydrocarbons is anticipated in the runoff entering a sand filter, the filter’s pre-treatment chamber should be designed to remove unemulsified hydrocarbons prior to their entrance into the primary filter chamber. An alternative option is to provide an upstream “treatment train” composed of a BMP(s) capable of reducing the level of hydrocarbons present in the runoff entering the sand filter.

12.1.11 Perennial and Chlorinated Flows

Sand filters must not be subjected to continuous or very frequent flows. Such conditions will lead to anaerobic conditions which support the export of previously captured pollutants from the facility. Additionally, sand filters must not be subjected to chlorinated flows, such as those from swimming pools or saunas. The presence of elevated chlorine levels can potentially kill the desirable bacteria responsible for the majority of nitrogen uptake in the facility.

12.1.12 Surface Loading

Sand filters constructed as underground vaults must have their load-bearing capacity evaluated by a licensed structural engineer. This evaluation is of paramount importance when the filter is to be located under parking lots, driveways, roadways, or adjacent to highways.

12.2 General Design Guidelines

The following presents a collection of design issues to be considered when designing a sand filter for improvement of water quality.

12.2.1 Isolation of the Water Quality Volume (WQV)

Sand filters should have only the site water quality volume directed to them. In Virginia, this is also true for the Delaware sand filter which has traditionally been installed online with stormwater conveyance systems. The most popular means of isolating the water quality volume is through the use of a diversion weir in the manhole, channel, or pipe conveying runoff to the BMP. Typically, the elevation of this weir is set equal with the water surface elevation in the BMP when the water quality volume is present. This approach ensures that flows beyond the water quality volume bypass the filter and are conveyed downstream by the storm drainage system. It is noted that the flow-splitter or diversion weir is used to convey a designated *volume* of runoff into the filter rather than to simply regulate the flow *rate* into the filter. The diversion structure may be prefabricated, or cast in place during construction. A schematic illustration of the flow-splitting weir is shown as follows:

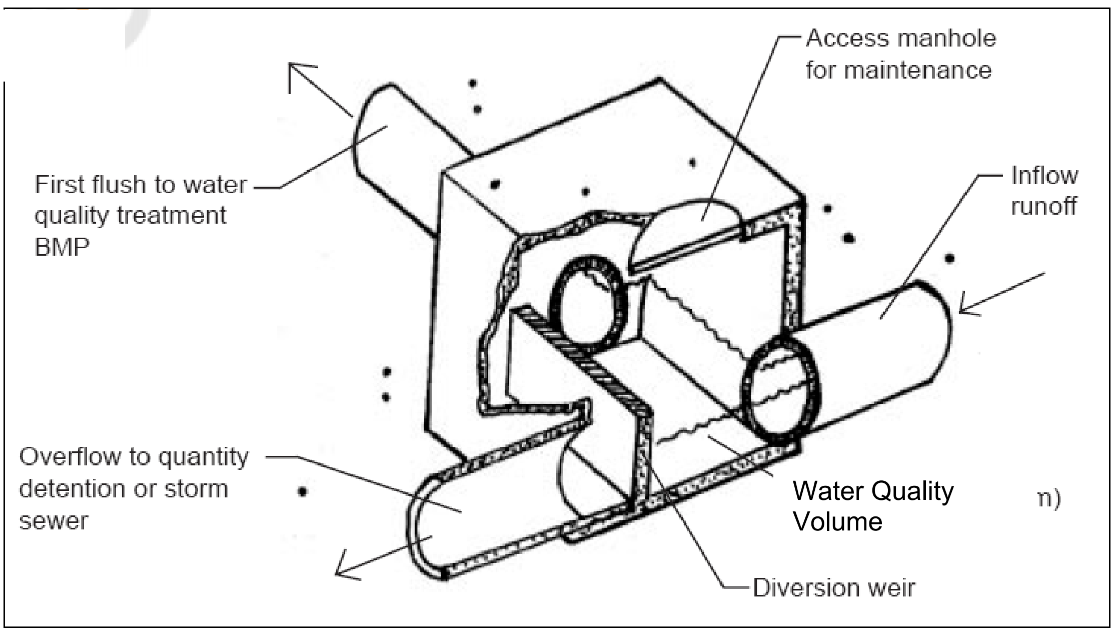


Figure 12.2.1 - Flow-splitting Diversion Weir (Bell, Warren, 1993)

Typically, the construction of the diversion weir will place its crest elevation equal to the maximum allowable ponding depth on the sand filter. This results in flow over the diversion weir when runoff volumes greater than the computed water quality volume enter the stormwater conveyance channel. This configuration results in minimal mixing between the held water quality volume and flows from large runoff producing events in excess of this volume.

An alternative approach is to provide a "low flow" pipe leading directly from the upstream structure to the sand filter. Water enters the BMP through this low-flow conduit, and once the water level rises to that equal with the allowable ponding depth on the filter, flow is conveyed downstream by a bypass pipe located at a higher elevation. A schematic illustration of this configuration is shown as follows:

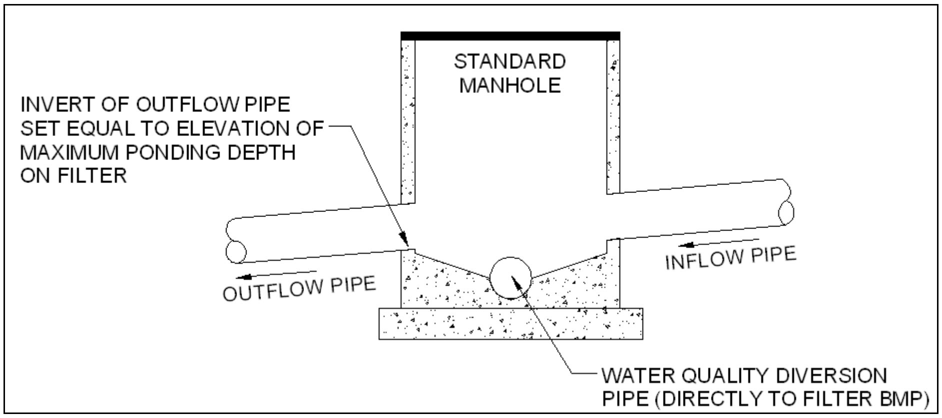


Figure 12.2.2 - Flow-Splitting Manhole Structure

12.2.2 Sand Filter Media

The sand filter media of an intermittent sand filter should meet the specifications of VDOT Grade A Fine Aggregate or as otherwise approved by the Materials Division.

12.2.3 Discharge Flows

All filter outfalls must discharge into an adequate receiving channel as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR/DEQ, 1992, Et seq.). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

12.2.4 Filter Sizing

Sand filters should be sized using a Darcy's Law approach, ensuring that the site water quality volume is filtered completely through the sand media within a maximum of 40 hours. Sizing the filter such that full drawdown of the water quality volume occurs within 40 hours ensures that aerobic conditions are maintained in the filter between storm events.

The coefficient of permeability of a filter's sand media may range as high as 3.0 feet/hour upon installation; however, due to filter clogging after only a few runoff producing events, the rate of permeability through the media has been observed to decrease considerably. Therefore, the coefficient of permeability employed in filter sizing calculations is a function of the degree to which pre-treatment is planned for the facility (full pre-treatment or partial pre-treatment). The following section presents specific sizing guidelines for each of the previously described types of sand filters in the context of a design scenario.

12.3 Design Process

This section presents the design process applicable to sand filters serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT *facilities* projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 4 of the Virginia Stormwater Management Handbook (DCR/DEQ, 1999, Et seq.) for expanded hydrologic methodology.

A design example is presented for each of the three aforementioned types of sand filter recommended for use in Virginia. The filter designs will meet the technology-based water quality requirements arising from a one-acre VDOT maintenance yard. The site water quality volume is directed into the filter by means of a diversion weir situated in the storm sewer. This example is an *offline* configuration. The design will include a Washington D.C. sand filter, a Delaware sand filter, and an Austin sand filter.

The total project site, including right-of-way and all permanent easements, consists of 1.0 acre. Pre and post-development land cover and hydrologic characteristics are summarized below in Table 12.2.

Table 12.3.1 - Hydrologic Characteristics of Example Project Site

	Pre-Development	Post-Development
Project Area (acres)	1.0	1.0
Land Cover	Unimproved Grass Cover	1.0 acres *new* impervious cover
Impervious Percentage	0	100

Site topography is such that the invert of the pipe exiting the sand filter from its clearwell chamber is 4.5’ lower than the invert of the storm sewer pipe discharging runoff into the filter’s pre-treatment chamber.

Step 1 - Compute the Required Water Quality Volume

The project site’s water quality volume is a function of the developed new impervious area. This *basic* water quality volume is computed as follows:

$$WQV = \frac{NIA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

NIA= New Impervious Area (ac.)

The project site in this example is composed of a total drainage area of 1.0 acres. The total new impervious area within the site is 1.0 acres. Therefore, the *basic* water quality volume is computed as follows:

$$WQV = \frac{1.0ac \times \frac{1}{2}in}{12\frac{in}{ft}} = 0.042\,ac \cdot ft \times \frac{43{,}560\,ft^2}{ac} = 1{,}830\,ft^3$$

Referencing Table 1.1, sand filters treating drainage sheds whose impervious fraction ranges between 67 and 100% should be sized for *twice* the basic water quality volume. Therefore, the filters in this example will be sized to treat a volume of 3,660 ft^3.

Upon evaluating various site constraints, cost, and maintenance considerations the designer will select which of the aforementioned types of sand filter best meets the site water quality needs. The following section demonstrates the sizing procedure for each of three types of intermittent sand filter.

Step 2A - Size Filter and Pre-Treatment Sedimentation Chamber – Washington D.C. Underground Vault Sand Filter

The variables expressed in the D.C. sand filter sizing equations are related to the following figure.

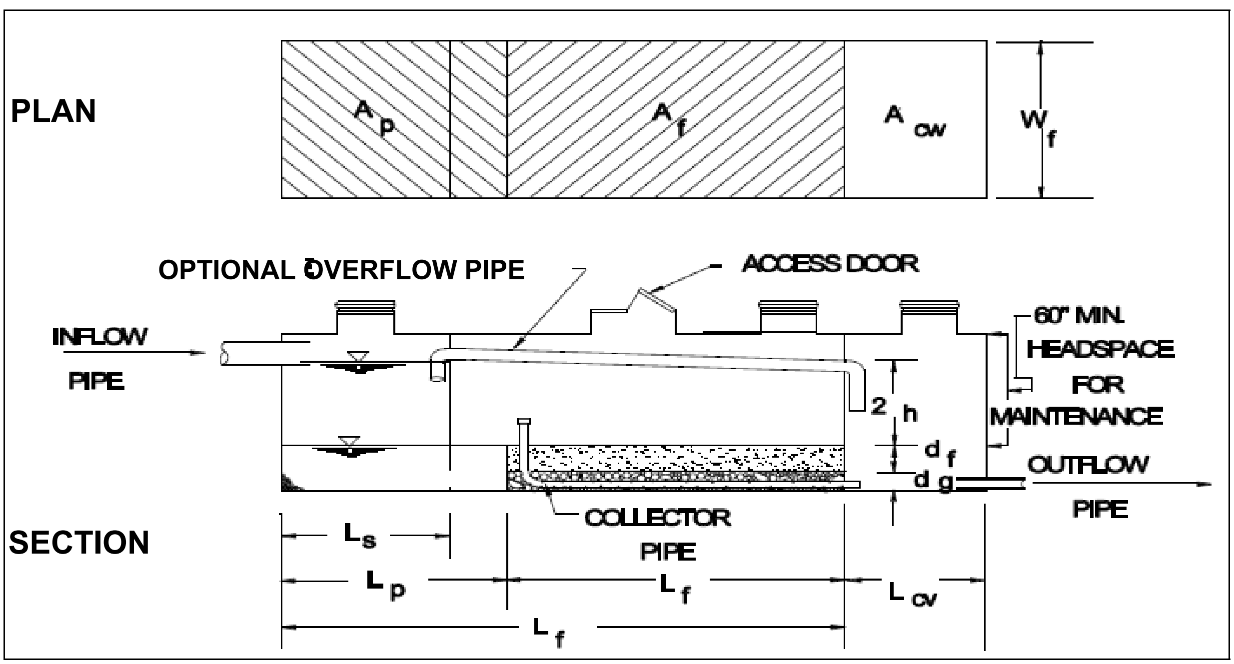


Figure 12.3.1 – DCR/DEQ D.C. Sand Filter – Cross Section
(Virginia Stormwater Management Handbook, 1999, Et seq.)

The D.C. sand filter is a *partial pre-treatment* intermittent sand filter. The total surface area of the sand media is computed by the following equation:

$$A_f = \frac{545 I_a d_f}{\left(h + d_f\right)}$$

A_f= Minimum surface area of sand bed (square feet)
I_a= Impervious fraction of contributing drainage shed (acres)
d_f= Sand bed depth (typically 1.5 to 2.0')
h= Average depth of water above surface of sand media (ft)

In this application, we will select a sand media depth of 2'. The sand filter media must be wrapped in a filter cloth approved by the Materials Division. Additionally, the sand layer is then underlain by a layer of ½" - 2" diameter washed gravel (10" thick) and overlain by a layer of 1" – 2" diameter washed gravel (1 – 2" thick).

The overall depth of all filter media is the sum of the sand media and the gravel underlay and overlay. This depth calculation is as follows:

$$d_m = d_f + d_g = 24in + 10in + 2in = 36in = 3ft$$

It was previously determined that the total elevation difference between the pipe discharging runoff into the filter and the pipe carrying effluent from the filter is 4.5'. Therefore, as shown in Figure 12.5, the *maximum* possible ponding depth, *2h*, on the filter is calculated by subtracting the total filter media depth from this total elevation difference:

$$2h = 4.5ft - 3ft = 1.5ft$$

Therefore, the average ponding depth on the filter, *h*, is determined to be *0.75'*.

The required surface area of the sand filter media is then computed as:

$$A_f = \frac{545(1.0ac)(2ft)}{(0.75ft + 2ft)} = 396.4ft^2$$

Next, the length and width of the filter are computed. This design will employ a rectangular configuration with at 2:1 length-to-width ratio.

$$L_f = 2W_f$$
$$2W_f^{\,2} = 396.4ft^2 \Rightarrow W_f = 14.1ft$$
$$L_f = 28.2ft$$

Rounding the computed dimensions to nominal values yields the following filter surface parameters:

Table 12.3.2 - D.C. Filter Surface Dimensions

L_f (ft)	W_f (ft)	A_f (ft2)
28.5	14	399

The next step is to compute the maximum available storage volume on the surface of the filter, V_{Tf}. This is computed based on the filter surface area and the maximum possible ponding depth, *2h* (1.5'):

$$V_{Tf} = 399\,ft \times 1.5\,ft = 598.5\,ft^3$$

Next, the total storage volume provided in the void space of the gravel and sand media is computed. The porosity of the sand and gravel filter media is typically taken to be 40%.

$$V_V = 0.4 \times A_f \times \left(d_f + d_g\right)$$
$$V_V = 0.4 \times 399\,ft^2 \times \left(2\,ft + 1\,ft\right) = 478.8\,ft^3$$

The next step is to compute the volume of inflow that passes through the filter media while the total water quality volume is accumulating in the BMP. This calculation is based on a coefficient of permeability, *k*, of 2 ft/day (0.0833 ft/hr) for the sand media and a total filling time of one hour. The pass-through volume during filling is computed by the following equation:

$$V_Q = \frac{kA_f\left(d_f + h\right)}{d_f}$$

For the design parameters previously established, the pass-through volume is computed as:

$$V_Q = \frac{0.0833\dfrac{ft}{hr}\left(399\,ft^2\right)\left(2\,ft + 0.75\,ft\right)}{2\,ft} = 45.7\,ft^3$$

The volume which must be stored awaiting filtration is computed from the following equation:

$$V_{st} = WQV - V_{Tf} - V_V - V_Q$$

For the design parameters previously established, the required storage volume, V_{st}, is computed as:

$$V_{st} = 3{,}660\,ft^3 - 598.5\,ft^3 - 478.8\,ft^3 - 45.7\,ft^3 = 2{,}537\,ft^3$$

The volume to be stored awaiting filtration dictates sizing of the filter's permanent pool volume. The length of this pool is defined as L_p (see Figure 12.6), and is computed as follows:

$$L_p = \frac{V_{st}}{(2h \times W_f)}$$

For the design parameters previously established, the permanent pool length, is computed as:

$$L_p = \frac{2{,}537\, ft^3}{(1.5\, ft \times 14\, ft)} = 120.8\, ft$$

The next design step is to compute the length of the sedimentation chamber, L_S, to provide storage for 20% of the site water quality volume (standard for a partial pre-treatment practice). The length of the sedimentation chamber is computed by the following equation:

$$L_s = \frac{0.2WQV}{(2h \times W_f)}$$

For the design parameters previously established, the length of the filter's sedimentation chamber is computed as:

$$L_s = \frac{0.2 \times 3{,}660\, ft^3}{(1.5\, ft \times 14\, ft)} = 34.9\, ft$$

The final design step is to adjust the length of the permanent pool. If the computed length of the permanent pool is greater than the length of the sedimentation chamber plus 2', then the permanent pool length is not adjusted; however, if the computed length of the permanent pool is less than the length of the sedimentation chamber plus 2', the permanent pool length should be increased to dimensions of $L_S + 2'$. In this example no adjustment is necessary.

Table 12.4 presents the final design summary of the Washington D.C. sand filter, with variables as defined in Figure 12.6.

Table 12.3.3 - Design Summary – D.C. Sand Filter

Filter Length (L_f) ft	Filter Width (W_f) ft	Filter Area (A_f) ft^2	Permanent Pool Length (L_p) ft	Sedimentation Chamber Length (L_s) ft
28.5	14	399	120.8	34.9

Special Considerations for Implementation of a Washington D.C. Intermittent Sand Filter

- For maintenance access, a minimum of 60” of headroom is required in the sedimentation and filter chambers. In the filtration chamber, this headroom should be measured from the top of the filter media.

- Passage of flow from the sedimentation chamber to the filter chamber should occur through an opening located a minimum of 18” below the depth of the weir dividing the two chambers. The cross-sectional area of this opening should, at a minimum, be 1.5 times the area of the pipe(s) discharging into the BMP.

- The total depth of the filter media must at least equal the height of weir separating the sedimentation and filtration chambers

- The filtration bed’s underdrain piping should consist of three 6” diameter schedule 40 perforated PVC pipes placed on 1% slope. Perforations should be 3/8” diameter with maximum spacing between perforated rows of 6”. The underdrain piping should be placed within the gravel filter media with a minimum of 2” of cover over the pipes.

- When the filter is placed underground, a dewatering drain controlled by a gate valve must be located between the filter chamber and the clearwell chamber.

- Access should be provided to each filter chamber through manholes of at least 22” in diameter.

Step 2B - Size Filter and Pre-Treatment Sedimentation Chamber – Delaware Sand Filter

The variables expressed in the Delaware sand filter sizing equations are related to the following figure:

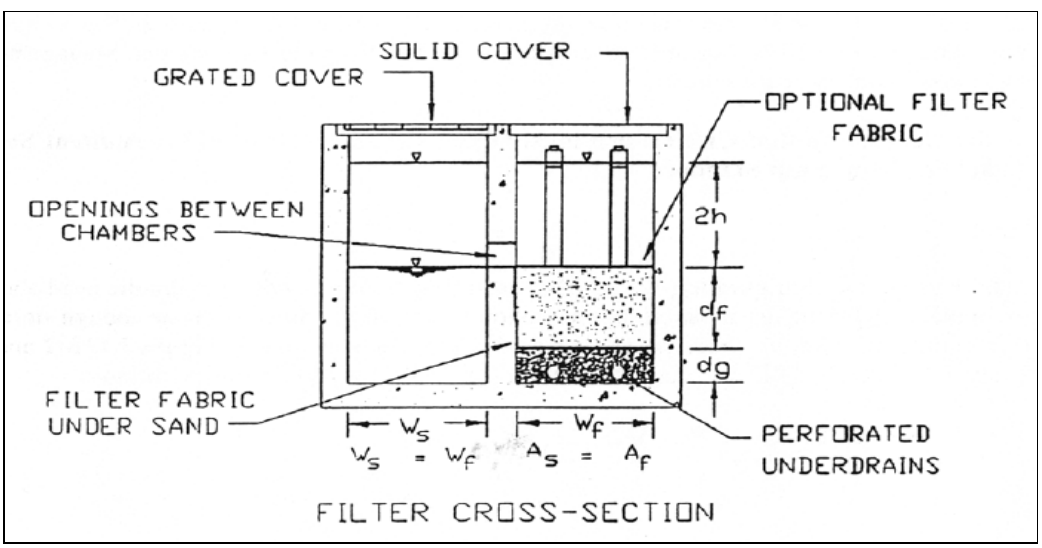


Figure 12.3.2 – DCR/DEQ Delaware Sand Filter – Cross Section
(*<u>Virginia Stormwater Management Handbook</u>, 1999, Et seq.)*

The Delaware sand filter's shallow configuration typically results in minimal hydraulic head acting on the filter. This configuration makes the Delaware filter ideal on sites with limited elevation difference between filter inflow and outflow points. Depending on site-specific constraints, and the maximum available hydraulic head, one of two different equations governs sizing of the filter surface area.

If the maximum hydraulic head acting on the filter (*2h* as shown in Figure 12.7) is less than 2'-8", the following equation should be used to compute the minimum filter surface area:

$$A_f = \frac{WQV}{(4.1h + d_f)}$$

WQV=Water quality volume
A_f= Minimum surface area of sand bed (square feet)
d_f= Sand bed depth (typically 1.5 to 2.0')
h= Average depth of water above surface of sand media (ft)

When the maximum available head is greater than 2'-8", the following equation governs sizing of the filter surface area:

$$A_f = \frac{545 I_a d_f}{(h + d_f)}$$

I_a= Impervious fraction of contributing drainage shed (acres)

It was previously determined that the total elevation difference between the pipe discharging runoff into the filter and the pipe carrying effluent from the filter is 4.5'. Therefore, the *maximum* possible ponding depth, *2h*, on the filter is calculated by subtracting the total filter media depth from this total elevation difference:

$$2h = 4.5\,ft - 3\,ft = 1.5\,ft$$

Therefore, the first equation applies as the available head on the filter is less than 2'-8". In this application, we will select a sand media depth of 2'. The average ponding depth on the filter, *h*, is determined to be *0.75'* and the filter surface area is computed as:

$$A_f = \frac{3{,}660\,ft^3}{((4.1)(0.75\,ft) + 2\,ft)} = 721.2\,ft^2$$

Next, the length and width of the filter are computed. This design will employ a rectangular configuration with a 2:1 length-to-width ratio.

$$L_f = 2W_f$$

$$2W_f^{\,2} = 721.2\,ft^2 \Rightarrow W_f = 19.0\,ft$$

$$L_f = 38.0\,ft$$

Rounding the computed dimensions to nominal values yields the following filter surface parameters:

Table 12.3.4 - Delaware Filter Surface Dimensions

L_f (ft)	W_f (ft)	A_f (ft2)
38	19	722

The Delaware sand filter is characterized by two parallel chambers, one serving as a pre-treatment sedimentation chamber and the other holding the sand filter media. The dimensions of the sedimentation chamber (L_s, W_s, and A_s) are identical to those of the filtration chamber shown in Table 12.5.

Special Considerations for Implementation of a Delaware Intermittent Sand Filter

- The filtration bed's underdrain piping should consist of two 4" diameter schedule 40 perforated PVC pipes placed on 1% slope. Perforations should be 3/8" diameter, minimum 4 holes per row, and row spacing a maximum of 6". The underdrain piping should be placed within the gravel filter media with a minimum of 2" of cover over the pipes.

- Weepholes are recommended between the filter chamber and the clearwell to permit draining if the underdrain piping should fail or become clogged.

- It is recommended that the sand filter media be wrapped in a filter cloth approved by the Materials Division. Additionally, the sand layer should be underlain by a layer of ½" - 2" diameter washed gravel (10" thick) and overlain by a layer of 1" – 2" diameter washed gravel (1 – 2" thick).

Step 2C - Size Filter and Pre-Treatment Sedimentation Chamber – Austin Surface Sand Filter

The Austin sand filter can be designed for full or partial pre-treatment of sediment. Full pre-treatment of inflow is characterized by capturing and detaining the entire WQV and releasing it into the filtration chamber over a period of not less than 24 hours. Partial pre-treatment of sediment entails providing pre-treatment storage for 20% of the WQV in a sedimentation chamber hydraulically connected to the filtration chamber (as with the D.C. and Delaware sand filters). Sizing of the sand media is a direct function of the volume of pre-treatment. The following equations govern filter sizing:

Filters equipped with full pre-treatment of inflow: $A_f = \dfrac{100\,ft^2}{\text{Acre Treated}}$

Filters equipped with partial pre-treatment of inflow: $A_f = \dfrac{545 I_a d_f}{(h + d_f)}$

This design example will employ full pre-treatment of inflow; therefore, the required filter area is computed as:

$$A_f = \frac{100\,ft^2}{acre} \times 1acre = 100\,ft^2$$

Austin sand filters should be sized with a minimum length-to-width ratio of 2:1. Employing this ratio, the following dimensions are computed for the filter:

$$L_f = 2W_f$$
$$2{W_f}^2 = 100\,ft^2 \Rightarrow W_f = 7.1\,ft$$
$$L_f = 14.2\,ft$$

Rounding the computed dimensions to nominal values yields the following filter surface parameters:

Table 12.3.5 - Austin Filter Surface Dimensions

L_f (ft)	W_f (ft)	A_f (ft2)
14.5	7	101.5

The next step is to size the pre-treatment sedimentation chamber. The surface area of the sedimentation basin is calculated from the Camp-Hazen equation as shown:

$$A_s = \frac{Q_o}{W} \times \left[-\ln(1 - E)\right]$$

With: A_s = sedimentation basin surface area (ft^2)
Q_o = discharge rate from basin (WQV / 24hr)
= $\dfrac{ft^3}{24hr} x \dfrac{1hr}{3600s} = cfs$; where WQV = water quality volume in ft^3
W = particle settling velocity (ft/sec)
E = sediment trapping efficiency of suspended solids (90%)

The particle settling velocity is a function of the impervious area contributing to the filtering practice. The following values are used in sizing the pretreatment basin:

Table 12.3.6 - Particle Settling Velocities (MDE, 2000)

Impervious Percentage	Particle Settling Velocity (ft/sec)
≤75	0.0004
>75	0.0033

The filter under design will serve a site with 100% impervious cover. Therefore, the filter area is computed as:

$$A_s = \frac{3{,}660\,ft^3}{24hour} \times \frac{1hr}{3{,}600\sec} \times \frac{1}{0.0033} \times \left[-\ln(1-0.9)\right] = 29.6\,ft^2$$

Pre-treatment must be provided for the entire WQV. Therefore, the depth of the sedimentation chamber is computed as:

$$d_s = \frac{3{,}660\,ft^3}{29.6\,ft^2} = 123.6\,ft$$

The depth of a sedimentation chamber should not exceed 10'. When the Camp- Hazen approach yields depths exceeding 10', the following equation should be used to size the filter's pre-treatment chamber:

$$A_s = \frac{WQV}{10\,ft}$$

$$A_s = \frac{3{,}660}{10\,ft} = 366\,ft^2$$

The filter pre-treatment chamber will be located parallel to the filter sedimentation chamber as shown in Figure 12.3. Therefore, the length of the pre-treatment chamber is set equal to the length of the sedimentation chamber, 14.5'. The width of the pre-treatment chamber is then computed as follows:

$$W_s = \frac{366\,ft^2}{14.5\,ft} = 25.2\,ft$$

Table 12.8 presents a design summary of the Austin sand filter.

Table 12.3.7. Design Summary – Austin Sand Filter

Filter Length (L_f) ft	Filter Width (W_f) ft	Filter Area (A_f) ft^2	Sedimentation Chamber Length (L_s) ft	Sedimentation Chamber Width (W_s) ft
14.5	7	101.5	14.5	25.2

The next step is to design an outlet configuration that will discharge the WQV from the pre-treatment chamber to the sedimentation chamber over a period of not less than 24 hours. Typically this conveyance occurs through a perforated stand pipe as shown in Figure 12.3. Control of flow should be dictated by a throttle plate or other flow-restricting mechanism, *not* the perforations in the stand pipe. The following steps illustrate sizing of the orifice.

Discharge of the water quality volume from the pre-treatment chamber to the filter chamber must occur over a period of not less than 24 hours. The Virginia Stormwater Management Handbook identifies two methods for sizing a water quality release orifice. The VDOT preferred method is METHOD 2, “average head/average discharge.”

The water quality volume is attained at a ponded depth of 10’ in the pre-treatment chamber, therefore the average head associated with this volume is computed as:

$$h_{avg} = \frac{10\,ft}{2} = 5\,ft$$

$$Q_{avg} = \frac{WQV}{(24hr)(3{,}600\sec/hr)} = \frac{3{,}660\,ft^3}{(24hr)(3{,}600\sec/hr)} = 0.04cfs$$

Next, the orifice equation is rearranged and used to compute the required orifice diameter.

$$Q = Ca\sqrt{2gh}$$

Q= discharge (cfs)
C= orifice Coefficient (0.6)
a= orifice Area (ft^2)
g= gravitational acceleration (32.2 ft/sec^2)
h= head (ft)

The head is estimated as that acting upon the *invert* of the water quality orifice when the total water quality volume of 1,830 ft^3 is present in the chamber. While the orifice equation should employ the head acting upon the center of the orifice, the orifice diameter is presently unknown. Therefore, the head acting upon the orifice invert is used. The small error incurred from this assumption does not compromise the usefulness of the results.

Rearranging the orifice equation, the orifice area is computed as

$$a = \frac{Q_{avg}}{C\sqrt{2gh}} = \frac{0.04}{0.6\sqrt{(2)(32.2)(5)}} = 0.004\,ft^2$$

The diameter is then computed as:

$$d = \sqrt{\frac{4a}{\pi}} = \sqrt{\frac{(4)(0.004)}{3.14}} = 0.071\,ft = 0.852in$$

An orifice with an outlet diameter of 0.75” will be employed to release the water quality volume into the filter chamber over the minimum 24-hour period.

Special Considerations for Implementation of an Austin Intermittent Sand Filter

- The depth of the sand filter media should range between 18” and 24”
- When constructed as an underground vault, a minimum of 60” of headroom is required in the sedimentation and filter chambers. In the filtration chamber, this headroom should be measured from the top of the filter media.
- The minimum length-to-width ratio of the filter chamber is 2:1.
- The pre-treatment sedimentation chamber should include a sediment sump for accumulation and subsequent removal of filtered sediment.

Step 3 - Establish the Crest Elevation of the Water Quality Diversion Weir

The intermittent sand filters presented in this design should have *only* the site water quality volume directed to them. The most popular means of isolating the water quality volume is through the use of a diversion weir in the manhole, channel, or pipe conveying runoff to the BMP. The crest elevation of the weir should be set equal with the water surface elevation corresponding to the maximum available ponding depth on the filter(s), *2h*, as previously defined. This approach ensures that flows beyond the water quality volume bypass the filter and are conveyed downstream by the storm drainage system with minimal mixing of the water quality volume held in the BMP. The weir and downstream receiving structures should typically be sized to accommodate the 10-year return frequency storm

13.1 Vegetated Roofs - Overview of Practice

The following example presents design guidance for Vegetated Roof applications serving runoff quality and quantity needs on VDOT facilities buildings. A vegetated roof cover is a veneer of vegetation that is grown on and completely covers an otherwise conventional roof, thus more closely matching surface vegetation than that of the impervious roof. (PADEP, January 2005)

The vegetated roof veneer may range between 2" and 6" in thickness, and may be comprised of multiple layers including waterproofing membranes, synthetic insulation, engineered and non-engineered soil media. With proper installation and selection of materials, even thin vegetated covers are capable of providing significant rainfall retention, runoff reduction, and water quality improvement.

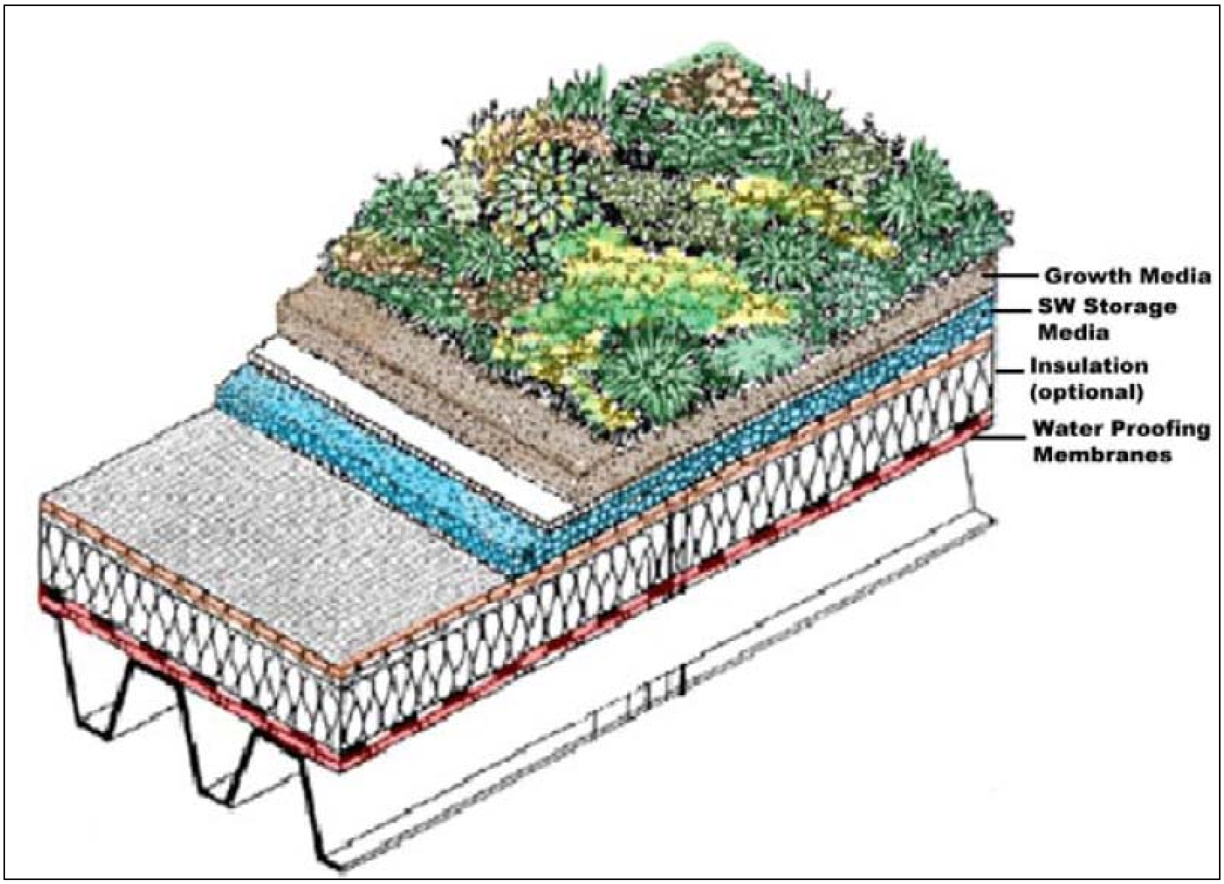


Figure 13.1.1 - Vegetated Roof Schematic (Roofscapes, Inc.)

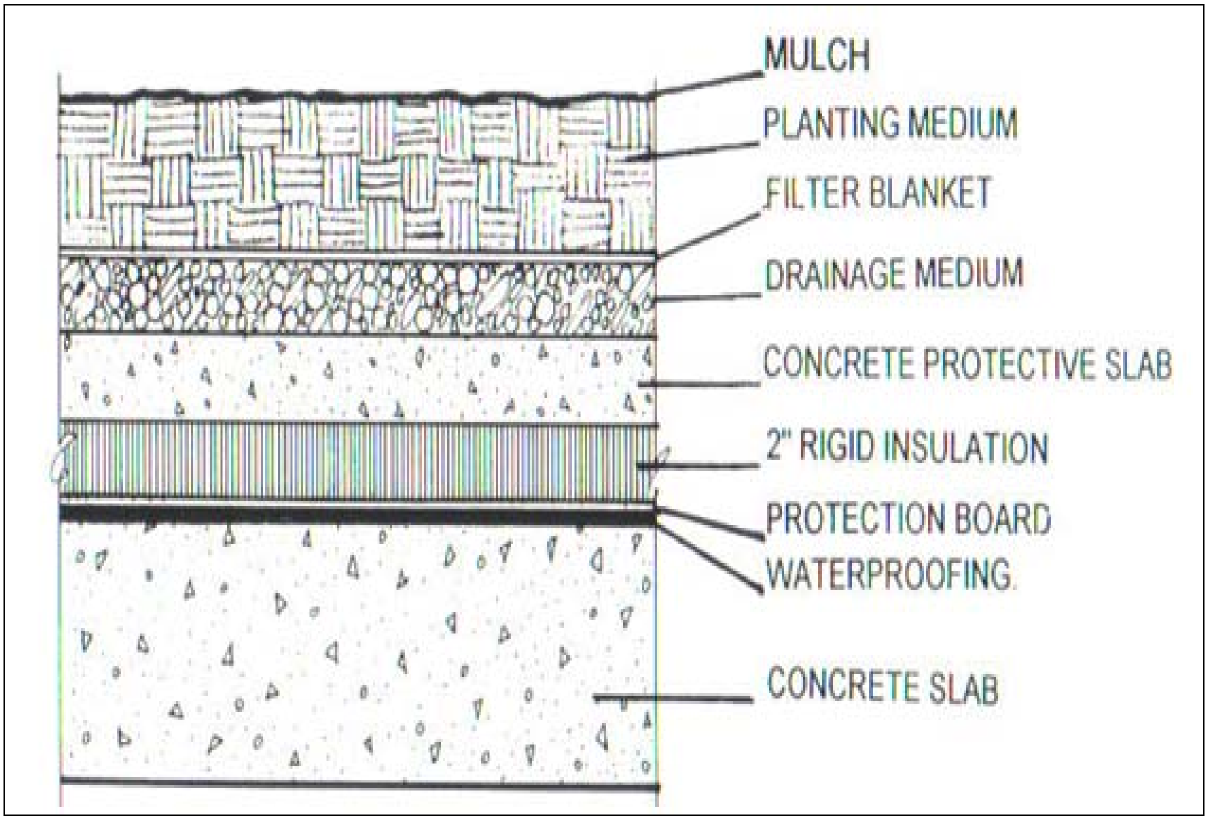


Figure 13.1.2 - Typical Vegetated Roof Section
(Osmundson, 1999)

13.2 General Application Considerations

Vegetated roofs may be applied as part of new construction or in retrofit applications.

Vegetated assemblies on roofs with pitches steeper than 2V:12H must be supplemented with additional structural measures to protect against sliding.

The roof structure of the building for which a vegetated roof practice is planned must be evaluated for compatibility with the anticipated maximum dead and live loads. Typical dead loads for wet vegetated covers range from 8 to 36 lbs/sf. Live loading values can vary considerably and are a function of rainfall retention. Actual design weights should be established using a standardized laboratory procedure.

The application of a vegetated roof system, in all application scenarios, requires a premium waterproofing system.

The chosen vegetation must create a vigorous, drought-tolerant cover. The most successful and commonly used ground covers for un-irrigated roof installations are varieties of Sedum and Delosperma. Vegetated roof designs deeper than 4” to 6” are able to incorporate a wider array of vegetation, including Dianthus, Phlox, Antennaria, and Carex.

Roof access must be provided to ensure proper maintenance and replanting of vegetative cover as necessary.

Source: Pennsylvania DEP *Draft Stormwater Best Management Practices Manual*, January 2005.

13.3 Design Guidelines

Vegetated roof installations intended to serve as water quality BMPs must not be fertilized. Generally, non-irrigated assemblies are strongly preferred, even though they preclude the use of certain, otherwise acceptable, plant species.

Internal building drainage, including provisions to cover and protect deck drains or scuppers (small openings to permit the drainage of water from a floor or rooftop), must anticipate the need to manage large rainfall events without inundating the vegetated cover.

When the selected waterproofing membrane is not root-fast, a supplemental root-barrier must be installed.

National Roofing Contractors Association (NRCA) and American Society for the Testing of Materials (ASTM) standards should be employed when choosing and testing the roof's waterproofing membrane.

Roof flashing should extend 6" higher than the top of the growth media surface and be protected by counter-flashings.

Care must be taken during installation of the vegetated cover to ensure that the waterproofing membrane is not damaged.

The vegetated layer should provide an internal drainage capacity capable of accommodating the two-year return frequency event without generating surface runoff.

Deck drains and scuppers serving to discharge water from the roof area should be equipped with access chambers. These enclosures should include removable lids to allow ready access for inspection.

A vegetated roof's engineered soil media should contain no clay particles and should contain no more than 15% organic matter.

The engineered media employed in vegetated roof applications should have a maximum moisture capacity ranging between 30 and 40%.

If insulation is included in the roof covering system, it may be located above or below the primary waterproofing membrane.

The International Code Council (ICC) and all other applicable standards should be considered for ballasted roofs.

Source: Pennsylvania DEP Draft Stormwater Best Management Practices Manual. January 2005.

13.4 Types of Vegetated Roofs

Vegetated roof systems that exceed 10” in depth are considered *intensive* roof covers. Intensive assemblies are intended primarily to achieve aesthetic and architectural objectives, with only secondary consideration of stormwater management function. These deep intensive systems may be called “roof gardens”. *Extensive* roof covers, by contrast, are usually 6” or less in depth and have a well-defined stormwater management objective as their primary function. The focus in this example is on the design of an extensive vegetated roof BMP.

Vegetated roof BMPs generally fall into three design categories:

- Single media with synthetic underdrain layer
- Dual media
- Dual media with synthetic retention/detention layer

13.4.1 Single Media Assemblies

Single media assemblies are most often used in pitched roof applications, and for thin and lightweight applications. The plants are selected from very drought-tolerant species, and the engineered media is of very high permeability. The profile of a single media vegetated roof assembly is typically as follows:

- Waterproofing membrane
- Root barrier (optional, depending upon the root resistance properties of the waterproofing membrane)
- Semi-rigid plastic geotextile drain or mat
- Separation geotextile
- Engineered growth media
- Foliage layer

Single media vegetated roof assemblies installed on pitched roofs may require the use of slope bars, rigid slope stabilization panels, cribbing, reinforcing mesh, or other provisions to prevent sliding and instability.

Single media assemblies used on flat roofs typically require a network of perforated internal drainage conduits to effectively convey percolated rainfall to deck drains and scuppers.

Assemblies with rigid geotextile drains or mats can be irrigated from beneath, while assemblies with drainage composites will require direct watering.

13.4.2 Dual Media Assemblies

In contrast to single media assemblies, dual media vegetated roof assemblies utilize two types of non-soil media. Fine-grained media with some organic content is placed over a basal layer of coarse lightweight mineral aggregate. Dual media assemblies do not include a geocomposite drain. The objective of a dual media assembly is to improve the drought resistance of the system by attempting to replicate a natural growth environment in which sandy topsoil overlies gravelly subsoil. These assemblies are typically 4” to 6” thick and are comprised of the following layers:

- Waterproofing membrane
- Protection layer
- Coarse-grained drainage media
- Root-permeable non-woven separation geotextile
- Fine-grained engineered growth media layer
- Foliage layer

Dual media assemblies are less versatile than their single media counterparts, and their implementation is restricted to roof pitches of 1.5:12 or less.

Large dual media assemblies should incorporate a network of perforated internal drainage piping to convey percolated rainfall.

Dual media assemblies are optimally suited to base irrigation methods.

13.4.3 Dual Media with Synthetic Retention / Detention Layer

Dual media assemblies employ plastic panels (geocomposite drain sheets) with cup-like receptacles on their upper surfaces. These sheets are then filled with coarse lightweight mineral aggregate. The cups trap and retain precipitation. The profile of a dual media system implementing a synthetic holding layer is as follows:

- Waterproofing membrane
- Felt fabric
- Retention / detention panel
- Coarse-grained drainage media
- Separation geotextile
- Fine-grained growth media layer
- Foliage layer

The complexity of the dual media synthetic assembly typically results in a total BMP depth of 5” or greater. These assemblies should only be considered for roof pitches less than or equal to 1:12.

Dual media assemblies equipped with synthetic retention / detention layers are best irrigated by surface spraying or mid-level drip.

13.5 Drainage Provisions

Adequate drainage is essential to the proper functioning of a vegetated roof. Failure of the roof drainage system can lead to loss of vegetation as well as penetration of water into surrounding structures. (Osmundson, 1999) Adequate drainage is a product of two key elements of the vegetated roof – the drainage medium and the drainage piping.

The drainage medium must consist of rot-proof material through which water can percolate and eventually enter the roof drains. In the United States, as early as the 1930's, pebbles and broken rock were being applied in rooftop gardens as a drainage medium.

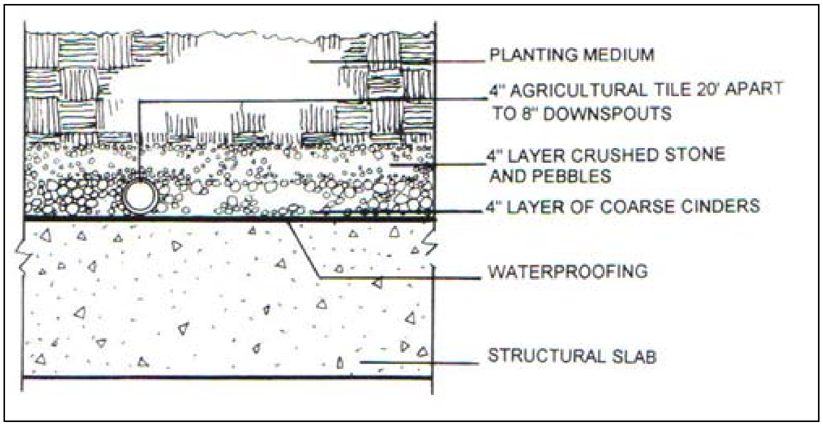


Figure 13.5.1 - Crushed Stone Drainage Medium
(Osmundson, 1999)

The most notable shortcoming of the crushed stone drainage medium shown in Figure 13.3 is its weight. Modern proprietary materials have been developed to provide superior drainage function without the excessive weight of aggregate material with comparable void space. Today, crushed stone drainage mediums are considered obsolete.

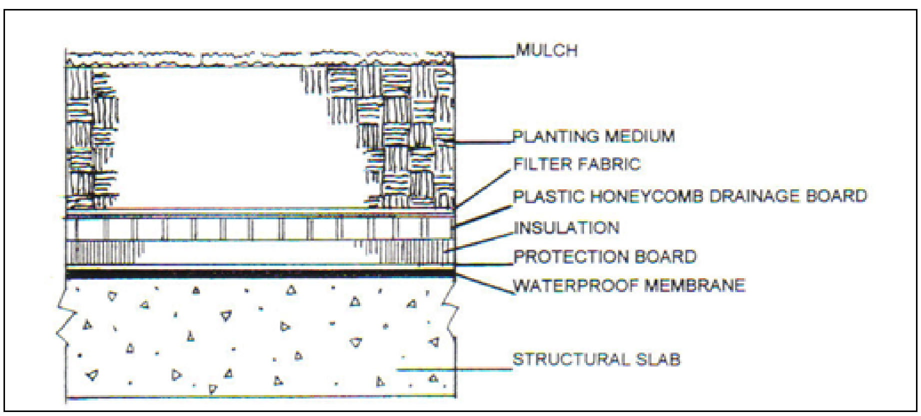


Figure 13.5.2 - Proprietary Structural Drainage Medium
(Osmundson, 1999)

One popular proprietary drainage device is the Grass-Cel system. When topped with a layer of plastic filter fabric (necessary to prevent clogging by the fines contained in overlying planting media), the Grass Cel system provides a strong, easily handled and cut, lightweight drainage layer. Other varieties of proprietary drainage medium are Enkadrain and Geotech.

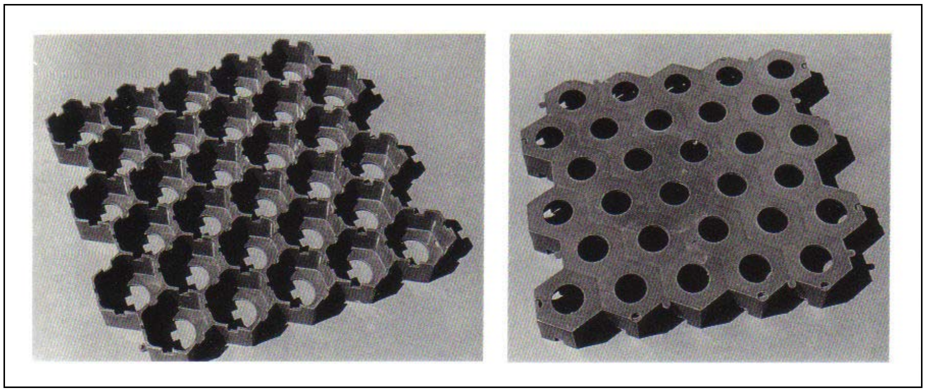


Figure 13.5.3 - Two Types of Grass Cel Drainage Medium
(Osmundson, 1999)

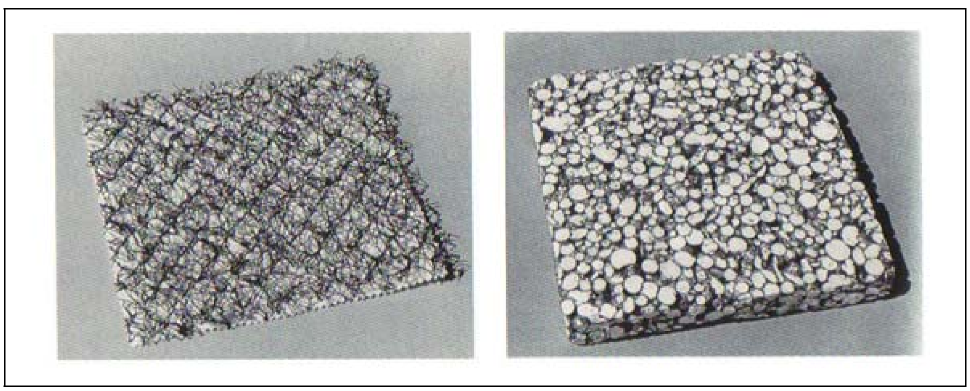


Figure 13.5.4 - Enkadrain (left) and Geotech (right)
(Osmundson, 1999)

Typically, the drainage piping for a vegetated roof assembly will be plastic, cast iron, or brass. A number of different drain types exist.

One type of vegetated roof drain is the *round* or *deck* drain. The round drain is characterized by a grated horizontal top surface and perforated side surfaces. They are useful because their design allows flow to enter at the ground surface level as well as through the sides.

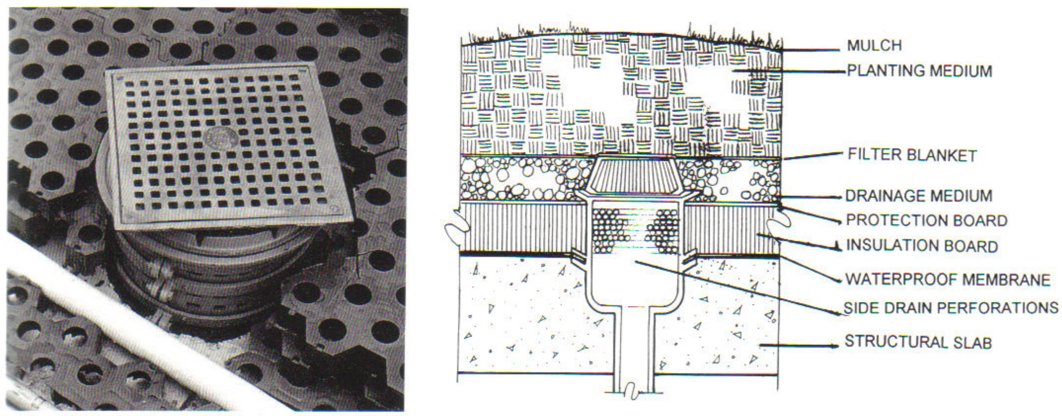


Figure 13.5.5 - Round Drain Situated in Grass Cel Drainage Medium
(Osmundson, 1999)

Another type of vegetated roof drain is the *dome* drain. The dome drain is characterized by its raised dome-shaped surface. It is particularly useful because its elevated surface permits water to enter even when the lower perforations become clogged by leaves and other debris.

A type of drain popular in Europe consists of a combination of sloping concrete trough or gutter in the concrete protective slab covered by a “half-section” of perforated plastic pipe covered in filter fabric. Water entering the system flows through the protective slab, into the gutter, eventually reaching the building downspouts.

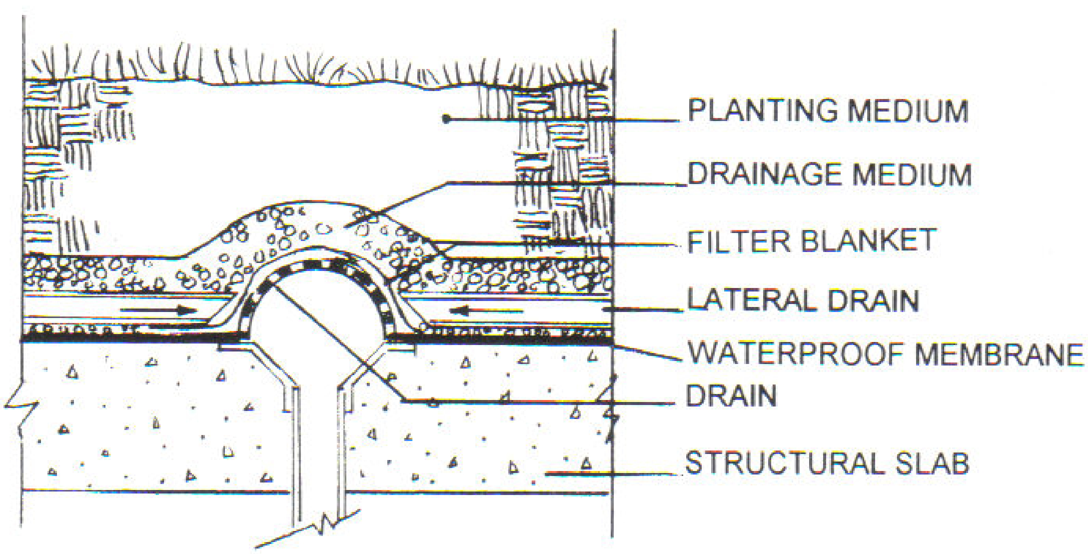


Figure 13.5.6 - Perforated Half Pipe Drain
(Osmundson, 1999)

The filter fabric/blanket chosen to prevent clogging of the drainage medium should meet the following specifications:

- Grab Tensile Strength (ASTM-D4632) 120lbs
- Mullen Burst Strength (ASTM-D3786) 225psi
- Flow Rate (ASTM-D4491) 95 gal/min/ft^2
- UV Resistance after 500 hours (ASTM-D4355)
- Heat-set or heat –calendared fabrics are not permitted.

(Pennsylvania DEP *Draft Stormwater Best Management Practices Manual* – January 2005)

The following is a non-exhaustive list of filter fabric manufacturers:

- Mirafi
- Supac
- Typar
- AMOCO
- EXXON
- TerraTex

U.S. Department of Transportation. Federal Highway Administration. *Evaluation and Management of Highway Runoff Water Quality*. Washington, D.C., 1996

Regardless of the type of drain employed, the system should be equipped with debris-collection basins to avoid clogging of the drainage piping by the inherent presence of debris and fine soil matter. (Osmundson, 1999) The pipes to which the drainage system connects are part of the building drainage system. Therefore, design of the vegetated roof drainage system will require an iterative design approach, working closely with the architect and structural engineer.

13.6 Growth / Planting Media

It is nearly impossible to classify a given soil mixture as optimal for all vegetated roof applications. Detailed performance data for a particular growth media requires long-term, controlled monitoring. In general, however, the growing media should adhere to certain guidelines, described as follows (Source: Osmundson, 1999):

- The optimum planting media consists of 45% sand, 45% soil and 10% humus.

- The presence of silt should be kept to a minimum. Silt possesses the ability to clog the system's filter fabric.

- Mulching should be avoided, as wash-off is likely during severe rainfall producing events.

- The growth media must provide a permanent means of supplying internal aeration to prevent compaction of the mix.

- The selected media must drain completely and efficiently over a 24-hour period.

- The media must be suitable for the plant species chosen. It must be able to supply or absorb water and nutrients for the vegetation to use over time.

- The media should exhibit very little shrink / swell phenomena, retaining its original volume over time.

13.7 Stormwater Peak Rate and Volume Mitigation

While conventional hydrologic methods are used to estimate the runoff from a vegetated roof system, one must consider that the runoff released from the system is not surface runoff, but rather percolated water. The rate and quantity of water released from a vegetated roof assembly during a particular return frequency storm is dependent upon the following physical properties of the assembly.

- Maximum media water retention
- Field capacity
- Plant cover type
- Saturated hydraulic conductivity
- Non-capillary porosity

The assembly's maximum water retention is a product of the quantity of water that the media can hold against gravity in a drained condition.

In the absence of continuous simulation modeling or detailed laboratory performance data, a reasonable approach to assessing peak mitigation performance of a vegetated roof assembly is to compare its performance to that of a conventional impervious roof.

A general rule of thumb when computing runoff from vegetated roof systems is that *for storm events in which the total rainfall depth is no more than three times the maximum media water retention for the assembly, the rate of runoff from the roof will be less than or equal to that of open space*. (PADEP, 2005)

The maximum moisture content of a vegetated roof drainage media is 40%. In the following tables, the required depth of a vegetated roof drainage media layer located in Henrico County is shown by return frequency storm. Vegetated roof assemblies whose drainage media depth and maximum moisture content achieve the target values shown will exhibit runoff patterns similar to undeveloped, open cover conditions.

Table 13.7.1 - Twenty four Hour Rainfall Depths, Henrico County
Virginia Stormwater Management Handbook, (DCR/DEQ, 1999)

Return Frequency (yrs)	24-Hr. Rainfall (in)
2	2.8
10	4.5
25	6.0
100	7.8

For runoff patterns to behave similarly to those of undeveloped open space, the available water retention within the drainage media of a vegetated roof assembly must be greater than or equal to *one third* of the rainfall depth for the return frequency storm for which peak mitigation is desired. These equivalent depths are presented as follows.

Table 13.7.2 - Required Media Moisture Retention Depth for Roof Assembly to Behave as Open Space (Henrico County)

Return Frequency (yrs)	Required Media Moisture Retention (in)
2	0.9
10	1.5
25	2.0
100	2.6

The physical depth of a vegetated roof assembly drainage media needed to achieve the moisture retention depths presented in Table 13.2 is a function of the maximum moisture content available within the media. Below are the required media depths for drainage medium exhibiting moisture contents of 30 and 40% respectively.

Table 13.7.3a - Required Drainage Media Depth for Roof Assembly to Behave as Open Space (30% moisture content)

30% Maximum Moisture Retention	
Return Frequency (yrs)	**Required Drainage Media Depth (in)**
2	3.0
10	5.0
25	6.7
100	8.7

Table 13.7.3b - Required Drainage Media Depth for Roof Assembly to Behave as Open Space (40% moisture content)

40% Maximum Moisture Retention	
Return Frequency (yrs)	**Required Drainage Media Depth (in)**
2	2.3
10	3.8
25	5.0
100	6.5

13.8 Pollutant Removal Performance

While various claims for pollutant removal performance of rooftop gardens have been made, it is not clear at this point that there is a sufficient database to support them. What is clear is that the opportunity of this BMP to intercept overland flow with its associated load of suspended sediment, phosphorous and nitrogen is non-existent. The only true source of pollutants on the rooftop garden will be atmospheric deposition, assuming there is no fertilizer application, as recommended in virtually all guidance documents. We can only surmise there has been little to no investigation of the removal process in the case of atmospheric deposition.

13.9 Vendor Websites

The book by Theodore Osmundson (1999) provides an excellent reference on the landscaping details of rooftop gardens, with many photographs of outstanding installations. However, this reference provides little guidance on the engineering aspects of rooftop drainage and structural design so critical to the success of the rooftop garden. Therefore, we believe it is imperative that the drainage engineer contact various vendors regarding engineered roof top systems, together with the architect and structural engineer for the site development well before the design of any roof top garden system. We have provided a partial list of vendors and their website addresses to assist in this process, recognizing that this list is not exhaustive and that there are other proprietary systems. Our list of vendors does not in any way constitute an endorsement of any one product.

American Hydrotech, Inc
www.hydrotechusa.com

Building Logics
www.buildinglogics.com

Elevated Landscape Technologies Inc. (ELT)
www.eltgreenroofs.com

Green Grid
www.greengridroofs.com

Henry Company
www.henry-bes.com/greenroofing.asp .

Prairie Technologies
www.prairie-tech.com

Roofscapes, Inc.
www.roofscapes.com

Xero Flor America, LLC
www.xeroflora.com

14.1 Rainwater Capturing Systems - Overview of Practice

Capture and Reuse BMP measures include a number of devices intended to intercept precipitation, store it for a period of time, and provide a means for reuse of the water. These capture devices include cisterns, rain barrels, and vertical storage or "fat downspouts." The capture and reuse approach to stormwater management can be applied in both site development and retrofit applications. Use as a BMP for highway runoff is limited. Generally, use of stored rainwater in potable applications is not advised in the absence of treatment; however, in addition to reducing stormwater runoff, the intercepted water is ideal for fire protection and irrigation.

14.1.1 Types of Capture and Storage Devices

Cisterns are containers designed to hold large volumes of water (by definition, cistern volumes are typically 500 gallons or more). Cisterns may be located underground or on the surface. Cisterns are available in a variety of sizes and materials, including fiberglass, concrete, plastic, and brick.

Figure 0.1 - Various Size Cisterns (PADEP, January 2005)

Rain Barrels are containers designed exclusively to capture runoff from roof leaders and downspouts. Rain barrels vary in volume, and are sized based on the roof area from which they are receiving runoff or as a minimum volume computed by a water budget approach, as discussed later in this document.

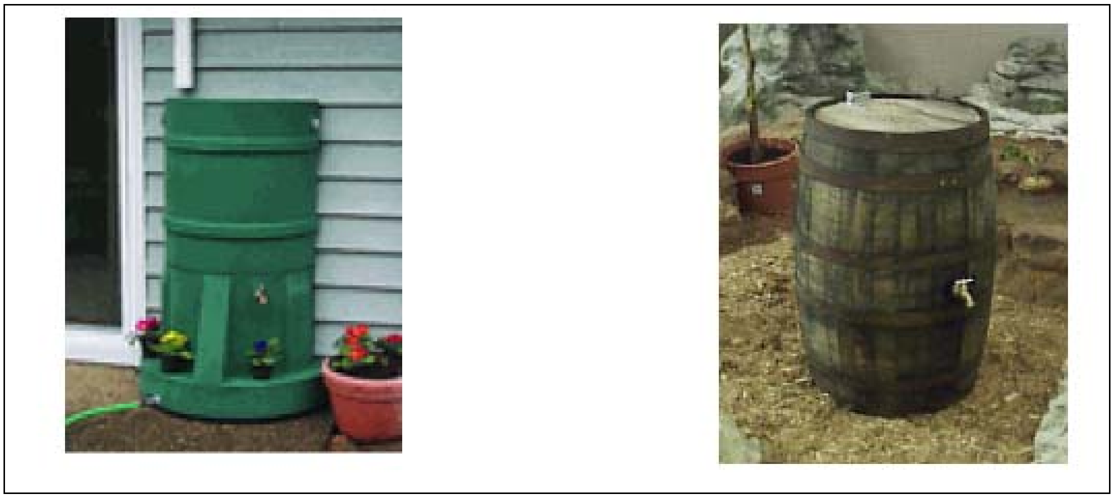


Figure 0.2 - Rain Barrels (PADEP, January 2005)

Vertical Storage units or “fat downspouts” function in the same manner as cisterns and rain barrels, but are typically much larger and usually rest against the building from which they are intercepting runoff. Often, the water stored in these vertical storage units is used to provide fire protection. When employed as storage for fire protection, the storage volume is dictated by applicable codes. The design and sizing of vertical storage units and fat downspouts must be accomplished by working closely with both the architect and structural engineer.

Figure 0.3 - Vertical Storage (Fat Downspouts)
(PADEP, January 2005)

Proprietary storage units, such as RainStore, may be located beneath paths and walkways. These storage devices often provide a supplemental irrigation supply.

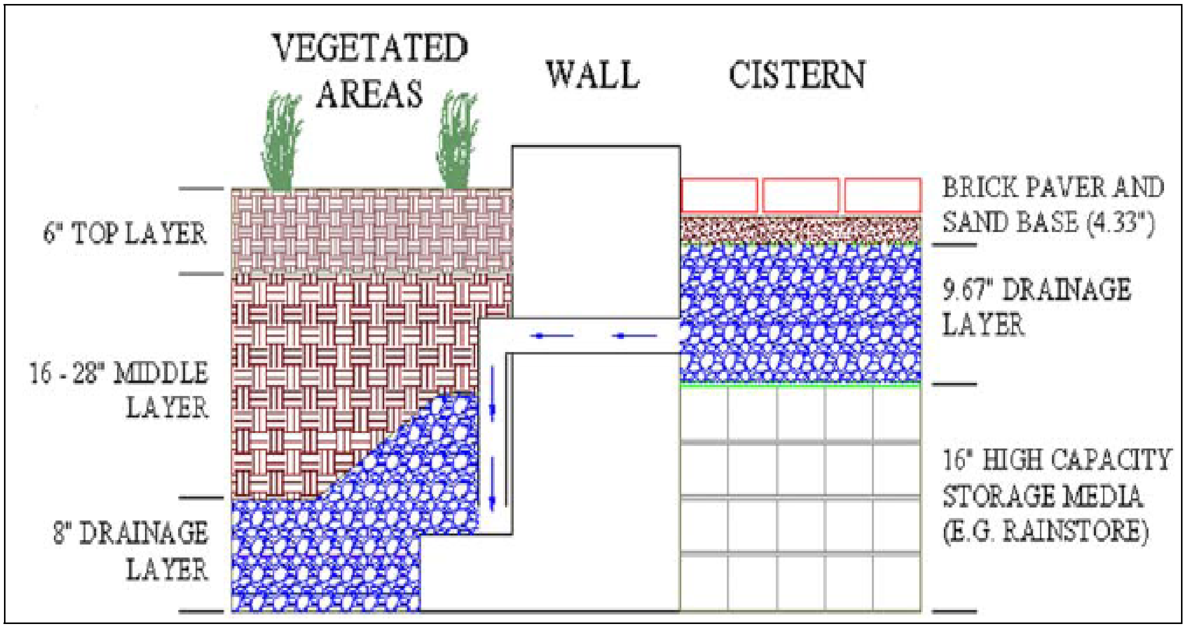


Figure 0.4 - Storage of Runoff Beneath Brick Walkway
(specifications from PADEP, January 2005)

14.1.2 Application of Stored Rainwater

While the use of stored rainwater as a potable supply is not recommended, a number of non-potable needs may be addressed by a capture and reuse approach. These include:

- Irrigation of landscaped areas and gardens
- Storage for fire protection needs
- “Greywater” needs such as flushing toilets
- Athletic field irrigation

In addition to satisfying non-potable water needs, rainwater capture devices can serve to reduce runoff volume and the frequency of surcharge events in urban combined sewer systems.

14.2 Design Considerations

The first step in the consideration of a capture and reuse system is to determine the water demand for the proposed reuse application. The demand is critical in determining the feasibility and size of the harvesting system. The volume of water harvested and stored, at a minimum, must equal the computed demand.

The capture and storage system must provide drawdown between storm events such that the required stormwater storage volume is available.

The conveyance system that delivers reused stormwater or greywater from the storage system must not cross connect with domestic or commercial potable water systems.

Storage units and conveyance systems must be clearly marked as non-potable water.

Screens may be used as a means to filter debris from capture and storage units.

Rainfall storage units should be protected from direct sunlight by positioning and landscaping.

When providing an overflow outlet for the storage unit, the proximity to building foundations must be considered.

In cold climates, capture and reuse systems should be disconnected during the winter months to prevent freezing.

Underground cisterns must be watertight.

Rain barrels and surface cisterns should have a cover with a tight fit capable of keeping out unwanted surface water, animals, dust, and light.

Cisterns, rain barrels, and vertical storage systems should be equipped with a means for overflow in the event of heavy runoff producing events.

Buried cisterns should possess observation risers extending to at least 6” above grade.

Re-use applications may require that the stored rainwater be pressurized. Stored water will exhibit a pressure of 0.43 psi per foot of elevation. Irrigation systems will usually require a minimum of 15 psi.

Source: PADEP, January 2005

14.3 Stormwater Performance

The employment of capture and reuse systems exhibits a positive impact on the volume, peak rate, and quality of stormwater runoff from a site.

The volume reduction is simply the volume of runoff from a single storm event that is captured and stored by the harvesting system. If the cistern or barrel is empty at the start of the precipitation event, the maximum potential volume reduction is the actual volume of the capture device.

Because capture and reuse devices take a volume of water out of the total site runoff, the reduced volume may result in a reduced rate of runoff from the site.

The removal of pollutants from stormwater entering a capture device takes place through filtration of the recycled primary storage, and natural filtration through soil and vegetation of any overflow discharge. A number of factors influence the pollutant removal performance of a rainwater harvesting system. These include the volume below the outlet of the system allocated to sediment accumulation, the hydraulic residence time, and the frequency of maintenance.

14.4 Design Approach

The first design element to consider in the installation of a capture and reuse system is that of a first flush diverter. Rooftops can collect dust, leaves, twigs, insect bodies, animal feces, pesticides, and other airborne residue. A first flush diverter routes the first flush of stormwater from the catchment surface away from the storage tank. A number of factors influence the recommended volume of water that should be diverted. These include the frequency of dry days, amount of accumulated debris, and the catchment area. One rule of thumb for first flush diversion is to divert a minimum of 10 gallons for every 1,000 square feet of collection surface. (Texas Water Development Board, 2005)

The most basic first flush diverter is a 6" or 8" PVC standpipe. The diverter fills with the first-flush volume, backs up, and then allows water to enter the conveyance and storage system. A pinhole drilled at the bottom of the pipe or a hose bib fixture left slightly open permits the gradual leakage of the first-flush volume (TWDB, 2005). The following lengths of PVC piping are required for first flush storage.

Table 14.4.1 - Length of Piping Per Gallon of Storage (TWDB, 2005)

Diameter (in)	Length (inches) per Gallon of Storage
3	33
4	18
6	8
8	5

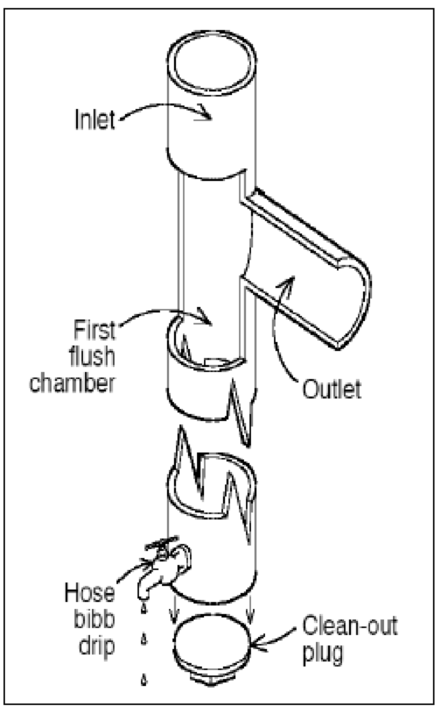


Figure 14.4.1 - Simple Standpipe First Flush Diverter
(TWDB, 2005)

Another variation of first flow capture devices is the standpipe equipped with a ball valve. In this configuration, as the chamber fills, the ball floats up and seals on the seat, trapping the first flush water and routing additional inflow into the storage tank.

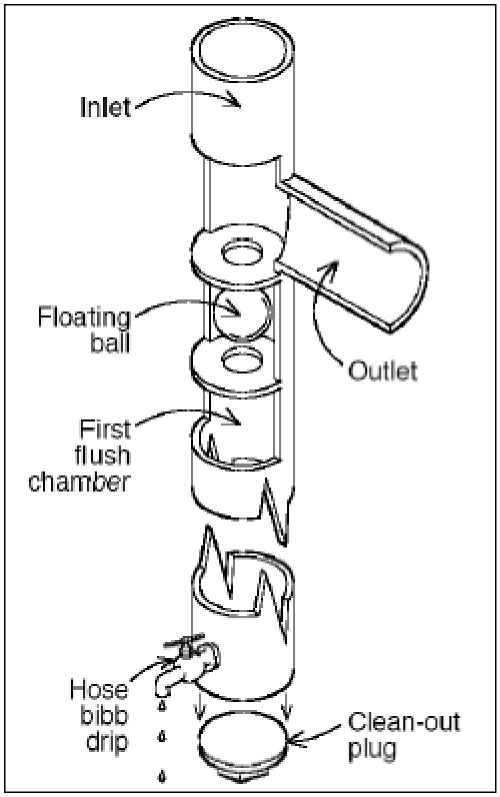


Figure 14.4.2 - Standpipe With Ball Valve
(TWDB, 2005)

The next step in the design process is to size the capture system. Typically, the system must be designed such that the volume of water captured and stored equals or exceeds the volume of water for which anticipated use is planned (demand). The first consideration is that of how much water can be collected. Theoretically, about 0.62 gallons of water per square foot per inch of rainfall can be collected; however, in practice, some precipitation is lost to the first-flush bypass, evaporation, splash-out, and leakage. Rough catchment surfaces are less efficient at conveying water, as water trapped in pore spaces tends to be lost to evaporation. Additionally, intense rainfall events often result in the inability of the system to capture the entire volume of water landing on the catchment surface. Obviously, once storage cisterns or barrels are full, rainwater is lost as overflow. For design purposes, collection efficiencies of 75 to 90% should be considered. The catchment area is the “footprint” of the roof. Regardless of the roof pitch, the total area covered by the collection surface should be considered in estimating the supply of captured water. Only catchment areas whose runoff is collected by a conveyance system (roof gutter) should be considered. (TWDB, 2005)

One popular method for sizing a rainwater harvesting and storage system is to employ the monthly water balance method. This method begins by assuming a volume of rainwater already in storage, adding the volume of water captured each month, and subtracting the demand. Two different methods of estimating monthly rainfall are commonly used; the *average rainfall method*, and the *median rainfall method*. The Virginia State Climatology Office maintains an online database with monthly climate information from various stations across the state. This information can be obtained at:
http://climate.virginia.edu/online_data.htm#monthly

Average rainfall is computed by summing historical rainfall and dividing it by the period of record. Median rainfall is the amount of rainfall that occurs in the midpoint of all historic rainfall totals for any given month. When the data is available, employing the median rainfall provides for the most conservative approach to sizing rainfall harvesting systems. The following example shows a typical water budget approach to determining the feasibility and sizing of a rainfall harvesting system.

Given Data: Average monthly rainfall for Louisa County
2,500 sf catchment area
85% assumed catchment efficiency
Demand as shown in Table 13.2 on the following page

The supply of monthly rainfall is computed as the product of average rainfall, catchment area, catchment efficiency, and the 0.62 gal/sf/in of rainfall constant. The calculation of monthly supply is shown below for January with an average precipitation of 3.14”:

Monthly Supply = (Catchment Area)(Average Rainfall)(Rainfall Constant)(Catchment Efficiency)

$$2{,}500\,ft^2 \times 3.14 in \times 0.62 \frac{gal/ft^2}{in} \times 0.85 = 4{,}137\,gal$$

This value is added to the initial storage volume at the beginning of the month (1,000 gallons for this example), and then the monthly demand is subtracted. The result becomes the initial volume for the month of February, and the calculation is repeated. The monthly budget calculation is presented in the following table with column (A) water demand is in gallons; (B) average rainfall is in inches; (C) rainfall collected is in gallons; and (D) end-of-month storage is in gallons.

Table 14.4.2 - Monthly Water Budget

Month	A Water Demand (gal)	B Average Rainfall (in)	C Rainfall Collected (gal)	D End of Month Storage (1,000 gal to start)
January	4,500	3.14	4,137	637
February	4,500	3.04	4,005	142
March	4,500	3.80	5,007	649
April	4,500	3.06	4,032	180
May	4,500	3.68	4,848	529
June	4,500	3.69	4,862	890
July	4,500	4.36	5,744	2,134
August	4,500	4.26	5,613	3,247
September	4,500	3.65	4,809	3,556
October	4,500	3.57	4,703	3,759
November	4,500	3.58	4,717	3,976
December	4,500	3.32	4,374	3,850

Employing the average monthly rainfall and the monthly water budget approach, we see from Table 13.2 that the storage unit(s) in this scenario would be sized to hold a *maximum* of 3,976 gallons (observed at the end of November) in order to retain all excess rainwater and meet the demand for each month. Alternatively, the *minimum* size storage would only have to be 1,126 gallons [3,976 - (3,850 – 1000)] if the goal is to meet all monthly demands *and* have 1,000 gallons in storage at the end of December each year. In this scenario we must be willing to spill some water during heavy rainfall months.

15.1 Catch Basin Inserts - Overview of Practice

The following design example provides guidance for the implementation of manufactured water quality inlets and catch basin inserts for purposes of runoff quality management on VDOT facilities projects.

Catch basins are chambers or sumps which provide the entrance point for surface runoff into a stormwater conveyance system. Catch basin inserts are employed to intercept coarse sediments, oils, grease, litter, and debris from the runoff prior to its entrance into the storm sewer. Catch basin inserts are well suited to parking lots, maintenance yards, and other locations where runoff travels directly from an impervious surface into the stormwater conveyance system. (VTRC, 2004)

Water quality inlets encompass a broad spectrum of BMPs designed to remove non-point source pollutants from runoff. These structural BMPs vary in size and treatment capacity, but typically employ some form of settling and filtration to remove particulate pollutants. Water quality inlets may exist as hydrodynamic separator systems (see Design Example 15), multi-chambered treatment trains, and a wide array of proprietary products discussed later in this design example.

Many types of catch basin inserts/water quality inlets exist; however, these different configurations generally exhibit similar strengths and shortcomings. The following presents the most common variations of water quality inlet filtering systems.

15.1.1 Tray Type

Tray type filters function by passing stormwater through a filter media situated in a tray located around the perimeter of the inlet. Runoff enters the tray and exits via weir flow under design conditions. Runoff from large storms simply passes over the tray into the inlet unobstructed.

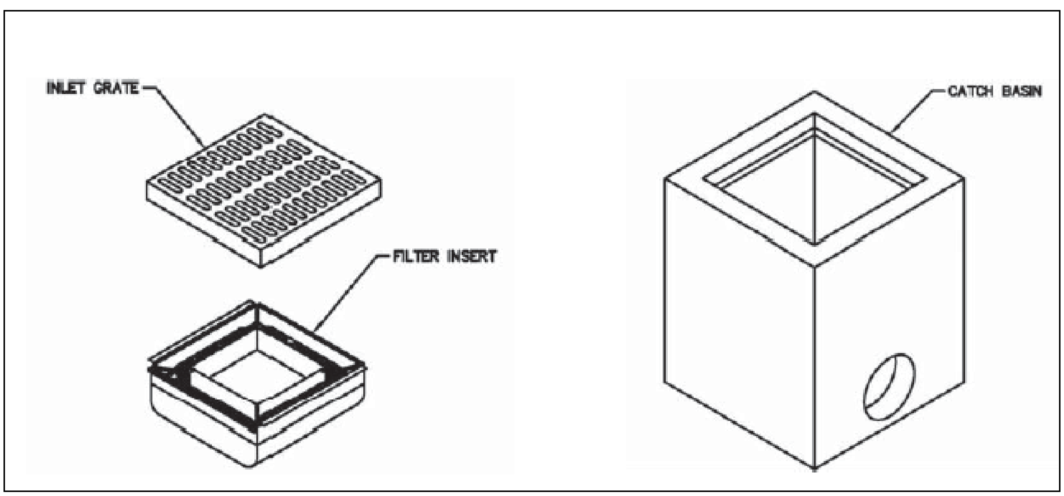


Figure 0.1 - Water Quality Inset Tray (PADEP, 2005)

15.1.2 Bag Type

Bag type inserts are made of fabric and placed in the drain inlet around the perimeter of the grate. Runoff entering the drain must pass through the bag prior to exiting through the drain pipe outlet. The system is usually equipped with overflow holes to prevent backwater conditions during heavy runoff producing events.

Figure 0.2 - Bag Type Inlet Filter and Installation (PADEP, 2005)

15.1.3 Basket Type

Basket type inserts set into the inlet and can be removed for periodic maintenance. Small orifices permit small storm events to weep through, while larger storms overflow the basket. Basket type inserts are useful for filtering trash, debris, and large sediment, but require consistent maintenance.

Figure 0.3 - Basket Type Inlet Filter (PADEP, 2005)

15.1.4 Sumps in Inlets

Inlets can be designed such that space is created below the invert of the outlet pipe(s) for sediment and debris to deposit. Generally, this space will be 6" to 12" deep. Small weep holes should be drilled into the bottom of the inlet to prevent standing water for long periods of time. Note that if weep holes are used to drain a sumped inlet, the inlet must conform to applicable design requirements for infiltration facilities. Inlets equipped with a sump require regular maintenance and sediment removal.

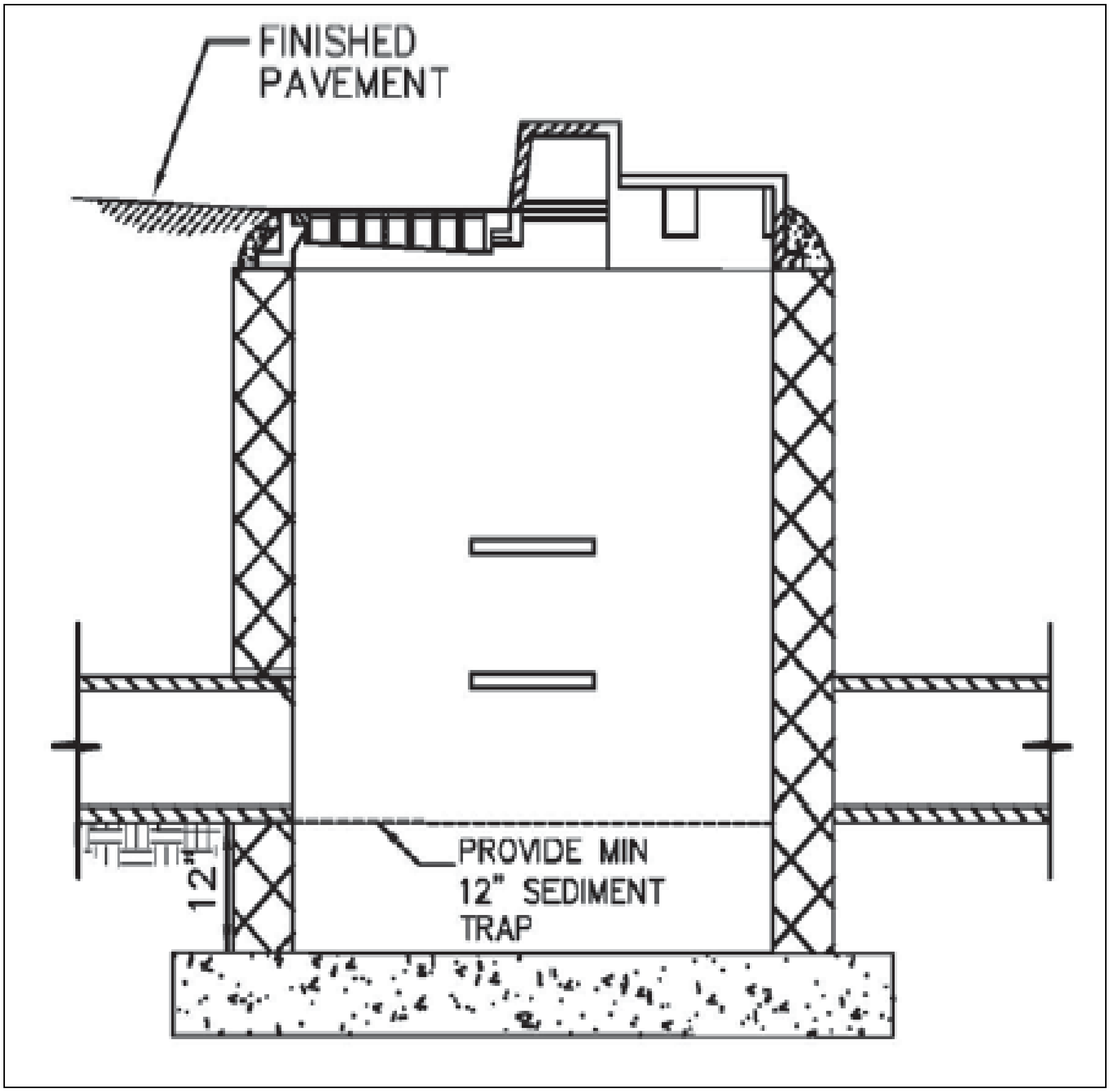


Figure 0.4 - Catch Basin Equipped With Sediment Sump (PADEP, 2005)

15.2 Design Considerations

The design process for a specific installation of a water quality inlet or catch basin insert usually begins with a review of various vendor publications and use of preliminary sizing guidelines provided by the vendor. The specific design criteria for the proprietary system being considered should be obtained from the manufacturer or vendor to ensure that the latest design and sizing criteria are used. At the very least, the design for a particular site should be reviewed by the manufacturer to ensure that the system is adequately sized and located.

15.2.1 Key Considerations Unique to Manufactured Products

Independent performance data must be available to prove a demonstrated capability of meeting stormwater management goals.

The chosen system or device must be appropriate for use in the geographic region for which implementation is planned.

Installation and operations/maintenance requirements must be understood by all parties approving and using the system or device in question.

15.2.2 General Design Guidance

Specific site conditions must be matched with the manufacturer/vendor guidelines and specifications. Geographic location and land use will determine the specific pollutants and their associated loading rates.

The re-suspension of particles and sediment is of concern. To avoid such re-suspension, the drainage area to each water quality inlet or catch basin should be restricted to no more than one acre of impervious cover. Regular maintenance and removal of accumulated debris is essential.

Retrofits should be designed specifically for the existing inlet.

Location of the water quality inlet or catch basin should provide ease of maintenance, and be at the forefront of the design process.

If the inlet is used during construction operations for erosion and sedimentation control, the insert should be reconfigured and cleaned per manufacturer guidelines prior to its implementation in the final site design.

Overflow should be provided such that storms in excess of the device capacity (typically the computed water quality volume) are bypassed.

Source: PADEP, 2005

15.3 Maintenance

The manufacturer's guidelines for maintenance should be followed for any proprietary system. The expected pollutant type and loading rate for the specific site of interest must also be considered. During construction operations, water quality inlets should be inspected a minimum of once per week, and cleaned as needed. Post-construction, they should be emptied when full of sediment and trash / debris. Thorough cleaning should occur at least twice per year. Water quality inlets and catch basins equipped with filtering devices should also be inspected after all heavy runoff producing events. Regular maintenance is critical to ensuring the continued functioning of water quality inlet systems. Studies have shown that water quality inlets storing in excess of 60% of their total sediment capacity may resuspend the stored sediments into the runoff entering the inlet. (PADEP, 2005)

15.4 Manufactured Products

The following discussion of manufactured water quality filters is intended only to serve as a description of the most widely used proprietary systems. The products discussed in this design example are not intended to constitute an exhaustive list of all catch basin / inlet filtering systems available. Presentation of the following products does not preclude the use of other available systems, nor does it constitute an endorsement of any one system.

The Virginia Transportation Research Council, via contract with University of Virginia, has constructed the following information matrices for the most widely used catch basin inserts and water quality inlets, as of 2004. The user is referred to the following for the originally published matrices:

Virginia Transportation Research Council. VDOT Manual of *Practice for Stormwater Management.* Charlottesville, Virginia, 2004.

Table 15.4.1 - Catch Basin Inserts Information Matrix (VTRC, 2004)

System Type	Manufacturer	Operation	Sizing and/or Area Treated	Maintenance	Cost	General Performance	Comments
Catch Basin Inserts							
Sorbant™	Sorbant Environmental Corp. Aventura, FL.	Flow cascades over 3 tiers of sorbent pads. Primarily for hydrocarbon removal	Structure drops into standard inlets.	ND	ND	Sorbs 16 to 22 times its weight in hydrocarbons. Does not leach in flooded conditions.* (Corcoran and Rich, 1995)	Catch basin or curb inlet design.
BMP Filter "CB" Series Catch Basin Insert	StormWater Compliance International Oroville, CA (www. stormwatercompintl.com/)	Insert directs flow through mesh screens for sediment removal, then through proprietary media filters.	Applied to catch basins or curb inlets. Overflow allows up to 0.63 cfs through the system.	Hydrocarbon media changes color when saturated. Replacement of other media filters every 6 months. More frequent cleaning of debris.	$900	Oil and grease removal to less than 5 mg/L. Neutral pH: 6-8, BOD & COD reduced to less than 50 mg/L; TSS removal over 90%.*	Company also manufactures oil/water separators, curb inlet filters, inline filters.
Hydro-Kleen™ Filtration System	Hydro Compliance Management, Inc. Brighton, MI (www. hydrocompliance.com/)	Multi-chambered system. Flow through sedimentation chamber to 2 media filters: proprietary material for hydrocarbon removal then activated carbon for final polishing.	Treats first-flush, with bypass available.	Filter change every 4-6 months. More frequent sediment cleanout by vacuum truck.	$1,200 - $2,500 per unit. Filter change: $400 including labor. Low installation cost.	Reduces hydrocarbons, pesticides, herbicides, VOCs to below detection limits.*	Can customize media for site-specific loads. Can be catch basin or burn inlet system. Vendor claims product satisfies structural BMP requirements for NPDES compliance.
Aqua-Guard™	AquaShield, Inc.	Flow through sedimentation chamber and filter media.	ND	Sediment removal by shop-vac or vacuum truck. Filter media changes color to black when replacement is needed.	ND	Effective removal of TSS, soluble and insoluble O&G, phosphorus, nitrogen, VOCs, sulfides, heavy metals. Certified by CA EPA 90-95% removal of dissolved petroleum and oils.*	Standard sizing for drop-in application.
StreamGuard™	Bowhead Manufacturing Co. Address: P.O. Box 80327 Seattle, WA 98108	The insert's universal skirt adapter is installed under a storm drain grate and provides water quality treatment through filtration.	Size based on flow rates from 20 to 40 gpm.	Remove trash and debris when accumulation becomes significant.	$56 to $93 each, depending on size.	Independent testing by King County Surface Water Management Division of	Installed at the U.S. Coast Guard Station in Chesapeake, VA.

Table 15.1 Catch Basin

Table 15.1 Cont'd. – Catch Basin Inserts Information Matrix (VTRC, 2004)

System Type	Manufacturer	Operation	Sizing and/or Area Treated	Maintenance	Cost	General Performance	Comments
		gravity settling and absorption.				Washington State demonstrated oil removal efficiencies of 88% when tested in a park-and-ride lot catch basin. Catch basin inserts installed at SeaTac International Airport's passenger pick-up area show average removal efficiencies for Total Suspended Solids of 80%, and for oil & grease of 94%.	
The SNOUT™	Best Management Products, Inc.	Simple hood covers outlet structure. Bottom of hood sites below static water level. Keeps floatables (including trash) above outlet.	ND	SNOUT itself does not require maintenance. Remove trash and debris when accumulation becomes significant.	Low hundreds	Inspections show significant accumulation of gross pollutants.*	Suitable for use with catch basins or water quality inlets. Can be equipped with flow restriction and/or odor control filter.
Filter bag inserts – general	*Multiple Vendors:* DrainPac™ by Drain Works; Drainguards by Ultra Tech; Ultra-Urban Filters by AbTech Industries.	Heavy filter fabric held in place by inlet grate.	Standard sizes for drop-in installation	Frequent inspection and cleanout	ND	Mainly designed to capture trash and sediment. Some also claim sorption of O&G. Can be effective if frequently maintained.	Improper installation causes leaks/bypass of runoff around filter media.

Table 15.4.2 - Water Quality Inlets Information Matrix (VTRC, 2004)

System Type	Manufacturer	Operation	Sizing and/or Area Treated	Maintenance	Cost	General Performance	Comments
Water Quality Inlets							
Oil/Water Separator (OWS)	*Multiple Vendors:* Areo-Power®; Flo-Trends, Inc.; PSI International, Inc.	Coalescing plate or tube separator. Flow-through system.	Usually designed for specific applications.	ND	ND	Low to negative removal of TSS, TPH, and O&G. (Othmer et al., 2001)	General inability to reduce low levels of hydrocarbons. Not generally recommended.
MCTT (Multi-Chambered Treatment Train)	Developed at the University of Alabama-Birmingham. Specifications are given for cast-in-place construction.	Flow through 3 chambers: screening, tube settling, media filtration. Provides some detention. Customize	Surface area of unit typically 0.5 – 1.5% of the drainage area.	Six-month inspections. Replace sorbent pillows & clean catch basin	$10,000 - $20,000 per 0.25 acre. (Schueler, 1994)	Treats 95% of annual rainfall. Toxicity reduced by filtration. Flow restrictions can	May be able to customize system depending on site pollutant
		with aerators, sorbent pads, multi-media filters	Criteria can be expanded to include storm characteristics and anticipated loads.	every 6 – 12 months. Media replacement after 3 – 5 years. Ensure mosquito control.		provide up to 24 hrs settling (US EPA, 1999c)	characteristics.
BaffleBox	Multiple Vendors: Suntree Technologies, Inc., or Cast-in-place construction	Large sediment trap comprised of multiple concrete or fiberglass chambers separated by weirs. Usually with trash screens and skimmers.	Usually 10 – 15 ft. long by 6 – 8 ft. wide. (2 ft. wider than inlet pipe)	Monthly during wet season, 2 – 3 months during dry season.	Installation: $20,000 - $30,000 Maintenance: $0.24/kg removed (avg. $450 per event)	Approx. 2,500 – 3,800 kg/yr sediment removal but highly site-specific. Model performance: removed at least 90% sand or sandy clay, but reduced to only 28% for fly ash. Differences in accumulated material noted between chambers.	Better performance with larger boxes. Systems become septic and odorous without base flow. Many systems installed in Florida. Wash-out can be a problem with larger events.
Oil/Grit Separators (OGS)	Usually cast-in-place construction.	On-line system. Flow through three chambers: sediment & trash, oil containment, energy dissipation. Inverted elbow in oil chamber retains floatables.	Treat 0.1" runoff. Recommended as a last resort for treatment area less than 1 acre.	Quarterly	$5,000 - $16,000; average $8,500 (US EPA, 1999d)	Of 109 systems investigated, the average residence time was less than 30 minutes. Poor retainment of trash and debris. 10 – 40% solids removal with 1	Used mainly at gas stations, fast food restaurants and other small, but highly-developed sites. Hundreds installed in the DC metro area. Better performance

Figures 14.5 through 14.9 are representative of many vendor products which can be viewed at the following EPA Region 1 New England website:
http://www.epa.gov/NE/assistance/ceitts/stormwater/techs.html

Additional vendor products and preliminary design information can be found at the US EPA NPDES/STORMWATER/BMPMENU website:
http://cfpub.epa.gov/npdes/stormwater/menuofbmps/post_7.cfm

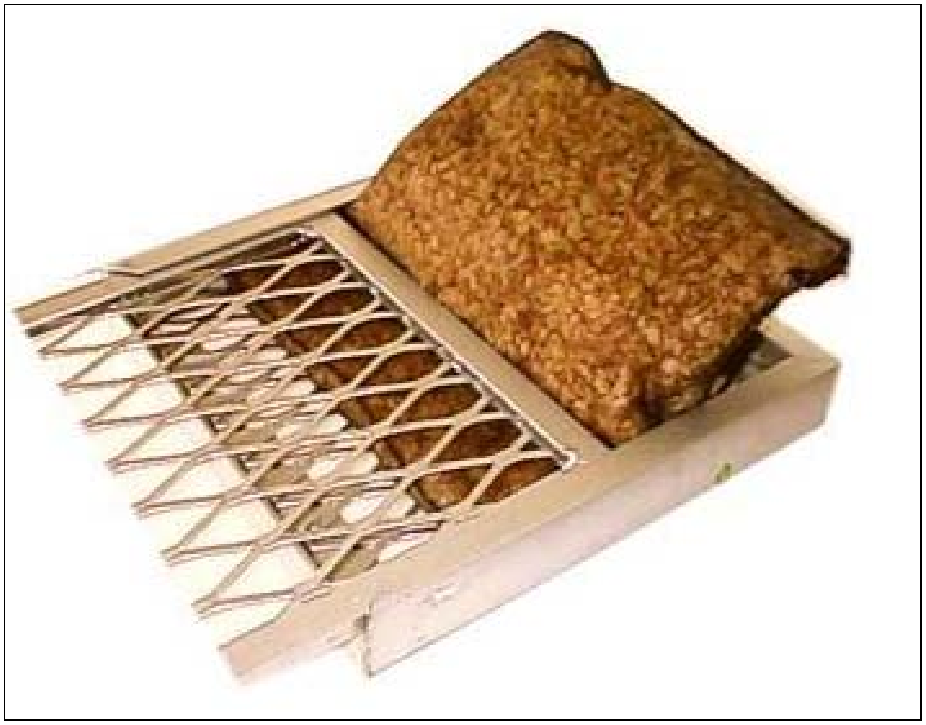


Figure 15.4.1 - Sorbant Filter Pillow System

Source: Sorbant Environmental Corp
P.O. Box 80-2505 • Aventura, FL 33280
305-655-9911 - Fax: 305-655-0470

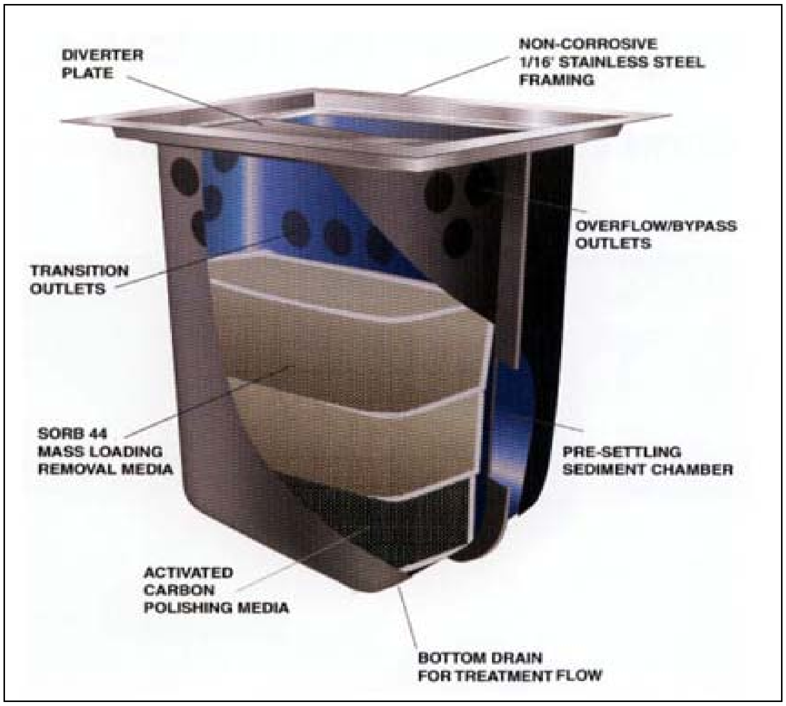


Figure 15.4.2 - Hydro-Kleen Filtration System

Source: Hydro Compliance Management, Inc. Brighton, MI

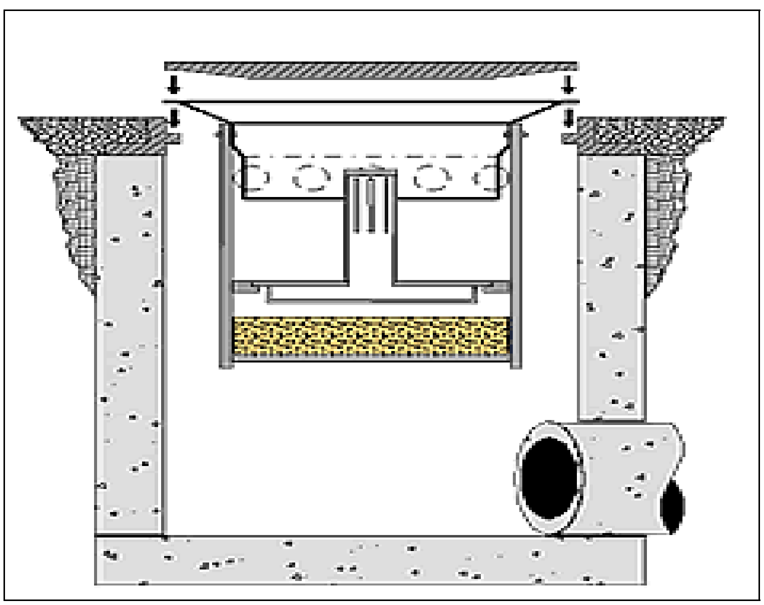


Figure 15.4.3 - Aqua-Guard Catch Basin Insert

Source: Aquashield, Inc.;Water Services Inc. 1102 C. Montalona Rd.
Dunbarton, NH 03046

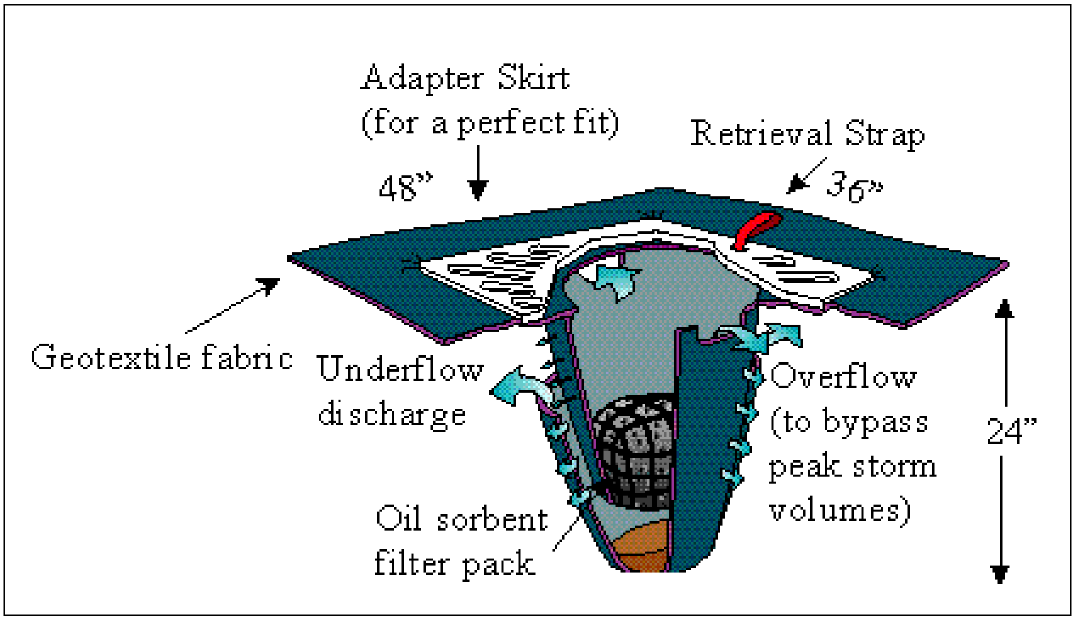


Figure 15.4.4 - StreamGuard Catch Basin Insert

Source: Bowhead Manufacturing Co.
P.O. Box 80327
Seattle, WA 98108

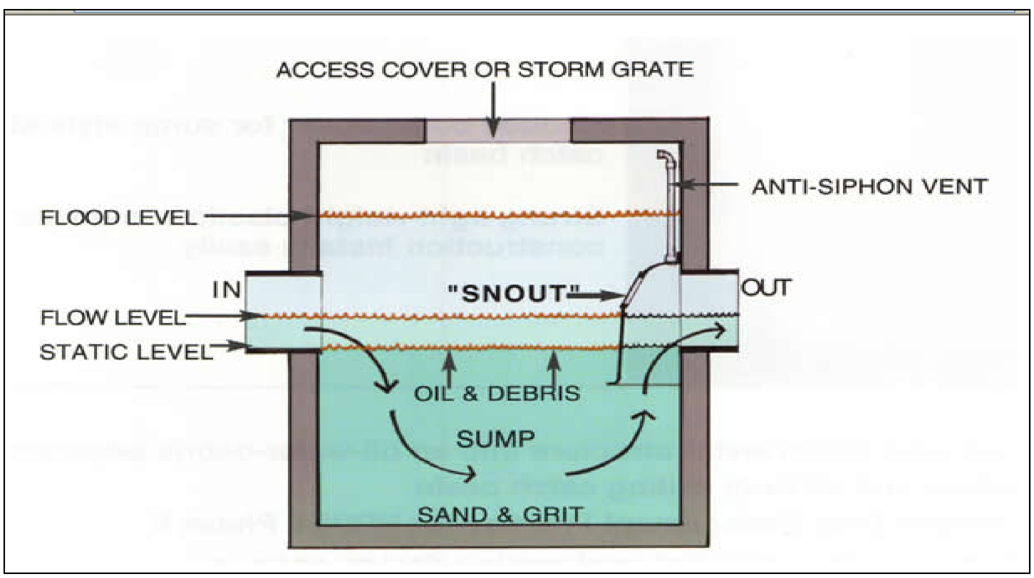


Figure 15.4.5 - The SNOUT Catch Basin Insert

Source: Best Management Products, Inc., 53 Mount Archer Road, Lyme, CT 06371

16.1 Hydrodynamic Separators - Overview of Practice

The following design example provides guidance for the implementation of manufactured oil / water hydrodynamic separation devices for purposes of runoff quality management on VDOT facilities projects.

Hydrodynamic separation devices are designed to remove settleable solids, oil and grease, debris, and floatables from stormwater runoff through gravitational settling. Oil / water separation devices are not intended to mitigate the peak rate of runoff from their contributing watershed. Their implementation is solely for water quality enhancement in urban and ultra-urban areas where surface BMPs are not feasible. These manufactured systems are designed as flow-through structures. In contrast to conventional BMP measures capable of storing a designated water quality volume, flow into a manufactured hydrodynamic separator is regulated by its inflow pipe or other structural hydraulic devices. When the maximum design inflow is exceeded, the inflow may be regulated by a pipe restrictor, causing stormwater to back up into the upstream conveyance system or associated storage facility. When structural devices are employed to regulate flow into the hydrodynamic separator, flows in excess of the desired treatment volume either bypass the structure completely or bypass the separator's treatment chamber (VADCR/DEQ, 2000).

Hydrodynamic separators are often employed as pretreatment measures for high-density or ultra-urban sites, or for use in hydrocarbon hotspots, such as gas stations and areas with high vehicular traffic. Hydrodynamic separators *cannot be used for the removal of dissolved or emulsified oils and pollutants such as coolants, soluble lubricants, glycols and alcohol* (Georgia Stormwater Manual 2001). Hydrodynamic separators are limited in application by the following:

- Hydrodynamic separators are not capable of removing more than 80% of total suspended solids TSS.

- Dissolved pollutants are not effectively removed by these BMPs.

- Frequent maintenance is required to maintain desired pollutant removal performance levels.

- Hydrodynamic separators do not reduce peak rates of runoff to pre-developed levels.

Hydrodynamic separation devices are generally categorized as *Chambered Separation Structures* or *Swirl Concentration Structures*.

Chambered separation devices rely on gravitational settling of particles and, to a lesser degree, centrifugal forces to remove pollutants from stormwater. Chambered systems exhibit an upper bypass chamber and a lower storage / separation chamber. Runoff enters the structure in the upper bypass chamber and is channeled through a downpipe into the lower storage / separation, or treatment chamber. The system is designed such that when inflow exceeds the operating capacity, flow “jumps” the downpipe and completely bypasses the lower treatment chamber (VADCR/DEQ, 1999).

Swirl separation structures are characterized by an internal mechanism that creates a swirling motion. This motion results in the settling of solids to the bottom of the chamber. Additional chambers serve to trap oil and other floating pollutants. Swirl separators do not exhibit a means for bypassing large runoff producing events. Larger flows simply pass through the structure untreated; however, due to the swirling motion within the structure, large flow events do not re-suspend previously trapped particulates. (VADCR/DEQ, 1999)

16.2 Design Considerations

The design process for a specific installation of a hydrodynamic separator usually begins with a review of various vendor publications and use of preliminary sizing guidelines provided by the vendor. The specific design criteria for the hydrodynamic separator being considered should be obtained from the manufacturer or vendor to ensure that the latest design and sizing criteria are used. At the very least, the design for a particular site should be reviewed by the manufacturer to ensure that the system is adequately sized and located. The following criteria are intended to serve only as general guidelines.

- The use of oil-grit hydrodynamic separators should be limited to the following applications:
  - Pretreatment for other structural controls.
  - High-density, ultra-urban or other space-limited development sites.
  - Hotspot areas where the control of grit, floatables, and/or oil and grease is required.
    - Hydrodynamic separators are typically limited in use to drainage areas less than five acres. It is recommended that the contributing drainage area to any single separator be limited to one acre or less of impervious cover.
    - Manufactured separation systems can be used in almost any soil or terrain. Additionally, since located underground, aesthetic and public safety issues are rarely encountered.
    - Separation devices are sized based on *rate* of runoff. This design criteria contrasts with most BMPs, which are sized for a designated runoff *volume*.
    - Hydrodynamic separators are typically designed to bypass runoff flows in excess of the design flow rate. This bypass may be accomplished by a built in bypass mechanism or a diversion weir or flow splitter located upstream of the separator in the runoff conveyance system. As with all runoff control structures, an adequately stabilized outfall must be provided at the separator's discharge point.
    - The separator units should be watertight to prevent possible groundwater contamination.

- The separation chamber must provide three distinct storage volumes:
  - o Volume for separated oil storage at the chamber top
  - o Volume for settleable solids at the chamber bottom
  - o Volume to provide adequate flow-through detention time (volume to ensure maximum horizontal velocity of 3 ft/min through the chamber)
- The total wet storage of the gravity separator unit should be at least 400 ft^3 per contributing impervious acre.
- The minimum depth of the permanent pools should be 4'.
- Hydrodynamic separators require a much more intensive maintenance schedule than other BMP measures. A typical maintenance schedule is shown as follows:

Table 0.1 - Typical Maintenance Activities for Gravity Separators

Activity	Schedule
Inspect the gravity separator unit.	Quarterly
Clean out sediment, oil and grease, and floatables, using catch basin cleaning equipment (vacuum pumps). Manual removal of pollutants may be necessary.	As needed

All specific design criteria should be obtained from the manufacturer.

Source: Georgia Stormwater Management Manual, published by the Atlanta Regional Commission, Atlanta, Georgia, 2001

16.3 Manufactured Products

The following discussion of manufactured hydrodynamic separators is intended only to serve as a description of the most widely used proprietary systems. The products discussed in this design example do not constitute an exhaustive list of all hydrodynamic separation devices available. Presentation of the following products does not preclude the use of other available systems, nor does it constitute an endorsement of any one system.

16.3.1 Stormceptor

Stormceptor is a precast, modular, vertical cylindrical tank divided into an upper bypass and lower storage chamber. The Stormceptor functions by diverting flow through a downpipe into the lower storage / separation chamber. Flow is then routed horizontally around the circular walls of the separation chamber. The circular flow motion, along with gravitational settling, traps sediments and other particulate pollutants. Flow then exits the Stormceptor through an outlet riser pipe. The outlet pipe is submerged, thus preventing trapped floatables from exiting the structure. The configuration also prevents turbulent flow in the storage / separation chamber, thus preventing resuspension of trapped particulates. The Stormceptor has no moving parts, and requires no external power source. (VADCR/DEQ, 1999)

During large runoff producing events, flow entering the Stormceptor floods over the diversion weir and through the bypass chamber into the downstream conveyance system. The overflow of the system is controlled by the incoming stormwater velocity and the hydraulics of the diversion weir. The bypass configuration does result in a backwater condition in the upstream conveyance system. (VADCR/DEQ, 1999)

It is generally recommended that Stormceptor systems be fully pumped a minimum of once per year. This frequency must be increased if high levels of sediment loading are observed. Schematic details of the Stormceptor system are presented as follows.

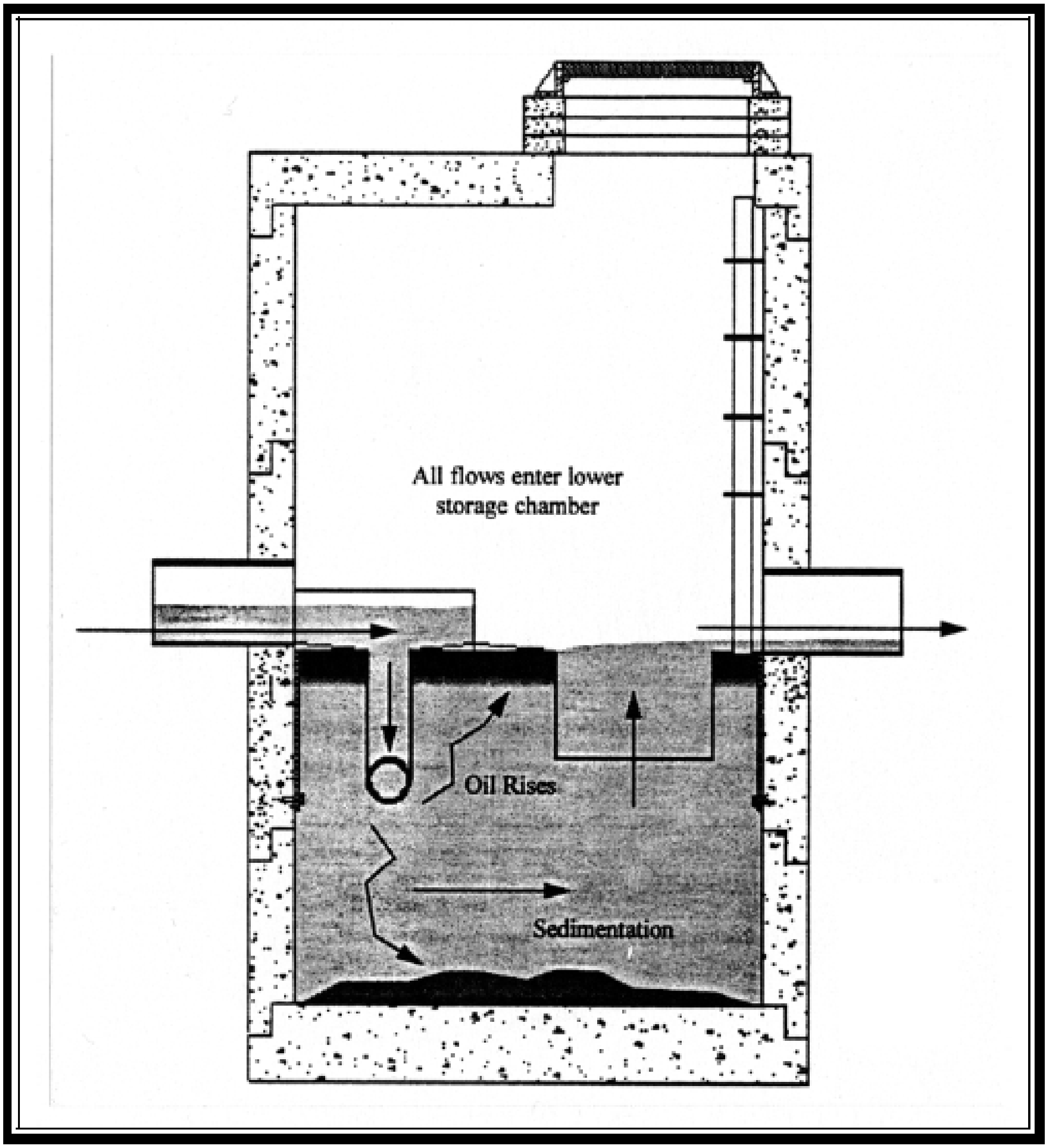


Figure 0.1 - Stormceptor During Normal Flow Conditions

Source: Virginia Department of Conservation and Recreation/Environmental Quality. *Virginia Stormwater Management Handbook*. Richmond, Virginia, 1999.

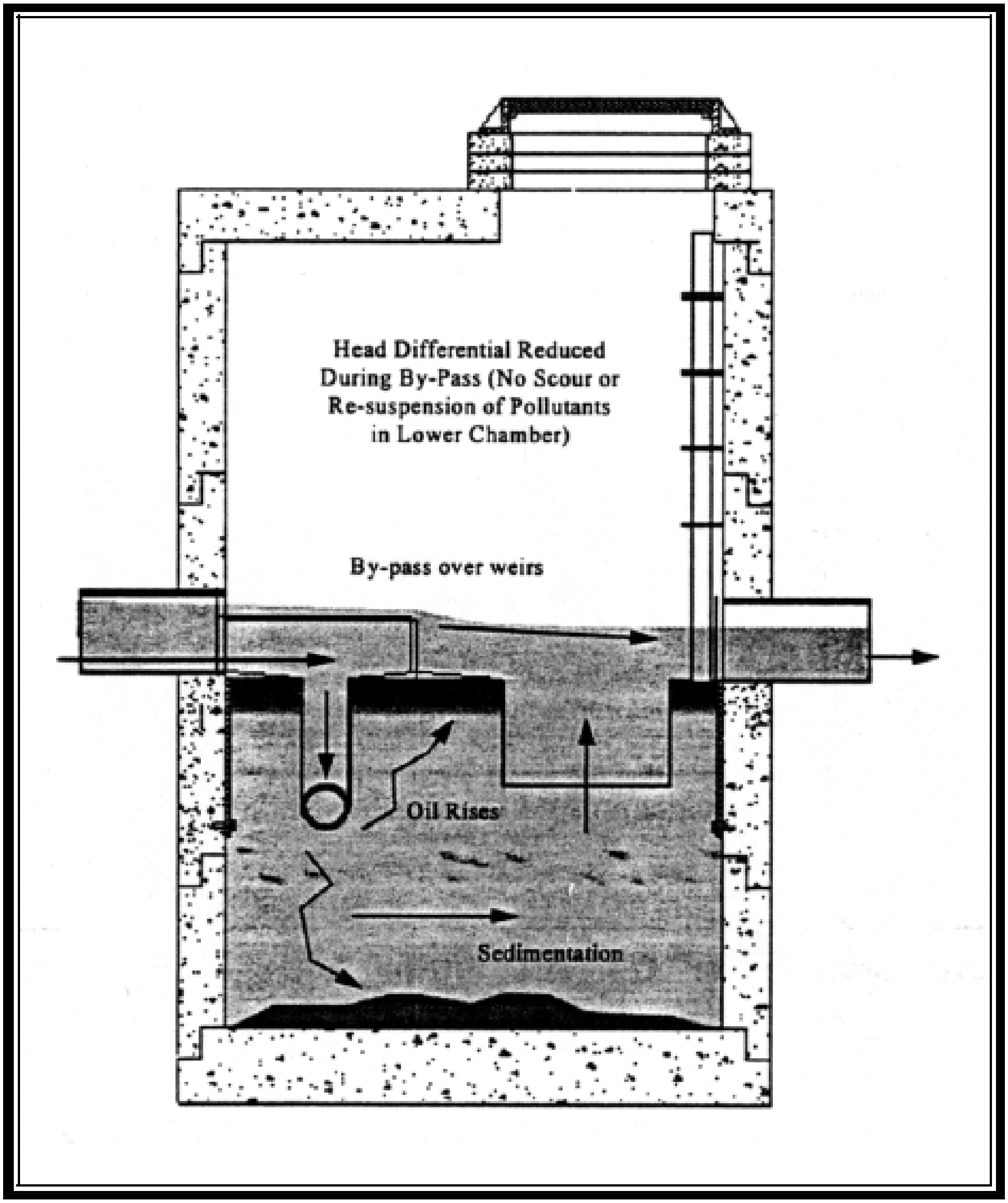


Figure 0.2 – Stormceptorlet's golll
During High Flow Conditions

Source: Virginia Department of Conservation and Recreation/Environmental Quality. *Virginia Stormwater Management Handbook.* Richmond, Virginia, 1999.

Current Stormceptor product information and vendor contacts can be obtained at:

http://www.stormceptor.com

16.3.2 Vortechs Stormwater Treatment System

The Vortechs Stormwater Treatment System is a precast rectangular unit composed of three chambers. The first chamber serves as a grit chamber, and creates a swirling motion that directs settleable solids toward the center where they become trapped. The Vortechs system is an all-inclusive proprietary system, with the swirl-inducing mechanism self-contained within the unit. Flow is then slowly released from this chamber into the oil chamber. The oil chamber contains a barrier which traps oil and grease and other floatable pollutants. The final chamber is the flow control chamber, which forces water to back up, thus reducing velocities and turbulence. The Vortechs Stormwater Treatment System contains no moving parts and requires no external power source. (VADCR/DEQ, 1999)

During large runoff producing events, the flow control chamber of the Vortechs system forces runoff to fill the structure. As this occurs, the swirling action in the grit chamber increases, keeping sediment concentrated at the center of the chamber. Because the swirling action of the system increases as the volume of runoff entering the structure increases, the resuspension of previously deposited material is eliminated. The Vortechs system is capable of providing limited flow attenuation within its storage capacity. When the volume of runoff entering the structure exceeds the capacity of the three chambers, the conveyance system leading to the Vortechs system will experience a backwater condition.

To ensure proper performance, the Vortechs system must be cleaned when it becomes full of pollutant material. During the first year of operation, the manufacturer recommends monthly inspections since contaminant loading rates vary greatly. Cleaning of the system is most readily accomplished by use of a vacuum truck.

Schematic details of the Vortechs system are presented as follows.

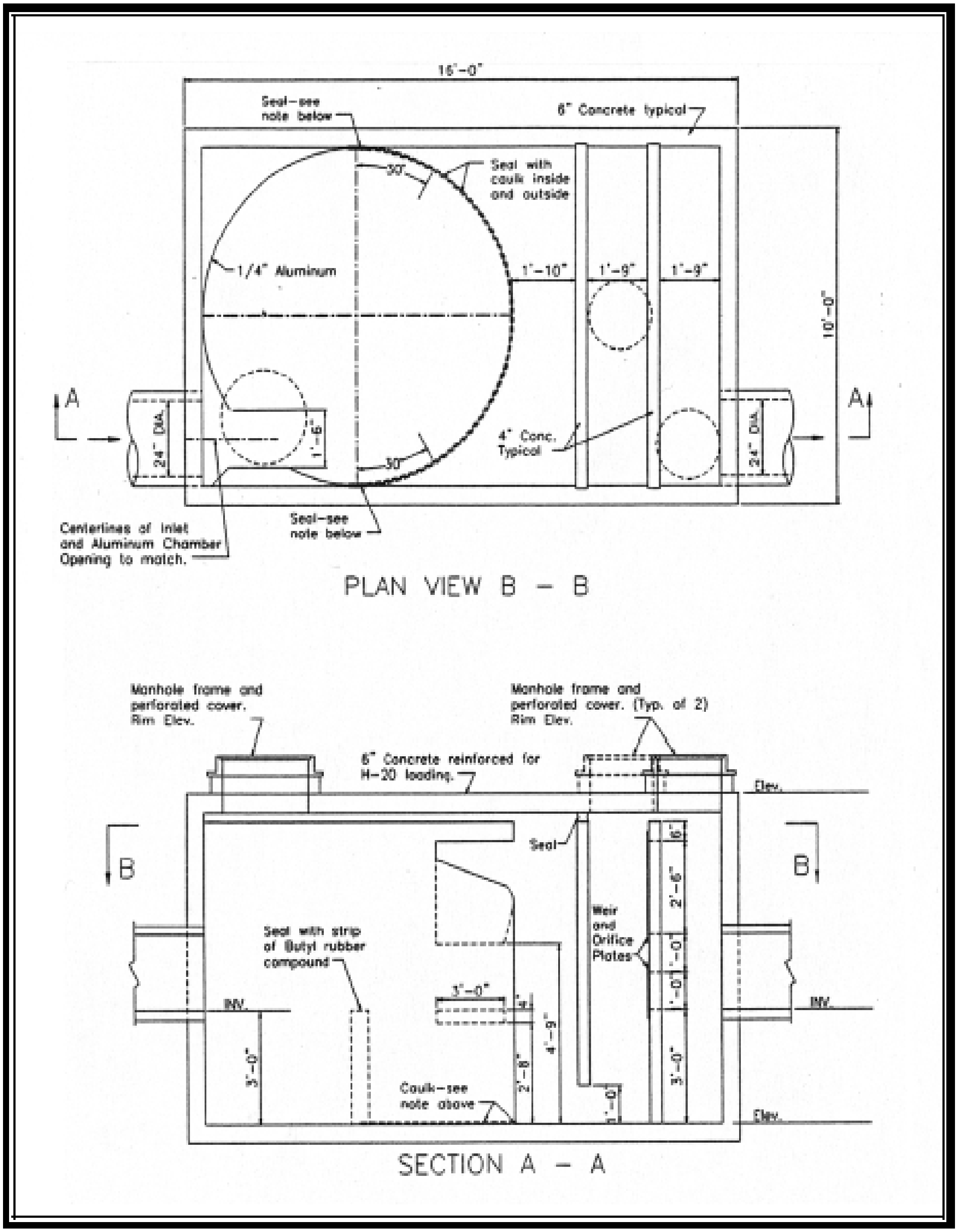


Figure 0.3 - Vortechs Stormwater Treatment System

Source: Virginia Department of Conservation and Recreation/Environmental Quality. *Virginia Stormwater Management Handbook*. Richmond, Virginia, 1999.

Current Vortechs product information and vendor contacts can be obtained at:
www.vortechnics.com

16.3.3 Downstream Defender

The Downstream Defender system is adaptable to all types of land uses. Additionally, the Downstream Defender can be installed in existing pipe systems as a retrofit.

The Downstream Defender is characterized by a concrete cylindrical structure with stainless steel components, and an internal 30^{o} sloping base. Runoff entering the structure passes through a tangential inlet pipe, resulting in a swirling motion. The flow then spirals downward along the perimeter of the structure. During this downward path, heavier particles settle out by gravity and by drag forces exerted along the wall and base of the structure. As flow rotates about the vertical axis, these solids are directed toward the base of the structure, where they are stored. The system's internal components direct the main flow away from the structure's perimeter and back up the middle of the vessel as a narrower spiraling column rotating at a slower velocity than the outer downward flow. When this upward flow reaches the top of the structure, it is virtually free of solids, and is then discharged through the outlet pipe. The Downstream Defender has no moving parts and requires no external power source.

During the first 12 months of operation, inspections should be conducted frequently following runoff-producing events in order to determine the sediment loading rate. After this time, a probe may be used after storm events to determine a maintenance schedule. H.I.L. Technology, Inc. recommends inspection and clean-out of the Downstream Defender system a minimum of twice per year.

Schematic details of the Downstream Defender system are presented as follows:

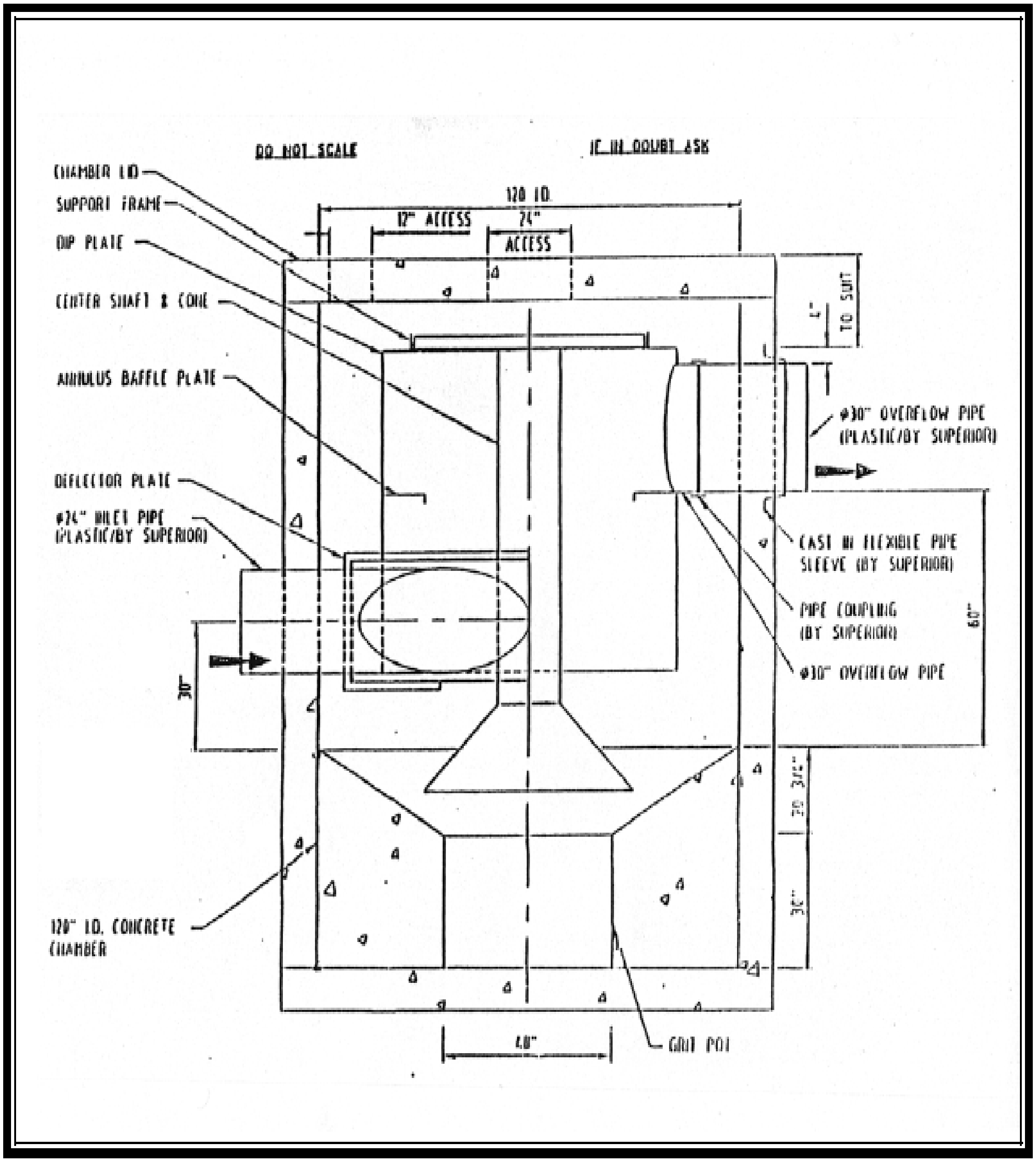


Figure 0.4 - Section View of Downstream Defender System

Source: Virginia Department of Conservation and Recreation/Environmental Quality. *Virginia Stormwater Management Handbook*. Richmond, Virginia, 1999.

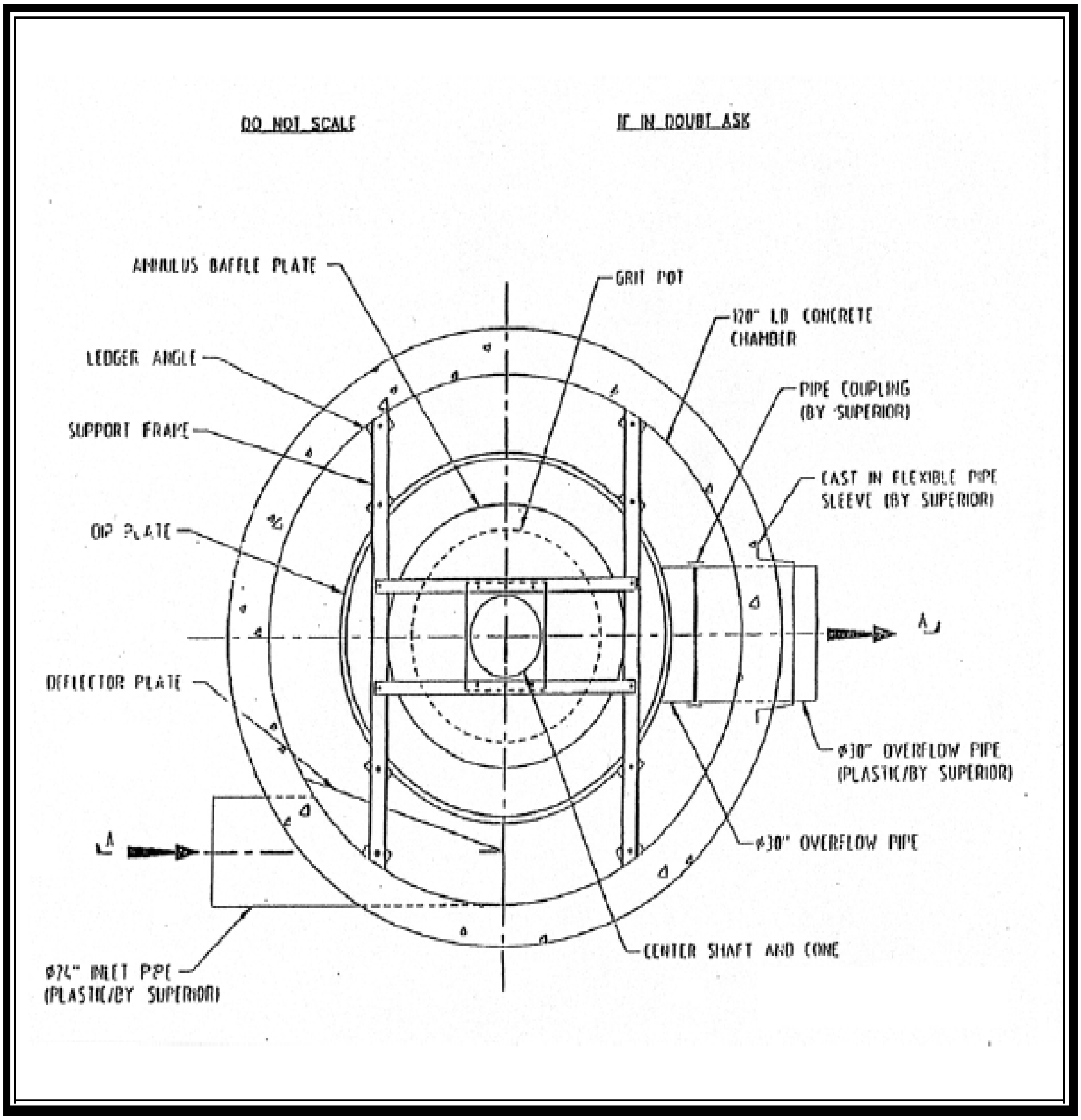


Figure 0.5 - Plan View of Downstream Defender System

Source: Virginia Department of Conservation and Recreation/Environmental Quality. *Virginia Stormwater Management Handbook*. Richmond, Virginia, 1999.

Current Downstream Defender product information and vendor contacts can be obtained at: www.hil-tech.com

16.3.4 BaySaver

The BaySaver system is composed of three main components: the primary separation manhole, the secondary storage manhole, and the BaySaver Separator Unit. Runoff enters the system through the primary separation manhole. The larger sediments contained in the runoff settle into the primary separation manhole whose flow exits through a trapezoidal weir. The runoff leaving the primary separation manhole carries with it floating contaminants, debris, and fine sediment which are then treated in the secondary storage manhole. The BaySaver system employs three potential flowpaths for runoff entering the system. First flush and low flows are diverted into the second manhole for the most efficient treatment. As the water level rises in the primary separation manhole, more water flows over the skimming weir and into the secondary manhole. The majority of oils and fine sediments are removed by this flow path. During more intense storms, water can flow through 90-degree elbow pipes located in the primary separation manhole. Because the elbows are situated below the surface, the water entering the secondary storage manhole is free from floating contaminants. During large, infrequent storm events, the BaySaver system bypasses the treatment stages, conveying water directly from inlet to outlet. Bypassed flows are prevented from entering the sedimentation manholes, and thus resuspension of contaminants does not occur. The BaySaver system contains no moving parts and requires no external power source. (VADCR/DEQ, 1999)

It is generally recommended that BaySaver systems be fully pumped a minimum of once per year. This frequency may be increased if high levels of sediment loading are observed.

Schematic details of the BaySaver system are presented as follows.

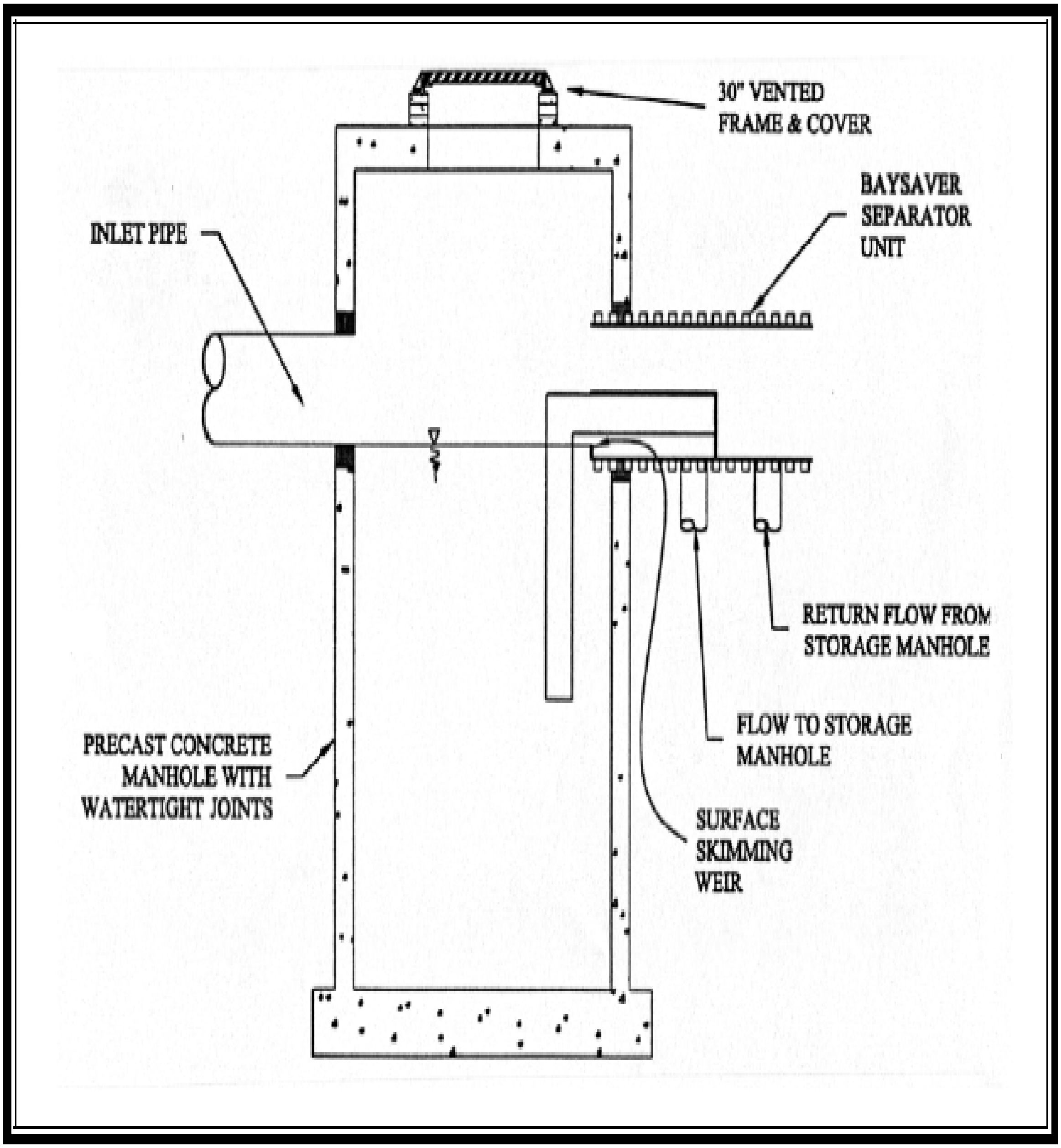


Figure 0.6 - BaySaver Primary Separation Manhole

Source: Virginia Department of Conservation and Recreation/Environmental Quality. *Virginia Stormwater Management Handbook*. Richmond, Virginia, 1999.

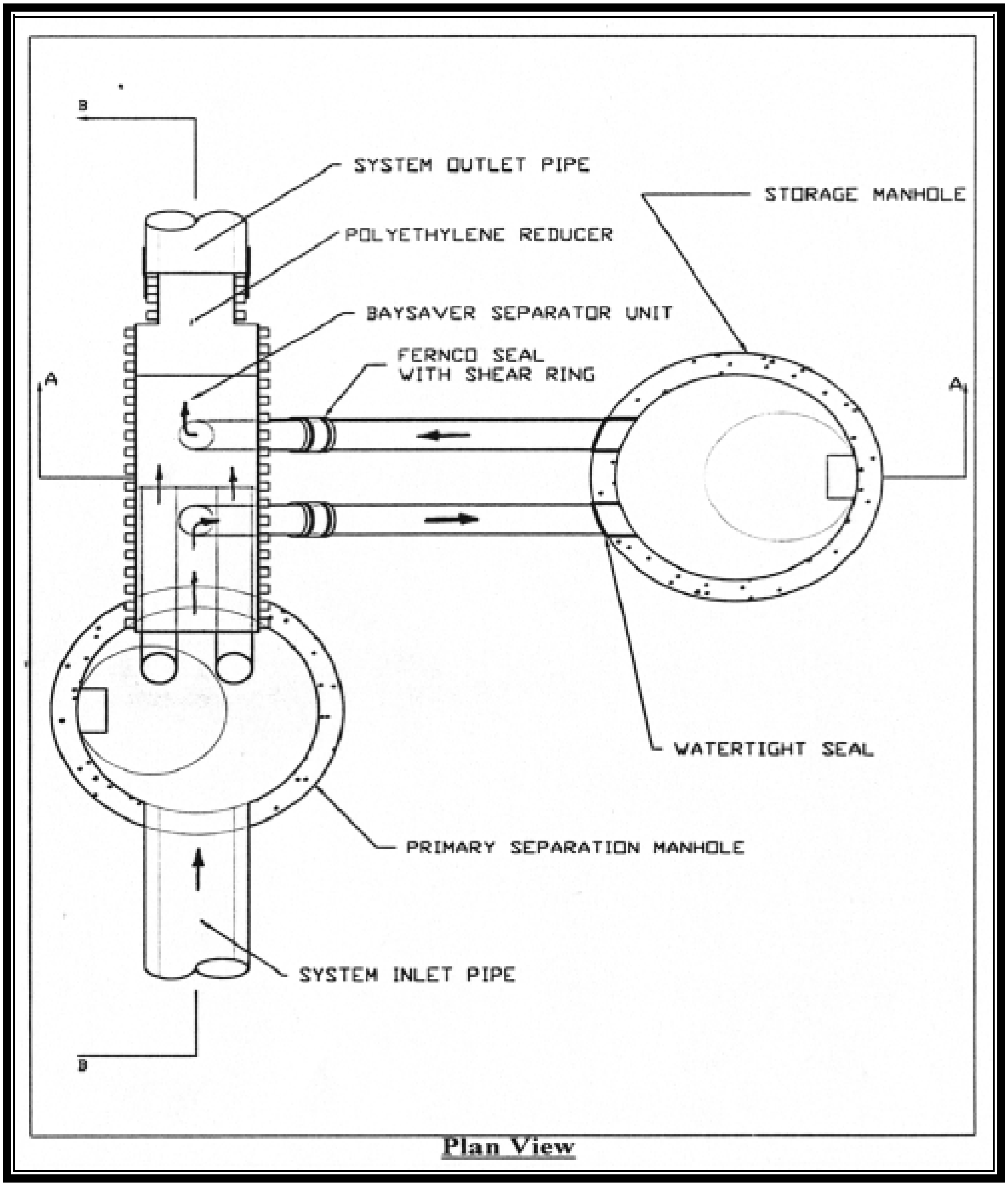


Figure 0.7 - Plan View of BaySaver System

Source: Virginia Department of Conservation and Recreation/Environmental Quality. *Virginia Stormwater Management Handbook*. Richmond, Virginia, 1999.

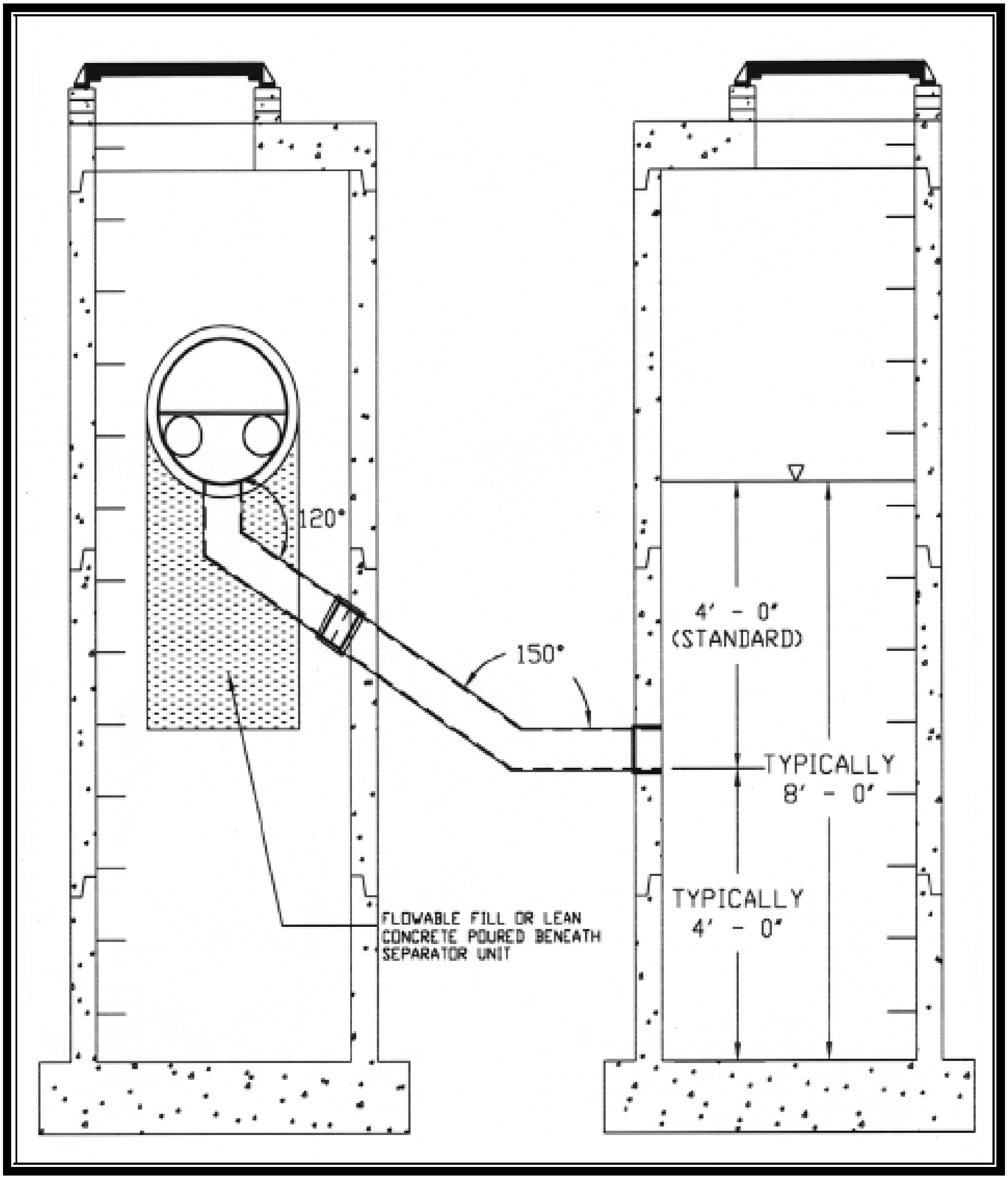


Figure 0.8 - Section through BaySaver Storage Manhole

Source: Virginia Department of Conservation and Recreation/Environmental Quality. <u>*Virginia Stormwater Management Handbook*</u>. Richmond, Virginia, 1999

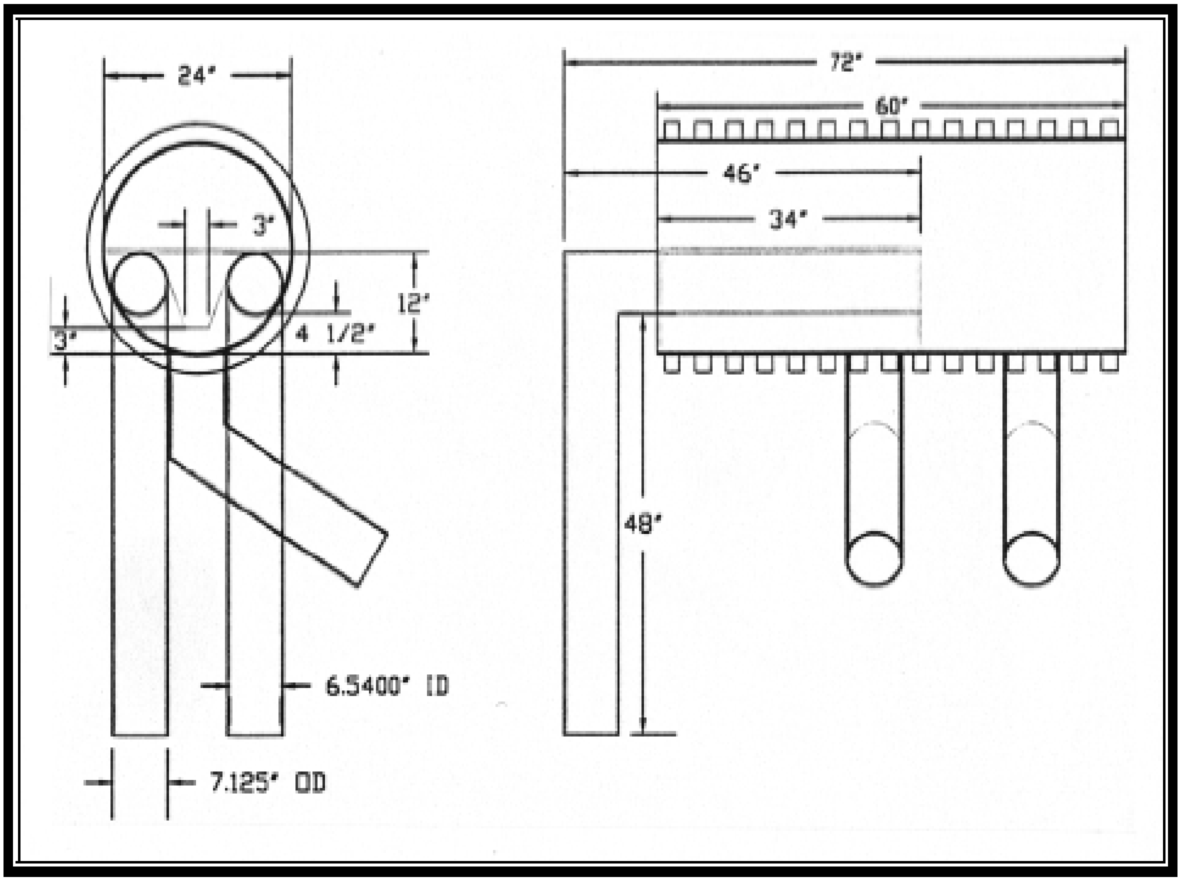


Figure 0.9 - BaySaver Separation Unit

Source: Virginia Department of Conservation and Recreation/Environmental Quality. _Virginia Stormwater Management Handbook._ Richmond, Virginia, 1999

Current Baysaver product information and vendor contacts can be obtained at: http://www.baysaver.com/

The Virginia Transportation Research Council, via contract with University of Virginia, has constructed the following information matrices for the most widely used hydrodynamic separators, as of 2004. The user is referred to the following for the originally published matrices:

Virginia Transportation Research Council. _VDOT Manual of Practice for Stormwater Management_. Charlottesville, Virginia, 2004.

Table 16.2

Table 0.1 - Hydrodynamic Separators Information Matrix (VTRC, 2004)							
System Type	**Manufacturer**	**Operation**	**Sizing and/or Area Treated**	**Maintenance**	**Cost**	**General Performance**	**Comments**
Hydrodynamic Separators							
V2B1	Kistner, Inc.	Swirl concentrator in 2 chambers. Second chamber collects floatables and has outlet. Maintains wet pool. Treats only first flush.	1 – 25 cfs treatment capability. Sized for local 2-month storm. Flows greater than first flush diverted directly to outlet.	Required only in first chamber if regular maintenance. Residuals removed by vacuum truck.	ND	80% TSS removal for first flush.	Floating pollutants isolated from peak storm flows.
Bay Saver®	Bay Saver, Inc.	Gravity treatment in 2 manholes connected by HDPE separator. Primary manhole in-line with the	Either according to flow rate or impervious area. Three units	Required in either chamber when accumulation reaches 2 ft.	$7,000 - $18,000 (materials only)	Designed to remove TSS, O&G, and debris,*	
		storm drain. First-flush or low-flow diverted to storage chamber for settling and O&G removal. Outflow from center of static wter column to retain floatables back to primary manhole. Maintains wet pool in storage chamber.	available correspond to range of treatable areas: 1.2 – 8.0 acres impervious area. Largest systems treats maximum up to 11 cfs.				
Stormceptor®	CSR America	Manhole-shaped device. First-flush or low flows divered beneath high-flow platform to settling chamber. Outflow from center of static water column to retain floatables. Maintains wet pool.	8 units available: 900 – 7,200 gal.; 0.55 – 6.7 acres of impervious area. Sized to treat 90% of annual rainfall.	Perform maintenance when stored material reaches 15% total system volume. Recommend quarterly inspections during first year to establish schedule.	Typical installation is $9,000 for 1 acre drainage area. Unit cost: $7,600 - $33,560 per unit. (US EPA, 1999e)	Varying reports. Vendor claims 50 – 80% removal of TSS based on field testing by contracted agencies. Canada ETV reports 81-94% TSS removal; 42-67% TKN removal.	Improper installation compromises system performance. Also, available with inflow configured for curb inlet or submerged application. Over 4,000 installations.
Stormvault™	Jensen Precast	Rectangular footprint. Interior baffles minimize horizontal velocity to enhance settling and prevent resuspension. Bypass available.	Variable sizes afforded by adding modular sections. Sized to treat 85% annual rainfall or runoff. Variable outlet structure allows extended detention.	Large footprint allows extended periods between maintenance. Recommended inspections to establish schedule.	ND	Laboratory testing indicates low horizontal velocity near vault bottom to minimize resuspension. Extensive evaluation provided in Brisbane et al., 2000	Several field monitoring studies are being performed.
Vortechs™	Vortechnics, Inc.	Rectangular footprint comprised of 3 chambers; swirl concentrator, O&G removal, underflow to energy dissipator. Maintains wet pool.	10 units available to treat maximum 10-yr design storms of 1.6 – 25 cfs without bypassing. On-line system sizing criteria based on 1 ft^2 grit chamber surface area per 100 gpm peak flow rate.	Monthly inspection during first year after installation or whenever loading have been high.	$10,000 - $40,000 per unit, not including shipping or installation (US EPA, 1999e)	Vendor claims 80% TSS removal for flow less than or equal to design events. Sediment storage capacity 0.75 – 7.0 yd^3 depending on model.*	Improper installation compromises system performance. 1998 US EPA Environmental Technology Innovator Award.
CDS®	CDS Technologies, Inc.	Non-mechanical screening system. Circular flow maintained within unit.	Treats first-flush with bypass option. Precast systems	3 – 4 times per year. Frequent inspection is required	$2,300 - $7,200 per cfs capacity (including	100% of particle size of mesh opening; Over 90% for	Vendor has won several engineering awards in Australia.

Table 16.2 – Cont'd

Table 16.2 Cont'd. – Hydrodynamic Separators Information Matrix (VTRC, 2004)

System Type	Manufacturer	Operation	Sizing and/or Area Treated	Maintenance	Cost	General Performance	Comments
		Pollutants settle to sump or remain floating and trapped in center column. Radial flow cleans screens. Maintains wet pool.	available up to 62 cfs. Cast-in-place options can treat up to 300 cfs. Screen size and unit diameter determined for specific applications.	especially during first month after installment. Maintenance includes inspection of screens for damage and measurement of sediment depth.	installation)	particles ½ the size of opening; over 85% for particles 1/3 size of opening; 80-90% O&G using sorbent materials. Complete trash removal*	Installations in the US, Australia and New Zealand.
Downstream Defender™	H.I.L. Technology, Inc.	Swirl concentrator creates a 3D flow path. Sediment settles to bottom of storage area. O&G also stored outside treatment path to prevent re-entrainment. Maintains a wet pool.	4 units range from 0.74 to 13 cfs design flows with corresponding 3 – 25 ft^3 capacity.	Clean-out after 1 – 2.5 ft of sediment accumulates – or annually.	$10,000 - $35,000 per unit (including installation)	PSD trapped sediments 0.001 – 0.01 mm (over 95% measured less than 75 µm). Estimate total solids removal was over 80% for theoretical design flows. Oil storage capacity 70 – 1050 gal.; sediment storage capacity of 0.7 – 8.7 yd^3.	ND

References

Virginia Department of Conservation and Recreation/Environmental Quality. Virginia Erosion and Sediment Control Handbook. Richmond, Virginia, 1992.

Virginia Department of Conservation and Recreation/Environmental Quality. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Virginia Department of Transportation. Virginia Department of Transportation Road and Bridge Standards. Richmond, Virginia, 2016.

Virginia Tech & Penn State Urban Hydrology Model. T.A. Seybert, D.F. Kibler, A.S. Chamberlain, T.F. Smith. Version 6.2. June 2005.

U.S. Army Corps of Engineers. Technical Memorandum Number 9. Washington, D.C., 1987.

U.S. Department of Transportation. Federal Highway Administration. Hydraulic Design of Highway Culverts. Washington, D.C.: 1985 (revised 2001).

Federal Highway Administration. Evaluation and Management of Highway Runoff Water Quality, FHWA-PD-96-032, Washington, D.C., 1996.

U.S. Bureau of Reclamation. Design of Small Dams. 1960 (Revised 1977).

Virginia Transportation Research Council (Yu, Et al.). VDOT Manual of Practice for Stormwater Management. Charlottesville, Virginia, 2004.

Akan, A. O. Open Channel Hydraulics. Elsevier/Butterworth-Heinemann, 2006.

Federal Highway Administration (Chen and Cotton). Design of Roadside Channels with Flexible Linings, Hydraulic Engineering Circular 15, FHWA-IP-87-7, 1988.

Federal Highway Administration. Evaluation and Management of Highway Runoff Water Quality, FHWA-PD-96-032, Washington, D.C., 1996.

Mays, L. W. Water Resources Engineering. John Wiley & Sons, Inc., 2005.

Bell, Warren. A Catalog of Stormwater Quality Best Management Practices for Heavily Urbanized Watersheds. National Conference on Urban Runoff Management: Enhancing Urban Watershed Management at the Local, County, and State Levels, 1993.

Maryland Department of the Environment. Maryland Stormwater Design Manual. Baltimore, MD 2000.

Osmundson, Theodore. Roof Gardens History, Design, and Construction. New York: W.W. Norton & Company, 1999.

Pennsylvania Department of Environmental Protection. Stormwater Best Management Practices Manual. Harrisburg, PA, 2006.

Texas Water Development Board. The Texas Manual on Rainwater Harvesting. Austin, Texas. Third Edition, 2005.

Virginia Transportation Research Council (Yu, Et al.). VDOT Manual of Practice for Stormwater Management. Charlottesville, Virginia, 2004.

AMEC Earth and Environmental, Center for Watershed Protection, Debo and Associates, Jordan Jones and Goulding, Atlanta Regional Commission. Georgia Stormwater Management Manual. Atlanta, Georgia, 2001.

Appendix 11A-2 Part IIB Best Management Practices

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Figure 3.1 - Schematic Grass Channels - Typical Plan 24
Figure 3.2 - Schematic Grass Channels -Typical Profile.. 24
Figure 3.3 - Schematic Grass Channels - Typical Section.. 25

1.1 Introduction

This Appendix was prepared for the Virginia Department of Transportation by Virginia Tech under contract for the Virginia Center for Transportation Innovation & Research. It provides guidance in the design of Best Management Practices capable of contributing to the goal of stormwater management as defined in Instructional and Informational Memorandum of General Subject *"Virginia Stormwater Management Program"* (IIM-LD-195), which states:

"Inclusive of this stormwater management program is a post-construction component that inhibits the deterioration of the aquatic environment by maintaining the post-development water quantity and quality runoff characteristics, as nearly as practicable, equal to or better than pre-development runoff characteristics."

Additionally, the design examples apply the BMP design methodologies found in the *Virginia Stormwater Management Handbook, 2nd Edition, Draft (DCR/DEQ, 2013)* to the site conditions and constraints typically encountered in linear development projects.

It is assumed that the readers of this document are knowledgeable in the engineering disciplines of hydrology and hydraulics and will understand fundamental fluid flow principles used in this manual.

This Appendix does not constitute a standard, specification, or regulation.

1.2 Project Site

The *project site* is defined as:

The area of actual proposed land disturbance (i.e., construction limits) plus any right of way acquired in support of the proposed land disturbance activity/project. Any support areas within existing or proposed VDOT right of way associated with the proposed land disturbance activity/project and identified in the pre-construction SWPPP for the proposed land disturbance activity/project shall also be considered a part of the site. Permanent easements and/or other property acquired through the right of way acquisition process in conjunction with the proposed land disturbance activity/project may be considered a part of the site and utilized in the determination of the post-development water quality requirements, provided such property will remain under the ownership/control of the VDOT and providing such property is so identified/designated on the proposed land disturbance activity/project plans and is legally encumbered for the purpose of stormwater management.

1.3 Design Treatment Volume

Treatment volume for practices discussed in this manual is related to a 1" rainfall over the contributing drainage area. The treatment volume is related to the area, a volume coefficient, and the composite runoff coefficient, as shown in **Equation 1.1**, below:

$$T_v = \left[\frac{(C_v)(1.0\text{ in.})\left(R_{v_{composite}}\right)(A)}{12}\right] \qquad (\mathbf{1.1})$$

where C_v is the volume coefficient (dependent on design level), T_v is the computed treatment volume (acre-ft), and A is the contributing drainage area (acres). The composite runoff coefficient, $R_{v_{composite}}$ is derived from the runoff reduction method for the contributing drainage area, A (acres). The *Virginia Stormwater Management Handbook, 2nd Edition, Draft (DCR/DEQ, 2013), Chapter 11* defines $R_{v_{composite}}$ as:

$$R_{v_{composite}} = \left(R_{v_I} \times \%I\right) + \left(R_{v_T} \times \%T\right) + \left(R_{v_F} \times \%F\right) \qquad (\mathbf{1.2})$$

where:

$R_{v_{composite}}$	=	Composite weighted runoff coefficient
R_{v_I}	=	Runoff coefficient for Impervious cover (**Table 1.1**)
R_{v_T}	=	Runoff coefficient for Turf cover (**Table 1.1**)
R_{v_F}	=	Runoff coefficient for Forested cover (**Table 1.1**)
$\%I$	=	Percent of site in Impervious cover (fraction)
$\%T$	=	Percent of site in Turf cover (fraction)
$\%F$	=	Percent of site in Forested cover (fraction)

Equation 1.2 and Table 1.1 are used to calculate $R_{v_{composite}}$ for the post-development condition.

Table 1.1 - Land Cover Volumetric Runoff Coefficients (Rv)
(Virginia Stormwater Management Handbook, Chapter 11, 2014, Et seq.)

Land Cover	Runoff Coefficients			
	HSG-A	HSG-B	HSG-C	HSG-D
Forest/Open Space	0.02	0.03	0.04	0.05
Disturbed Soil or Managed Turf	0.15	0.20	0.22	0.25
Impervious Cover	0.95			

1.4 Water Quality and Quantity Standards

For new projects, water quality and quantity standards shall conform to Part IIB (9VAC25-870-62) of the Virginia Stormwater Management Regulations.

"Part IIB (9VAC25-870-62 et. seq.) contains the "new" technical criteria that include the Runoff Reduction methodology (for determining compliance with water quality requirements) and the Energy Balance Equation (for determining compliance with the stream channel flooding and erosion requirements). Part IIB technical criteria are applicable to non-grandfathered projects."

For projects that have been grandfathered, the requirements of Part IIC (9VAC25-870-93 et. seq.) of the Virginia Stormwater Management regulations shall apply. VDOT shall determine if a project is grandfathered prior to design.

"Part IIC(9VAC25-870-93 et. seq.) contains the "old" technical criteria that include the Performance/Technology-Based methodology (for determining compliance with water quality requirements) and MS-19 criteria (for determining compliance with stream channel flooding and erosion requirements). Part IIC technical criteria are applicable to grandfathered projects.

2.1 Sheet Flow - Overview of Practice

Filter strips are used to treat runoff from areas that generate and deliver sheet flow from adjacent impervious and managed turf areas by slowing the velocity of runoff, which allows sediment and pollutants to be filtered by vegetation and/or settled out of stormwater runoff. Two variations of sheet flow practices as outlined by *Virginia DCR/DEQ Stormwater Design Specification No. 2, (2013)* are *Conserved Open Space* and *Vegetated Filter Strips.* Although Conserved Open Space is allowed in principal, it is unlikely that the right of way associated with a VDOT project will contain the required minimum conservation space to ensure long term viability of the practice; therefore information regarding use of conserved open space is not included in this document.

Due to the requirement of a uniform linear edge to maintain runoff as sheet flow, these practices are applicable to a wide array of road construction projects.

Table 2.1 - Summary of Stormwater Functions Provided by Filter Strips [1]
Modified from *Virginia Stormwater Design Specification No. 2, Draft (DCR/DEQ, 2013)*

Stormwater Function	Vegetated Filter Strip	
	HSG Soils A	HSG Soils B [4], C and D

	No CA [3]	With CA [2]
Annual Runoff Vol. Reduction (RR)	50%	50%
Total Phosphorus (TP) EMC Reduction[5] by BMP Treatment Process	0	
Total Phosphorus (TP) Mass Load Removal	50%	50%
Total Nitrogen (TN) EMC Reduction by BMP Treatment Process	0	
Total Nitrogen (TN) Mass Load Removal	50%	50%
Channel Protection and Flood Mitigation	**Partial.** Designers can use the VRRM Compliance spreadsheet to adjust curve number for each design storm for the contributing drainage area; *and* designers can account for a lengthened Time-of-Concentration flow path in computing peak discharge.	

[1] CWP and CSN (2008); CWP (2007)

[2] CA = Compost Amended Soils (see Design Specification No. 4)

[3] Compost amendments are generally not applicable for undisturbed A soils, although it may be advisable to incorporate them on mass-graded A or B soils and/or filter strips on B soils, in order to maintain runoff reduction rates.

[4] The plan approving authority may waive the requirement for compost amended soils for filter strips on B soils under certain conditions (see Section 6.2 below)

[5] There is insufficient monitoring data to assign a nutrient removal rate for filter strips at this time.

2.2 Site Constraints and Siting of the Facility

When sheet flow is proposed to either conserved open space or managed turf, the designer must consider a number of site constraints to ensure that the practice is applicable to the suggested use.

2.2.1 Filter Strip Location

Ideally, the vegetated filter strip shall be located within the VDOT property or right-of-way or, if not, then subject to a drainage easement to which VDOT has appropriate access to ensure proper inspection and continued proper function of the practice.

2.2.2 Maximum Drainage Area (CDA) and Contributing Flow Path

Vegetated filter strips should be restricted to treatment scenarios where the contributing drainage area is small, typically 5,000 ft^2 or less. It is important to design vegetative filter strips within the limits established for contributing drainage areas. Too much or too little runoff can result in performance issues and the need for subsequent repairs. Typically, the crucial design factor is the length of the contributing flow path, which is shown in Table 2.2. The overall contributing drainage area must be relatively flat to ensure sheet flow draining into the filter area. Where this is not possible, alternative measures, such as an Engineered Level Spreader (ELS), can be used.

2.2.3 Site Slopes

Slopes approaching vegetated filter strips shall be kept to a minimum in order to maintain sheet flow. Typically, for many applications, the maximum slope entering pretreatment shall be the maximum shoulder slope (typically 8%) as allowed in the *VDOT Road and Bridge Standards*, latest edition.

2.2.4 Site Soils

Filter strips are allowable in all soil types. Use in fill soils and HSG B, C, and D soils will likely require the use of compost amendments (see **Table 2.2**). Engineer shall indicate on plans for the Contractor to keep filter strip area off-line and free from construction vehicle traffic, in accordance with VDOT Special Provision for Sheet Flow to Vegetated Filter Strip (2014). The runoff reduction associated with the measure shall be associated with the underlying Hydrologic Soil Group (HSG) and whether or not composted soil amendments are used to supplement existing soils in the area of the filter strip.

2.2.5 Depth to Water Table

Generally, vegetated filter strips will not function to optimum levels in the presence of a seasonally high water table. If a high water table is encountered, the designer and/or Contractor shall notify VDOT immediately to determine any corrective actions necessary to address the lever of groundwater on the site.

2.2.6 Karst Areas

Vegetated filter strips may be used in karst areas. However, an adequate receiving system down grade of the filter must be evaluated to be consistent with requirements of the *Virginia Stormwater Management Handbook, 2nd Edition, Draft (DCR/DEQ, 2013)* as they relate to stormwater discharge in karst areas.

2.2.7 Utilities

Vegetated filter strips may be constructed over existing and proposed utilities. Generally utilities that cross (perpendicular) a vegetated filter strip are preferred. Long longitudinal runs of utilities (parallel to road) through a grass filter strip should be discussed with VDOT prior to incorporating on plans due to long term issues with maintenance on utilities affecting operation and maintenance of the vegetated filter.

Table 2.2 - Filter Strip Design Criteria

Modified from *Virginia Stormwater Design Specification No. 2, Draft (DCR/DEQ, 2013)*

Design Issue	Vegetated Filter Strip
Soil and Vegetative	Amended soils and dense turf cover or landscaped with herbaceous cover,

Cover	shrubs, and trees
Overall Slope and length (parallel to the flow)	1% [1] to 4% Slope – Minimum 35' length 4% to 6% Slope – Minimum 50' length 6% to 8% Slope – Minimum 65' length The first 10' of filter must be 2% or less in all cases
Contributing Area of Sheet Flow	Maximum flow length of 150' from adjacent pervious areas; Maximum flow length of 75' from adjacent impervious areas
Level Spreader for dispersing Concentrated Flow	Length of ELS [3] Lip = 13 lin.ft. per each 1 cfs of inflow (13 lin.ft. min; 130 lin.ft. max.)
Construction Stage	Prevent soil compaction by heavy equipment
Typical Applications	Treat small areas of Impervious Cover
Compost Amendments	Optional (A soils) Yes (B, C, and D soils) [2]
Boundary Spreader	GD [3] at top of filter PB [3] at toe of filter

[1] A minimum of 1% is recommended to ensure positive drainage.
[2] The plan approving authority may waive the requirement for compost amended soils for filter strips on B soils under certain conditions
[3] ELS = Engineered Level Spreader; GD = Gravel Diaphragm; PB = Permeable Berm.

2.3 General Design Guidelines

The following presents a collection of design considerations when designing and installing a vegetated filter strips for improvement of water quality. Cross-section details for specific design features, including material specifications, can be found in the *VDOT BMP Standard Detail SWM-2—Sheet Flow to Vegetated Filter Strip.*

2.3.1 Vegetated Filter Strip General Design Requirements

Filter strips should be used to treat small sections of impervious cover, 5,000 ft^2, or less, adjacent to road shoulders. They may be used as pretreatment for other BMPs and may be incorporated into a treatment train.

Vegetated strips shall be designed to meet the following criteria:

- Soils compacted during installation will be restored through compost amendments over the full length and width of the filter strip and according to recommendations listed in *VDOT BMP Standard Detail SWM-2*—Sheet Flow to Vegetated Filter Strip.
- The proposed strip shall be identified on the project's erosion and sediment control plan.
- After construction is complete, maintenance of the strip shall follow procedures outlined in the *VDOT BMP Maintenance Manual (2015),*

unless prior approval from VDOT project manager is received for alternative maintenance procedures.

2.3.2 Slopes

The allowed range for slopes through a filter is typically 1.0%-8.0% slope, in order to maintain sheet flow throughout. In addition, upstream slopes should be relatively flat to maintain sheet flow conditions as runoff enters the filter. If restriction of upstream slopes is not possible, a level spreader meeting the requirements shown in VDOT BMP Standard Detail SWM-2—Sheet Flow to Vegetated Filter Strip may be used.

2.3.3 Flow Path

Flow lengths upstream of a filter shall be limited to those values shown in **Table 2.2**.

2.3.4 Hotspot Land Uses

Vegetated filters should not receive runoff directed from stormwater hotspots due to the risk of groundwater contamination.

2.3.5 Compost Amendments

Generally, compost amendments will be required for hydrologic soil group B, C, and D soils. The requirement for amendments in type B soils may be waived at the discretion of VDOT if the designer provides additional information to VDOT regarding soil type, texture, and profile, and the area will be protected from disturbance during construction. Compost amendments shall be installed according to the depths outlined in VDOT BMP Standard Sheet SWM-4. The media used for amending the soils shall be Engineered Soil Media Type 3, as found in the VDOT Special Provision for Soil Compost Amendment (2014). Installation of compost amendments shall be in accordance with the VDOT Special Provision for Sheet Flow to Vegetated Filter Strip (2014).

2.3.6 Planting

Vegetation shall be installed at an appropriate density to achieve a 90% grass/herbaceous cover after the second growing season. Sod shall not be applied in filter strip areas. Species utilized within filter strips shall be salt tolerant.

2.3.7 Diaphragms, Berms and Level Spreaders

Proper pre-treatment preserves a greater fraction of the Treatment Volume over time and prevents large particles from clogging orifices, filter material, and infiltration sites. Selecting an improper type of pre-treatment or designing and constructing the pre-treatment feature incorrectly can result in performance and maintenance issues. In that respect, a gravel diaphragm is required for all sheet flow entering a vegetated filter strip in roadway applications. The gravel diaphragm shall be installed in accordance with the Typical Gravel Diaphragm Sheet Flow to Vegetated Filter Strip detail shown in VDOT BMP Standard Detail SWM-2—Sheet Flow to Vegetated Filter Strip.

Sources of concentrated inflow upstream of a vegetated filter strip shall be returned to sheet flow through the use of a Type I or Type II level spreader, as outlined in VDOT BMP Standard Detail SWM-2—Sheet Flow to Vegetated Filter Strip. A Type I level spreader shall be used in applications where there is channel inflow, or in areas where upstream sheet flow has become partially concentrated, or violates contributing area length requirements shown in **Table 2.2**. Type II level spreaders can be utilized to transition pipe or channel inflow to sheet flow prior to runoff entering the vegetated filter strip. Receiving areas downstream of level spreaders shall be designed to withstand the shear force created by incoming flows. Stabilization downstream of the spreader will be accomplished using clean, washed VDOT #1 stone, underlain by non-woven geotextile filter fabric, as shown in VDOT BMP Standard Detail SWM-2—Sheet Flow to Vegetated Filter Strip.

2.3.8 Topsoil and Compost Requirements

If existing topsoil will not promote dense turf growth, imported topsoil with the following characteristics may be used:

- Loamy sand or sandy loam texture
- Less than 5% clay content
- Corrected pH between 6 and 7
- Soluble salt content not exceeding 500 ppm
- Organic matter content exceeding 2%
- Topsoil shall be of uniform depth between 6 and 8”

Compost shall be in accordance with the requirements set forth in *VDOT Special Provision for Soil Compost Amendments, 2014.*

2.3.9 Construction and Maintenance

Construction shall be in accordance with the requirements set forth in the VDOT Special Provision for Sheet Flow to Vegetated Filter Strip (2014), and maintenance shall be in accordance with procedures set forth in the *VDOT BMP Maintenance Manual (2015).*

2.4 Design Example

This section presents the design process applicable to vegetated filters serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 11 of the *Virginia Stormwater Management Handbook, 2nd Edition*, Draft (DCR/DEQ, 2013) for details on hydrologic methodology.

Typically, vegetated filters are designed as the first step in a treatment train approach to meeting water quality control requirements. However, due to the applicability for treating sheet flow, there will be applications, such as shoulder widening, that may exclusively use vegetated filters to potentially meet full stormwater quality control requirements. In order to meet the vegetated filter strip requirements, sufficient right-of-way must be present to meet the minimum lengths required (see **Table 2.2**).

A shoulder widening project is planned along I-66 near Front Royal, Virginia. The longitudinal slope along this section of I-66 is approximately 1.0%. The project consists of adding a 6’ paved shoulder along the interior side (into median) of the east bound lanes. In addition, a 50’ wide portion of the median will be regraded for drainage improvements. The presence of HSG D soils along this 1,000’ section of the project will require compost amendments to supplement the existing soil. The disturbed area of the project and the additional impervious area added is minimal. Since the vegetated filter strip can also treat existing runoff up to the road crown, it is particularly well suited for this application.

Due to a wide existing compacted gravel shoulder along the edge of the existing pavement, the proposed widening will only add an additional 0.03 acres of impervious area. Although the disturbed area (including median work) is 1.29 acres, the treatment area extends to the crown of the road, containing an additional 0.29 acres of impervious cover (HSG D), and sums to 1.58 acres total area. In the post-development condition, the time of concentration has been calculated to be 9 minutes. Geotechnical investigations reveal compacted soil with a high clay content. Lab tests confirm that infiltration cannot be performed at this location. The project site does not exhibit a high or seasonally high groundwater table.

Table 2.3 - Hydrologic Characteristics of Example Project Site

		Impervious	Turf
Pre	**Soil Classification**	**HSG D**	**HSG D**
	Area (acres)	0.00	1.29
Post	**Soil Classification**	**HSG D**	**HSG D**
	Area (acres)	0.03	1.26

Step 1 - Enter Data into VRRM Spreadsheet

The required site data from **Table 2.3** is input into the VRRM Spreadsheet for Redevelopment (2014) to compute load reductions for a linear project, resulting in site data summary information shown in **Table 2.4**. Note that using the redevelopment spreadsheet, the required reduction for linear projects is computed as the sum of the Post-Redevelopment Load and the Post-Development Load minus 80% of the Predevelopment Listed load.

Table 2.4 - Summary of Output from VRRM Site Data Tab

Site Rv	**0.27**
Post-development TP Load (lb/yr)	**0.78**
Total TP Load Reduction Required (lb/yr)	**0.20**

It is important to note that the values in Table 2.4 are only the values for the disturbed area of the project. Although other run-on areas (0.29 acres total) were described in the problem statement, they are not part of the disturbed area, and should not be entered as such in the VRRM Spreadsheet to compute required reductions (**Table 2.4**).

The vegetated filter will be used to treat runoff from the disturbed area and the run-on area (0.29 acres). Note that the VRRM Spreadsheet will warn the user that the area (1.58 acres) exceeds the disturbed area (1.29 acres); however, it is acceptable to treat adjacent run-on area as part of the project. Appropriate data for post-development conditions is input into the VRRM Spreadsheet Drainage Area tab, yielding compliance results summarized in **Table 2.5.**

Table 2.5 - Summary of Output from VRRM Site Data Tab for Full Treatment Area

Total Impervious Cover Treated (acres)	**0.32**
Total Turf Area Treated (acres)	**1.26**
Total TP Load Reduction Achieved in D.A. A (lb/yr)	**0.71**

In this case, the total phosphorus reduction required is 0.20 lbs/yr. The estimated removal is 0.71 lbs/yr; therefore, the target has been met.

Step 2 - Enter Data in Channel and Flood Protection Tab

Hydrologic computations for required design storms for flood and erosion compliance are not shown as part of this example. The user is directed to the VDOT Drainage Manual for appropriate levels of protection and design requirements related to erosion and flood protection.

Values for the 1-, 2-, and 10-year 24- hour rainfall depth should be determined from the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 and entered into the “Channel and Flood Protection” tab of the spreadsheet. For this site (Lat 38.942421, Long -78.138086), those values are shown in **Table 2.6**.

Table 2.6 - Rainfall Totals from NOAA Atlas 14

	1-year storm	2-year storm	10-year storm
Rainfall (inches)	2.51	3.02	4.47

Curve numbers used for computations should be those calculated as part of the runoff reduction spreadsheet (Virginia Runoff Reduction Spreadsheet for Redevelopment, 2013). For this site, computed adjusted curve numbers are 81, 81 and 82 for the 1-, 2- and 10-year storms, respectively (**Table 2.7**).

Table 2.7 - Adjusted CN from Runoff Reduction Channel and Flood Protection Sheet

	1-year Storm	2-year Storm	10-year Storm
$RV_{Developed}$ (in) with no Runoff Reduction	1.12	1.53	2.79
$RV_{Developed}$ (in) with Runoff Reduction	0.93	1.34	2.59
Adjusted CN	**81**	**81**	**82**

The values reported in Table 2.7 are only valid for the drainage area served by the proposed vegetated filter drainage subarea. The remaining portion of the site drainage area should use the appropriate curve numbers for those areas.

Input data is used in the Natural Resource Conservation Service Technical Release 55 (NRCS TR-55) Tabular method to calculate discharge hydrographs. (**Note that other hydrologic methodologies are suitable-see VDOT Drainage Manual, Hydrology for guidance**) Peaks of those hydrographs for the 1-, 2-, and 10-year storms are reported in **Table 2.8**. These values can be used to size the conveyance downstream of the vegetated filter (not shown in this design example).

Table 2.8 - Post-development Discharge Peaks

	1-year storm	2-year storm	10-year storm
Discharge (cfs)	1.70	2.43	4.93

Step 3 - Select Filter Length

Because the travel lane and proposed paved shoulder has a 2.08% cross slope and the filter will extend at a 4.0% grade cross-slope from the edge of shoulder, the required filter length (in the direction of flow) from **Table 2.2** is 35'. The designer should also confirm that the upstream length restrictions are not violated during the design. In this case, the length of the shoulder and existing travel lane to the crown total 20'; therefore, the maximum upstream length of 75' of paved surface is not violated.

Step 4 - Determine Compost Amended Soil Requirements

Because the underlying soil type is HSG D soils, the area where the filter will be implemented must be amended. Amendments will be according to specifications shown in the VDOT Special Provision for Soil Compost Amendments, 2013. Based on the requirements in that document, amendments for this project will require incorporating 10" of compost to a minimum incorporated depth of 11.6" (see detailed calculations in Section 4.4) using a tiller. Specific compost requirements and incorporation requirements are discussed in that document.

Step 5 - Seeding

The grass chosen should be able to withstand both wet and dry periods. The user is directed to the *Virginia Erosion Control Handbook (1992)* permanent seeding chapter for guidance. The selected seed mix combination should provide low maintenance, tolerance of moisture conditions, and be tolerant to high salt concentrations during the winter months.

Step 6 - Design of Overflow and Conveyance Structures

Overflow and conveyance structures must be designed to pass the specified design storm based on functional classification of the road. This includes calculations for overtopping of the check dams by storms of lower recurrence (i.e. 25-, 50-, and 100-year storms). These computations are beyond the scope of this design example. However, the user is directed to the VDOT Drainage Manual for guidance on flood and erosion compliance calculations.

3.1 Grass Channels - Overview of Practice

Grass channels are effective in providing moderate peak attenuation, volume reduction, and filtering of stormwater runoff. They are particularly effective as a first line treatment option in a treatment train, or for treatment of runoff prior to its entry into inlets or culverts. Although they cannot provide as much volume or pollutant reduction as dry swales, their performance can be improved through the use of soil amendments within the channel.

Grass channels are preferable to curb and gutter or storm drains due to their ability to treat the runoff, unlike the impervious alternatives. The *Virginia Stormwater Design Specification No. 3, Grass Channels, Draft (DCR/DEQ, 2013)* describes grass channels as particularly well suited to linear applications, such as transportation related projects.

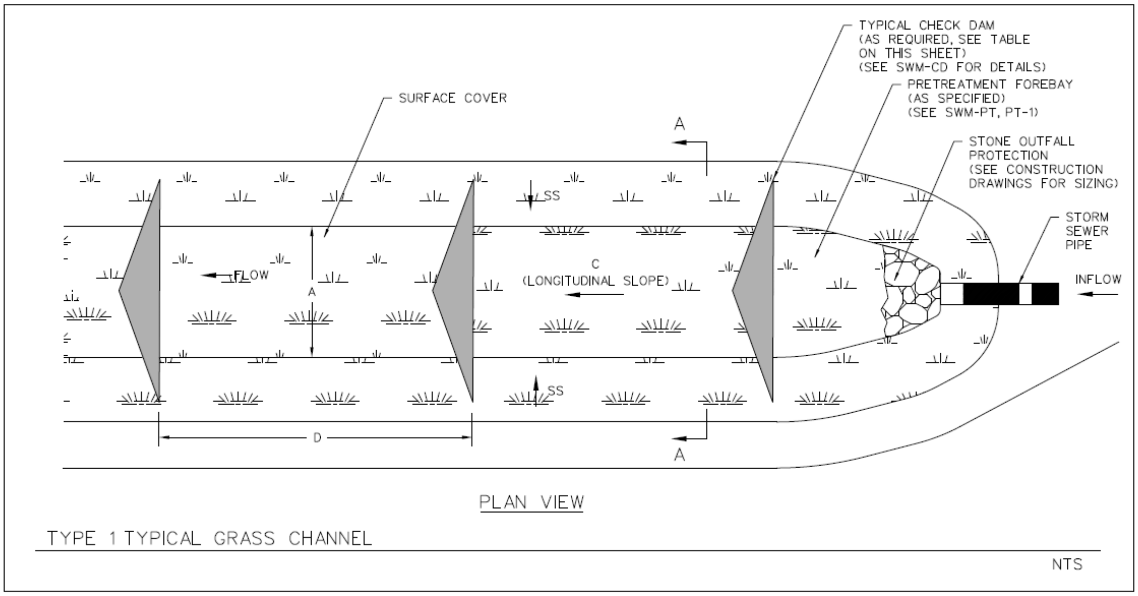


Figure 3.1 - Schematic Grass Channels - Typical Plan
VDOT SWM-3 Grass Channels, 2015

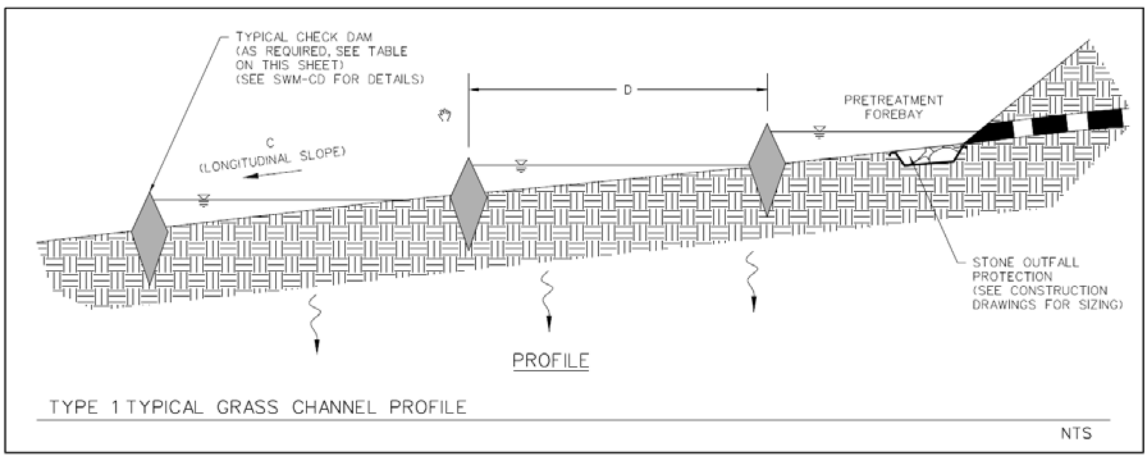


Figure 3.2 - Schematic Grass Channels -Typical Profile
VDOT SWM-3 Grass Channels, 2015

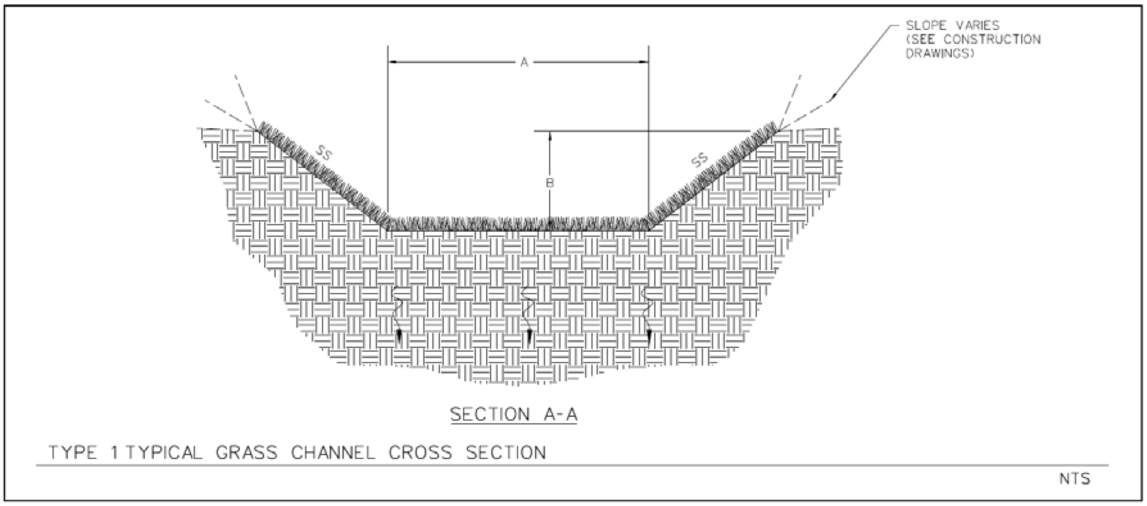


Figure 3.3 - Schematic Grass Channels - Typical Section
VDOT SWM-3 Grass Channels, 2015

Table 3.1 - Stormwater Functions Provided in Grass Channels[1]
Virginia Stormwater Design Specification No. 3, Grass Channels, Draft (DCR/DEQ, 2013)

<table>
<tr><th rowspan="2">Stormwater Function</th><th colspan="2">HSG Soils A and B</th><th colspan="2">HSG Soils C and D</th></tr>
<tr><th>No CA [2]</th><th>With CA</th><th>No CA</th><th>With CA</th></tr>
<tr><td>Annual Runoff Volume Reduction (RR)</td><td>20%</td><td>NA [3]</td><td>10%</td><td>20%</td></tr>
<tr><td>Total Phosphorus (TP) EMC Reduction[4] by BMP Treatment Process</td><td colspan="2">15%</td><td colspan="2">15%</td></tr>
<tr><td>Total Phosphorus (TP) Mass Load Removal</td><td colspan="2">32%</td><td colspan="2">24% (no CA) to
32% (with CA)</td></tr>
<tr><td>Channel & Flood Protection</td><td colspan="4">Partial.
● Use VRRM Compliance spreadsheet to calculate a Curve Number (CN) adjustment[5]; OR
● Design extra storage in the stone underdrain layer and peak rate control structure (optional, as needed) to accommodate detention of larger storm volumes.</td></tr>
</table>

[1] CWP and CSN (2008) and CWP (2007).
[2] CA= Compost Amended Soils, see Stormwater Design Specification No. 4.
[3] Compost amendments are generally not applicable for A and B soils, although it may be advisable to incorporate them on mass-graded and/or excavated soils to maintain runoff reduction rates. In these cases, the 20% runoff reduction rate may be claimed, regardless of the pre-construction HSG.
[4] Change in event mean concentration (EMC) through the practice. Actual nutrient mass load removed is the product of the pollutant removal rate and the runoff volume reduction rate (see Table 1 in the *Introduction to the New Virginia Stormwater Design Specifications*).

3.2 Site Constraints and Siting of the Facility

When a grass channel is proposed, the designer must consider a number of site constraints to ensure that the practice is applicable to the suggested use.

3.2.1 Maximum Drainage Area

The maximum drainage area of a grass channel is limited to 5 acres. Past this threshold, there is an increasing likelihood that the velocity of flow in the channel will reach a point that prevents the runoff treatment and effective filtering of the treatment volume. In addition, there is an increasing threat of erosion in the channel as the velocity increases.

3.2.2 Site Slopes

The design and installation of grass channels are limited to relatively shallow slopes due to increased velocity and the threat of erosion on steeper slopes. Soil conditions, turf type, and channel cross-section will affect the maximum sustained velocity that a channel can withstand without erosion. It is the responsibility of the designer to reduce the channel slope to a level that can sustain non-erosive flow. Check dams may be used on moderately steep slopes to reduce the effective channel slope—see **Section 3.3.3**.

3.2.3 Site Soils

Grass channels may be installed on all soil types. However, soil amendments will be required in areas with HSG C and D soils to enhance vegetative growth, improve long term functionality, and promote runoff reduction. Soil compost amendments shall be integrated into the project according to instructions found in the VDOT Special Provision for Grass Channels (2014) and Section 4 of this Appendix.

3.2.4 Depth to Water Table

Grass channels should not be installed on sites with a high groundwater table that results in frequently flowing water during all or part of the year. Grass channels are intended to be dry between storm events.

3.2.5 Separation Distances

A 50’ minimum separation from water supply wells is required. Additionally, a 35’ minimum separation from septic drain fields is required.

3.2.6 Karst Areas

Grass channels are an acceptable practice in karst terrain, as long as they do not treat hotspot runoff as defined in Table 8.10 of DCR/DEQ Stormwater

Specification #8, Infiltration (2013). The following design adaptations apply to grass channels in karst terrain:

- Soil compost amendments in conformance with VDOT Special Provision for Grass Channels (2015), may be incorporated into the bottom of grass channels to improve their runoff reduction capability.
- Check dams are discouraged for grass swales in karst terrain, since they pond too much water (although flow spreaders that are flush with the ground surface and spaced along the channel length may be useful in spreading flows more evenly across the channel width).
- The minimum depth to the bedrock layer is 18".
- A minimum slope of 0.5% must be maintained to ensure positive drainage.
- The grass channel may have off-line cells and should be tied into an adequate discharge point.

3.2.7 Existing Utilities

Grass channels that do not employ compost amendments may be installed over existing utilities. However, grass channels installed parallel over existing utilities should be avoided unless there is a minimum 2' separation between the bottom of channel and top of underlying utility.

3.2.8 Floodplains

Grass channels may be installed in 100 year floodplains if there is no negative impact to flood elevation as mandated by state and federal guidelines.

3.3 General Design Guidelines

Table 3.2 presents a collection of design considerations when designing a grass channel for conveyance of storm water and improvement of water quality. Cross-section details for specific design features are found in the *VDOT BMP Standard SWM-3: Grass Channels (2014).*

Table 3.2 - Grass Channel Design Guidance
Virginia Stormwater Design Specification No. 3, Grass Channels, Draft (DCR/DEQ, 2013)

Design Criteria
The bottom width of the channel shall be set to maintain the peak flow rate for the 1" storm design treatment volume (T_V)[1] at less than 4" in depth and ≤ 1 fps velocity.
The channel side-slopes should be 3H:1V or flatter.
The maximum total contributing drainage area to any individual grass channel is 5 acres.
The longitudinal slope of the channel should be no greater than 4%. (Check dams may be used to reduce the effective slope in order to meet the limiting velocity requirements.[Table 3.3])
The dimensions of the channel should ensure that flow velocity is non-erosive during the 2-year and 10-year design storm events and the 10-year design flow is contained within the channel (minimum of 4 inches-feet of freeboard).

[1] The design of grass channels should consider the entire T_V of the contributing drainage area (rather than the T_VBMP which would reflect a decrease in T_V based on upstream runoff reduction practices) in order to ensure non-erosive conveyance during all design storm conditions.

3.3.1 Channel Parameters

Grass channels are designed to provide conveyance based on peak rates of flow. Longitudinal slopes should typically be between 0.5 and 4.0%; however, the ideal slope is between 1% and 2%. Check dams shall be used in areas of higher slopes to create a lower effective slope that allows reductions in stormwater velocity and erosive potential see **Table 3.3**.

Manning's Equation is typically used to verify the hydraulic capacity of a grass channel based on physical parameters. **Equation 3.1** describes the Manning Equation for flow velocity:

$$V = \left[\left(\frac{1.49}{n}\right) R^{2/3} S^{1/2}\right] \qquad (3.1)$$

where:

V is flow velocity (ft./sec)
n is Manning's roughness coefficient (see discussion below)
R is hydraulic radius (ft), which is the cross sectional area divided by wetted perimeter
S represents the average longitudinal channel slope (ft/ft)

Note that for very shallow flows the hydraulic radius (R) may be approximated by the flow depth, D, in ft.

Grass channels are commonly used to convey runoff to secondary treatment practices. The flow depth for the 1" rainfall should be maintained at a depth of 4" or less. For flows under this depth, the manning coefficient ("n") is 0.2 for well-established grass channels. For a depth of 12", the manning coefficient is reduced to 0.03.

Channels shall be designed to convey runoff without eroding the channel for the 2 and 10-year flows. The 10-year peak flows shall be conveyed within the channel with a minimum of 4" of freeboard.

For linear highway projects, the grass channel shall be evaluated at every significant change in channel cross-section or slope to verify channel adequacy for both non-erosive conveyance and verification of adequate freeboard.

The residence time for the treatment volume (1" rainfall) shall be a minimum of 9 minutes *(Virginia Stormwater Design Specification No. 3, Grass Channels, Draft (DCR/DEQ,2013))*. When multiple inflow points exist, a 9 minute residence time must be demonstrated for each point through evaluation of **Equations 3.1 and**

3.2 (Equations 3-1 and 3-2, *Virginia Stormwater Design Specification No. 3, Grass Channels, Draft (DCR/DEQ, 2013))*

$$q_{pT_v} = VA = V(W \times D) \tag{3.2}$$

where:

q_{pT_v} is design treatment volume (1") peak flow rate (cfs)
A is cross sectional flow area (ft^2)
V is flow velocity (fps)
W represents the channel base width (ft)
D is the flow depth (ft)

Note that the substitution of cross sectional area in **Equation 3.2** with the product of channel width and flow depth is only valid as an approximation for shallow flows.

Combination and manipulation of **Equations 3.1 and 3.2** yields solutions for minimum channel widths and velocities as found in **Equations 3.3 and 3.4**.

$$W = (n)(q_{pT_v})/(1.49D^{5/3}S^{1/2}) \tag{3.3}$$

$$V = q_{pT_v}/(\mathrm{W \times D}) \tag{3.4}$$

The velocity calculated by **Equation 3.4** should be less than 1 fps. Equation parameters, n, W, and S may be adjusted, as necessary, for site conditions to decrease velocity, and thus, increase residence time. The minimum length of channel necessary to achieve a 9 minute residence time can be calculated using the velocity resulting from use of **Equation 3.5**.

$$L = 540V \tag{3.5}$$

where:

L is the minimum channel length (ft).
V is flow velocity (ft./sec.)

3.3.2 Geometry

Grass channels shall be either trapezoidal or parabolic in cross-section in order to facilitate mowing and maintenance. Side slopes should be kept to a maximum slope of 3:1 to facilitate mowing. Typically, the bottom width is between 4' to 8' in width. Wider cross-sections require use of measures (typically check dams) that prevent erosion along the channel bottom.

3.3.3 Check Dams

Check dams (see Figure 3.2) are installed within grass channels, as necessary, to provide temporary impoundment of runoff volume. Their purpose is to decrease velocity and decrease the effective longitudinal slope, which results in an increase of hydraulic residence time within the channel. The height of the check dam should not exceed 12” above the normal channel elevation. Check dams shall be securely anchored into the channel bottom a minimum of 6” and entrenched into the swale side slopes to prevent outflanking during high intensity storms. Soil plugs, which can reduce the chance for a blow out or erosion of the media under the dams, are typically used on slopes of 4% or greater or when maximum height (12”) check dams are used. A weir is designed and installed in the top of the dam to pass design storms (10-year), with appropriate armoring down the back side and at the downstream toe of the dam. A weep hole shall be provided at the base of the dam to allow dewatering after storms. Design and materials for check dam construction shall conform with those listed in the *VDOT BMP Standard –SWM-3: Grass Channels (2014).*

Check dams should be spaced (**Table 3.3**) to allow a minimum of 25’-40’ length between the toe of the upstream check dam and the face of a downstream check dam. Water impoundment on the downstream check dam shall not extend upstream to a point where impounded stormwater touches the toe of the upstream dam.

Table 3.3 - Typical Check Dam (CD) Spacing to Achieve Effective Swale Slope
Virginia Stormwater Design Specification No. 10, Dry Swales, Draft (2013)

Swale Longitudinal Slope	LEVEL 1	LEVEL 2		
	Spacing [1] of 12” High (max.) Check Dams [2, 3] to Create an Effective Slope of 2%	Spacing [1] of 12” High (max.) Check Dams [2, 3] to Create an Effective Slope of 0%	to	1%
0.5%	–	200’	to	–
1.0%	–	100’	to	–
1.5%	–	67’	to	200’
2.0%	–	50’	to	100’
2.5%	200’	40’	to	67’
3.0%	100’	33’	to	50’
3.5%	67’	30’	to	40’
4.0%	50’	25’	to	33’
4.5% [4]	40’	20’	to	30’
5.0% [4]	40’	20’	to	30’

Notes:
[1] The spacing dimension is half of the above distances if a 6” check dam is used.
[2] A Check dams requires a stone energy dissipater at its downstream toe.
[3] Check dams require weep holes at the channel invert. Swales with slopes less than 2% will require multiple weep holes (at least 3) in each check dam.

3.3.4 Runoff Pre-treatment

Upstream pre-treatment should be considered for grass channels to decrease velocity and filter runoff of excess sediments prior to being introduced into the conveyance system. Upstream pre-treatment for grass channels is typically achieved through use of one of the following options.

a. ***Check Dam Forebay***: These cells (**Figures 3.1 and 3.2**) act as forebays to allow sediment to settle out of stormwater runoff prior to entering the grass channel. In addition, it is used as an energy dissipater to reduce the velocity of incoming stormwater runoff and prevent erosive damage within the main channel.

b. ***Grass Filter Strips***: Runoff entering a grass channel as *sheet flow* may be treated by a grass filter strip. The purpose of the grass buffer strip/energy dissipation area is to reduce the erosive capabilities of runoff prior to its entrance into the main channel. The recommended minimum length of the grass filter strip should not be less than 10' when using the maximum side slope of 5:1. An alternative design may be used that integrates road shoulders, requiring a 5' minimum grass filter strip at 20:1 (5%), that is combined with 3:1 (or flatter) side slopes of the swale to provide pre-treatment. See VDOT BMP Standard SWM-PT: Pre-treatment (Pretreatment Forebay).

c. ***Gravel Diaphragms***: These pre-treatment measures are typically installed along the edge of the pavement or roadway shoulder draining into the channel, with the purpose of evenly distributing flow along the length of the channel. See VDOT BMP Standard SWM-PT: Pre-treatment (Gravel Diaphragm).

d. ***Pea Gravel Flow Spreader***: These measures are typically located at points of concentrated inflow, such as curb cuts, etc. There should be a 2" - 4" drop from the adjacent impervious surface into the flow spreader. Gravel/stone should extend along the entire width of the opening, creating a level stone weir at the bottom of the channel. Installation shall be in accordance with VDOT BMP Standard SWM-PT: Pre-treatment (Gravel Flow Spreader).

3.3.5 Compost Soil Amendments

Soil amendments should be considered for all soils that have a hydrologic soil classification of C or D and shall be installed as specified in VDOT Special Provision for Soil Compost Amendment (2014).

3.3.6 Surface Cover

Salt tolerant grass species that can resist erosion and withstand both wet and dry periods as well as high-velocity flows should be used in order to withstand concentrations of deicing solution used to treat roads during the winter. Species selection is based on several factors, including climate, soil type, topography, and sun or shade tolerance and should include those that will achieve a dense cover as quickly as possible. Furthermore, selected species should have the following characteristics: a deep root system to resist scouring; a high stem density with well-branched top growth; water-tolerance; resistance to being flattened by runoff; and an ability to recover growth following inundation. For turf selection, consult the Virginia Erosion and Sediment Control Handbook and **Table 3.4**.

Table 3.4 - Maximum Permissible Velocities for Grass Channels
Virginia Stormwater Design Specification No. 3, Grass Channels, Draft (DCR/DEQ, 2013)

Cover Type	Slope (%)	Erosion Resistant Soils (ft./sec.)	Easily Eroded Soils (ft./sec.)
Bermudagrass	0 – 5	6	4.5
Kentucky bluegrass Reed Canarygrass Tall fescue	0 – 5	5	3.8
Bermudagrass	5 – 10	5	3.8
Kentucky bluegrass Reed Canarygrass Tall fescue	5 – 10	4	3
Grass-legume mixture	0 – 5 5 - 10	4 3	3 2.3
Kentucky bluegrass Reed Canarygrass Tall fescue	> 10	3	2.3
Red fescue	0 - 5	2.5	1.9

3.4 Design Example

This section presents the design process applicable to grass channels serving as water quality BMPs. The pre- and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 11 of the *Virginia Stormwater Management Handbook, 2nd Ed*., Draft (DCR/DEQ, 2013) for details on hydrologic methodology.

A grass channel is proposed along a construction project for improvement of Route 652 in Stafford County. The project increases the pavement width 4' along each side of a 1,200' section of the road. Computations in this example are addressing a single side of the expansion. Similar computations would be required for expansion of the opposing lane. The longitudinal slope along this section of Route 652 is approximately 1.0%. Runoff from the crown to the side of the expansion for this section of the project can be redirected to a BMP location having a total cumulative contributing drainage area (at downstream end of swale) of 1.10 acres. The current lane (on the BMP side of the crown) and shoulder represent 0.35 impervious acres of the total drainage area. The remainder of the total drainage area is 0.75 acres of turf covered shoulder that drains to the area. The entire drainage area overlays HSG A soils (predominantly Kempsville fine sandy loam.

The proposed widening will add an additional 0.10 acres of impervious area (0.45 acres, total land disturbance), and reduce the turf area post-development to 0.65 acres. In the post-development condition, the time of concentration has been calculated to be 8 minutes.

Geotechnical investigations reveal a sandy loam soil that is well drained. Lab tests confirm that infiltration is possible at this location with Ksat ranging between 0.57 and 1.98 in/hr. The project site does not exhibit a high or seasonally high groundwater table.

Table 3.5 - Hydrologic Characteristics of Example Project Site

		Impervious	Turf
Pre	**Soil Classification**	**HSG B**	**HSG B**
	Area (acres)	0.35	0.75
Post	**Soil Classification**	**HSG B**	**HSG B**
	Area (acres)	0.45	0.65

Initially, the designer should use the Virginia Runoff Reduction Method (VRRM) spreadsheet (Virginia Runoff Reduction Spreadsheet for Redevelopment, 2014) to calculate removal for a linear project and ensure that the required water quality load reduction is met by using the proposed grass channel for treatment. Note that using the redevelopment spreadsheet, the required reduction for linear projects is computed as the sum of the Post-Redevelopment Load and the Post-Development Load minus 80% of the Predevelopment Listed load. In this case, the total phosphorus reduction required is 0.39 lbs/yr (**Table 3.6**). The estimated removal is 0.41 lbs/yr; therefore, the target has been met (**Table 3.7**).

Table 3.6 - Site Data Summary Table from VRRM showing Required Phosphorus Removal

Site Rv	0.51
Post-development Treatment Volume (ft^3)	2,024
Post-development TP Load (lb/yr)	1.27
Total TP Load Reduction Required (lb/yr)	0.39

Table 3.7 - Drainage Area Summary Table from VRRM showing Achieved Phosphorus Removal by Grass Channel A/B soils

Total Impervious Cover Treated (acres)	0.45
Total Turf Area Treated (acres)	0.65
Total TP Load Reduction Achieved in D.A. A (lb/yr)	0.41

Values for the 1, 2, and 10-year 24-hour rainfall depth should be determined from the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 and entered into the channel and flood protection tab of the VRRM spreadsheet. For this site (Lat 38.37167, Long -77.49431), those values are shown in **Table 3.8**. Curve numbers used for computations are the adjusted curve number calculated as part of the runoff reduction spreadsheet. For this drainage area, results from the channel protection tab of the runoff reduction spreadsheet are shown in **Table 3.9**, and result in adjusted curve numbers of 74, 74 and 75 for the 1, 2 and 10-year storms, respectively, which is a nominal reduction from the computed unadjusted curve number of 76 (for all return periods).

Table 3.8 - Rainfall Totals from NOAA Atlas 14

	1-year storm	2-year storm	10-year storm
Rainfall (inches)	2.57	3.11	4.79

Table 3.9 - Adjusted CN from Runoff Reduction Redevelopment Spreadsheet Channel and Flood Protection Tab

	1-year storm	2-year storm	10-year storm
$RV_{Developed}$ (in) with no Runoff Reduction	0.74	1.09	2.36
$RV_{Developed}$ (in) with Runoff Reduction	0.64	0.99	2.26
Adjusted CN	**74**	**74**	**75**

Input data (rainfall depths from **Table 3.8**, drainage area, time of concentration, and CN from **Table 3.9**) is used in the Natural Resource Conservation Service Technical Release 55 (NRCS TR-55) Tabular method to calculate discharge hydrographs. Peaks of those hydrographs for the 1, 2, and 10-year storms are reported in **Table 3.10**. These values will be used to evaluate residence time, adequacy, and size the conveyance downstream of the grass channel.

Table 3.10 - Post-development Discharge Peaks to BMP

	1-year storm	2-year storm	10-year storm
Discharge (cfs)	0.86	1.34	3.26

Step 1 - Compute the Treatment Volume Peak Discharge

The length of the project along Route 652 is approximately 1,200'. Since the proposed channel cross-section and longitudinal slope is consistent along the entire length, the channel will be evaluated for compliance at the most downstream end. In order to achieve this, the proposed treatment volume (q_{pT_v}) must be computed. An initial step in computing this value is determining an adjusted CN that generates runoff equivalent to the treatment volume from a 1" rainfall. Note that this adjusted curve number is different than the adjusted curve numbers associated with runoff reduction.

$$CN = \frac{1000}{[10 + 5P + 10Q_a - 10(Q_a^2 + 1.25Q_aP)^{0.5}]} \qquad \textbf{(3.6)}$$

where,

CN = Adjusted curve number
P = Rainfall (inches), (1.0" in Virginia)
Q_a = Runoff volume (watershed inches), equal to $Tv \div drainage\ area$

$$Q_a = \frac{2{,}024\,ft^3}{1.1\,ac\left(\frac{43{,}560\,ft}{1\,ac}\right)}\left(\frac{12\,in}{1\,ft}\right) = 0.51\,in$$

$$CN = \frac{1000}{\left[10 + 5(1\,in) + 10(0.51\,in) - 10\left((0.51\,in)^2 + 1.25(0.51\,in)(1\,in)\right)^{0.5}\right]}$$

$$CN = 94$$

$$q_{pTv} = (q_u)(A)(Q_a) \qquad \textbf{(3.7)}$$

where,

q_{pTv} = Treatment Volume peak discharge (cfs)
q_u = unit peak discharge (cfs/mi^2/in)
A = drainage area (mi^2)
Q_a = runoff volume (watershed inches = Tv/A)

All of the variables are known in the above equation with the exception of q_u. To determine its value, first the initial abstraction must be computed using the equation:

$$I_a = \frac{200}{CN} - 2 \qquad \textbf{(3.8)}$$

$$I_a = \frac{200}{94} - 2 = 0.13\ inches$$

Compute I_a/P where P is the 1" rainfall (inches), which equates to 0.13.

Read the unit peak discharge, q_u, from Exhibit 4-II of the <u>*SCS TR-55 Handbook (1986)*</u>. Reading the chart yields a value of 925 cfs/mi^2/in.

$$q_{pTv} = \left(\frac{925\,\frac{\text{cfs}}{\text{mi}^2}}{\text{in}}\right)\left(\frac{1.1\ ac}{640\ {}^{ac}/_{mi^2}}\right)\left(\frac{2{,}024\ \text{ft}^3}{1.1\ ac \times \left(43{,}560\ {}^{ft}/_{1\ ac}\right)}\right)\left(\frac{12 in}{1\ ft}\right)$$

$$q_{pTv} = 0.81\ cfs$$

Based on the requirements set forth in Virginia DCR/DEQ Stormwater Design Specification No. 3, Grass Channels (2013), the Manning 'n' coefficient is 0.2 for a depth of up to 4". Since a depth of 4" will result in the minimum bottom width estimate, it will be used as a first iteration of **Equation 3.3**.

$$W = (0.20)(0.81\ \text{cfs})/(1.49(0.33\ ft)^{5/3}(0.01\frac{\text{ft}}{\text{ft}})^{1/2})$$

$$W = 6.9\ ft$$

Velocity can now be computed using **Equation 3.4** as:

$$V = \frac{0.81\ cfs}{(6.9\ ft\ \times 0.33\ ft)} = 0.36\frac{ft}{s}$$

This velocity is less than the maximum velocity of 1 fps required and therefore is an acceptable design.

The minimum swale length is calculated using **Equation 3.5** as:

$$L = 540V = (540\ sec)\left(0.36\frac{ft}{s}\right) = 194\ ft$$

The total length of the swale will be a minimum of 1,194', which includes the length adjacent to the project (1,000') and the length downstream of the last inflow location (corresponding to the termination of the project). If an existing receiving channel exists downstream that approximates or exceeds the proposed channel cross-section, then the downstream 194' of channel will not be required.

Step 2 - Compute the Channel Geometry for Conveyance of 10-Year Storm

The peak 10-year flow at the most downstream location is 4.79 cfs, as shown in **Table 3.9**. To facilitate maintenance (mowing), the side slopes of the channel will be 3:1. Ditch computations to verify adequacy for conveyance of the 10-year storm shall meet guidelines shown in the *VDOT Drainage Manual*, latest edition.

Step 3 - Seeding

The grass chosen should be able to withstand both wet and dry periods. The combination should provide low maintenance, tolerance of moisture conditions, and be tolerant of high salt concentrations during the winter months. For compliance with methods specified in the VDOT Special Provision for Grass Channels (2014) temporary E&S controls are required during construction of the grass channel area to divert stormwater away from the grass channel area until it is completed and permanently stabilized. These may include diversions, temporary stormwater conveyance, or other standard methods for temporary diversion of runoff around disturbed areas. Special protection measures such as erosion control fabrics may be needed to protect vulnerable side slopes from erosion during the construction process.

4.1 Soil Compost Amendments - Overview of Practice

Soil compost amendments are used to improve the retention and infiltration characteristics of post-construction or in situ soils through deep tilling and composting. This allows heavily compacted post-construction fill or existing hydrologic soil classification (HSG) B, C, or D soils to be remediated in order to be suitable for receiving runoff from rooftop disconnections, grass channels and vegetated filter strips. Requirements shown herein are modifications to specifications found in Virginia Stormwater Design Specification No. 4, Soil Compost Amendment, Draft, (DCR/DEQ 2013) for specific application to VDOT projects.

Table 4.1 - Stormwater Functions Provided by Soil Compost Amendments [1]
Virginia Stormwater Design Specification No. 4, Soil Compost Amendment, Draft, (DCR/DEQ 2013)

Stormwater Function	HSG Soils A and B		HSG Soils C and D	
	No CA [2]	**With CA**	**No CA**	**With CA**
Annual Runoff Volume Reduction (RR)				
Simple Rooftop Disconnection	**50%**	**NA** [3]	**25%**	**50%**
Filter Strip	**50%**	**NA** [3]	**NA** [4]	**50%**
Grass Channel	**20%**	**NA** [3]	**10%**	**30%**
Total Phosphorus (TP) EMC Reduction[4] **by BMP Treatment Practice**	**0**		**0**	
Total Phosphorus (TP) Mass Load Removal	**Same as for RR (above)**		**Same as for RR (above)**	
Total Nitrogen (TN) EMC Reduction by BMP Treatment Practice	**0**		**0**	
Total Nitrogen (TN) Mass Load Removal	**Same as for RR (above)**		**Same as for RR (above)**	
Channel Protection & Flood Mitigation	**Partial.** Designers can use the RRM spreadsheet to adjust the curve number for each design storm for the contributing drainage area, based on annual runoff volume reduction achieved.			

[1] CWP and CSN (2008), CWP (2007)
[2] CA = Compost Amended Soils,
[3] Compost amendments are generally not applicable for A and B soils, although it may be advisable to incorporate them on mass-graded B soils to maintain runoff reduction rates.
[4] Filter strips in HSG C and D should use composted amended soils to enhance runoff reduction capabilities. See DEQ Stormwater Design Specification No. 2: Sheet Flow to Vegetated Filter Strip or Conserved Open Space.

4.2 Feasibility and Constraints

Compost amendments are suitable for compacted soils that been placed during construction, and in situ soils belonging the HSG C or D. Constraints on use of amendments are further defined in following sections.

4.2.1 Maximum Contributing Drainage Area (CDA) and Contributing Flow Path

The maximum impervious area draining to an area of compost amended soils should typically be less than the area of the amendment bed. In areas where this cannot be achieved, VDOT Hydraulics must be consulted to determine if amendments can be used to achieve the normal runoff reduction credit.

4.2.2 Site Slopes

Slopes approaching amendment areas shall be kept to a minimum in order to maintain sheet flow. Maximum slopes can vary based on the practice utilizing the amendments (i.e. rooftop disconnection, sheet flow, or grass channels). See those specifications for further guidance regarding maximum slopes. In addition, amendments should not be used upslope of existing buildings.

4.2.3 Depth to Bedrock and Water Table

Amendments may be used if depth to bedrock and/or water table exceeds a minimum of 1.5’ from final grade. Areas that are seasonally inundated within 1.5’ of the soil surface should not be used for amendment beds.

4.2.4 Utilities

Amendment areas may be placed above existing or proposed utilities. A minimum of 1.5’ clearance to top of utility line should be provided. However, keep in mind that if the utility needs to do its own maintenance at some point in time, the excavation may disrupt the benefit of the compost amendments, especially if the excavated amended soil is not use as backfill or if the surface is subsequently compacted. Therefore, it is probably wise to avoid amending soils above utility lines if at all possible.

4.2.5 Proximity to Tree Line

Amendments should not be placed below drip lines of existing trees that will remain due to likelihood of damage to root system during tilling operations.

4.3 General Design Guidelines

The following presents a collection of design guidelines to be followed when using amendments on VDOT projects. Specific material specifications and guidelines can be found in VDOT Special Provision for Soil Compost Amendments (2014). Installation shall also be in compliance with standard detail SWM-4 Soil Compost Amendment (2014).

4.3.1 Soil Testing

Test shall be performed prior to implementation of the amendment plan to determine existing soil properties in the amendment area. Results of this testing may indicate a larger or smaller amendment area than that indicated by USDA Soil Survey mapping. Tests should be performed to a depth of no less than 1' to report bulk density, pH, salts, and soil nutrients. Testing shall be performed at a minimum spacing of one test every 5,000 ft^2 of proposed bed area.

Post-construction testing will be performed at least one week after amendment placement and incorporation to determine if any additional adjustments must be made to meet the soil requirements as specified in the VDOT Special Provision for Soil Compost Amendments (2014).

4.3.2 Volume Reduction

Volume reductions for each Hydrologic Soil Group is outlined in Virginia Stormwater Design Specification No. 4, Soil Compost Amendment, Draft, (DCR/DEQ 2013). **Table 4.2**, as presented in that specification is reproduced below, for convenience. **Table 4.2** may be used to calculate reductions in the total treatment volume for areas within the right of way beyond shoulder areas treated with compost amendments.

Table 4.9 - Runoff Coefficients for Use for Different Pervious Areas

Virginia Stormwater Design Specification No. 4, Soil Compost Amendment, Draft, (DCR/DEQ 2013)

Hydrologic Soil Group	Undisturbed Soils [1]	Disturbed Soils [2]	Restored and Reforested [3]
A	0.02	0.15	0.02
B	0.03	0.20	0.03
C	0.04	0.22	0.04
D	0.05	0.25	0.05

Notes:

[1] Portions of a new development site, outside the limits of disturbance, which are not graded and do not receive construction traffic.

[2] Previously developed sites, and any site area inside the limits of disturbance as shown on the E&S Control plan.

[3] Areas with restored soils that are also reforested to achieve a minimum 75% forest canopy

4.3.3 Depth of Compost Incorporation

The depth of compost incorporation is shown in the VDOT Special Provision for Soil Compost Amendments (2014). **Table 4.3** is a reproduction of Table 1, as seen in the referenced VDOT publication.

Table 4.3 - Compost Incorporation Depths for Various Impervious Cover Ratios
VDOT Special Provision for Soil Compost Amendments (2014)

	Contributing Impervious Cover to Soil Amendment Area Ratio [1]			
	IC/SA = 0 [2]	**IC/SA = 0.5**	**IC/SA = 0.75**	**IC/SA = 1.0** [3]
Compost (in) [4]	2 to 4 [5]	3 to 6 [5]	4 to 8 [5]	6 to 10 [5]
Incorporation Depth (in)	6 to 10 [5]	8 to 12 [5]	15 to 18 [5]	18 to 24 [5]
Incorporation Method	Rototiller	Tiller	Subsoiler	Subsoiler

Notes:
[1] IC = contrib. impervious cover (ft^2) and SA = surface area of compost amendment (ft^2)
[2] For amendment of compacted lawns that do not receive off-site runoff
[3] In general, IC/SA ratios greater than 1 should be avoided, unless applied to a simple rooftop disconnection
[4] Average depth of compost added
[5] Lower end for B soils, higher end for C/D soils

An estimation of the total amount of compost required based on equations from TCC (1997) is shown below:

$$C = A \times D \times 0.0031 \qquad \textbf{(4.1)}$$

where:

C = required compost (cubic yards)
A = surface area of soil amendment (ft^2)
D = depth of compost amendment [determined from Table 4.3] (inches)

4.3.4 Compost Specifications and Installation

Compost specifications and installation procedures shall be in compliance with requirements listed in the VDOT Special Provision for Soil Compost Amendments (2014).

4.4 Design Example

Due to the nature of compost soil amendments, no detailed or lengthy design process is required. The designer is simply required to calculate the ratio between the impervious cover and the surface area of the amendment in order to complete the design using **Table 4.3**. An estimate of the required volume of compost may then be calculated using **Equation 4.1**.

The design example found in Section 2.4 (Sheet Flow to Vegetated Filter Strips) requires that HSG soils are compost-amended for compliance. Based on information given in Section 2.4, the total impervious area draining to the bed is 0.32 acres, made up of a widened lane, and the existing pavement section to the crown. The area of the bed itself is a minimum of 35' wide by 1,000' in length, encompassing an area of 0.80 acres. Therefore, the IC/SA ratio used in **Table 4.3** is computed as:

$$\frac{IC}{SA} = \frac{0.32\ acres}{0.80\ acres} = 0.40$$

Using **Table 4.3**, the IC/SA ratio can be compared to given table values and linearly interpolated to determine the incorporation depth. **Table 4.3** indicates that for an IC/SA of 0, the incorporation depth for HSG D soils is 10", while an IC/SA of 0.50 yields an incorporation depth of 12". Interpolation allows computation of actual required incorporation depth:

$$\frac{12\ in - 10\ in}{0.50 - 0.0} \times 0.4 + 10\ in = \mathbf{11.6\ inches}$$

Once the depth of the amendment has been computed, the estimated volume of compost in cubic yards is computed using **Equation 4.1** as:

$$C = (1{,}000\ ft \ \times 35\ ft \times 11.6\ inches \times 0.0031\ \frac{CY}{ft^2 \cdot in} = \mathbf{1{,}259\ CY}$$

Therefore, 1,259 cubic yards of compost will be required to be tilled into the amendment area to an average depth of 11.6".

5.1 Permeable Pavement - Overview of Practice

Permeable pavement are surfaces that allow for rapid filtration of rainfall through voids in pavement surfaces to a subsurface stone storage layer for discharge or infiltration. The result is a decrease in the effective impervious area of the site. The reservoir layer is designed to provide adequate structural support as well as sufficient storage for the design treatment volume. Permeable pavement should be designed to treat runoff that falls directly on the pavement and adjacent impermeable surfaces; however, treatment of adjacent pervious areas should be limited to the extent possible. Requirements shown herein are modifications to specifications found in Virginia Stormwater Design Specification No. 7, Permeable Pavement (DCR/DEQ, 2013), for specific application to VDOT projects. **Note that although limited Level 2 criteria is shown in this specification for consistency with DEQ specifications, currently VDOT does not allow the use of Level 2 designs for permeable pavement.**

Permeable pavement can be an important part of the stormwater quality treatment compliance for a site, but it requires special design considerations to minimize long-term maintenance. Otherwise, the pavement can become a maintenance burden, particularly if sediment is allowed to accumulate on the surface and fill the pore spaces, negating the pavement's runoff reduction and water quality benefits. Proper design (followed by proper construction) can eliminate (or at least minimize) such problems.

Table 5.1 - Summary of Stormwater Functions Provided by Permeable Pavement
Virginia Stormwater Design Specification No. 7, Permeable Pavement (DCR/DEQ, 2013)

<table>
<tr><th>Stormwater Function</th><th>Level 1 Design</th><th>Level 2 Design</th></tr>
<tr><td>Annual Runoff Volume Reduction (RR)</td><td>45%</td><td>75%</td></tr>
<tr><td>Total Phosphorus (TP) EMC Reduction[1] by BMP Treatment Process</td><td>25%</td><td>25%</td></tr>
<tr><td>Total Phosphorus (TP) Mass Load Removal</td><td>59%</td><td>81%</td></tr>
<tr><td>Total Nitrogen (TN) EMC Reduction[1]</td><td>25%</td><td>25%</td></tr>
<tr><td>Total Nitrogen (TN) Mass Load Removal</td><td>59%</td><td>81%</td></tr>
<tr><td>Channel Protection</td><td colspan="2">● Use VRRM Compliance spreadsheet to calculate a Curve Number (CN) adjustment[2]; OR
● Design extra storage in the stone underdrain layer and peak rate control structure (optional, as needed) to accommodate detention of larger storm volumes.</td></tr>
<tr><td>Flood Mitigation</td><td colspan="2">Partial. May be able to design additional storage into the reservoir layer by adding perforated storage pipe or chambers.</td></tr>
<tr><td colspan="3">[1] Change in event mean concentration (EMC) through the practice. Actual nutrient mass load removed is the product of the removal rate and the runoff reduction rate (see Table 1 in the Introduction to the New Virginia Stormwater Design Specifications).
[2] NRCS TR-55 Runoff Equations 2-1 thru 2-5 and Figure 2-1 can be used to compute a curve number adjustment for larger storm events based on the retention storage provided by the practice(s).</td></tr>
</table>

Sources: CWP and CSN (2008) and CWP (2007)

5.1.1 Typical Configurations

Permeable pavement applications used for VDOT projects are limited in nature due to restrictions in recommended use for high speed and high volume traffic areas in extreme weather conditions. Permeable pavement typically are used only for parking applications. Prior to use of permeable pavement in a road applications, VDOT shall be consulted to confirm acceptance of use.

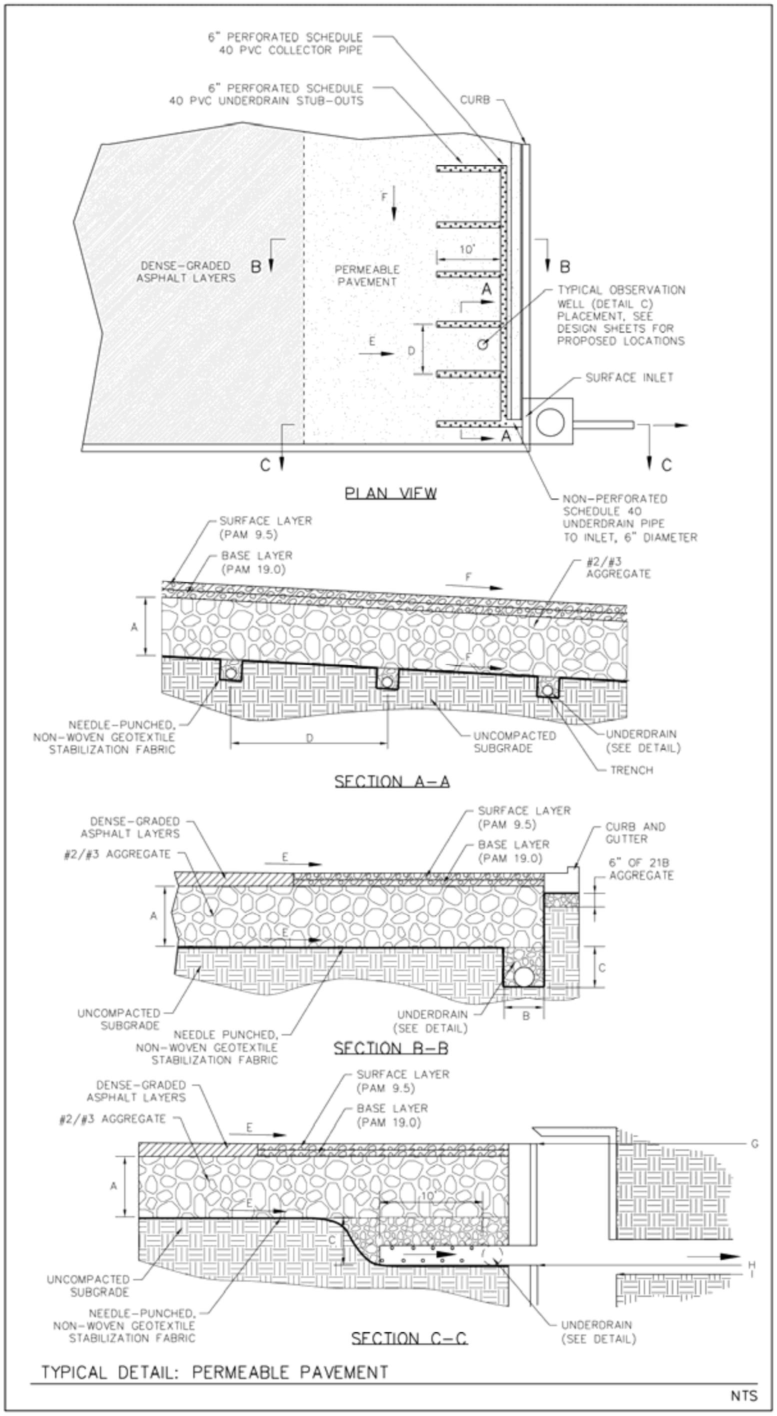


Figure 5.1 - Typical Permeable Pavement Detail (Parking Lots) (VDOT SWM-5, Filtering Practices, 2014)

5.2 Site Constraints and Siting of the Facility

When a permeable pavement system is proposed, the designer must consider a number of site constraints to ensure that the practice is applicable to the suggested use.

5.2.1 Site Soils

Site soils do not typically restrict the use of permeable pavement; however, based on the hydrologic soil group, an underdrain may be required. If permeable pavement is placed on compacted fill material, an underdrain must be present. Designs that propose full infiltration of the captured storage volume must be approved by VDOT prior to installation and have field-verified infiltration rates exceeding 0.5 in/hr. Native soils must have a silt/clay content of less than 40% and a clay content of less than 20%. Testing for infiltration shall be in accordance with standards outlined in the VDOT Special Provision for Stormwater Miscellaneous (2014). Level 1 designs using an underdrain to provide and outlet for the reservoir layer do not require infiltration testing. In addition, permeable pavement should never be situated above fill soils unless designed with an impermeable liner and underdrain, and should not be installed over underlying soils with a high shrink/swell potential.

5.2.2 Contributing Drainage Area (CDA)

Permeable pavement is not intended to treat sites with high sediment or trash/debris loads, since such loads may cause the practice to clog and fail. External drainage areas (areas draining to the surface of permeable pavement, excluding the permeable pavement area) are allowed only for applications using underdrains. When used, the external drainage area shall not exceed a loading ratio of 2.5:1 and should be nearly 100% impervious. Any design with an external drainage area contributing “run-on” to the permeable pavement section should include requirements for more frequent operation and maintenance inspections. It is important to design permeable pavement within the limits established for CDAs. Too much or too little runoff can result in performance issues and the need for subsequent repairs or reconstruction.

5.2.3 Pavement Slope

Generally, permeable pavement surface slopes should be less than 5%, and preferably less than 2%. Designers should consider using a terraced design of the sub-base for permeable pavement above sloped areas. The bottom bed slope under the storage layer shall be relatively flat, with longitudinal grades generally ranging between 0% and 1% for installations using infiltration or underdrains/overdrains, respectively. Laterally, the grade shall be even (0%) across the entire installation.

5.2.4 Hydraulic Head

Typically hydraulic head requirements are nominal, although they should be evaluated on a case by case basis. Level 1 installations should have underdrains installed at slopes greater than 0.5% in order to reduce the amount of hydraulic head necessary to drive stored runoff from the system.

5.2.5 Depth to Water Table

A minimum separation of 2’ is required between the base of the storage layer and the seasonal high groundwater table.

5.2.6 Setbacks

Although setbacks to structures are not applicable on many VDOT installations, projects at district or area headquarters, rest areas or park-and-ride facilities may propose permeable pavement in the vicinity of existing or proposed structures. Setbacks are dependent upon the surface area of the permeable installation. Requirements are as follows:

- 250-1,000 ft^2 : 5’ down-gradient, 25’ up-gradient
- 1,000-10,000 ft^2: 10’ down gradient, 50’ up-gradient
- >10,000 ft^2: 25’ down gradient, 100’ up-gradient

In cases where setbacks listed above cannot be met, those setbacks can be reduced if an impermeable liner is used to encase the installation, and with express permission from VDOT.

Due to the potential for contamination, a minimum setback of 100’ from all water supply wells shall be enforced. In areas having a higher risk for ground water contamination, ground water mapping should be used to determine interconnectivity of groundwater systems to wells on surrounding properties.

5.2.7 Existing and Proposed Utilities

Although it is feasible to construct permeable pavement systems near and over existing or proposed utilities, permission must be provided by VDOT during the design process. Typically, a minimum vertical separation of 1’ will be required below the stone layer and the top of the utility. A layer of impermeable clay, or an impermeable liner may be required to prevent migration of stored runoff from the pavement storage to the utility bedding. If ground water contamination is a concern, additional preventative measures may be required to prevent flow from exiting the system in utility bedding.

However, considering that maintenance of the utility lines will require excavation through the permeable pavement, and that it is unlikely that the utility contractor will backfill properly and replace the permeable pavement (due to the limited size of the backfill area, it is highly recommended that areas over utility lines be avoided for permeable pavement installations. Alternatively, VDOT should carefully monitor utility repairs under permeable pavement installations for appropriate quality control in replacing the pavement materials.

5.3 General Design Guidelines

Permeable pavement must be designed to support vehicular loads expected during design life. Structural design will be based on four primary criteria:

- Total traffic volume and load
- In-situ soil strength
- Environmental elements
- Surface materials, bedding and reservoir layer design

Typical structural designs for surface layers will include Porous Asphalt Mix (PAM) 9.5 and 19.0 components in thicknesses as specified through the design guidance set forth in the VDOT Special Provision for Permeable Pavement (2014). For parking applications, typical surface application will be 1.5” PAM-9.5 with underlying 3” PAM-19.0.

5.3.1 Sizing of Reservoir Layer

The hydraulic design to determine the depth of the stone layer used as storage in the system is reproduced below in **Equation 5.1** as found in the Virginia Stormwater Design Specification No. 7, Permeable Pavement (DCR/DEQ, 2013).

$$d_{stone} = \frac{\left(P \times A \times R_{v_{composite}}\right) + (P \times A_P)}{\eta_r \times A_P} \qquad \textbf{(5.1)}$$

where:

d_{stone} = Depth of stone reservoir layer (feet)
P = Rainfall depth (feet); Level 1 = 0.08’
A = Contributing drainage area (ft^2)
$R_{v_{composite}}$ = Composite runoff coefficient for contributing area
A_P = Area of permeable pavement (ft^2)
η_r = Porosity of stone layer (0.4)

Several assumptions are related to **Equation 5.1**, including:

- Drainage area, A, is limited to a ratio of 2.5:1 (external area to permeable pavement area) if an underdrain is used. If an underdrain is not used, then the drainage area must equal the area of permeable pavement.
- The stone reservoir footprint is equivalent to the area of permeable pavement, A_P.

Table 5.2 - Permeable Pavement Design Criteria

Level 1 Design
Tv = (1)(Rv)(A) / 12 – the volume reduced by an upstream BMP [1]
Soil infiltration is less than 0.5 in./hr.
Underdrain required
CDA[1] = The permeable pavement area plus upgradient parking, as long as the ratio of external contributing area to permeable pavement does not exceed 2.5:1.
[1] The contributing drainage area to the permeable pavements should be limited to paved surfaces in order to avoid sediment wash-on. When pervious areas are conveyed to permeable pavement, sediment source controls and/or pre-treatment must be provided; a gravel filter strip or sump should be used. The pre-treatment may qualify for a runoff reduction credit if designed accordingly.

The computed treatment volume is defined in **Section 1, Equation 1.1**. The composite runoff coefficient, $R_{v_{composite}}$ is computed through use of **Section 1, Equation 1.2**, and coefficients from **Section 1, Table 1.1**.

Permeable pavement can also be designed to address, in whole or in part, the detention storage needed to comply with channel protection and/or flood control requirements. The designer can model various approaches by factoring in storage within the stone aggregate layer, expected infiltration, and any outlet structures used as part of the pavement design. Routing calculations can also be used to provide a more accurate solution of the peak discharge and required storage volume. Oversizing the reservoir layer in this manner can also decrease the maintenance frequency of the BMP and, thus, its life-cycle cost.

The permeability of the pavement surface and that of the gravel media is very high. However, the permeable pavement reservoir layer will drain increasingly slower as the storage volume decreases (i.e., the hydraulic head decreases). To account for this change, a conservative stage discharge should be established for routing the stone reservoir. The underdrains can serve as a hydraulic control for limiting flows, or an external control structure can be used at the outlet of the system.

Keep in mind that designing the pavement to accomplish these additional purposes means that the designer should provide requirements for VDOT to maintain the pavement surface carefully to prevent clogging or other functional failure, which would then place the channel protection or flood protection aspects of the installation at risk as well.

5.3.2 Overdrains (High Flow Bypass)

An overdrain should be integrated in the design to prevent runoff from backing up onto the pavement surface. In VDOT installations, it is recommended that a DI-3 series inlet be installed along perimeter curb and gutter to function as an overdrain system (see VDOT BMP Detail SWM-5, Permeable Pavement). On pavement designs with a long grade, the designer should use a stepped design with an Overdrain in each cell in order to establish level reservoir storage areas and prevent flow from exiting the pavement through the surface at the low end.

5.3.3 Pretreatment

Pretreatment is typically not required for permeable pavement systems. However, pretreatment may be required if the pavement receives runoff from adjacent pervious areas. For example, a gravel filter strip can be placed along the receiving edge of the permeable pavement section to trap sediment particles before they reach the permeable pavement surface.

5.3.4 Reservoir Layer

The reservoir layer shall be in accordance with the standards set forth in Section II.(e) of the VDOT Special Provision for Permeable Pavement (2014). In general, the layer shall consist of VDOT #2 or #3 stone having a minimum thickness of 12”. When installed in karst regions, the minimum thickness shall be increased to 24”. The maximum thickness of the reservoir layer shall not exceed 36”.

5.3.5 Underdrain

Underdrains shall be installed in an underdrain trench, with typical dimensions of 12” by 12” (see detail on VDOT SWM-5, Permeable Pavement (2014)). The underdrain shall be 6” Schedule 40 PVC, with a minimum slope of 0.5%. Installation details are found in VDOT SWM-5, Permeable Pavement (2014), and specifications regarding installation are found in the VDOT Special Provision for Permeable Pavement (2014).

5.3.6 Maintenance Reduction Features

Maintenance is a crucial element to ensure the long-term performance of permeable pavement. The most frequently cited maintenance problem is surface clogging caused by organic matter and sediment, which can be reduced by the following measures:

- ***Subgrade Design and Construction is Very Important to the Long-Term Integrity of Permeable Pavement***. This can help prevent untimely deterioration of the pavement surface, thus extending the life-span and reducing life-cycle costs.

- ***Address Nearby Drainage Problems and Problems with Existing Pavement Conditions***. If the permeable pavement is being installed in a larger parking area as additional parking space or as a retrofit to replace a conventional paving surface, ensure that any existing drainage issues that may affect the permeable pavement are resolved. As well, worn pavement in areas that may drain toward the permable pavement can contribute pavement particles and other solid matter that could clog the pore space in the permeable pavement. Therefore, it is important to repair any such conditions prior to completion of the permeable pavement installation.

- ***Periodic Vacuum Sweeping.*** The pavement surface is the first line of defense in trapping and eliminating sediment that may otherwise enter the stone base and soil subgrade. The rate of sediment deposition should be monitored and vacuum sweeping done once or twice a year. This frequency should be adjusted according to the intensity of use and deposition rate on the permeable pavement surface. At least one sweeping pass should occur at the end of winter.

- ***Protecting the Bottom of the Reservoir Layer.*** There are two options to protect the bottom of the reservoir layer from intrusion by underlying soils. The first method involves covering the bottom with a barrier of choker stone and sand. In this case, underlying native soils should be separated from the reservoir base/subgrade layer by a thin 2” to 4” layer of clean, washed, choker stone (ASTM D 448 No. 8 stone) covered by a layer of 6” to 8” of course sand.

  The second method is to place a layer of filter fabric on the native soils at the bottom of the reservoir. Some practitioners recommend avoiding the use of filter fabric, since it may become a future plane of clogging within the system; however, designers should evaluate the paving application and refer to AASHTO M288-06 for an appropriate fabric specification. AASHTO M288-06 covers six geotextile applications: Subsurface Drainage, Separation, Stabilization, Permanent Erosion Control, Sediment Control and Paving Fabrics. However, AASHTO M288-06 is not a design guideline. It is the engineer's responsibility to choose a geotextile for the application that takes into consideration site-specific soil and water conditions. Fabrics for use under permeable pavement should, at a minimum, meet criterion for Survivability Classes (1) and (2). Permeable filter fabric is still recommended to protect the excavated sides of the reservoir layer, in order to prevent soil piping.

- ***Observation Well.*** An observation well shall be placed in all permeable pavement installations. The well shall be installed to conform with Detail C of VDOT SWM-5, Permeable Pavement (2014).

5.3.7 Karst Considerations

Level 1 designs can be used when an impermeable liner is placed below the reservoir layer and an underdrain is used. A detailed geotechnical investigation will be required prior to consideration of any installation in a karst area.

5.3.8 High Water Table

Permeable pavement should not be used in areas where the seasonally high water table is less than 2' from the bottom of the reservoir layer. If an underdrain is used beneath the pavement in such a setting, a minimum 0.5% slope must be maintained to ensure proper drainage.

5.3.9 Cold Weather Performance

Freeze-thaw action may affect the long term viability of permeable pavement installations. Therefore, the following considerations should be made during the design:

- Eliminate surface ponding using an overflow structure (typically a DI-3 series inlet)
- Extend the reservoir layer and underdrain to below the frost line when possible
- Do not store pushed snow on permeable pavement
- Sand should not be used for winter traction in the vicinity of permeable pavement installation.

5.3.10 Construction and Inspection

Construction and inspection shall be in conformance with the VDOT Special Provision for Permeable Pavement (2014). The designer should direct that the construction process should be carefully monitored (via regular inspections). Improper construction is the main cause of permeable pavement failure, resulting in costly repair/replacement.

5.4 Design Example

This section presents the design process applicable to permeable pavement serving as a water quality BMP. The pre- and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 11 of the *Virginia Stormwater Management Handbook, 2nd Edition, Draft (DCR/DEQ, 2013),* for details on hydrologic methodology.

Installation of a combination building is proposed for an existing VDOT Area Headquarters (AHQ) in Grayson County, near Galax, Virginia. The hydrologic classification of on-site soils over the entire site is HSG B. The site is approximately 4.9 acres, with the land cover parameters as listed in **Table 5.3**. The time of concentration to the porous pavement area is less than 5 minutes, and therefore will be set as 5 minutes for analysis. The project site does not exhibit a high or seasonally high groundwater table or indicate the presence of bedrock, based on geotechnical tests performed on site. Although a portion of the site will be treated using a bioretention basin (computations for bioretention basin not shown in this example), preliminary indications are that removal rates will not meet required levels; therefore, a secondary treatment BMP will be necessary. The contributing drainage area to the permeable pavement system is a small double loaded (spaces on both sides of a travelway) parking lot. The parking lot has thirty-two 9' x 18' spaces, and a 24' wide drive aisle. The total paved area draining to the stone reservoir is 8,712 ft^2, while the surface area of the 16 parking spaces to be constructed using permeable pavement is 2,592 ft^2. The expected pavement cross-slope toward overflow inlet is 2.08%.

Table 5.3 - Hydrologic Characteristics of Total Example Project Site*

		Impervious	Turf	Forest
Pre	Soil Classification	HSG B	HSG B	HSG B
	Area (acres)	2.35	2.55	0.0
Post	Soil Classification	HSG B	HSG B	HSG B
	Area (acres)	2.25	2.65	0.00

*Note: Only the portions of the site identified in the above description (0.20 ac) drain to the permeable pavement system. Areas shown above are for entire disturbed area of the site.

Step 1 - Enter Data into VRRM Spreadsheet

The required site data from **Table 5.3** is input into the **VRRM Spreadsheet for Redevelopment (2014)**, resulting in site data summary information shown in **Table 5.4**.

Table 5.4 - Summary of Output from VRRM Site Data Tab

Site Rv	**0.54**
Post-development Treatment Volume (ft^3)	**9683**
Post-development TP Load (lb/yr)	**6.08**
Total TP Load Reduction Required (lb/yr)	**1.08**

Prior to proceeding, the designer should make certain through VRRM calculations that the required water quality load reduction is met by using the proposed permeable pavement for treatment.

Based on site data described above, the total phosphorus reduction required for the entire site is 1.08 lbs/yr (**Table 5.4**). The estimated removal in the bioretention component (not shown) is 0.91 lbs/yr. In order to provide the remaining treatment, a Level 1 permeable pavement system is proposed. The 0.20 acres of impervious area for the Level 1 permeable pavement area is entered into the Runoff Reduction Spreadsheet Drainage Area tab. The estimated phosphorus removal reported by the spreadsheet for the permeable pavement treatment area is 0.25 lbs/year (**Table 5.5**).

Table 5.5 - Summary of Output from VRRM Summary Tab for Permeable Pavement Treatment Area

Total Impervious Cover Treated (acres)	0.20
Total Turf Area Treated (acres)	0.00
Total TP Load Reduction Achieved in D.A.A (lb/yr)	0.25

Thus, combined with the phosphorus removal from the bioretention component (0.91 lbs/year), the permeable pavement system will be sufficient to meet water quality requirements (1.08 lbs/year) for the project, resulting in a total phosphorus removal of 1.16 lbs/year.

Step 2 - Compute the Required Treatment Volume

The treatment volume can be calculated using **Section 1, Equation 1** or taken directly from the VRRM Spreadsheet Drainage Area tabs. For this example, the reported treatment volume on the drainage area tab (treating the 0.20 acre area) is 690 ft^3.

Step 3 - Enter Data in Channel and Flood Protection Tab

Hydrologic computations for required design storms for flood and erosion compliance are not shown as part of this example. The user is directed to the VDOT Drainage Manual for appropriate levels of protection and design requirements related to erosion and flood protection. However, hydrologic computations are necessary to compute overflow conveyance structures.

Values for the 1-, 2-, and 10-year 24-hour rainfall depth should be determined from the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 and entered into the “Channel and Flood Protection” tab of the spreadsheet. For this site (Lat 36.6289, Long -80.9873), those values are shown in **Table 5.6**.

Table 5.6 - Rainfall Totals from NOAA Atlas 14

	1-year storm	2-year storm	10-year storm
Rainfall (inches)	2.49	3.01	4.39

Curve numbers used for computations should be those calculated as part of the runoff reduction spreadsheet (Virginia Runoff Reduction Spreadsheet for Redevelopment, 2014). For runoff draining to the permeable pavement, results from the runoff reduction spreadsheet are shown in **Table 5.7**, and result in adjusted curve numbers of 94, 94 and 94 for the 1, 2 and 10-year storms, respectively. Note that although areas draining to the bioretention facility would also result in volume reduction and adjusted curve numbers, that the bioretention portion should be entered as a separate drainage area in the RRM spreadsheet in order to properly segregate the design parameters in order to design the storage system and overflow for the permeable pavement system.

Table 5.7 - Adjusted CN from Runoff Reduction Channel and Flood Protection

	1-year Storm	2-year Storm	10-year Storm
$RV_{Developed}$ (in) with no Runoff Reduction	2.26	2.78	4.15
$RV_{Developed}$ (in) with Runoff Reduction	1.83	2.35	3.73
Adjusted CN	94	94	94

Input data is used in the Natural Resource Conservation Service Technical Release 55 (NRCS TR-55) Tabular method to calculate discharge hydrographs. (**Note that other hydrologic methodologies are suitable-see VDOT Drainage Manual, Hydrology for guidance**). Peaks of those hydrographs for the 1, 2, and 10-year storms are reported in **Table 5.8**. These values will be used to size the conveyance downstream of the filtering practice.

Table 5.8 – Post-development Discharge Peaks Exiting BMP

	1-year Storm	2-year Storm	10-year Storm
Discharge (cfs)	0.59	0.74	1.17

Step 4 - Compute Minimum Reservoir Depth

Based on the input parameters, a Level 1 design, and using **Equation 5.1**, the required depth of the reservoir layer is calculated as:

$$d_{stone} = \frac{(0.08\ feet \times 8{,}712\ ft^2 \times 0.95) + (0.08\ feet \times 2{,}592\ ft^2)}{0.40 \times 2{,}592\ ft^2} = 0.84\ feet$$

Step 5 - Specify Underdrains

The depth of the system's underdrain trench should be installed along the end of the storage reservoir, parallel to the gutter pan (see **Figure 5.1**). Dimensions of stone trench shall be 12" x 12" x 144' (width of 16 parking spaces). As specified in section 5.3.5, the pipe shall be perforated and constructed using 6" schedule 40 PVC at the minimum slope of 0.5%. Perforated underdrain stubouts shall extend out into the permeable pavement section a distance of 10' perpendicular to the underdrain main line (see detail, **Figure 5.1**). Spacing between stubouts shall be maintained at 20' on center. Computations should be completed to verify that the underdrain system draws down the reservoir within a 48-hour period.

Step 6 - Design Overflow Structure

The overflow structure for this application will be a single DI-3A sump inlet at the lower end of the parking lot. Capacity for this overflow structure should be verified for the 10-year storm to determine adequacy. As seen in **Table 5.8**, the overflow peak for the 10-year storm is 1.17 cfs.

The interception capacity of the DI-3A curb inlet operating as a weir can be calculated using **Equation 9.10** of the ***VDOT Drainage Manual*** as shown below:

$$Q_t = C_w(L + 1.8W)d^{1.5}$$

where:

Q_t	=	Intercepted flow, cfs
C_w	=	Weir coefficient, use 2.3
L	=	Length of curb opening, ft
W	=	Width of local depression, ft
d	=	Depth of water at curb from a point where the normal pavement cross slope would intercept the curb face, ft

Allowable spread should be at least 1” below the top of curb, or 5” (0.42’).

The depth of allowable ponding = 8(0.0208) = 0.17’, which extends 8’ into the adjacent parking space.

Using a factor of safety of 2, the depth of ponding is less than 1” below the top of curb, or (2x(0.17’) < 0.42’).

If d/h<1.2, where h is the opening of the curb inlet then the inlet is in weir control. With the factor of safety, the depth, d, is 0.34’ (4”) as shown above. From specifications, the opening of the curb inlet is 5”. Therefore, d/h=4/5=0.80. Since 0.8 < 1.2 then operation under weir control is confirmed.

Equation 9.10 from the VDOT Drainage Manual, and the length of the opening of a DI-3A of 2.5’ is used to compute the flow capacity of a DI-3A:

$$Q_t = 2.3\big(2.5 + 1.8(2)\big)0.34^{1.5}$$

$$Q_t = 2.3\big(2.5 + 1.8(2)\big)0.34^{1.5} = 2.78\ cfs$$

Because the theoretical capacity is greater than the design flow, 2.78 cfs > 1.17 cfs, then a DI-3A may be used as the overflow. Otherwise, the design would need to be upsized to use a DI-3C sump curb inlet. The proposed outlet pipe from the DI-3A manhole is a 12” RCP pipe at 1.0% slope. Using Manning’s equation, the pipe full capacity of a 12” reinforced concrete pipe at 1% slope is 3.57 cfs; therefore, the system will be adequate to convey the 10-year overflow.

Step 7 - Specify Pavement Section

The structural design of the surface and intermediate pavement sections are not shown. Based on geotechnical analysis, CBR testing, and the expected pavement loading, these two components have been determined to be 1.5” of PAM 9.5 and 3.0” of PAM 19.0. As computed above, the reservoir layer (stone bedding) will be just over 10”, at 0.84’. See VDOT Special Provision for Permeable Pavement, 2014 for additional specification and design elements related to permeable pavement systems.

6.1 Infiltration - Overview of Practice

Infiltration practices typical employ a surface and subsurface storage volume to temporarily store a design volume of runoff prior to exfiltration into underlying soils. The infiltration process treats the design volume through physical and chemical absorption processes for pollutant removal. On sites with suitable soils, infiltration basins are used to promote groundwater recharge, and they aid the designer in mimicking predevelopment hydrology in the post-development condition. Due to removal of a volume of stormwater from the post-development runoff hydrograph, infiltration practices result in the highest rate of runoff reduction of any of the best management practices. Requirements shown herein are modifications to specifications found in Virginia Stormwater Design Specification No. 8, Infiltration Practices, Draft (DCR/DEQ, 2013), for specific application to VDOT projects. Infiltration can be an important part of the stormwater quality treatment solution for a site, but it requires special design considerations to minimize long-term maintenance. Otherwise, the BMP can become a maintenance burden, particularly if sediment is allowed to accumulate on the surface and clog the pore spaces, negating the BMP’s runoff reduction and water quality benefits. Proper design (followed by proper construction) can eliminate (or at least minimize) such problems.

Table 6.1 - Summary of Stormwater Functions Provided by Infiltration Practices
Virginia Stormwater Design Specification No. 8, Infiltration Practices, Draft (DCR/DEQ, 2013)

Stormwater Function	Level 1 Design	Level 2 Design
Annual Runoff Volume Reduction (RR)	50%	90%
Total Phosphorus (TP) EMC Reduction[1] by BMP Treatment Process	25%	25%
Total Phosphorus (TP) Mass Load Removal	63%	93%
Total Nitrogen (TN) EMC Reduction[1] by BMP Treatment Process	15%	15%
Total Nitrogen (TN) Mass Load Removal	57%	92%
Channel and Flood Protection	• Use the Virginia Runoff Reduction Method (VRRM) Compliance Spreadsheet to calculate the Curve Number (CN) Adjustment OR • Design for extra storage (optional; as needed) on the surface or in the subsurface storage volume to accommodate larger storm volumes, and use NRCS TR-55 Runoff Equations [2] to compute the CN Adjustment.	
[1] Change in the event mean concentration (EMC) through the practice. The actual nutrient mass load removed is the product of the removal rate and the runoff reduction (RR) rate (see Table 1 in the *Introduction to the New Virginia Stormwater Design Specifications*). [2] NRCS TR-55 Runoff Equations 2-1 thru 2-5 and Figure 2-1 can be used to compute a curve number adjustment for larger storm events, based on the retention storage provided by the practice(s).		

Sources: CWP and CSN (2008) and CWP (2007)

6.1.1 Typical Configurations

Due to the nature of the practice, infiltration facilities are applicable to a wide variety of projects, including linear highway projects. Infiltration practices are typically subdivided into three categories: micro-infiltration (250 to 2,500 ft^2), small-scale infiltration (2,500 to 20,000 ft^2), and conventional infiltration (20,000 to 100,000 ft^2). Specific criteria generally associated with each category is found in **Table 6.2**. A typical configuration and various cross-sections typically associated with infiltration facilities are found in **Figures 6.1-6.4**.

Table 6.2 - Characteristics of Three Design Scales of Infiltration Practices

Virginia Stormwater Design Specification No. 8, Infiltration Practices, Draft (DCR/DEQ, 2013)

Design Factor	Micro-Infiltration	Small-Scale Infiltration	Conventional Infiltration
Impervious Area Treated	250 to 2,500 ft^2	2,500 to 20,000 ft^2	20,000-100,000 ft^2
Typical Practices	Dry Well French Drain Paving Blocks	Infiltration Trench Permeable Paving [1]	Infiltration Trench Infiltration Basin
Min. Infiltration Rate	1/2 in/hr field verified		
Design Infil. Rate	50% of measured rate		
Observation Well	No	Yes	Yes
Type of Pretreatment (see **Table 8.6**)	External (leaf screens, grass filter strip, etc)	Vegetated filter strip or grass channel, forebay, etc.	Pretreatment Cell
Depth Dimensions	Max. 3' depth	Max. 5' depth	Max. 6' depth,
UIC Permit Needed	No	No	Only if the surface width is less than the max. depth
Head Required	Nominal: 1 to 3'	Moderate: 1 to 5'	Moderate: 2 to 6'
Underdrain Requirements?	An elevated underdrain only on marginal soils	None required	Back up underdrain
Required Soil Tests	Based on surface area of practice; minimum of one soil profile, one infiltration tests per location [3]	Varies based on surface area of practice [3]	Varies based on surface area of practice [3]
Building Setbacks	10' down-gradient [2]	10' down-gradient 50' up-gradient	25' down-gradient 100' up-gradient

[1] Although permeable pavement is an infiltration practice, a more detailed specification is provided in Section 5, Permeable Pavement.
[2] Note that the building setbacks are intended for simple foundations. The use of a dry well or french drain adjacent to an in-ground basement or finished floor area or any building should be carefully designed and coordinated with the design of the structure's water-proofing system (foundation drains, etc.), or avoided altogether.
[3] Refer to VDOT Special Provision for Stormwater Miscellaneous (2014)

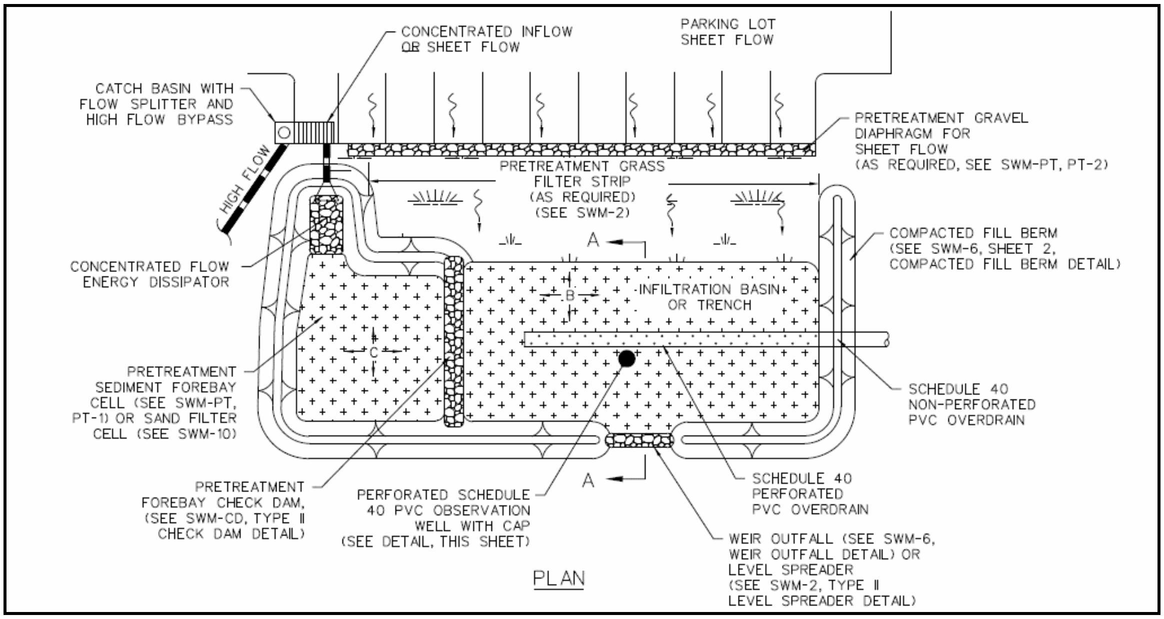


Figure 6.1 - Typical Infiltration Basin
VDOT SWM-6, Infiltration Practices (2014)

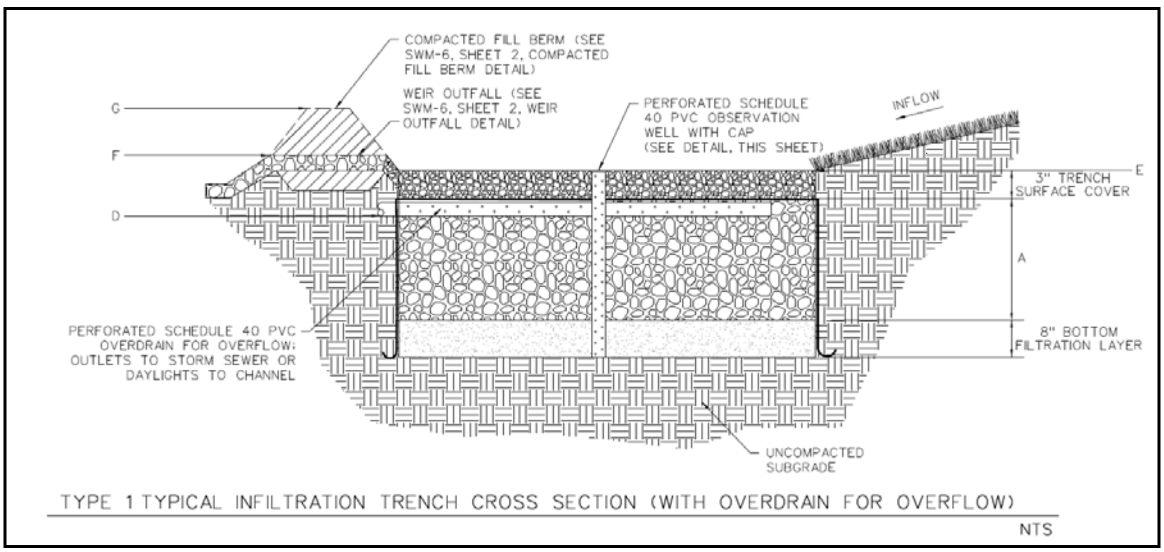


Figure 6.2 - Typical Infiltration Trench Cross Section (with overdrain)
VDOT SWM-6, Infiltration Practices (2014)

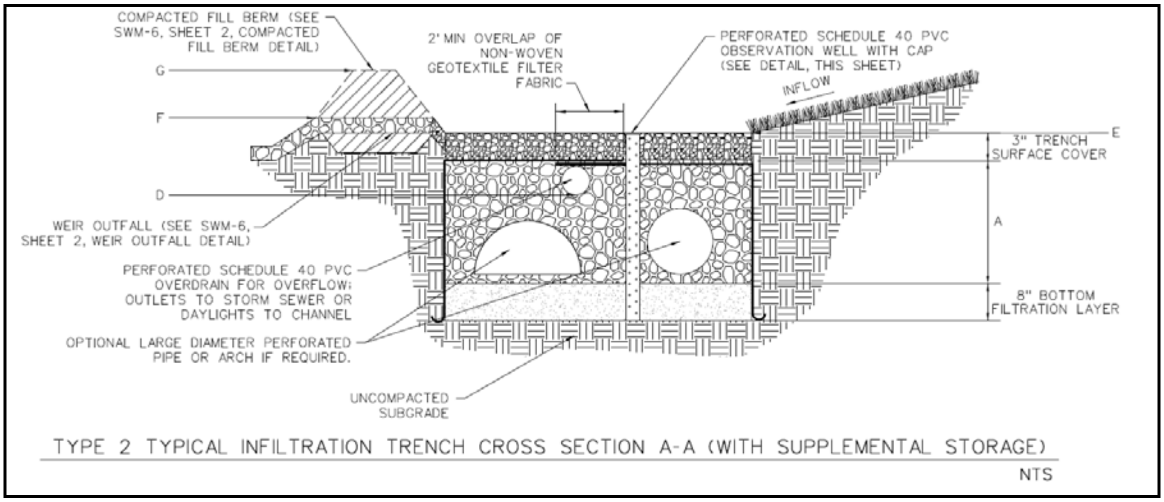


Figure 6.3 - Infiltration Trench Cross Section (with supplemental storage)
VDOT SWM-6, Infiltration Practices (2014)

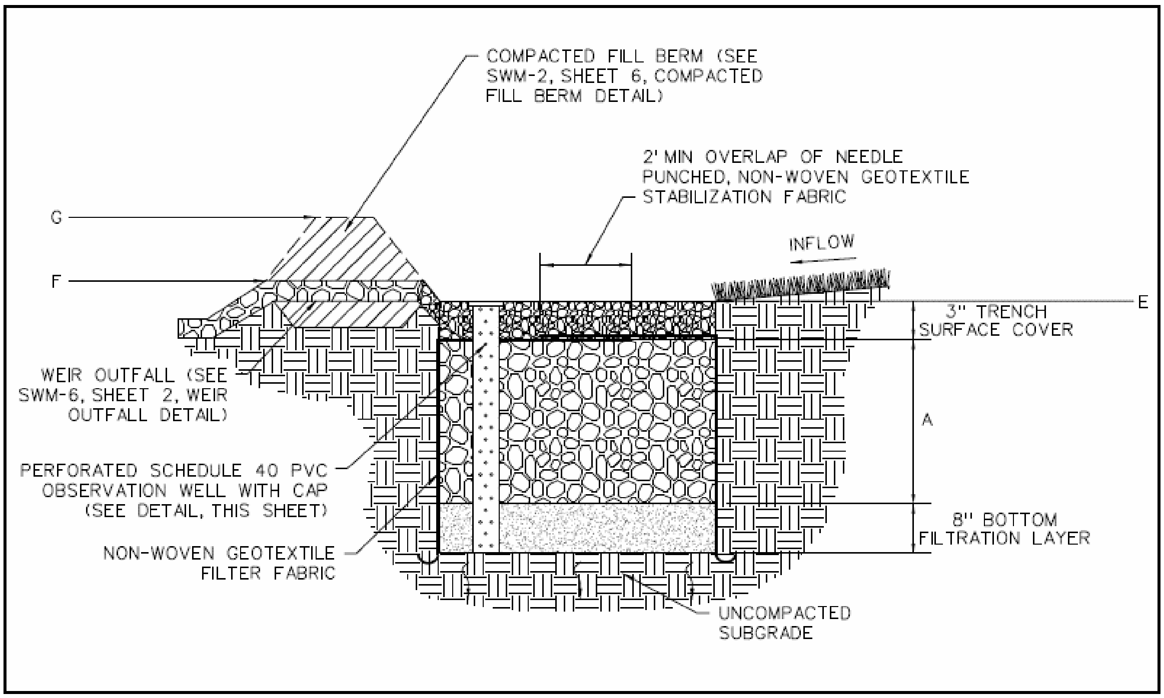


Figure 6.4 - Infiltration Trench Cross Section (without subdrainage)
VDOT SWM-6, Infiltration Practices (2014)

6.2 Site Constraints and Siting of the Facility

Typically infiltration facilities can be considered if the proposed location has a hydrologic soil classification of A or B. Infiltration should generally not be used in areas prone to hotspot runoff due to the possibility of contaminating the ground water table in the vicinity of the site.

6.2.1 Contributing Drainage Area (CDA)

Typically contributing drainage area to an infiltration facility should be highly impervious (approaching 100%), and should not exceed 2.0 acres on any single installation. Various scales related to the impervious area treated are found in **Table 6.2**. It is important to design infiltration facilities within the limits established for CDAs. Too much or too little runoff can result in performance issues and the need for subsequent repairs or reconstruction.

6.2.2 Site Slopes in Vicinity of Practice

Infiltration practices should typically have flat bottoms (0% slope) in order to promote evenly distributed infiltration over the practice area. The average slope of upstream contributing areas shall be 15% or less.

6.2.3 Depth to Bedrock

Separation of at least 2’ is required between bedrock and the invert of the infiltration bed.

6.2.4 Depth to Water Table

Separation of at least 2’ is required between the seasonally high groundwater table and the invert of the infiltration bed.

6.2.5 Hydraulic Head

Minimal hydraulic head is typically required to drive flow through infiltration practices; however, up to 2’ may be necessary for optimal functioning of conventional infiltration practices, to be evaluated on a case by case basis.

6.2.6 Soils

Soils in the infiltration area are required to have infiltration tests in accordance with the VDOT Special Provision for Stormwater Miscellaneous (2014), and performed using the ASTM D 2434 Tube Permeameter Method. A field-tested infiltration rate of 0.5 in/hr or greater is required for use of infiltration practices on VDOT projects.

6.2.7 Setbacks

In order to prevent damage by seepage, infiltration practices should not be hydraulically tied into base stone in pavement cross section or connected to structure foundations. Setbacks from adjacent roads and structures are found in Table 6.2 for each scale of infiltration practice. Setbacks from wells shall be a minimum of 100', and setbacks from septic drainfields shall be a minimum of 50'. Infiltration practices shall be installed a minimum of 5' down gradient of utility lines. When located near down-gradient slopes of 20% or greater, infiltration practices shall be located a minimum distance of 200' from those slopes.

6.2.8 Karst Areas

Conventional infiltration practices shall not be allowed in karst regions. Micro-scale or small-scale infiltration areas (Table 6.2) can be permitted if geotechnical tests indicate a separation of 4' to bedrock, an underdrain and impermeable liner are used, and permission is acquired from VDOT prior to design. In Karst areas, Bioretention is typically a preferred alternative to Infiltration.

6.2.9 Coastal Plain

The flat terrain, low head and high water table of many coastal plain sites can constrain the application of conventional infiltration practices. However, such sites are still suited for micro-scale and small-scale infiltration practices. Designers should maximize the surface area of the infiltration practice, and keep the depth of infiltration to less than 24" plus the necessary separation distance from the groundwater table. Where soils have a very high infiltration rate (more than 4.0" in/hr), shallow bioretention is a preferred alternative. Where soils are more impermeable (i.e., marine clays with an infiltration rate of less than 0.5 in/hr a constructed wetland practice may be more appropriate.

6.2.10 Cold Climate and Winter Performance

Infiltration practices can be designed to withstand moderate winter conditions. The main problem is caused by ice forming in the voids or the subsoils below the practice, which may briefly result in nuisance flooding when spring melting occurs. The following design adjustments are recommended for infiltration practices installed in colder parts of the state (higher elevations, etc.):

- The bottom of the practice should extend below the frost line.
- Infiltration practices are not recommended at roadside locations that are heavily sanded and/or salted in the winter months (to prevent movement of chlorides into groundwater and prevent clogging by road sand).
- Pre-treatment measures can be oversized to account for the additional sediment load caused by road sanding (up to 40% of the T_v).

Infiltration practices must be set back at least 25' from roadways to prevent potential frost heaving of the road pavement.

6.2.11 Groundwater Hotspots

Stormwater hotspots are designated as areas with a higher potential for high concentrations of stormwater pollutants, particularly toxic pollutants. Virginia Stormwater Design Specification No. 8, Infiltration Practices, Draft (DCR/DEQ, 2013) contains information regarding specific types of hotspots. For purposes of VDOT projects these include wash pads, maintenance facilities, and fueling areas of VDOT area headquarters, parking lots or park and ride lots containing 40 spaces or more, and roads with 2,500 or higher average daily trips (ADT). Contributing drainage areas that contain maintenance facilities, fueling stations, wash facilities, or VDOT fleet storage facilities shall not use stormwater infiltration practices. Parking lots, highways with more than 2,500 ADT, or any other VDOT practice as specified by the District or Richmond offices, will require restricted infiltration. In these areas, a minimum of 50% of the total treatment volume must be treated using a filtering practice or bioretention prior to direction to an infiltration practice.

6.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing an infiltration practice for improvement of water quality. Cross-section details for specific design features, including material specifications, can be found in the VDOT BMP SWM-6, Infiltration Practices. General guidance for filtering practices can be found in **Table 6.3.**

Table 6.3 - Stormwater Infiltration Practice Design Guidance
Virginia Stormwater Design Specification No. 8, Infiltration Practices, Draft (DCR/DEQ, 2013)

Level 1 Design (RR:50; TP:25; TN:15)	**Level 2 Design (RR:90; TP:25; TN:15)**
<u>Sizing:</u> $T_v = [(Rv)(A)/12]$ – the volume reduced by an upstream BMP	<u>Sizing:</u> $T_v = [1.1(Rv)(A)/12]$ – the volume reduced by an upstream BMP
At least two forms of pre-treatment (see **Table 6.6**)	At least three forms of pre-treatment (see **Table 6.6**)
Soil infiltration rate 1/2 to 1 in/hr number of tests depends on the scale (**Table 6.2**)	Soil infiltration rates of 1.0 to 4.0 in/hr number of tests depends on the scale (**Table 6.2**)
Minimum of 2' between the bottom of the infiltration practice and the seasonal high water table or bedrock	
T_v infiltrates within 36 to 48 hours	
Building Setbacks – see **Table 6.2**	
All Designs are subject to hotspot runoff restrictions/prohibitions	

***** The Virginia DEQ Office of Water Supply (OWS) has taken the position that stormwater infiltration BMPs designed in accordance with this design specification are acceptable related to their potential impacts on groundwater quality and will not require a Virginia Pollution Abatement (VPA) Permit. However, the DEQ Division of Land Protection and Revitalization, which includes the OWS, may change the approach to evaluating impacts to groundwater from stormwater infiltration BMPs in the future. In addition, stormwater infiltration BMPs designed according to other specifications will require a case-by-case determination by DEQ of VPA Permit requirements for the facility.

6.3.1 Sizing

The measured infiltration rate on site shall be in accordance with Virginia Stormwater Design Specification No. 8, Infiltration Practices, Draft (DCR/DEQ, 2013), and VDOT Special Provision for Infiltration Practices (2014).

Actual dimensions are determined from **Equations 8.1 to 8.4** of the Virginia Stormwater Design Specification No. 8, Infiltration Practices, Draft (DCR/DEQ, 2013). For convenience, those equations are reproduced below.

Infiltration basins may be designed as surface or subsurface facilities. If the facility is designed as a surface basin, the maximum depth is defined as:

$$d_{max} = \frac{\left(\frac{f}{2}\right)(t_d)}{12} \qquad \textbf{(6.1)}$$

where:

d_{max} = maximum depth of the infiltration practice (feet)
f = measured infiltration rate (inches/hour)
t_d = maximum draw down time (usually 48 hours)

If the facility is designed as a subsurface basin, the maximum depth is calculated using **Equation 6.2**:

$$d_{max} = \frac{\left(\frac{f}{2}\right)(t_d)}{\eta \times 12} \qquad \textbf{(6.2)}$$

where:

η = porosity of the stone reservoir (assume 0.4)

After calculation with **Equation 6.1 or 6.2**, **Table 6.4** shall be used for comparison. The allowable depth that is less (**Equations 6.1/6.2** or **Table 6.4**) shall be used for final design.

Table 6.4 - Maximum Depth (in feet) for Infiltration Practices
Virginia Stormwater Design Specification No. 8, Infiltration Practices, Draft (DCR/DEQ, 2013)

Mode of Entry	Scale of Infiltration		
	Micro Infiltration	Small Scale Infiltration	Conventional Infiltration

Surface Basin	1.0	1.5	2.0
Underground Reservoir	3.0	5.0	varies

Once the depth has been chosen, the surface area is computed using either **Equation 6.3** for surface basins or **Equation 6.4** for subsurface basins:

$$SA = {(T_{vBMP})}/{d} + \left[{\left((1/2) f \times t_f\right)}/{12} \right] \quad \textbf{(6.3)}$$

where:

SA = Surface Area (ft^2)
T_{vBMP} = Treatment Volume from drainage area plus remaining volume from upstream practices (ft^3)
d = Infiltration depth (feet), cannot exceed maximum allowable
f = Measured infiltration rate (inches/hr)
t_f = Time to fill the infiltration facility (2 hours)

$$SA = {(T_{vBMP})}/{(\eta \times d)} + \left[{\left((1/2) f \times t_f\right)}/{12} \right] \quad \textbf{(6.4)}$$

where:

η = porosity of the stone reservoir (assume 0.4)

The computed treatment volume used in **Equations 6.3 or 6.4** is as defined in **Section 1, Equation 1.1**, with adjustment for any remaining upstream volume from BMPs that is to be infiltrated.

Required infiltration tests shall be according to surface area thresholds shown in **Table 6.5**.

Table 6.5 - Number of Soil Profiles and Infiltration Tests Required
Virginia Stormwater Design Specification No. 8, Infiltration Practices, Draft (DCR/DEQ, 2013)

Area of Practice	# of Soil Profile Explorations	# of Infiltration (Permeability) Tests
Up to 2,500 ft^2	1	2*
2,500 ft^2 to 5,000 ft^2	2	3
5,000 ft^2 to 7,500 ft^2	2	4
7,500 ft^2 to 10,000 ft^2	2	5
Greater than 10,000 ft^2	Add 1 soil profile and 2 infiltration tests for each additional 5,000 ft^2 of practice	
Linear practices should add 1 additional soil profile for each 100 LF of practice, and 1 additional infiltration test for each additional 50 LF of practice. *Micro-scale applications with a small footprint (<500 ft^2), such as a downspout disconnection (Design Specification No. 1) require only one infiltration test per location.		

6.3.2 Pretreatment

Pretreatment, including minimum pretreatment volume, required for infiltration practices is as specified in **Table 6.6**.

Table 6.6 - Required Pre-treatment Elements for Infiltration Practices
Virginia Stormwater Design Specification No. 8, Infiltration Practices, Draft (DCR/DEQ, 2013)

Pre-treatment [1]	Scale of Infiltration		
	Micro Infiltration	Small-Scale Infiltration	Conventional Infiltration
Number and Volume of Pre-treatment Techniques Employed	**2** external techniques; no minimum pre-treatment volume required.	**3** techniques; **15%** minimum pre-treatment volume required (inclusive).	**3** techniques; **25%** minimum pre-treatment volume required (inclusive); at least **one** separate pre-treatment cell.
Acceptable Pre-treatment Techniques	Leaf gutter screens Grass filter strip Upper sand layer Washed bank run gravel	Grass filter strip Grass channel Plunge pool Gravel diaphragm	Sediment trap cell Sand filter cell Sump pit Grass filter strip Gravel diaphragm
[1] A minimum of 50% of the runoff reduction volume must be pre-treated by a filtering or bioretention practice *prior* to infiltration *if* the site is a restricted stormwater hotspot			

6.3.3 Infiltration Basins

Ponding depth is restricted to 24” over an infiltration area. Side slopes entering the basin shall be no steeper than 4H:1V. If the contributing drainage area is greater than 20,000 ft^2, a surface pretreatment cell must be provided. This cell may be a dry sediment collection area or a sand filter.

6.3.4 Drawdown

Drawdown should typically be complete in 36 to 48 hours.

6.3.5 Infiltration Rate Adjustment

Measured infiltration rates are adjusted by a factor of 2 to allow a factor of safety for long term operation. This adjustment has been applied to the measured infiltration rate in **Equations 6.1 – 6.4**.

6.3.6 Porosity

Porosity, used in **Equations 6.2 and 6.4**, should be assumed to be 0.4; however, if additional storage in the form of subsurface pipes or similar structures are used, the porosity coefficient may be adjusted, as appropriate.

6.3.7 Construction and Inspection

Construction and inspection shall be in conformance with the VDOT Special Provision for Infiltration Practices, 2014. The designer should direct that the construction process should be carefully monitored (via regular inspections). Improper construction is the main cause of Infiltration BMP failure, resulting in costly repair/replacement.

6.3.8 Maintenance Reduction Considerations

Maintenance is a crucial element that ensures the long-term performance of infiltration practices. The most frequently cited maintenance problem for infiltration practices is clogging of the surface stone by organic matter and sediment. The following design features can either minimize the risk of clogging or help to identify maintenance issues before they cause failure of the facility:

Pre-treatment Filter Strip of Low Maintenance Vegetation - Regular mowing of turf generates a significant volume of organic debris that can eventually clog the surface of an infiltration trench or basin located in a turf area; similarly, mulch from landscaped areas can migrate into the infiltration facility. Landscaping vegetation adjacent to the infiltration facility should consist of low maintenance ground cover.

Observation Well - Small-scale and conventional infiltration practices should include an observation well, consisting of an anchored 6” diameter perforated PVC pipe fitted with a lockable cap installed flush with the ground surface, to facilitate periodic inspection and maintenance.

Filter Fabric - Geotextile filter fabric **should not be installed** along the bottom of infiltration practices. Experience has shown that filter fabric is prone to clogging, and a layer of coarse washed stone (choker stone) is a more effective substitute. However, permeable filter fabric must be installed on the trench sides to prevent soil piping. A layer of fabric may also be installed along the top of the practice to help keep organic debris or topsoil from migrating downward into the stone. Periodic maintenance to remove and replace this surface layer will be required to ensure that surface runoff can get into the infiltration practice.

Direct Maintenance Access - Access must be provided to allow personnel and equipment to perform non-routine maintenance tasks, such as practice reconstruction or rehabilitation. While a turf cover is permissible for micro- and small-scale infiltration practices, the surface should not be covered by an impermeable material, such as asphalt or concrete.

6.4 Design Process

This section presents the design process applicable to infiltration practices serving as water quality BMPs. The pre- and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 11 of the *Virginia Stormwater Management Handbook, 2nd Edition*, Draft (DCR/DEQ, 2013) for details on hydrologic methodology.

A Level 2 infiltration basin is being proposed to treat runoff from a 1.25 acre addition (120 new parking spaces) to a park and ride lot near the U.S. 311 and Interstate 81 interchange in Salem, VA. The hydrologic classification of on-site soils is a mix of HSG B and HSG D soils. Infiltration tests indicate that the HSG B soils are suitable for infiltration. Post-development conditions within the disturbed area indicate 1.05 acres of impervious area, and 0.20 acres of managed turf. Summaries of these parameters are found in **Table 6.7**. The time of concentration to the infiltration practice has been computed as 6 minutes. The project site does not exhibit a high or seasonally high groundwater table or indicate the presence of bedrock, based on geotechnical tests performed on-site. Due to the scale of the facility, it is classified as “conventional infiltration” according to the impervious treatment areas shown in **Table 6.2**. In this case, because there will be over 40 new parking spaces, the site is considered a stormwater hotspot. Therefore, a Level 1 sand filter will be installed to provide additional pretreatment prior to infiltration. Full design of the sand filter pretreatment is not shown in this example. The user is directed to the Section 10 for computational methodology used to size the Level 1 sand filter pretreatment cell.

Table 6.7 - Hydrologic Characteristics of Example Project Site

		Impervious	Turf	Forest	Impervious	Turf
Pre	Soil Classification	HSG B	HSG B	HSG B	HSG D	HSG D
	Area (acres)	0.00	0.15	0.20	0.00	0.90
Post	Soil Classification	HSG B	HSG B	HSG B	HSG D	HSG D
	Area (acres)	0.25	0.10	0.00	0.80	0.10

Step 1 - Enter Data into VRRM Spreadsheet

The required site data from **Table 8.2** is input into the **VRRM Spreadsheet for New Development (2014)**, resulting in site data summary information shown in **Table 6.8**.

Table 6.8 - Summary of Output from VRRM Site Data Tab

Site Rv	**0.83**
Post-development Treatment Volume (ft^3)	**3,784**
Post-development TP Load (lb/yr)	**2.38**
Total TP Load Reduction Required (lb/yr)	**1.87**

Appropriate data for post-development conditions is input into the VRRM Spreadsheet Drainage Area tab, yielding compliance results summarized in **Table 6.9. Note that this includes used the Level 1 sand filter as the first BMP in a treatment train, with effluent directed to a Level 2 Infiltration facility.**

Table 6.9 - Summary Data from Treatment Train Treatment

Total Impervious Cover Treated (acres)	1.05
Total Turf Area Treated (acres)	0.20
Total TP Load Reduction Achieved in D.A. A (lb/yr)	2.30

In this case, the total phosphorus reduction required is 1.87 lbs/yr (**Table 6.8**). The estimated removal is 2.30 lbs/yr; therefore, the target has been met.

Step 2 - Compute the Required Treatment Volume

The treatment volume can be calculated using **Section 1, Equation 1** or taken directly from the VRRM Spreadsheet Drainage Area tabs. For this example, the reported treatment volume on the drainage area tab (treating the 1.25 acre area described by data in **Table 6.7**) is 3,784 ft^3. Because the infiltration facility is a Level 2, this treatment volume must be multiplied by a factor of 1.1 to yield the BMP treatment volume. This calculation yields a value of 4,162 ft^3.

Step 3 - Enter Data in Channel and Flood Protection Tab

Values for the 1-, 2-, and 10-year 24-hour rainfall depth should be determined from the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 and entered into the “Channel and Flood Protection” tab of the spreadsheet. For this site (Lat 37.3170, Long -80.0553), those values are shown in **Table 6.10**. Curve numbers used for computations should be those calculated as part of the runoff reduction spreadsheet (Virginia Runoff Reduction Spreadsheet for New Development, 2014). For this site, results from the runoff reduction spreadsheet are shown in **Table 6.11**, and result in adjusted curve numbers of 84, 85 and 87 for the 1, 2 and 10-year storms, respectively.

Table 6.10 - Rainfall Totals from NOAA Atlas 14

	1-year storm	2-year storm	10-year storm
Rainfall (inches)	2.56	3.10	4.63

Table 6.11 - Adjusted CN from Runoff Reduction Channel and Flood Protection

	1-year Storm	**2-year Storm**	**10-year Storm**
$RV_{Developed}$ (in) with no Runoff Reduction	1.93	2.45	3.94
$RV_{Developed}$ (in) with Runoff Reduction	1.18	1.70	3.19
Adjusted CN	84	85	87

Site data and adjusted curve numbers are used in the Natural Resource Conservation Service Technical Release 55 (NRCS TR-55) Tabular method to calculate discharge hydrographs. (**Note that other hydrologic methodologies are suitable-see VDOT Drainage Manual, Hydrology for guidance**). Resulting peaks of hydrographs for the 1-, 2-, and 10-year storms are reported in **Table 6.12**. These values can be used to size the conveyance downstream of the infiltration practice.

Table 6.12 – Post-development Discharge Peaks Exiting BMP

	1-year storm	2-year storm	10-year storm
Discharge (cfs)	2.25	3.28	6.35

Step 4 - Calculate Maximum Allowable Depth

The measured infiltration rate at the site is 1.2 in/hr. Based on guidelines by DEQ, a factor of safety of 2 will be applied to this infiltration rate. Therefore, the design rate will by 0.6 in/hr. The facility will be a subsurface facility; therefore, the maximum depth is calculated using **Equation 6.2** as:

$$d_{max} = \frac{(0.6\ in/hr)(48\ \text{hrs})}{0.4 \times 12} = 6\ feet$$

Step 5 - Calculate Underground Reservoir Surface Area

Six inches of temporary surface storage will be used above ground for use during larger storms. However, due to the presence of an over drain at the top of the stone reservoir layer (see VDOT SWM-6, Type I), this surface area cannot be used as part of the treatment volume. An initial assumed depth of the facility is taken as 75% of the maximum depth. Therefore, **Equation 6.4** is evaluated using an assumed reservoir depth of 4.5' to determine surface area as:

$$SA = {(4{,}162 ft^3)}\Big/{(0.4 \times 4.5\ ft)} + \left[\left((1/2) 1.2 \frac{in}{hr} \times 2hr \right) \Big/ 12 \right] = 2{,}312\ ft^2$$

Because the assumed depth (4.5') does not exceed the maximum allowed depth of 6', and the calculated surface area of the facility does not exceed the available area in the HSG B soils, the design is appropriate. Therefore, the facility will have a bed area of 2,312 ft^2 and a stone reservoir depth of 4.5'.

Step 4 - Pretreatment

Parking lot runoff drains directly to a gravel diaphragm that runs along the edge of the proposed pavement to introduce stormwater runoff into a small perimeter grass channel, where it is conveyed into the pretreatment sediment forebay, spilling into the sand filter cell. The minimum sand filter treatment volume is calculated to be 0.50T_v (due to hotspot restrictions) of the infiltration practice, which is 2,081 ft^3. However, VDOT, in conversations with the City of Salem, has determined that maximum removal of hotspot contaminants from this site is desired; therefore the entire treatment volume will be treated through the sand filter prior to entering the infiltration bed. Sizing of the sediment forebay and sand filter will be according to guidelines found in Section 10, but are not shown in this example.

Step 5 - Specify Full Cross Section and Geometry

Due to width constraints, the final dimensions of the facility will be 26.6' wide and 90' long. The vertical cross section shall conform to the VDOT SWM-6 Infiltration Practices (2014) detail for a Type I Infiltration Practice. The surface shall consist of 3" of river stone. The stone reservoir shall have a depth of 3.83', consisting of VDOT #1 open graded course aggregate. Below this, an 8" filtration layer consisting of grade A VDOT fine aggregate shall be installed. Finally, directly above the bed, a 4" choker layer of #8 stone shall be installed. Note that due to the location of the overflow drain (VDOT SWM-6), the surface layer of river stone and the top 4” of VDOT #1 stone in the reservoir layer cannot be used as part of the storage volume calculation.

Step 6 - Design Overflow Structure

Discharges for design storms are found in **Table 6.12**. Per the requirements of VDOT SWM-6 Infiltration Practices (2014), an overflow weir shall be installed to allow outflow of design storms. In this case, a weir shall be installed at a 6” elevation above the surface (river stone) of the infiltration bed, with a base width of 3’ and side slopes of 3:1. The overflow structure must be evaluated based on design peaks.

One purpose of the Runoff Reduction Method is to produce adjusted curve numbers for use in estimating peak runoff downstream of a practice. Although this method can be used, due to additional above-ground storage in the infiltration facility, the peaks generated using this method (**Table 6.12**) would be slightly conservative for this design example. An alternative method is to perform a routing of the storms through the facility using common hydrologic modeling software and hydrographs that have not been adjusted for the volume reduction in the practice, in this case a curve number of 94. Use of TR-55 methodology, using all other information and a curve number of 94, yields hydrographs with peaks for the 2- and 10- year 24-hour design storms of 4.83 cfs and 7.78 cfs, respectively. Routing of the 2- and 10-year storms has been performed through the (assumed) empty facility. The first step in routing is the development of a storage-elevation curve. Using information from Step 5, the resulting storage-elevation data is shown in **Table 6.13**.

Table 6.13 - Storage Elevation Data

Elevation (feet)	Storage (cubic feet)
1700.00	0
1702.00	1,830
1705.08	4,661
1705.58	5,811
1707.00	9,078

Using a 4” perforated riser exiting the bed at a 1.0% slope at invert 1704.50’, and an overflow weir with crest of 1705.58’ (see geometry above), a rating curve can be generated using standard hydrologic modeling software. Once the rating curve is developed, hydrologic routing calculations can occur. Abbreviated routing results for the 2- and 10-year design storms are found is **Tables 6.14 and 6.15**, respectively.

Table 6.14 - Routing of 2-Year Storm Through Facility

Event Time (hrs)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation MSL (feet)	Basin Outflow (cfs)
0.90	1.60	1.57	0.029	1701.38	0.000
1.00	3.09	3.06	0.048	1702.29	0.000
1.10	4.83	4.80	0.081	1703.83	0.000
1.20	3.00	2.97	0.112	1705.17	0.219
1.30	1.04	1.01	0.126	1705.45	0.250
1.40	0.70	0.67	0.131	1705.54	0.269
1.50	0.59	0.56	0.134	1705.59	0.288
1.60	0.50	0.47	0.135	1705.62	0.361
1.70	0.41	0.38	0.136	1705.63	0.389
1.80	0.36	0.33	0.136	1705.62	0.373
1.90	0.34	0.31	0.135	1705.62	0.348
2.00	0.32	0.29	0.135	1705.61	0.326
2.10	0.29	0.26	0.135	1705.60	0.302
2.20	0.27	0.24	0.134	1705.60	0.290
2.30	0.26	0.23	0.134	1705.59	0.287
2.40	0.24	0.21	0.133	1705.58	0.283
2.50	0.23	0.20	0.133	1705.56	0.279

The infiltration rate of the facility has been converted to a constant outflow rate of 0.032 cfs by using an adjusted infiltration rate of 0.6 in/hr (half of measured rate) and the bed surface area (2,312 ft^2). This infiltration rate must be implemented as part of the routing to compensate for exfiltration during the course of the runoff event. Note that the 2-year storm is completely contained within the facility until the subsurface storage volume is overwhelmed and the overdrain is activated. The 2-year storm overflow peak computed using this method is 0.39 cfs. Note that this is partially due to the additional storage available above the level of the overdrain and under the crest of the overflow weir.

Table 6.15 - Routing of 10-Year Storm Through Facility

Event Time (hrs)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation MSL (feet)	Basin Outflow (cfs)
0.60	0.41	0.38	0.012	1700.58	0.000
0.70	1.13	1.10	0.018	1700.87	0.000
0.80	1.85	1.82	0.030	1701.45	0.000
0.90	2.57	2.54	0.048	1702.30	0.000
1.00	4.98	4.95	0.079	1703.77	0.000
1.10	7.78	7.75	0.131	1705.53	0.266
1.20	4.84	4.81	0.164	1706.15	4.340
1.30	1.67	1.64	0.158	1706.05	3.350
1.40	1.13	1.10	0.149	1705.87	1.750
1.50	0.95	0.92	0.145	1705.79	1.210
1.60	0.80	0.77	0.143	1705.75	0.980
1.70	0.66	0.63	0.141	1705.72	0.800
1.80	0.59	0.56	0.140	1705.70	0.670

Note that routing the 10-year storm using this method results in a peak of 4.34 cfs, vs. the peak of 6.35 cfs that is calculated using the adjusted curve numbers. During the design, the VDOT project manager and VDOT Hydraulics shall be consulted to determine the methodology to be used for final analysis. The receiving channel downstream of the overflow weir must be evaluated for adequacy using standard methodologies, such as the Manning equation.

Step 7 - Design of Overflow and Downstream Conveyance Structures

Overflow and conveyance structures must be designed to pass the specified design storm based on functional classification of the road. This includes calculations for storms of lower recurrence (i.e. 25-, 50-, and 100-year storms). These computations are beyond the scope of this design example. However, the user is directed to the VDOT Drainage Manual for guidance on flood and erosion compliance calculations.

7.1 Bioretention - Overview of Practice

Bioretention practices form a class of both filtration and infiltration BMPs whose function is to improve the quality of stormwater runoff by means of adsorption, filtration, volitilization, ion exchange and microbial decomposition. The soil media and stone bed also contribute to partial volume reduction as calculated through the runoff reduction methodology. In the most general sense, a bioretention BMP can be thought of as a modified infiltration area comprised of a *specific* mix of trees, plants, and shrubs intended to mimic the ecosystem of an upland (non-wetland) forest floor. There are two categories of bioretention BMP: *basins* and *filters.*

Bioretention *basins* are planting areas constructed as shallow basins in which stormwater inflow is treated by filtration through the surface plant material, biological and chemical reactions within the soil and basin vegetation, and the eventual infiltration into the underlying soil media. Bioretention *filters* function much the same as bioretention basins, but are used in locations where full infiltration is not feasible due to inadequate soil permeability or the proximity to wells, drainfields, or structural foundations. Bioretention filters are equipped with a connection, via underdrain, to a local storm sewer system such that water enters the storm sewer after it has filtered through the bioretention cell. Figures 9.1 and 9.2 present the general configuration of a bioretention basin and filter.

The Virginia Stormwater Design Specification No. 9, Bioretention, Draft (DCR/DEQ, 2013) lists several bioretention applications, including parking lot islands, parking lot edges, road median, roundabouts, interchanges and cul-de-sacs, right-of-way or commercial setback, or courtyards. Due to the ability to construct the practice in irregular shapes, including linear formations, and the relatively high pollutant removal efficiency, bioretention facilities are applicable on a wide array of transportation related projects.

Bioretention can be an important part of a stormwater quality treatment train, but these BMPs require special design considerations to minimize maintenance. Otherwise, they can become a maintenance burden, particularly if sediment accumulates within the basin, where it can clog the media pore space. Good design can eliminate or at least minimize such problems.

Table 7.1 - Summary of Stormwater Functions Provided by Bioretention Basins

Stormwater Function	Level 1 Design	Level 2 Design
Annual Runoff Volume Reduction (RR)	40%	80%
Total Phosphorus (TP) EMC Reduction[1] by BMP Treatment Process	25%	50%
Total Phosphorus (TP) Mass Load Removal	55%	90%
Total Nitrogen (TN) EMC Reduction[1] by BMP Treatment Process	40%	60%
Total Nitrogen (TN) Mass Load Removal	64%	90%
Channel and Flood Protection	• Use the Virginia Runoff Reduction Method (VRRM) Compliance Spreadsheet to calculate the Curve Number (CN) Adjustment ***OR*** • Design extra storage (optional; as needed) on the surface, in the engineered soil matrix, and in the stone/underdrain layer to accommodate a larger storm, and use NRCS TR-55 Runoff Equations[2] to compute the CN Adjustment.	

[1] Change in event mean concentration (EMC) through the practice. Actual nutrient mass load removed is the product of the removal rate and the runoff reduction rate (see Table 1 in the *Introduction to the New Virginia Stormwater Design Specifications*).
[2] NRCS TR-55 Runoff Equations 2-1 thru 2-5 and Figure 2-1 can be used to compute a curve number adjustment for larger storm events based on the retention storage provided by the practice(s).

Sources: CWP and CSN (2008) and CWP (2007)

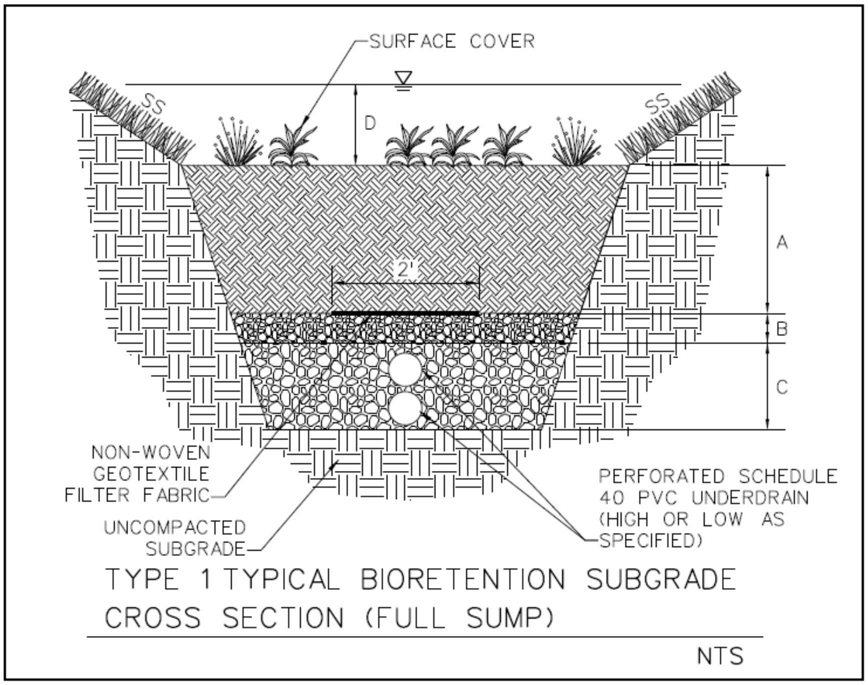


Figure 7.1 - Bioretention Cross-section – Type 1
VDOT SWM-7 Bioretention, 2014

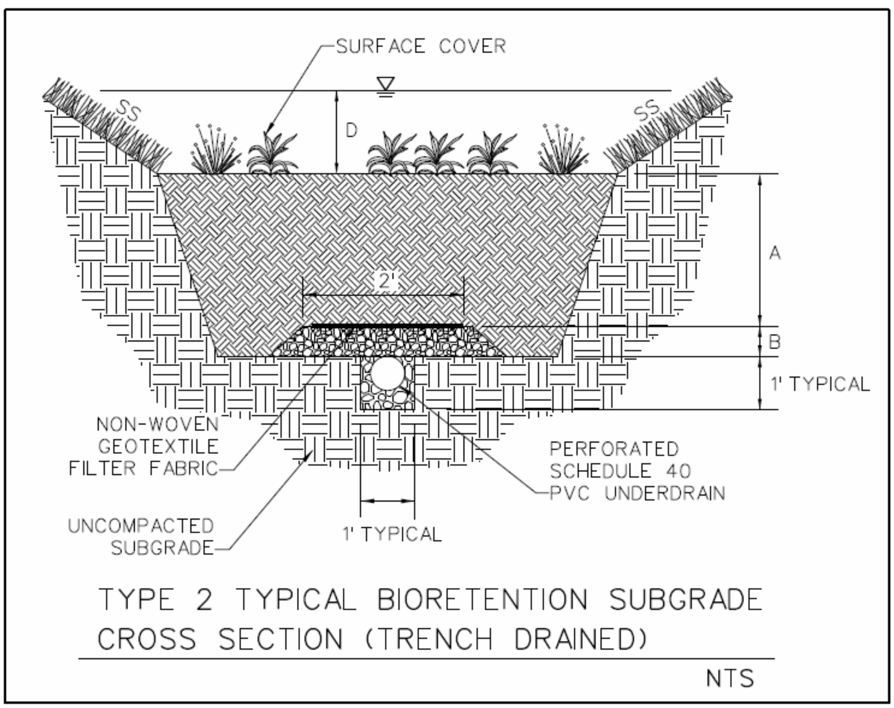


Figure 7.2 - Bioretention Cross-section – Type 2
VDOT SWM-7 Bioretention, 2014

7.2 Site Constraints and Siting of the Facility

When a bioretention facility is proposed the designer must consider a number of site constraints in addition to the contributing drainage area's new impervious cover. These constraints are discussed as follows.

7.2.1 Minimum Drainage Area

The minimum drainage area contributing runoff to a bioretention cell should typically be no smaller than 0.1 acres. Bioretention basins and filters are well suited to relatively small drainage areas.

7.2.2 Maximum Drainage Area

The maximum drainage area to a single bioretention facility is dependent on the type/level of bioretention proposed. Bioretention basins typically have an upper limit of 2.5 acres. Under special circumstances, the upper limit of bioretention basins may be increased to a maximum area of 5 acres (no more than 50% impervious) if additional low flow diversions or other pre-treatment and flow regulation measures are included. Any decision to exceed the maximum threshold of 2.5 acres should be discussed with VDOT prior to submittal. It is important to design bioretention facilities within the limits established for CDAs. Too much or too little runoff can result in performance issues and the need for subsequent repairs or reconstruction.

7.2.3 Site Slopes

Bioretention facilities are most suited for sites with upstream slopes between 1% and 5%. Steep upstream slopes are typically indicative of higher runoff velocities and higher probability of erosion and sediment transport into the facility, which is to be avoided. Installation on sites with greater upstream slopes (up to 15%) than those recommended will require energy dissipation measures integrated with required pre-treatment to ensure that runoff that is laden with high concentrations of sediment is not entering the facility.

7.2.4 Site Soils

This section refers to the native site soils underlying a bioretention facility. The planting soil mix of a bioretention facility is governed by specific guidelines discussed later in this Section and also in the Virginia Stormwater Design Specification No. 9, Bioretention, Draft (DCR/DEQ, 2013).

Although an underdrain will be required on most VDOT facilities, a bioretention system without underdrains may be acceptable under certain circumstances and only upon approval of the VDOT project manager. When such a facility is proposed, *a subsurface analysis and permeability test is required*. The required subsurface analysis should investigate soil characteristics to a depth of no less than 3' below the proposed bottom of the engineered media. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the final design characteristics of the proposed facility.

The soil infiltration rate should be measured according to the requirements in VDOT Special Provision for Stormwater Miscellaneous (2014) – see "Infiltration and Soil Testing". Soil infiltration rates which are deemed acceptable for bioretention basins without an underdrain are typically greater than *0.50 in/hr*. Soils exhibiting a clay content of greater than 20% and silt/clay content of more than 40% are typically unacceptable for bioretention facilities without an underdrain. Sites categorized as stormwater hotspots should not be used for infiltrative bioretention facilities due to higher likelihood of groundwater contamination.

7.2.5 Depth to Water Table

Bioretention basins should not be installed on sites with a high groundwater table. Inadequate separation between the BMP bottom and the surface of the water table may result in contamination of the water table. This potential contamination arises from the inability of the soil underlying the BMP to filter pollutants prior to their entrance into the water table. Additionally, a high water table can flood the bioretention cell and render it inoperable during periods of high precipitation and/or runoff. A separation distance of no less than 2' is required between the bottom of a bioretention basin and the surface of the *seasonally* high water table unless the site is located in coastal plain residential settings where the distance may be reduced to 1'.

7.2.6 Separation Distances

Setbacks from buildings and streets should be in accordance with **Table 7.2**. When using a liner, a 50' minimum separation from wells is required, which is increased to 100' if no liner is present. Additionally, a 20' minimum separation from septic drain fields is required when using a liner, and is increased to 50' if a liner is not present. Bioretention facilities must maintain a minimum down-gradient separation of 5' from wet utilities; however, dry utilities may pass **beneath** a bioretention facility if the utilities are encased. In the latter case, the utilities do not have to be encased if they can be routinely accessed without disturbing the bioretention basin.

7.2.7 Karst Areas

Infiltrative bioretention facilities (Level 2) should not be used in karst areas, or in areas with a prevalence of bedrock or fractured rock. However, a bioretention filter (Level 1) with an underdrain and liner may be considered if a separation distance of 3' is maintained between the bottom of the facility and the top of rock. In addition, drainage areas to Level 1 practices in these areas should be limited to 20,000 ft^2, and setbacks from structures should be discussed with VDOT and should generally be larger than standards shown in **Table 7.1**.

7.2.8 Placement on Fill Material

Bioretention basins should not be constructed on or nearby fill sections due to the possibility of creating an unstable subgrade. Fill areas are vulnerable to slope failure along the interface of the in-situ and fill material. The likelihood of this type of failure is increased when the fill material is frequently saturated, as anticipated when a bioretention basin.

7.2.9 Existing Utilities

Bioretention facilities can often be constructed over existing utility easements, provided permission to construct the facility over these easements is obtained from the utility owner *prior* to design of the facility. However, keep in mind that if the utility needs to do its own maintenance at some point in time, the excavation may disrupt the benefit of the filter media, especially if the excavated media mix is not used as backfill or if the surface is subsequently compacted. Therefore, it is generally advisable avoid locating bioretention BMPs above utility lines if at all possible.

7.2.10 Perennial, Chlorinated, Toxic and Irrigation Flows

Bioretention facilities must not be subjected to continuous or very frequent flows. Such conditions will lead to anaerobic conditions which support the export of previously captured pollutants from the facility. Additionally, bioretention facilities must not be subjected to chlorinated flows, such as those from swimming pools or saunas or toxic pollutants from stormwater hotspots, such as gasoline stations. The presence of elevated chlorine levels or toxic pollutants can kill the desirable bacteria responsible for the majority of nitrogen uptake in the facility. In general, bioretention facilities should not be subjected to any flows that are not stormwater runoff.

7.2.11 Floodplains

Bioretention facilities shall not be located in 100-year floodplains as designated on applicable FEMA flood maps for the project area.

7.2.12 Access

It is vital to provide adequate access to the BMP site. Site access must be safe and must provide enough room and appropriate gradients (ideally 4H:1V or flatter) for construction vehicles to install the BMP and for crews and equipment to perform maintenance. Ideally, access should include a dedicated easement that guarantees right-of-entry. Access requirements for underground versus above-ground BMPs are slightly different.

It is also important to consider alternative surface treatments for access ways, when appropriate, such as reinforced turf that do not increase the site's impervious cover. Maintenance access should extend to all critical elements of the BMP, such as the forebay, safety bench, inlet and riser/outlet structures, flow splitters, by-pass manholes and chambers, and emergency spillways. Risers should be located in embankments for access from land, and they should include access to all elements via a manhole and steps.

7.2.13 Security

To the degree feasible, the BMP should be located so that appropriate security can be provided – to minimize the risk to the facility of physical damage caused by outside sources, to minimize access to the facility by unauthorized persons (particularly children), and to thus reduce VDOT's liability for potential damages and physical harm. Where fencing is considered appropriate, ensure that gates are large enough to allow equipment necessary to perform maintenance to pass through and maintenance crews have keys/codes to unlock the gates.

7.3 General Design Guidelines

The proposed location of a bioretention facility must be established early in the project design phase and remain an integral component of the site design throughout to ensure runoff intended for treatment can be directed to the bioretention facility, pretreatment can be provided, and that slopes within the drainage area are appropriate.

Table 7.2 - Bioretention Filter and Basin Design Criteria
Virginia Stormwater Design Specification No. 9, Bioretention, Draft (DCR/DEQ, 2013)

Level 1 Design (RR 40 TP: 25)	Level 2 Design (RR: 80 TP: 50)
Sizing (**Section 7.3.2**): Tv_{BMP} = [(1)(Rv)(A) / 12] + any remaining volume from upstream BMP Surface Area (ft^2) = Tv_{BMP} / Storage Depth [1]	Sizing (**Section 7.3.2**): Tv_{BMP} = [(1.25)(Rv)(A) / 12] + any remaining volume from upstream BMP Surface Area (ft^2) = Tv_{BMP} /Storage Depth [1]
Recommended maximum contributing drainage area = 2.5 acres, or with local approval up to 5 acres and a maximum of 50% impervious	
Maximum Ponding Depth = 6 to 12 inches [2]	Maximum Ponding Depth = 6 to 12 inches [2]
Filter Media Depth minimum = 24”; recommended maximum = 48”	Filter Media Depth minimum = 36”; recommended maximum = 48”
Media & Surface Cover (**Section 7.3.10**) = supplied by vendor; tested for acceptable hydraulic conductivity (or permeability) and phosphorus content	
Sub-soil Testing (**Section 7.3.5**): not needed if an underdrain used; Min infiltration rate > 1/2 in/hr in order to remove the underdrain requirement.	Sub-soil Testing (**Section 7.3.5**): one soil profile and two infiltration tests per facility (up to 2,500 ft^2 of filter surface); Min infiltration rate > 1/2 in/hr in order to remove the underdrain requirement.
Underdrain (**Section 7.5, Step 5**) = Schedule 40 PVC with clean-outs	Underdrain & Underground Storage Layer (**Section 7.5, Step 5**) = Schedule 40 PVC with clean outs, and a minimum 12” stone sump below the invert
Inflow: sheet flow, curb cuts, trench drains, concentrated flow, or the equivalent	
Geometry (**Section 7.3.6**): Length of shortest flow path/Overall length = 0.3; ***OR***, other design methods used to prevent short-circuiting; a one-cell design (not including the pre-treatment cell).	Geometry (**Section 7.3.6**): Length of shortest flow path/Overall length = 0.8; ***OR***, other design methods used to prevent short-circuiting; a two-cell design (not including the pre-treatment cell).
Pre-treatment (**Section 7.3.7**): a pre-treatment cell, grass filter strip, gravel diaphragm, gravel flow spreader, or another approved (manufactured) pre-treatment structure.	Pre-treatment (**Section 7.3.7**): a pre-treatment cell *plus* one of the following: a grass filter strip, gravel diaphragm, gravel flow spreader, or another approved (manufactured) pre-treatment structure.
Conveyance & Overflow (**Section 7.3.9**)	Conveyance & Overflow (**Section 7.3.9**)
Planting Plan (**Section 7.3.11**): a planting template to include turf, herbaceous vegetation, shrubs, and/or trees to achieve surface area coverage of at least 75% within 2 years.	Planting Plan (**Section 7.3.11**): a planting template to include turf, herbaceous vegetation, shrubs, and/or trees to achieve surface area coverage of at least 90% within 2 years. If using turf, must combine with other types of vegetation.
Building Setbacks [3] (**Section 7.2.6**): 10’ if down-gradient from building or level (coastal plain); 50’ if up-gradient.	
Deeded Maintenance O&M Plan (**VDOT maintains per BMP Maintenance Manual**)	
[1] Storage depth is the sum of the porosity (η) of the soil media and gravel layers multiplied by their respective depths, plus the surface ponding depth. (**Section 7.3.4**). [2] A ponding depth of 6” is preferred. Ponding depths greater than 6” will require a specific planting plan to ensure appropriate plant selection (**Section 7.3.4**). [3] These are recommendations for simple building foundations. If an in-ground basement or other special conditions exist, the design should be reviewed by a licensed engineer. Also, a special footing or drainage design may be used to justify a reduction of the setbacks noted above.	

7.3.1 Basin Size

For preliminary sizing and space planning, a general rule of thumb is that the surface area of the facility will be 3%-6% of the contributing drainage area (dependent on imperviousness and design level). To avoid performance issues, the facility must be sized properly for the target Treatment Volume. However, oversizing the storage provided in the BMP, as compared to what is required to achieve the BMP's performance target, can decrease the frequency of maintenance needed and, thus, potential life-cycle costs. Oversizing, where feasible, can also help VDOT achieve its broader pollution reduction requirements associated with its DEQ MS4 Permit and the Chesapeake Bay TMDL. Oversizing options are likely to involve the adjustment of detention times and may require prior approval by DEQ.

Equation 7.1 describes the bioretention design storage depth as:

$$SD = (0.25)M_d + (0.40)G_d + (1.0)SS_d \qquad \textbf{(7.1)}$$

where

SD = storage depth (ft);
M_d = proposed media depth (ft);
G_d , = proposed gravel depth (ft) and
SS_d = the proposed surface storage depth (ft).

Coefficients in front of each correspond with void ratios associated with each layer as defined in Virginia DEQ Stormwater Design Specification No.9, (2013, et seq).

Equation 7.2 describes the calculation of the required minimum bioretention surface area as:

$$SA = \frac{[C_v \times T_v - V_u]}{SD} \qquad \textbf{(7.2)}$$

where

SA = computed surface area (ft^2)
C_v = volume coefficient (1.0 for level 1 design and 1.25 for level 2 design);
T_v , = computed treatment volume (acre-ft);
V_u = volume reduced by an upstream BMP (in a treatment train);
SD = storage depth (ft).

The computed treatment volume in **Equation 7.2** is further defined in **Section 1, Equations 1.1 and 1.2.**

7.3.2 Media Depth

The depth of the facility's planting soil should be determined from **Table 7.2**, according to the specified design level (Level 1 or Level 2).

7.3.3 Surface Ponding Depth

The depth of ponding on the facility surface should be restricted to no less than 6" and no more than 12" to preclude the development of anaerobic conditions within the planting soil. Further, for elevated outlet structures, a minimum of 1' of freeboard should be provided from the crest elevation to the top of the berm. The 10-year storm is required to pass through the primary outlet without overtopping the berm.

7.3.4 Soil Infiltration Rate

Level 1 designs do not require soil infiltration rate testing due to the presence of an underdrain. Subsoil infiltration rates must exceed 1/2 in/hr for bioretention basins if an underdrain is not installed. The soil infiltration rate should be measured according to the requirements in VDOT Special Provision for Stormwater Miscellaneous (2014) – see "Infiltration and Soil Testing".

7.3.5 Basin Geometry

Basins should be configured to prevent short circuiting or bypassing of runoff from the edge of the facility to the overflow structure. In addition, the overall efficiency of the facility is contingent upon even distribution of inflow across the surface of the facility (flat filter surface). In order to prevent short circuiting, the ratio of the shortest flow path to the longest flow path in the facility should not fall below 0.3 for Level 1 designs, or 0.8 for Level 2 designs (see **Figure 7.3**). If some inlets are unable to meet this criteria, the drainage areas served by these inlets should be 20% or less of the contributing drainage area. Further, this requirement may be waived by VDOT Hydraulics on a case by case basis if the design incorporates methods to prevent short circuiting such as landscape baffling or other methods.

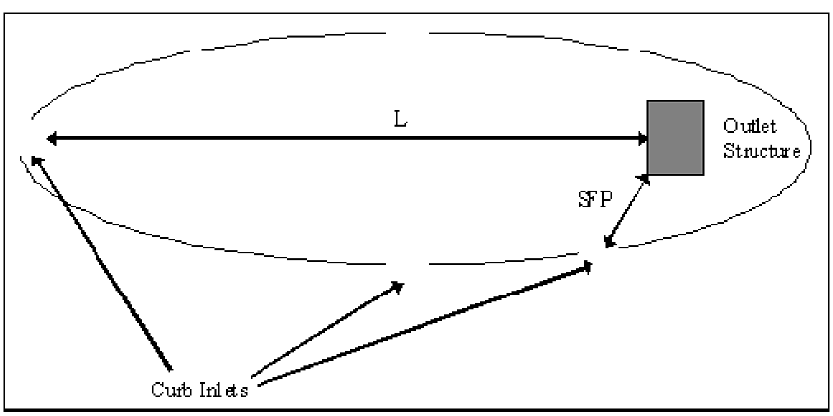


Figure 7.3 - Basin Geometry Relating Shortest and Longest Flow Paths. (Virginia DEQ Stormwater Design Specification No. 9, 2013, Et seq.)

7.3.6 Runoff Pre-treatment

Bioretention facilities *must* be preceded upstream by some form of runoff pre-treatment. For Level 1 designs, at least one of the pre-treatment options below must be chosen. A Level 2 design requires the installation of a pre-treatment forebay **in addition to** one of the other options. Roadways and parking lots often produce runoff with high levels of sediment, oil, and other pollutants. These pollutants can potentially clog the pore space in the facility, thus greatly reducing its pollutant removal performance. The selection of runoff pre-treatment is primarily a function of the type of flow entering the facility, as discussed below. Proper pre-treatment preserves a greater fraction of the Treatment Volume over time and prevents large particles from clogging orifices and filter media. Selecting an improper type of pre-treatment or designing and constructing the pre-treatment feature incorrectly can result in performance and maintenance issues.

e. ***Pre-treatment Forebay***: These cells act as forebays to allow sediment to settle out of stormwater runoff prior to entering the bioretention cell. Concentrating sediment settling in one location simplifies maintenance significantly. In addition, the forebay functions as an energy dissipater to reduce the velocity of incoming stormwater runoff and prevent erosive damage within the treatment cell. A pre-treatment cell must have a minimum volume of at least 15% of the bioretention cells total treatment volume. Installation shall be in accordance with VDOT BMP Standard SWM-PT: Pre-treatment (Pre-treatment Forebay).

f. ***Grass Filter Strips***: Runoff entering a bioretention basin or filter as *sheet flow* may be treated by a grass filter strip. The purpose of the grass buffer strip/energy dissipation area is to reduce the erosive capabilities of runoff prior to its entrance into the bioretention area. The recommended length of the grass buffer strip is a function of the land cover of the contributing drainage area and its slope. The recommended minimum length of the grass buffer strip should not be less than 10'. The maximum side slope of a grass filter strip is 5:1 for Pre-treatment Level 1 and 3:1 for Pre-treatment Level 2. For Pre-treatment Level 2, a minimum 5' length of 5:1 or shallower slope is required prior to sloping to the surface of the facility.

g. ***Gravel Diaphragms***: These pre-treatment measures are typically installed along the edge of pavement or road shoulder with the purpose of evenly distributing flow onto the cell surface. The diaphragm should be oriented perpendicular to flow, as shown in VDOT BMP Standard SWM-PT: Pre-treatment (Gravel Diaphragm).

h. ***Gravel Flow Spreader***: These measures are typically located at points of concentrated inflow, such as curb cuts, downspouts, etc. There should be a 2"-4" drop from the adjacent impervious surface. Gravel/stone should extend along the entire width of the opening, creating a level stone weir at the bottom of the channel. Installation shall be in accordance with VDOT BMP Standard SWM-PT: Pre-treatment (Gravel Flow Spreader).

7.3.7 Offline Configurations

Whenever possible, bioretention facilities should be placed off-line so that flow is diverted onto it. This permits the facility to fill with only the desired treatment volume and bypass any remaining flow to the storm drainage system. Because offline bioretention BMPs are sized to accommodate only the designated water quality volume, a flow-splitter or diversion weir must be designed to restrict inflows to the bioretention area. The flow-splitter or diversion weir must be designed to admit a designated *volume* of runoff into the basin rather than to simply regulate the flow *rate* into the basin. The diversion structure may be prefabricated, or cast in place during construction.

Typically, the construction of the bypass channel invert (or diversion weir) will place its crest elevation equal to the maximum allowable ponding depth in the bioretention area. Flow over the diversion weir will occur when runoff volumes exceed the depth in the cell that corresponds with the computed water quality volume. These overflows then enter the stormwater conveyance channel. This configuration results in minimal mixing of the held water quality volume with flows from large runoff producing events. A modified design referred to as a *dual pond system* is characterized by a diversion weir/channel which directs the computed water quality volume into the bioretention area, while conveying excess volumes downstream to a peak mitigation detention pond.

7.3.8 Overflow/Bypass Structure

When a bioretention facility is constructed online, or the maximum volume of flow entering the facility is not otherwise restricted, an overflow structure *must* be provided. This structure provides bypass for excess runoff when the bioretention subsurface and surface capacity is met. A maintenance bypass also allows storms to be re-routed around the BMP during maintenance cycles (from several days to a week). Maintenance bypasses should typically be located either at the inlet or slightly upstream of the BMP. In piped systems, this is accommodated by fitting sluice gates to the by-pass pipe and BMP inlet pipe in an upstream manhole. For maintenance operations, the gate to the BMP can be closed and the gate to the by-pass pipe opened. This type of system can also be used for the seasonal operation of infiltration systems that accept roadway runoff.

Common overflow structures include domed risers, grate or slot inlets (such as DI-7), and weir structures. Budget, site aesthetics, and maintenance will govern the selection of the overflow structure. The sizing of the overflow structure must consider the flow rate for the design storm of interest, typically the 10-year runoff producing event. The crest or discharge elevation of the overflow structure should be set an elevation of 6” to 12” above the elevation of the filter bed. A typical riser overflow structure is shown in **Figure 7.4**.

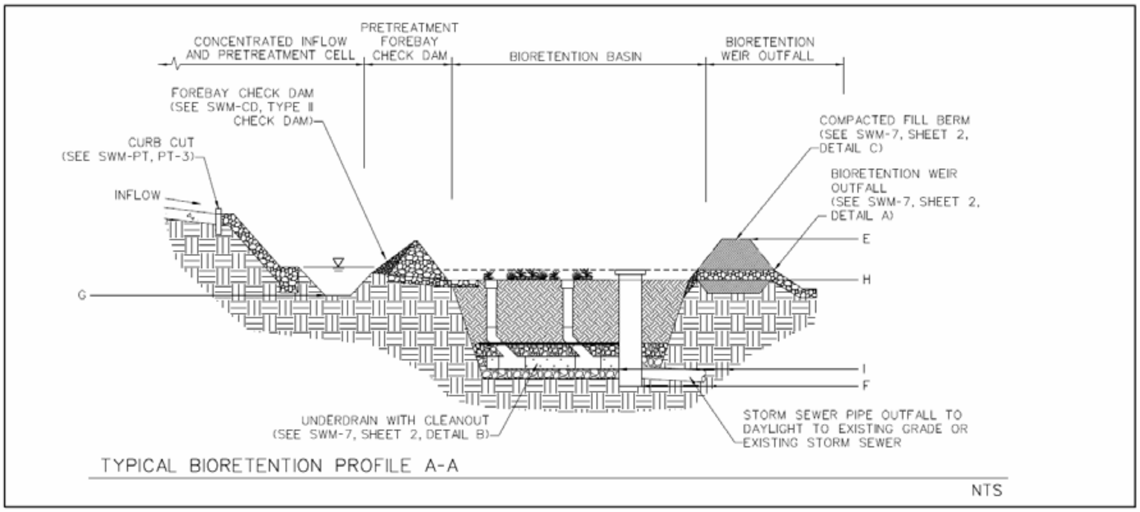


Figure 7.4 - Typical Bypass Structure Configuration
VDOT BMP Standard SWM-7: Bioretention (2014)

7.3.9 Filter Media and Surface Cover

Installation of correct filter media and surface cover are critical to the functionality and long term maintenance of a bioretention facility. Media shall be installed according to the requirements in VDOT BMP Standard SWM-7: Bioretention and the VDOT Special Provision for Bioretention Facilities (2014). Surface cover shall be either a 2”-3” layer of shredded, aged, hardwood mulch, or alternative covers such as turf, perennials/herbaceous shrubs, or a combination as recommended by a landscape architect or plant specialist for application in specific region and based on salt tolerance and/or other specific project considerations. It is critical to specify and install the correct type and depth of filter media; doing otherwise is likely to result in performance and maintenance issues. See VDOT Special Provision for Bioretention Facilities (2014) for material specifications.

7.3.10 Planting Considerations

The ultimate goal in the selection and location of vegetation within a bioretention facility is to, as closely as possible, mimic an upland (non-wetland) terrestrial forest ecosystem. This type of planting scheme is based on a natively-occurring forest’s ability to effectively cycle and assimilate nutrients, metals, and other pollutants through the plant species, underlying soil, and also the system’s organic matter. It is crucial that a planting plan be prepared and that plant selection includes a range of robust species capable of handling frequent inundation and within the ability to withstand expected concentrations of pollutants (salt, oil, VOCs, etc.). If designed correctly, planting plans can reduce future maintenance liabilities. For example, proper landscaping can stabilize banks and prevent upland erosion.

Aesthetics is an important concern as well. Bioretention BMPS can often be incorporated into the stormwater management plans of high profile areas, providing a desirable site amenity in the form of landscaping. The design of bioretention facilities requires a working knowledge of indigenous horticultural practices, and it is recommended that a landscape architect or other qualified professional participate in the design process.

Typically, one of six planting templates should be used to maintain the function and appearance of a bioretention bed. The six most common bioretention templates are as follows:

- ***Turf.*** This option is typically restricted to on-lot micro-bioretention applications, such as a front yard rain garden. Grass species should be selected that have dense cover, are relatively slow growing, and require the least mowing and chemical inputs (e.g., fine fescue, tall fescue).
- ***Perennial garden.*** This option uses herbaceous plants and native grasses to create a garden effect with seasonal cover. It may be employed in both micro-scale and small scale bioretention applications. This option is attractive, but it requires more maintenance in the form of weeding.
- ***Perennial garden with shrubs.*** This option provides greater vertical form by mixing native shrubs and perennials together in the bioretention area. This option is frequently used when the filter bed is too shallow to support tree roots. Shrubs should have a minimum height of 30”.
- ***Tree, shrub and herbaceous plants.*** This is the traditional landscaping option for bioretention. It produces the most natural effect, and it is highly recommended for bioretention basin applications. The landscape goal is to simulate the structure and function of a native forest plant community.
- ***Turf and tree.*** This option is a lower maintenance version of the tree-shrub-herbaceous option 4, where the mulch layer is replaced by turf cover. Trees are planted within larger mulched islands to prevent damage during mowing operations.
- ***Herbaceous meadow.*** This is another lower maintenance approach that focuses on the herbaceous layer and may resemble a wildflower meadow or roadside vegetated area (e.g., with Joe Pye Weed, New York Ironweed, sedges, grasses, etc.). The goal is to establish a more natural look that may be appropriate if the facility is located in a lower maintenance area (e.g., further from buildings and parking lots). Shrubs and trees may be incorporated around the perimeter. Erosion control matting can be used in lieu of the conventional mulch layer.

The goal is to provide a planting plan that will provide cover for the filter surface in a short amount of time. Plants should be tolerant and able to withstand periods of inundation and drought. Species more tolerant of wet conditions should be located towards the center of the bed, with those less tolerant toward the perimeter. If trees are used, a spacing of approximately 15’ on center, and density of approximately one tree per 250 ft^2 is suggested. Shrubs should be planted approximately 10’ on center, and herbaceous vegetation should be planted at 1 to 1.5' on center. Where trees and shrubs are recommended (typically Level 2 designs), the designer should consider the long-term growth habit of the plants – trees can dominate a facility and require extensive maintenance. Maintenance is crucial when selecting plant species, and non-maintenance intensive species are preferred. ***All bioretention facilities installed for VDOT facilities or in rights of way shall be planted with salt-tolerant, herbaceous perennials due to the propensity of salt laden runoff occurring during winter months.***

7.4 Design Example

This section presents the design process applicable to bioretention facilities serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT *facilities* projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to *Chapter 11 of the Virginia Stormwater Management Handbook, 2nd Edition (DCR/DEQ, 2013)* for details on hydrologic methodology.

A bioretention basin design is being proposed to treat runoff from a 3,000' long section of a lane widening project along I-81 in Montgomery County Virginia. The current shoulder in the area that will be disturbed includes 1.10 acres of impervious (gravel and paved) area [0.80 acres overlaying HSG B soils and 0.30 acres overlaying HSG C soils]. Note that the milled areas on the remaining lanes are not counted in the disturbed area for calculations. In addition there is 1.20 acres of turf covered shoulder that drains to the area (0.90 acres in HSG B soils and 0.30 acres in HSG C soils) within the area of disturbance. The proposed widening will add an additional 0.40 acres of impervious area (total 1.10 acres HSG B and 0.40 acres HSG C), and reduce turf area post-development to 0.80 acres (0.60 acres HSG B and 0.20 acres HSG C). See **Table 7.3** for disturbed area characteristics.

Table 7.3 - Hydrologic Characteristics of Example Project Site

		Impervious		Turf	
Pre	**Soil Classification**	**HSG B**	**HSG C**	**HSG B**	**HSG C**
	Area (acres)	0.80	0.30	0.90	0.30
Post	**Soil Classification**	**HSG B**	**HSG C**	**HSG B**	**HSG C**
	Area (acres)	1.10	0.40	0.60	0.20

Due to geographic and topographic constraints, only a portion of the disturbed area (1,500') can be caught and treated at the proposed BMP location. Treatment calculations should include drainage from the disturbed area in addition to any additional treatment area (undisturbed by project) that drains to the proposed BMP location. For this project, additional run-on occurs from the existing lane that was milled and resurfaced (0.40 acres in HSG B soils). A summary of the runoff characteristics to the proposed BMP location is shown in **Table 7.4**.

Table 7.4 - Hydrologic Characteristics of Contributing Drainage Area to BMP

		Impervious		Turf	
Treatment	**Soil Classification**	**HSG B**	**HSG C**	**HSG B**	**HSG C**
	Area (acres)	0.80	0.20	0.30	0.10

The time of concentration for the BMP location subarea has been calculated to be 12 minutes. Geotechnical investigations reveal compacted soil with a high clay content. Lab test confirm that infiltration cannot be performed at this location. The project site does not exhibit a high or seasonally high groundwater table.

Step 1 - Enter Data into VRRM Spreadsheet

The required site data from **Table 7.3** is input into the **VRRM Spreadsheet for Redevelopment (2014)** to compute load reductions for a linear project, resulting in site data summary information shown in **Table 7.5**. Note that using the redevelopment spreadsheet, the required reduction for linear projects is computed as the sum of the Post-Redevelopment Load and the Post-Development Load minus 80% of the Predevelopment Listed load.

Table 7.5 - Summary Data from VRRM Site Data Analysis

Site Rv	0.69
Post-development TP Load (lb/yr)	3.62
Total TP Load Reduction Required (lb/yr)	1.27

Step 2 - Select Candidate BMP and Enter Information into Drainage Area Tab

A Level 1 bioretention has been selected as the candidate BMP for treatment of captured runoff. The land cover characteristics from **Table 7.4** is input into the **VRRM Spreadsheet for Redevelopment (2014)** drainage area tab, resulting in site data summary information shown in **Table 7.6**.

Table 7.6 - Summary Data from Level 1 Bioretention Treatment

Total Impervious Cover Treated (acres)	1.00
Total Turf Area Treated (acres)	0.40
Total TP Load Reduction Achieved in D.A. A (lb/yr)	1.29

Step 3 - Compute the Required Treatment Volume

The treatment volume can be calculated using **Section 1, Equation 1** or taken directly from the VRRM Spreadsheet Drainage Area tabs. For this example, the reported treatment volume on the drainage area tab (treating the 1.40 acre area described by data in **Table 7.6**) is 3,746 ft^3.

Step 4 - Enter Data in Channel and Flood Protection Tab

Hydrologic computations for required design storms for flood and erosion compliance are not shown as part of this example. The user is directed to the VDOT Drainage Manual for appropriate levels of protection and design requirements related to erosion and flood protection. However, hydrologic computations are necessary to compute peaks to design overflow components of the Level 1 Bioretention.

Values for the 1-, 2-, and 10-year 24- hour rainfall depth should be determined from the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 and entered into the “Channel and Flood Protection” tab of the spreadsheet. For this site (Lat 37.1538, Long -80.3265), those values are shown in **Table 7.7**. For the 1-, 2-, and 10-year 24-hour storms, adjusted curve numbers supplied by the VRRM spreadsheet should be used for conveyance and overflow sizing related to the proposed BMP.

Table 7.7 - Rainfall Totals from NOAA Atlas 14

	1-year storm	2-year storm	10-year storm
Rainfall (inches)	2.31	2.80	4.17

For this site, results from the runoff reduction spreadsheet are shown in **Table 7.8**, and result in adjusted curve numbers of 83, 84 and 85 for the 1-, 2- and 10-year storms, respectively.

Table 7.8 - Adjusted CN from Runoff Reduction
Channel and Flood Protection Sheet

	1-year storm	2-year storm	10-year storm
$RV_{Developed}$ (in) with no Runoff Reduction	1.22	1.64	2.89
$RV_{Developed}$ (in) with Runoff Reduction	0.93	1.35	2.59
Adjusted CN	**83**	**84**	**85**

Input data obtained in Tables 7.7 and 7.7 is used in the Natural Resource Conservation Service Technical Release 55 (NRCS TR-55, 1986) Tabular method to calculate discharge hydrographs. Peaks of those hydrographs for the 1-, 2-, and 10-year storms are reported in **Table 7.9**. These values will be used to size the overflow structures and downstream conveyance from the bioretention.

Table 7.9 - Post-development Discharge Peaks to BMP

	1-year storm	2-year storm	10-year storm
Discharge (cfs)	1.52	2.31	4.57

Step 5 - Design of BMP Geometry

The depth of the facility's planting soil should be a minimum or 24", as specified in **Table 7.2**. While this is the minimum allowed, the minimum should not be exceeded except under special circumstances (such as site area constraints), and should be discussed with VDOT during design. Site grading and placement of the facility's overflow structure must ensure a minimum surface ponding depth of 6" and a maximum ponding depth of 12".

Because the proposed design is for a Level 1 facility, using standard values of M_d, G_d, and SS_d in **Equation 7.1** as 2', 1', and 0.5', respectively, yields a storage depth of 1.40'.

$$SD = (0.25)M_d + (0.40)G_d + (1.0)SS_d$$
$$SD = (0.25)(2\text{ ft}) + (0.40)(1\text{ ft}) + (1.0)(0.5\text{ ft}) = 1.40\text{ ft}$$

From Step 3 above, the treatment volume is:

$$T_v = 3{,}746\ ft^3$$

The basin minimum surface area is determined through use of **Equation 7.2**.

$$SA = \frac{[1.0 \times 3{,}746 ft^3 - 0]}{1.40\text{ ft}} = 2{,}676\text{ sq. ft}$$

Note in the above calculation that the upstream treatment volume was assumed to be 0. A coefficient of 1.0 is used when multiplying by the treatment volume since this is a Level 1 facility. If this were part of a treatment train, the volume treated by the upstream BMP would be subtracted from the treatment volume.

In order to prevent short circuiting, for a Level 1 design, the SFP/L ratio is required to be 0.30 or greater. In order to determine an initial estimate of the width and length of the basin to meet this ratio, the following calculations can be performed (initially assuming a rectangular basin).

$$L \times W = 2{,}676\text{ sq. ft}$$

If the overflow structure is centered lengthwise ($0.5L$) along the perimeter of the basin opposite the inflow (side of facility opposite the road shoulder), then a second equation relating the two parameters is:

$$\frac{W}{0.5L} = 0.30$$

Solving the two equations yields:

$$L \times (0.30)(0.5L) = 2{,}676\text{ sq. ft}$$

$$L = 133.6\text{ ft}$$

$W = (0.30)(0.50)(133.6 \text{ ft})$

$W = 20 \text{ ft}$

These calculations yield an initial estimate of 134' x 20' for the basin surface area. However, based on site and right of way constraints, modifications may be required to these preliminary dimensions.

Step 5 - Design of Pretreatment

Level 1 bioretention facilities are required to be pre-treated by one of the methods discussed in ***Section 9.3.7*** *Runoff Pre-treatment*. In this case, pre-treatment forebays will be used to dissipate energy and remove some sediment prior to discharge into the facility.

Pre-treatment forebays are required to contain a minimum of 15% of the treatment volume of the facility. For this case, the volume required is calculated as:

$(0.15)(2{,}676 \, ft^3) = 401 \, ft^3$

This volume can be achieved through many geometric configurations, and should be evaluated to best fit the site grades, channel cross sections, etc. If stone or rip-rap is included within the calculated pre-treatment volume section, the designer must ensure that only voids within the rip-rap are used to calculate available volume.

Step 6 - Underdrains

Underdrains will be designed in accordance with the VDOT Special Provision for Stormwater Miscellaneous (2014). Based on specification in that document, underdrains shall be 6" rigid Schedule 40 PVC with 4 rows of 3/8" (9.5 mm) holes with a hole spacing of 3.25 +/- 0.25". A non-woven geotextile fabric shall be installed over the top of the underdrain, extending 2' to either side prior to installation of the stone layers. Filter fabric shall be non-woven and shall have 0.08" thick equivalent opening size of #80 sieve, and maintain 125 GPM/ft^2 flow rate and meet ASTM D-751 (Puncture strength of 125 lbs), ASTM D-1117 (Mullen Burst Strength of 400 PSI, and ASTM D-1682 (tensile strength of 300 lbs).

Step 7 - Design Overflow Structure

Overflow and conveyance structures must be designed to pass the specified design storm based on functional classification of the road. This includes calculations for overtopping of the check dams by storms of lower recurrence (i.e.

25-, 50-, and 100-year storms). These computations are beyond the scope of this design example. However, the user is directed to the VDOT Drainage Manual for guidance on flood and erosion compliance calculations.

Step 8 - Specify the Number of Vegetative Plantings

Specification of plant materials in a bioretention area should be designed by a landscape architect, or someone with extensive knowledge of plant species. Depending of the planting scenario [one of the six discussed in **Section 7.3.10** and in the Virginia DCR/DEQ Stormwater Design Specification No. 9, Bioretention, Draft, (2013)] trees may or may not be part of the planting plan. Due to the nature of this site (adjacent to an interstate), it is expected that there will be significant salt laden runoff during winter months. Therefore, the plant specialist should ensure that salt resistant varieties are used during plant selection. The goal is to achieve at least 80% cover within a three (3) year period. Select a planting plan, as described above in **Section 7.3.10**.

8.1 Overview of Practice

Dry swales are effectively a modification of bioretention facilities that are designed to fit into long and narrow linear configurations and covered with turf or surface material rather than mulch and ornamental plants. Due to this, dry swales are a desirable BMP option for linear highway projects. Dry swales form a class of both filtration and infiltration BMPs whose function is to improve the quality of stormwater runoff by means of adsorption, filtration, volitilization, ion exchange and microbial decomposition. The soil media and stone bed also contribute to partial runoff volume reduction as calculated through the runoff reduction methodology.

Dry swales may be configured as a *Dry Conveyance Swale* or a *Dry Treatment Swale*. The primary difference between the two is that the Dry Conveyance Swale is used to convey runoff in the direction of a downstream discharge point along a linear impervious area, while a Dry Treatment Swale may treat more non-linear impervious areas and may be used instead of a bioretention facility due to space constraints. Both configurations are used to store and filter the calculated treatment volume through soil media that is similar to that used for bioretention practices, having a high sand content. Based on soil testing results, the practice may be designed to infiltrate into underlying soils, but on VDOT projects, requires the installation of an underdrain. This underdrain will discharge to grade with appropriate outlet protection or to a local storm sewer system, such that water enters the storm sewer after it has filtered through the soil media. **Figures 8.1 - 8.5** present the general configuration of Level 1 and 2 dry swales, to be installed in accordance with VDOT Special Provision for Dry Swales (2014).

The Virginia Stormwater Design Specification No. 10, Dry Swale, Draft (DCR/DEQ, 2013) lists several dry swale applications, including road medians and shoulders, in commercial setbacks, parking lots, and along buildings to accept and treat runoff from roofs. Due to the linear nature of the practice and the relatively high pollutant removal efficiency, dry swales are applicable on a wide array of transportation related projects.

Dry Swales can be an important part of the stormwater quality treatment train, but they require special design considerations to minimize maintenance. Otherwise, they can become a maintenance burden, particularly if sediment accumulates within the channel or if flows cause erosion within the channel. Good design can eliminate or at least minimize such problems.

Also, while check dams or inter-channel berms may be useful flow control devices, they can also increase the maintenance burden, clogging quickly with sediment and debris that must be removed to ensure conveyance of design flows. Therefore, only use these devices when they are absolutely necessary.

Table 10.1 - Summary of Stormwater Functions Provided by Dry Swales

Stormwater Function	Level 1 Design	Level 2 Design
Annual Runoff Volume Reduction (RR)	40%	60%
Total Phosphorus (TP) EMC Reduction[1] by BMP Treatment Process	20%	40%
Total Phosphorus (TP) Mass Load Removal	52%	76%
Total Nitrogen (TN) EMC Reduction[1] by BMP Treatment Process	25%	35%
Total Nitrogen (TN) Mass Load Removal	55%	74%
Channel Protection	Use the Virginia Runoff reduction Method (VRRM) Compliance Spreadsheet to calculate the Curve Number (CN) Adjustment ***OR*** Design for extra storage (optional; as needed) on the surface, in the engineered soil matrix, and in the stone/underdrain layer to accommodate a larger storm, and use NRCS TR-55 Runoff Equations[2] to compute the CN Adjustment.	
Flood Mitigation	Partial. Reduced Curve Numbers and Time of Concentration	
[1] Change in the event mean concentration (EMC) through the practice. The actual nutrient mass load removed is the product of the removal rate and the runoff reduction rate (see Table 1 in the *Introduction to the New Virginia Stormwater Design Specifications*). [2] NRCS TR-55 Runoff Equations 2-1 thru 2-5 and Figure 2-1 can be used to compute a curve number adjustment for larger storm events, based on the retention storage provided by the practice(s).		

Sources: CWP and CSN (2008), CWP, 2007

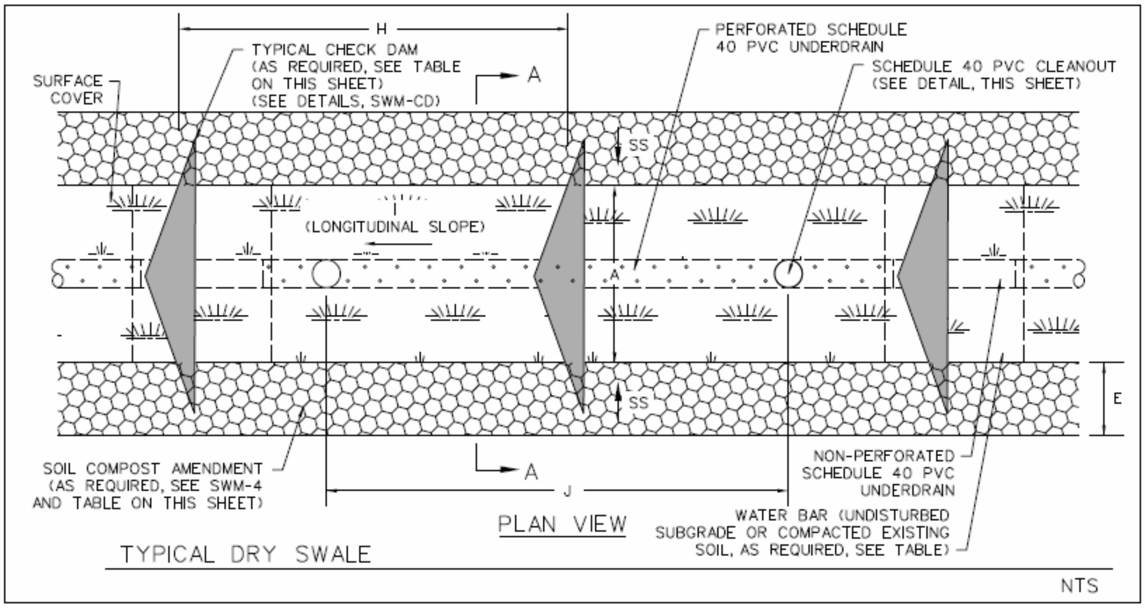


Figure 8.1 - Typical Dry Swale [Plan View]
VDOT BMP Standard Detail SWM-8, 2014

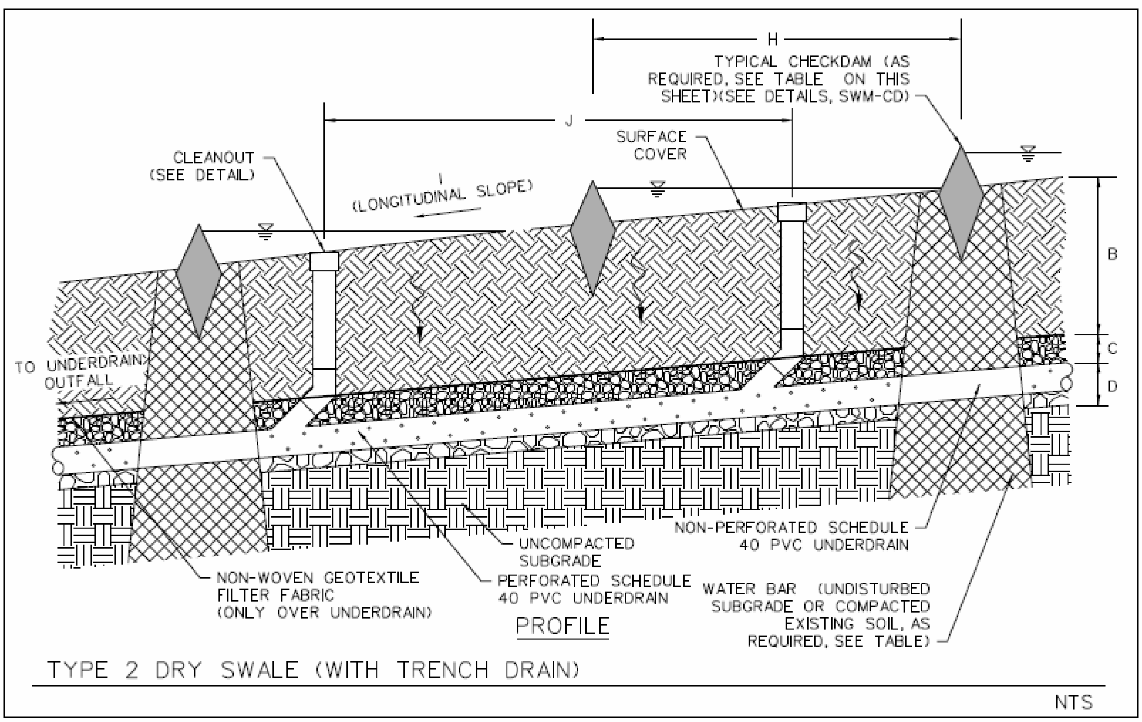


Figure 8.2 - Typical Dry Swale – Level 1 [Profile View]
VDOT BMP Standard Detail SWM-8, 2014

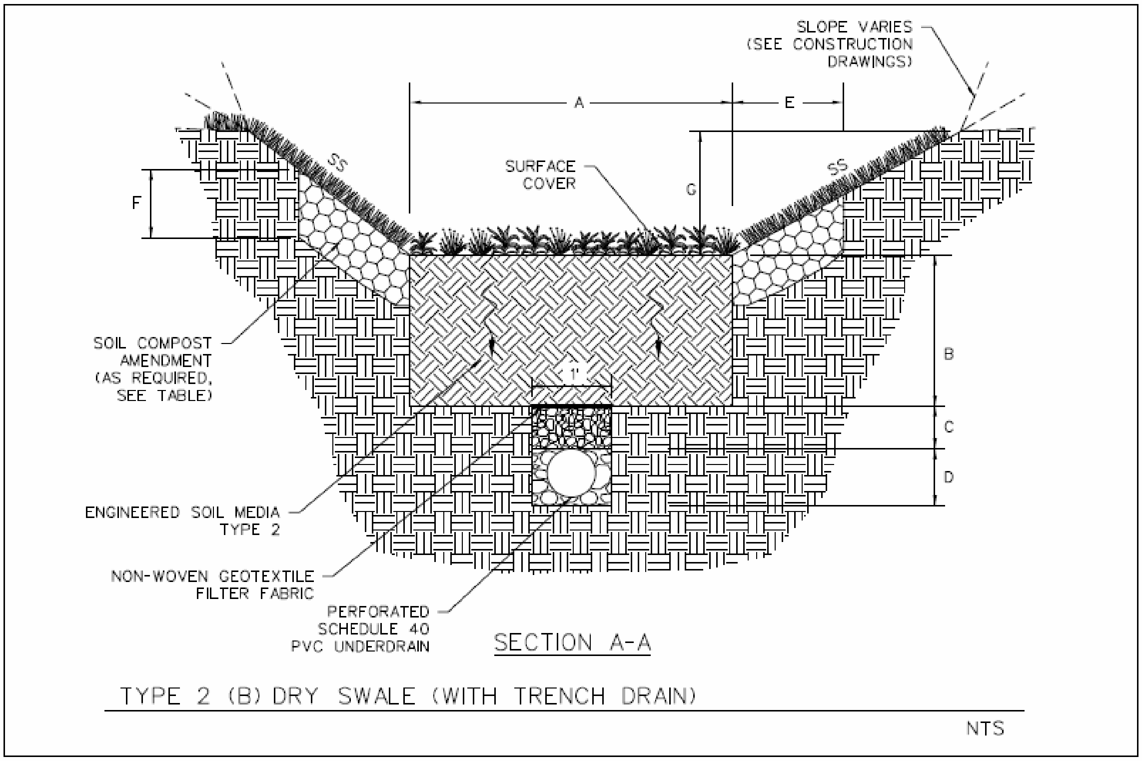


Figure 8.3 - Typical Dry Swale – Level 1 [Cross-section View]
VDOT BMP Standard Detail SWM-8, 2014

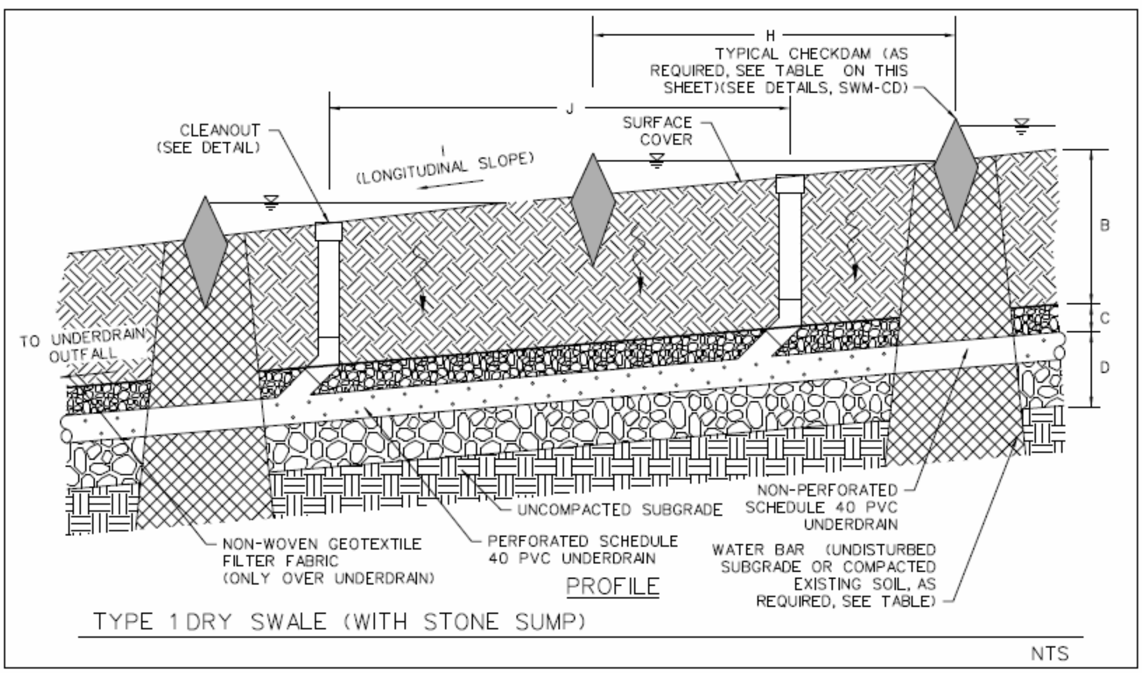


Figure 8.4 - Typical Dry Swale – Level 2 [Profile View]
VDOT BMP Standard Detail SWM-8, 2014

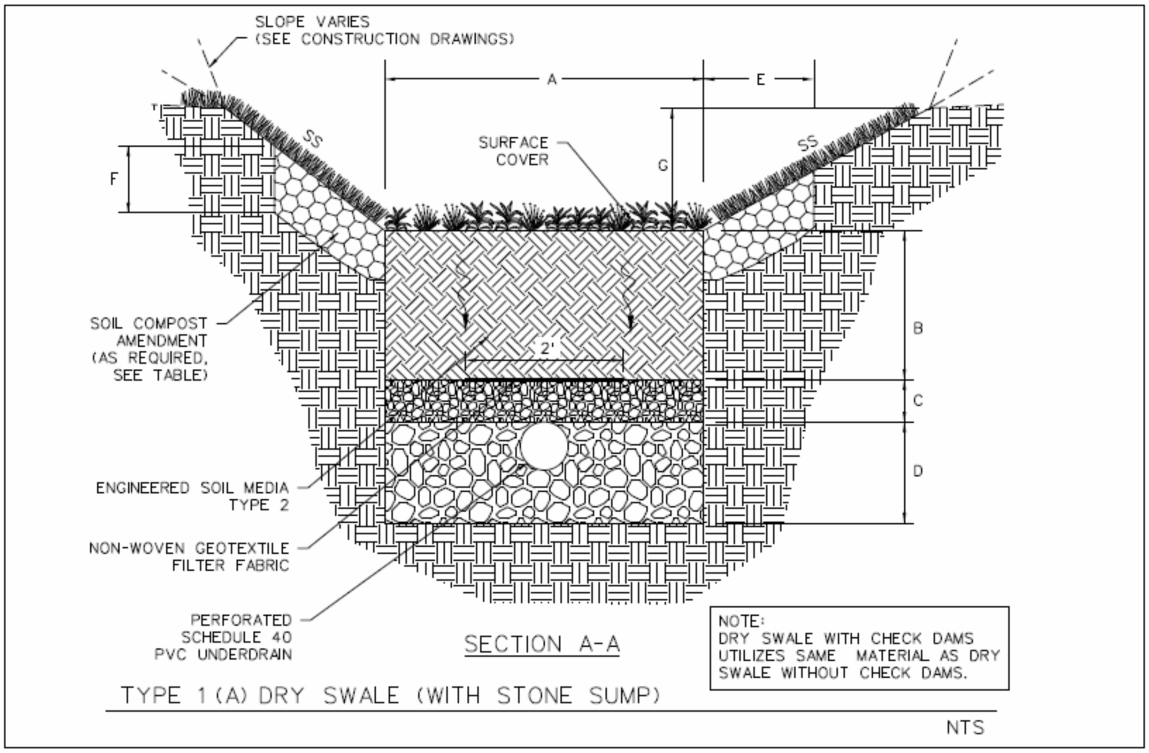


Figure 8.5 - Typical Dry Swale – Level 2 [Cross-section View]
VDOT BMP Standard Detail SWM-8, 2014

8.2 Site Constraints and Siting of the Facility

When a dry swale is proposed the designer must consider a number of site constraints to ensure that the practice is applicable to the suggested use.

8.2.1 Maximum Contributing Drainage Area (CDA)

The maximum drainage area to a dry swale should be limited to 5 acres. Past this threshold, there is an increasing likelihood that the velocity of flow in the swale will reach a point causing difficulty to prevent erosion in the channel and hydraulic overloading through the underlying sections. It is important to design dry swales within the limits established for CDAs. Too much or too little runoff can result in performance issues and the need for subsequent repairs or reconstruction.

8.2.2 Site Slopes

Dry swales are suited to sites with slopes up to 4%, with a preference for slopes 2% or less. Steep upstream slopes are typically indicative of higher runoff velocities and higher probability of erosion and sediment transport into the facility, which is to be avoided. Steep downstream slopes can be subject to seepage and failure, and should be avoided in close proximity to the edge of the dry swale when possible.

8.2.3 Site Soils

The soil mix of a dry swale is governed by specific guidelines in the VDOT Special Provision for Dry Swales (2014).

Soil conditions do not endorse nor preclude the use of dry swales; however, they do determine if a liner must be installed. Therefore, in situ soil infiltration rate is a critical design element in a dry swale for a level 2 design since the underdrain is situated above the stone sump. When such a facility is proposed, *a subsurface analysis and permeability test is required in support of a level 2 design*. The required subsurface analysis should investigate soil characteristics to a depth of no less than 3' below the proposed bottom of the engineered media. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the final design characteristics of the proposed facility.

The soil infiltration rate should be measured according to the requirements in VDOT Special Provision for Stormwater Miscellaneous (2014) – see "Infiltration and Soil Testing". Soil infiltration rates which are deemed acceptable for dry swales are typically greater than *0.50 in/hr*. Soils exhibiting a clay content of greater than 20% and silt/clay content of more than 40% are typically unacceptable for level 2 dry swales. Sites categorized as stormwater hotspots should not be used for infiltrative bioretention facilities due to the higher likelihood of groundwater contamination.

8.2.4 Depth to Water Table

Dry swales should not be installed on sites with a seasonally high groundwater table. Inadequate separation between the BMP bottom and the surface of the water table may result in contamination of the water table. This potential contamination arises from the inability of the soil underlying the BMP to filter pollutants prior to their entrance into the water table. Additionally, a high water table can flood the media underlying the dry swale and render it inoperable during periods of high precipitation and/or runoff. A separation distance of no less than 2' is required between the bottom of the dry swale and the surface of the *seasonally* high water table unless the site is located in coastal plain residential settings where the distance may be reduced to 1'. Unique site conditions may arise which require an even greater separation distance.

8.2.5 Separation Distances

Setbacks from buildings and streets should be in accordance with the distances shown in **Table 8.1**. A 50' minimum separation from wells is required. Additionally, a 20' minimum separation from septic drain fields is required when using a liner, and this is increased to 35' if a liner is not present. Dry swales must maintain a minimum down-gradient separation of 5’ from wet utilities; however, dry utilities may pass **beneath** a dry swale if utilities are encased. Bottom elevations of swales should be a minimum of 1’ below the bottom elevation of an adjacent road or parking lot bed.

8.2.6 Karst Areas

Infiltrative dry swales should not be used in karst areas, or in areas with a prevalence of bedrock, or fractured rock. However, a dry swale with underdrain and liner may be considered if a separation requirement of 2’ is maintained between the bottom of the facility and the top of rock. In addition, setbacks between structures and karst features should be discussed with VDOT, and should generally be larger than standards shown in **Table 8.1**.

Table 8.1 - Dry Swale Design Criteria
Virginia Stormwater Design Specification No. 10, Dry Swale, Draft (DCR/DEQ, 2013)

<table>
<tr><th>Level 1 Design (RR:40; TP:20; TN:25)</th><th>Level 2 Design (RR:60; TP:40; TN: 35)</th></tr>
<tr><td>Sizing (Sec. 8.3.1):
Surface Area (ft^2) = (T_v– the volume reduced by an upstream BMP) / Storage depth [1]</td><td>Sizing (Sec. 8.3.1):
Surface Area (ft^2) = {(1.1)(T_v) – the volume reduced by an upstream BMP } / Storage Depth [1]</td></tr>
<tr><td>Effective swale slope ≤ 2%[2]</td><td>Effective swale slope ≤ 1%[2]</td></tr>
<tr><td>Media Depth: minimum = 18”; Recommended maximum = 36”</td><td>Media Depth minimum = 24”
Recommended maximum = 36”</td></tr>
<tr><td>Sub-soil testing (Section 8.3.4): not needed if an underdrain is used; min. infiltration rate must be > 1/2 in/hr to remove the underdrain requirement;</td><td>Sub-soil testing (Section 8.3.4): one soil profile and two infiltration tests for dry swales up to 50 LF; add one additional infiltration test for dry swales up to 100 LF; Refer to Section 8.3.4 for swales longer than 100 LF; min. infiltration rate must be > 1/2 in/hr to remove the underdrain requirement</td></tr>
<tr><td>Underdrain (Section 8.3.8): Schedule 40 PVC with clean-outs</td><td>Underdrain and Underground Storage Layer (Section 8.3.8): Schedule 40 PVC with clean outs, and a minimum 12” stone sump below the invert; OR none if the soil infiltration requirements are met (see Section 8.3.4)</td></tr>
<tr><td colspan="2">Media (Section 8.3.10): supplied by the vendor; tested for an acceptable hydraulic conductivity (or permeability) and phosphorus content [3]</td></tr>
<tr><td colspan="2">Inflow: sheet or concentrated flow with appropriate pre-treatment</td></tr>
<tr><td colspan="2">Pre-Treatment (Section 8.3.9): a pre-treatment cell, grass filter strip, gravel diaphragm, gravel flow spreader, or another approved (manufactured) pre-treatment structure.</td></tr>
<tr><td>On-line design</td><td>Off-line design or multiple treatment cells</td></tr>
<tr><td>Turf cover</td><td>Turf cover, with trees and shrubs</td></tr>
<tr><td colspan="2">Building Setbacks [4] :
10’ if down-gradient from building or level (coastal plain); 50’ if up-gradient.
(Refer to additional setback criteria in Section 8.2.5)</td></tr>
<tr><td colspan="2">[1] The storage depth is the sum of the Void Ratio (V_r) of the soil media and gravel layers multiplied by their respective depths, plus the surface ponding depth (Refer to Section 8.3.1)
[2] The effective swale slope can be achieved through the use of check dams – 12” height maximum
[3] Refer to VDOT Special Provision for Dry Swales (2014)
[4] These are recommendations for simple building foundations. If an in-ground basement or other special conditions exist, the design should be reviewed by a licensed engineer. Also, a special footing or drainage design may be used to justify a reduction of the setbacks noted above.</td></tr>
</table>

8.2.7 Placement on Fill Material

Dry swales that are to be constructed on or nearby fill sections shall be discussed with VDOT prior to design due to the possibility of creating an unstable subgrade. Fill areas are vulnerable to slope failure along the interface of the in-situ and fill material. The likelihood of this type of failure is increased when the fill material is frequently saturated, as anticipated with a dry swale. The practice may be used if an impermeable liner and underdrain is present, with the approval of VDOT.

8.2.8 Existing Utilities

Dry swales can often be constructed over existing *vacant* easements, provided permission to construct the strip over these easements is obtained from the utility owner *prior* to design of the strip. However, conflicts with utilities should be avoided where possible due to concerns over future access and maintenance to both the swale and utility lines.

8.2.9 Wetlands

When the construction of a dry swale is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation in their vicinity.

8.2.10 Floodplains

Dry swales should not be located in 100-year floodplains for project areas as defined by applicable FEMA flood maps.

8.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing a dry swale for improvement of water quality. Cross-section details for specific design features, including material specifications, can be found in the VDOT BMP Standard SWM-8: Dry Swale (2014).

8.3.1 Swale Size

For preliminary sizing and space planning, a general rule of thumb is that the surface area of the facility will be 3%-10% of the contributing drainage area (dependent on imperviousness and design level).

Equation 8.1 describes the dry swale design equivalent subsurface storage depth as:

$$SD = (0.25)M_d + (0.40)G_d \qquad \textbf{(8.1)}$$

where,

SD = equivalent storage depth (ft);
M_d = media depth (ft);
G_d = gravel depths (ft).

Coefficients in front of each correspond to the void ratio associated with each layer, as defined in Virginia DEQ Stormwater Design Specification No.10, (2013, et seq). In a typical Level 1 design, the depths for these two layers (M_d and G_d) are 1.5' and 0.25', respectively, which yields an effective subsurface storage depth of 0.5'; however, equation 10.1 should be used to calculate the design-specific equivalent storage depth if a situation results in the modification of the standard design.

Similarly, a Level 2 dry swale design also uses **Equation 8.1**. However, in a Level 2 design, the depths for the two subsurface layers (M_d and G_d) are typically 2' and 1', respectively, which yields an equivalent storage depth of 0.90'. Again, **Equation 8.1** should be used to calculate the actual equivalent depth if a situation results in the modification of this standard design.

Equation 8.2 below is used to calculate the required surface area, SA, of the Level 1 and Level 2 swales described above. If the dry swale includes check dams to decrease the effective swale longitudinal slope, or to simply create storage volume, it is recommended that the designer estimate the design width of the swale, compute the storage volume retained by the check dams (V_{ss}), and subtract it from the BMP design treatment volume, T_v, of Dry Swale. This will be an iterative computation if the design width of the Dry Swale is different from that which is used to estimate the surface storage.

Equation 8.2 describes the calculation of the required minimum dry swale surface area as:

$$SA = \frac{[T_v - V_{ss}]}{SD} \qquad \textbf{(8.2)}$$

where,

SA = surface area (ft^2)
T_v = computed treatment volume (ft^3), **Section 1, Equation 1.1** (C_v = 1.0 for Level 1 and 1.1 for level 2)
V_{ss} = volume of surface storage (ft^3)
SD = storage depth (ft), as computed by **Equation 8.1**.

8.3.2 Media Depth

Media depth should be determined from **Table 8.1**, according to the specified design level (Level 1 or Level 2).

8.3.3 Surface Ponding Depth

The depth of ponding on the facility surface should be restricted to no more than 12" at the most downstream point to preclude the development of anaerobic conditions within the planting soil.

8.3.4 Soil Infiltration Rate

Level 1 designs do not require soil infiltration rate testing due to the presence of an underdrain. Subsoil infiltration rates must exceed 1/2 in/hr for Level 2 dry swales if an underdrain is not installed. The soil infiltration rate should be measured according to the requirements in VDOT Special Provision for Stormwater Miscellaneous (2014) – see "Infiltration and Soil Testing". A minimum of one soil profile and infiltration test shall be collected if attempting obtain level 2 credit. One additional infiltration test shall be necessary for dry swales between 50' and 100' long, with one additional soil profile for each 100' of length above the first 100' and one additional infiltration test for each 50' of length above the first 100'.

8.3.5 Dry Swale Geometry

Dry swale cross-sectional geometry is assumed to be trapezoidal or parabolic. Side slopes are to be 3:1 or flatter. Flatter slopes (5H:1V) act to enhance pre-treatment of sheet flow entering the swale. The minimum bottom width should be between 2' and 4'. Swales wider than 6' require incorporation of check dams, berms, level spreaders, etc. to prevent excessive erosion of the bottom. Recommended bottom slopes are less than 2% for a Level 1 design and less than 1% for a Level 2 design. The minimum recommended slope for an inline dry swale is 0.5%. Off line dry swales may function similarly to bioretention facilities and have very flat (less than 0.5%) slopes.

8.3.6 Check Dams

Check dams (see **Figure 8.2**) are installed within dry swales to provide upgrade impoundment of runoff volume for filtration through subsurface media. The height of the check dam should not exceed 18" above the normal channel elevation. Check dams shall be securely entrenched into the swale side slopes to prevent outflanking during high intensity storms. Soil plugs can reduce the chance for a blow out or erosion of the media under the dams. They are typically used on slopes of 4% or greater or when maximum height (18") check dams are used. A weir should be installed in the top of the dam to pass design storms, with appropriate armoring down the back side of the dam.

Check dams may also be used for velocity reduction. Velocities in dry swales should not exceed 3 fps to prevent erosion. Typical check dam spacing to achieve effective swale slopes may be found in the **Section 3, Table 3.3.**

8.3.7 Drawdown

Drawdown of the treatment volume should occur within a 6 hour period. Filtration may be accomplished through the soil media mix or in situ soils with verified adequate permeability. This drawdown time can be achieved by using the soil media mix specified in the VDOT Special Provision for Dry Swales (2014) and an underdrain along the bottom of the swale, or native soils with adequate permeability, as verified through testing.

8.3.8 Underdrains

Underdrains shall be installed in accordance with material, size, and installation specifications found in the VDOT Special Provision for Stormwater Miscellaneous (2014), VDOT Special Provision for Dry Swales (2014), and VDOT Standard Detail SWM-8: Dry Swales (2014).

8.3.9 Runoff Pre-treatment

Dry swales *must* be preceded upstream by some form of runoff pre-treatment. For dry conveyance swales, pre-treatment typically consists of a 10' wide (minimum) grass filter strip. Pre-treatment for dry treatment swales is typically integrated at inflow locations along the swale. Roadways and parking lots often produce runoff with high levels of sediment, grease, and oil. These pollutants can potentially clog the pore space in the media mix, thus greatly reducing its pollutant removal performance. The selection of runoff pre-treatment is primarily a function of the type of flow entering the facility, as discussed below. Proper pre-treatment preserves a greater fraction of the Treatment Volume over time and prevents large particles from clogging orifices, filter material, and infiltration sites. Selecting an improper type of pre-treatment or designing and constructing the pre-treatment feature incorrectly can result in performance and maintenance issues.

a. ***Pre-treatment Forebay***: These cells act as forebays to allow sediment to settle out of stormwater runoff prior to entering the dry swale. Concentrating sediment settling in one location simplifies maintenance significantly. In addition, a forebay is used as an energy dissipater to reduce the velocity of incoming stormwater runoff and prevent erosive damage within the treatment cell. A pre-treatment cell must have a 2:1 length to width ratio and a minimum storage volume of at least 15% of the dry swale total treatment volume. Installation shall be in accordance with VDOT BMP Standard SWM-PT: Pre-treatment (see Pre-treatment Forebay).

b. ***Grass Filter Strips***: Runoff entering a dry swale as *sheet flow* may be treated by a grass filter strip. The purpose of the grass filter strip/energy dissipation area is to reduce the erosive capabilities of runoff prior to its entrance into the bioretention area. The recommended length of the grass filter strip is a function of the land cover of the contributing drainage area and its slope. The recommended minimum length of the grass filter strip should not be less than 10’ when using the maximum side slope of 5:1. An alternative design may be used that integrates road shoulders, requiring a 5’ minimum grass filter strip at 20:1 (5%), that is combined with 3:1 (or flatter) side slopes of the swale to provide pre-treatment.

c. ***Gravel Diaphragms***: These pre-treatment measures are typically installed along the edge of a swale with the purpose of evenly distributing flow along the length of the swale and, of course, to pre-treat that flow. The diaphragm should be oriented perpendicular to flow, with a drop of 2”-4” from adjacent edge of the impervious surface, as shown in VDOT BMP Standard SWM-PT: Pre-treatment (Gravel Diaphragm).

d. ***Pea Gravel Flow Spreader***: These measures are typically located at points of concentrated inflow, such as curb cuts, etc. There should be a 2”-4” drop from the adjacent impervious surface. Gravel/stone should extend along the entire width of the opening, creating a level stone weir at the bottom of the channel. Installation shall be in accordance with VDOT BMP Standard SWM-PT: Pre-treatment (Gravel Flow Spreader).

8.3.10 Filter Media

Filter media shall be installed as specified in VDOT Special Provision for Dry Swale (2014). It is critical to specify and install the correct type and depth of filter media; doing otherwise is likely to result in performance and maintenance issues.

8.3.11 Overflow

The dry swale shall be designed to convey the 10-year storm within the banks with a minimum of 3” of freeboard. Overflow from the dry swale may discharge into an overflow structure (such as a VDOT Standard DI-7), and overflow channel, or an overflow pipe. Discharge of overflow shall be to an adequate channel per state and local requirements.

8.3.12 Surface Cover

Surface cover shall be in a 3”- 4” layer of topsoil having a loamy sand or sandy loam texture, with less than 5% clay content, a pH (corrected) of 6-7, and at least 2% organic matter. Cover will typically be turf or river stone, but may also include bioretention plants, if required and/or approved by VDOT.

Salt tolerant grass and plant species should be used in order to withstand concentrations of deicing solution used to treat roads during the winter.

8.4 Design Example

This section presents the design process applicable to dry swales serving as water quality BMPs. The pre- and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 11 of *the Virginia Stormwater Management Handbook, 2nd Edition*, Draft (DCR/DEQ, 2013) for details on hydrologic methodology.

A Level 1 dry conveyance swale design is being proposed to treat runoff from a 2,100' long section of a road improvement project along I-64 near Waynesboro. The longitudinal slope along this section of I-64 is approximately 1.5%. Runoff from the crown to the side of the expansion for this section of the project can be redirected to a BMP location having a total cumulative contributing drainage area (at the downstream end of swale) of 2.30 acres. The current lane (on BMP side of crown) and shoulder represent 1.40 acres of impervious area (1.00 acres overlaying HSG B soils and 0.40 acres overlaying HSG C soils). In addition there is 0.90 acres of turf covered shoulder that drains to the area (0.60 acres in HSG B soils and 0.30 acres in HSG C soils).

The proposed widening will add an additional 0.60 acres of impervious area (total 1.40 acres HSG B and 0.60 acres HSG C), and reduce turf area post-development to 0.30 acres (0.20 acres HSG B and 0.10 acres HSG C). In the post-development condition, the time of concentration has been calculated to be 13 minutes.

Geotechnical investigations reveal compacted soil with a high clay content. Lab tests confirm that infiltration rates necessary for a Level 1 design a at this location. The project site does not exhibit a high or seasonally high groundwater table.

Step 1 - Enter Data into VRRM Spreadsheet

The required site data from **Table 8.2** is input into the **VRRM Spreadsheet for Redevelopment (2014)** to compute load reductions for this linear project, resulting in site data summary information shown in **Table 8.3**. Note that using the redevelopment spreadsheet, the required reduction for linear projects is computed as the sum of the Post-Redevelopment Load and the Post-Development Load minus 80% of the Predevelopment Listed load.

Table 8.2 - Hydrologic Characteristics of Example Project Site

		Impervious		Turf	
Pre	**Soil Classification**	**HSG B**	**HSG C**	**HSG B**	**HSG C**
	Area (acres)	0.00	0.00	0.60	0.30
Post	**Soil Classification**	**HSG B**	**HSG C**	**HSG B**	**HSG C**
	Area (acres)	0.40	0.20	0.20	0.10

It is important to note that the values in **Table 8.2** are only the values for the disturbed area of the project. Although other run-on areas (2.30 acres total) were described in the problem statement, they are not part of the disturbed area, and should not be entered as such in the VRRM Spreadsheet to compute required reductions (**Table 8.3**).

Table 8.3 - Summary Data from VRRM Site Data Analysis

Site Rv	0.63
Post-development TP Load (lb/yr)	1.52
Total TP Load Reduction Required (lb/yr)	1.12

The drainage area is for this outfall is roughly symmetrical, with flow approaching a common central discharge point from both directions. The Level 1 dry swale will be used to treat runoff from one direction only (a total of 1.20 acres) for water quality compliance. Note that the VRRM Spreadsheet will warn the user that the area (1.20 acres) exceeds the disturbed area (1.05 acres); however, it is acceptable to treat adjacent run-on area as part of the project. Appropriate data for post-development conditions is input into the VRRM Spreadsheet Drainage Area tab, yielding compliance results summarized in **Table 8.4.**

Table 8.4 - Summary Data from Level 1 Dry Swale Treatment

Total Impervious Cover Treated (acres)	1.00
Total Turf Area Treated (acres)	0.20
Total TP Load Reduction Achieved in D.A. A (lb/yr)	1.17

In this case, the total phosphorus reduction required is 1.12 lbs/yr. The estimated removal is 1.17 lbs/yr; therefore, the target has been met.

Step 2 - Compute the Required Treatment Volume

The treatment volume can be calculated using **Section 1, Equation 1** or taken directly from the VRRM Spreadsheet Drainage Area tabs. For this example, the reported treatment volume on the drainage area tab (treating the 1.20 acre area described by data in **Table 8.4**) is 3,594 ft^3.

Step 3 - Enter Data in Channel and Flood Protection Tab

Hydrologic computations for required design storms for flood and erosion compliance are not shown as part of this example. The user is directed to the VDOT Drainage Manual for appropriate levels of protection and design requirements related to erosion and flood protection. However, hydrologic computations are necessary to compute peaks to design components of the Dry Swale. In particular, the 10-year 24-hour design storm is used to size the rectangular notch is check dams.

Values for the 1-, 2-, and 10-year 24-hour rainfall depth should be determined from the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 and entered into the "Channel and Flood Protection" tab of the spreadsheet. For this site (Lat 38.0522, Long 78.9162), those values are shown in **Table 8.5**. For the 1-, 2-, and 10-year 24-hour storms, adjusted curve numbers supplied by the VRRM spreadsheet should be used for conveyance and overflow sizing related to the proposed BMP.

Table 8.5 - Rainfall Totals from NOAA Atlas 14

	1-year storm	2-year storm	10-year storm
Rainfall (inches)	2.58	3.12	4.64

Table 8.6 - Adjusted CN from Runoff Reduction Channel and Flood Protection Sheet

	1-year Storm	2-year Storm	10-year Storm
$RV_{Developed}$ (in) with no Runoff Reduction	1.49	2.27	3.74
$RV_{Developed}$ (in) with Runoff Reduction	1.16	1.94	3.41
Adjusted CN	**87**	**88**	**89**

The values reported in Table 8.6 are only valid for the drainage area served by the proposed dry swale. The remaining portion of the site drainage area should use the appropriate curve numbers for those areas.

Input data is used in the Natural Resource Conservation Service Technical Release 55 (NRCS TR-55, 1986) Tabular method to calculate discharge hydrographs. (**Note that other hydrologic methodologies are suitable-see VDOT Drainage Manual, Hydrology for guidance**). Peaks of those hydrographs for the 1-, 2-, and 10-year storms are reported in **Table 8.7**.

Table 8.7 - Post-development Discharge Peaks (cfs) based on Adjusted CN

	1-year storm	2-year storm	10-year storm
Discharge (cfs)	1.96	2.77	4.92

Step 4 - Compute Minimum Basin Floor Area

Because the proposed design is for a Level 1 facility, using standard values of M_d and G_d in **Equation 8.1** as 1.5' and 0.25', respectively, yields an equivalent storage depth of 0.48'.

$$SD = (0.25)M_d + (0.40)G_d$$
$$SD = (0.25)(1.5\text{ ft}) + (0.40)(0.25\text{ ft}) = 0.48\text{ ft}$$

Although not required (due to the low slope of 1.5%), check dams will be installed to increase surface storage and decrease the required width of the dry swale. Based on the check dam spacing table in **Section 3, Table 3**, to achieve an effective channel slope of 1.0%, a spacing of 67' to 200' should be used if the actual channel slope is 1.5%. If 3:1 side slopes are assumed, the surface storage volume may be approximated by:

$$V_S = \frac{H^2}{S}\left(\frac{W}{2} + H\right) \qquad \textbf{(8.3)}$$

where,

V_S = surface volume in ft^3 (between check dams);
W = channel bottom width (ft);
H = check dam height (ft);
S = channel slope (ft/ft);
****Note: Equation 8.3 has been derived specifically for the geometry used in this example. It is not a general equation that may be used for all applications.***

An assumed bottom width must be used ultimately to verify storage requirements. One way to get an initial estimate of bottom width is to base the minimum width on the required with of the weir through the check dam that is required to pass to the 10-year storm (VDOT Special Provision for Stormwater Miscellaneous (2014) requires a central weir to pass the 10-year storm for in-line check dams). The weir (assumed to be rectangular) discharge, Q, can be calculated by:

$$Q = 3.33(L - 0.2h)h^{3/2} \qquad \textbf{(8.4)}$$

where,

Q = design flow (cfs)
L = weir length
h = height of flow over the weir.
3.33 is used for weir coefficient for rectangular broad crested weir

The notch weir length should be a minimum of 1' less (6" clearance on each side) than the channel bottom width to reduce the chance of erosion to channel banks.

The weir should also be centered in the check dam. An assumed h of 0.50' (6") and the Q_{10} discharge of 4.92 cfs (**Table 8.7**) is used in rearranged **Equation 8.4** to compute weir length:

$$L = \frac{Q_{10}}{3.33h^{\frac{3}{2}}} - 0.2h = \frac{4.92\ cfs}{3.33(0.50ft)^{3/2}} - 0.2(0.50\ ft) = 4.1\ ft$$

Adding 1' clearance to the notch weir length (to prevent erosion), yields a minimum bottom width of ~5.1'. Therefore, 5.1' will be used as the assumed bottom width for surface storage computations.

The surface storage requirement is based on volume behind check dams, and must initially be calculated by assuming the number of check dams necessary for the application. If nine check dams are assumed, then the length of dry swale media bed is estimated as 67' (distance between dams) x 9 (dams), or approximately 603'. The 67' distance assumption stems from the spacing criteria shown in Section 3, Table 3.3, which suggests of spacing of 67' when on a 1.5% swale slope to decrease the effective slope to 0%. If the media bed is assumed to extend across the entire width of the channel bottom, the required minimum surface storage can be calculated as:

$$3{,}594\ ft^3 - (0.48ft)(603ft)(5.10ft) = 2{,}118\ ft^3$$

The 2,118 ft^3 is divided between storage areas behind each proposed check dam. The volume calculated above after being divided by 9 dam areas (235 ft^3) is equivalent to V_S from **Equation 8.3**. Substituting into that equation, and assuming an effective check dam height, H, of 12" (1'), the required minimum surface storage volume, V_S is computed as:

$$235\ ft^3 = \frac{H^2}{S}\left(\frac{W}{2} + H\right) = \frac{(1.0\ ft)^2}{0.015}\left(\frac{W}{2} + 1.0\ ft\right)$$

$$W = 5.1\ ft$$

This value confirms the assumed channel width that was based on the weir length calculated by **Equation 8.4** (with 1' added for erosion clearance) of 5.1'. Therefore, the assumption of 9 check dams is valid, and produces sufficient surface storage for the design.

The final treatment bed will encompass an area along the channel of 600' x 5.10', with 9 check dams spaced evenly at 67' intervals. A 4.1' wide weir will be centered in each check dam with a crest elevation of 12" above channel bottom, and height of 6". The total height of the check dam will be the maximum allowable height of 18" (**Figure 8.6**).

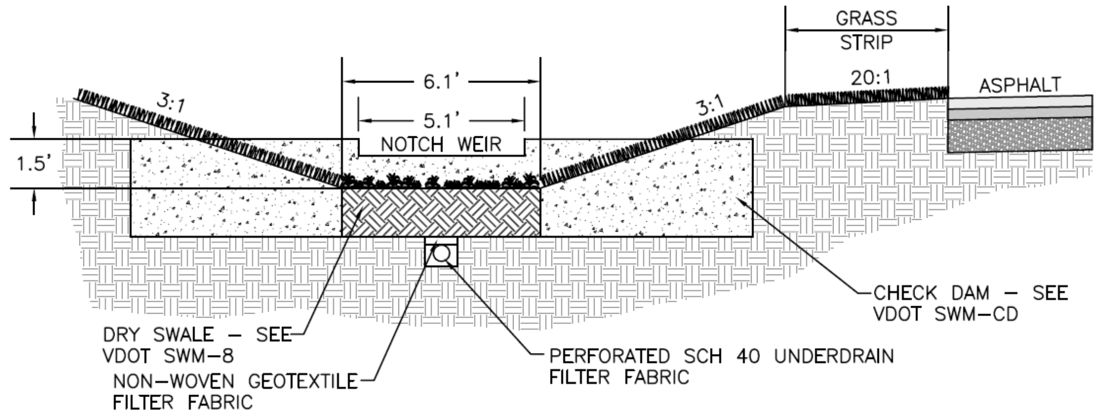


Figure 8.6 - Cross-section through Downstream Check Dam

Step 5 - Pre-treatment

Pre-treatment requirements will be met through the use of a grass filter for sheet flow. The filter is shown in **Figure 10.3** as the 5' 20:1 shoulder along the pavement, with a 3:1 slope to the bottom of the swale. No other pre-treatment is required for this installation.

Step 6 - Specify Media Depth

The depth of the facility's filtering media should be a minimum or 18", as specified in **Table 10.2**. for a Level 1 design. While this is the minimum allowed, the minimum should not be exceeded except under special circumstances, and should be discussed with VDOT during design. Media type and specifications are as found in the VDOT Special Provision for Dry Swales (2014)**.**

Step 7 - Underdrains

Based on VDOT guidelines, an underdrain is required for the installation. Due to the minimal width of the facility, a single 6" perforated underdrain pipe will be required along the length of the facility. Discharge will be routed to a storm sewer, adequate channel, or stormwater management facility downstream. Observation wells and cleanouts shall be placed along the length of the channel for observation and maintenance. Cleanouts should be placed at a minimum spacing of approximately 100'.

Step 8 - Seeding

The grass chosen should be able to withstand both wet and dry periods. The user is directed to the *Virginia Erosion Control Handbook (1992)* permanent seeding chapter for guidance. The selected seed mix combination should provide low maintenance, tolerance of moisture conditions, and be tolerant to high salt concentrations during the winter months.

Step 9 - Design of Overflow and Conveyance Structures

Overflow and conveyance structures must be designed to pass the specified design storm based on functional classification of the road. This includes calculations for overtopping of the check dams by storms of lower recurrence (i.e. 25-, 50-, and 100-year storms). These computations are beyond the scope of this design example. However, the user is directed to the VDOT Drainage Manual for guidance on flood and erosion compliance calculations.

9.1 Wet Swales - Overview of Practice

Wet swales are effectively a hybrid treatment device that is a cross between a swale and a constructed wetland. The purpose of the practice is to intercept the high groundwater table and detain runoff. Wet swales provide pollutant removal though gravitational settling, pollutant uptake, and microbial activity.

Due to the presence of shallow groundwater present in locations where the practice is viable, wet swales do not provide a runoff volume reduction credit and, therefore, are typically used in a treatment train. According to Virginia Stormwater Design Specification No. 11, Wet Swales, Draft (DCR/DEQ, 2013), use of wet swales "**should therefore be considered only if there is remaining pollutant removal required after all other upland runoff reduction options have been considered and properly credited.**"

The Virginia Stormwater Design Specification No. 11, Wet Swales, Draft (DCR/DEQ, 2013) describes wet swales as well-suited for use in linear applications to treat highway or residential street runoff.

Wet Swales can be an important part of the stormwater quality treatment train, but they require special design considerations to minimize maintenance. Otherwise, they can become a maintenance burden, particularly if sediment accumulates within the channel or if flows cause erosion within the channel. Good design can eliminate or at least minimize such problems.

Also, while check dams or inter-channel berms may be useful flow control devices, they can also increase the maintenance burden, clogging quickly with sediment and debris that must be removed to sustain design flows. Therefore, only use these devices when they are absolutely necessary, because they make the maintenance worker's job more difficult.

Table 9.1 - Summary of Stormwater Functions Provided by Wet Swales
Virginia Stormwater Design Specification No. 11, Wet Swales, Draft (DCR/DEQ, 2013)

Stormwater Function	Level 1 Design	Level 2 Design
Annual Runoff Volume Reduction (RR)	0%	0%
Total Phosphorus (TP) EMC Reduction[1] by BMP Treatment Process	20%	40%
Total Phosphorus (TP) Mass Load Removal	20%	40%
Total Nitrogen (TN) EMC Reduction[1] by BMP Treatment Process	25%	35%
Total Nitrogen (TN) Mass Load Removal	25%	35%
Channel Protection	Limited – reduced Time of Concentration; and partial detention volume can be provided above the Treatment Volume (*Tv*), within the allowable maximum ponding depth.	
Flood Mitigation	Limited	
[1] Change in event mean concentration (EMC) through the practice.		

Sources: CWP and CSN (2008), CWP, 2007

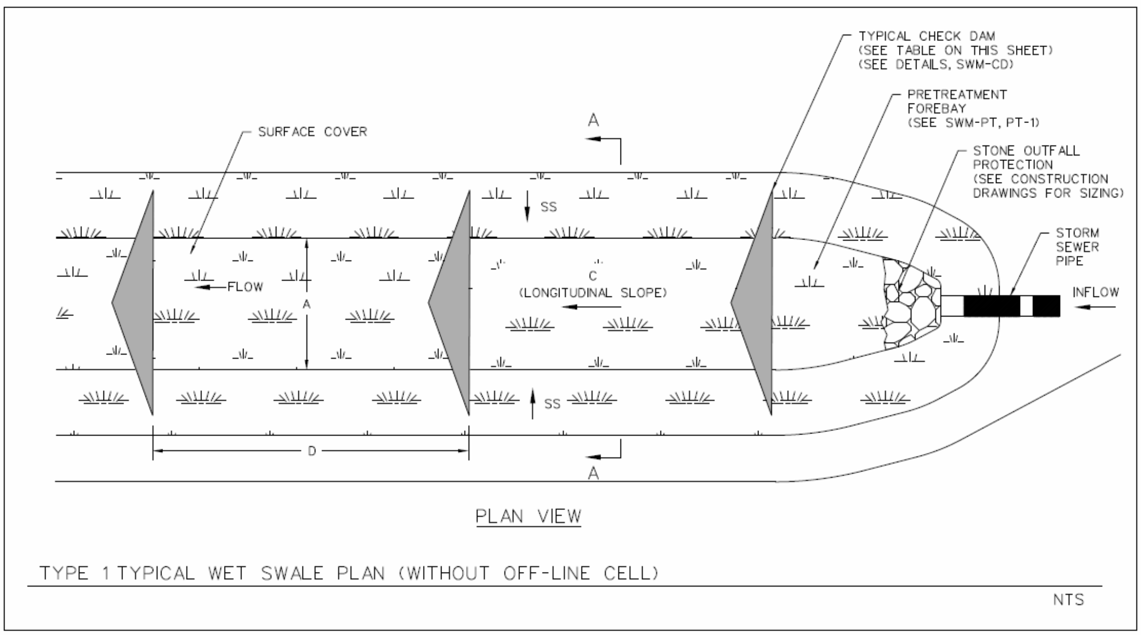


Figure 9.1 - Wet Swale with Check Dams
VDOT BMP Standard Detail SWM-9 Wet Swale, (2014)

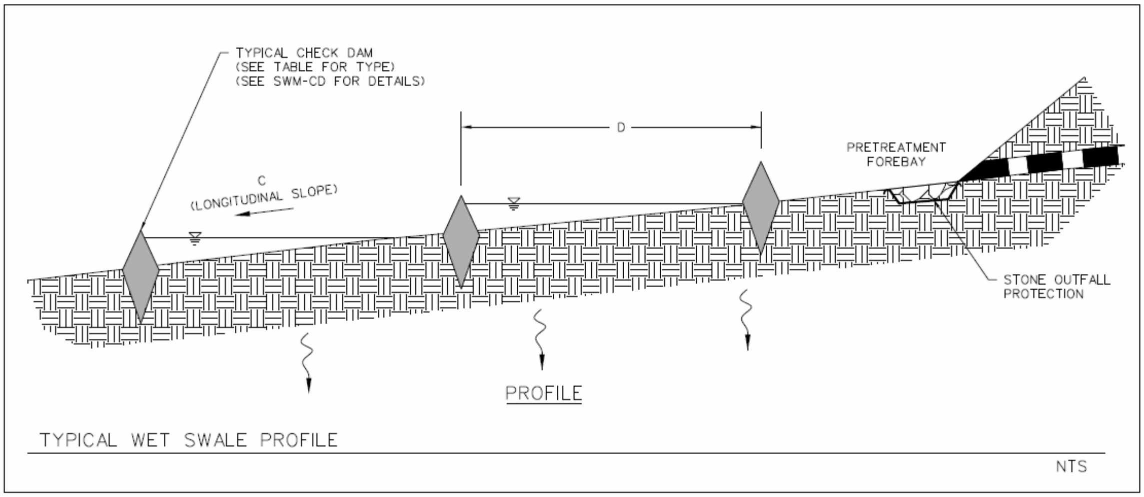


Figure 9.2 - Typical Wet Swale Profile
VDOT BMP Standard Detail SWM-9 Wet Swale, (2014)

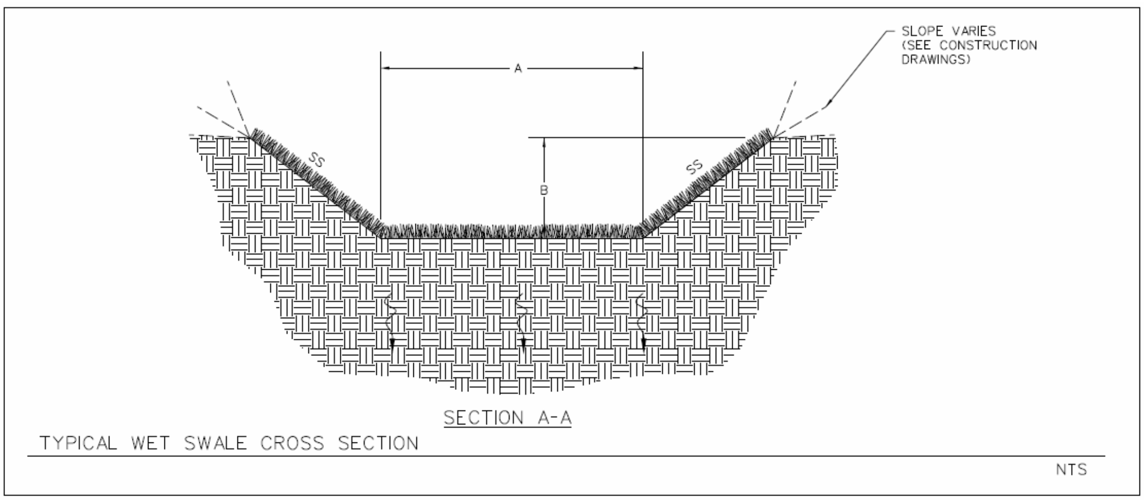


Figure 9.3 - Typical Wet Swale Cross-section
VDOT BMP Standard Detail SWM-9 Wet Swale, (2014)

9.2 Site Constraints and Siting of the Facility

When a wet swale is proposed the designer must consider a number of site constraints to ensure that the practice is applicable to the suggested use.

9.2.1 Maximum Contributing Drainage Area (CDA)

The maximum drainage area of a wet swale is limited to 5 acres, but preferably less. Past this threshold, there is an increasing likelihood that the velocity of flow in the swale will reach a point that prevents the residence time needed to provide effective settling of the treatment volume. It is important to design wet swales within the limits established for CDAs. Too much or too little runoff can result in performance issues and the need for subsequent repairs or reconstruction.

9.2.2 Site Slopes

Wet swales are suited to sites with slopes up to 2%. Although some gradient is necessary to establish positive drainage, typically, wet swales are limited to very shallow slopes. If wet swales are proposed in locations of steeper slopes, it may be possible to use a regenerative conveyance system (see section 9.3.8) upon approval by the VDOT Project Manager.

9.2.3 Site Soils

Typically wet swales are more suited on HSG C and D soils since they are generally more impermeable in nature.

9.2.4 Depth to Water Table

Wet swales are allowed to intersect the groundwater table. Typically, this intersection should be limited to approximately 6” on the bottom of the swale.

9.2.5 Hotspot Runoff

Wet swales should not be used for treatment of runoff from hotspots (areas that produce higher than normal concentrations of toxic pollutants) due to the potential direct contamination of the ground water table.

9.2.6 Karst Considerations

Wet swales are not feasible in karst areas.

9.2.7 Wetlands

When the construction of a wet swale is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, the feasibility of BMP implementation in their vicinity, and potential permit requirements.

Table 9.2 - Wet Swale Design Criteria
Virginia Stormwater Design Specification No. 11, Wet Swales, Draft (DCR/DEQ, 2013)

Level 1 Design (RR:0; TP:20; TN:25)	Level 2 Design (RR:0; TP:40; TN:35)
Tv = [(1")(R_v)(A)] / 12 – the volume reduced by an upstream RR BMP	Tv = [(1.25")(R_v)(A)] / 12 – the volume reduced by an upstream RR BMP
Swale slopes less than 2% [1]	Swale slopes less than 1% [1]
On-line design	Off-line swale cells
Minimal planting; volunteer vegetation	Wetland planting within swale cells
Turf cover in buffer	Trees, shrubs, and/or ground cover within swale cells and buffer
[1] Wet Swales are generally recommended only for flat coastal plain conditions with a high water table. A linear wetland is always preferred to a wet swale. However, check dams or other design features that lower the effective longitudinal grade of the swale can be applied on steeper sites, to comply with these criteria.	

9.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing a wet swale for improvement of water quality. Cross-section details for specific design features, including material specifications, can be found in the VDOT BMP Standard SWM-9 Wet Swale (2014).

9.3.1 Swale Sizing

For preliminary sizing and space planning, a general rule of thumb is that the surface area of the facility will be approximately 5% of the contributing drainage area (dependent on imperviousness and design level).

Actual dimensions are determined from the requirement to capture and treat the T_v (treatment volume) remaining from upstream runoff reduction practices (if any). Treatment credit is applied to both the permanent wet storage below the normal pool level and any temporary storage created through the installation of check dams or other features.

The design must also demonstrate that *on-line* wet swales also have sufficient capacity above the T_v to safely convey the 10-year design storm and be non-erosive during both the 2-year and 10-year design storms. When a Wet Swales is used as an off-line practice (Level 2 design), a bypass or diversion structure must be designed to divert the large storm (e.g., when the flow rate and/or volume exceeds the water quality Treatment Volume) to an adequate channel or conveyance system.

Design guidance shown in **Sections 3.3.1, 3.3.2, and 3.3.4 of Section 3, Grass Channels**, should be used for design of pre-treatment and swale geometry.

9.3.2 Normal Pool Depth

The normal pool depth (average) should be less than or equal to 6”.

9.3.3 Surface Ponding Depth

The maximum temporary ponding depth in any single Wet Swale cell should not exceed 12” at the most downstream point (e.g., at a check dam or driveway culvert).

9.3.4 Basin Geometry

Wet swale cross-sectional geometry is assumed to be trapezoidal or parabolic. Side slopes are to be 4:1 or flatter. Recommended longitudinal bottom slopes are less than 2% for a Level 1 design and less than 1% for a level 2 design. Individual cells formed by the installation of check dams shall generally be greater than 25’ but less than 40’ in length.

9.3.5 Check Dams

Materials and sizing guidelines for check dam construction shall conform with that listed in the VDOT BMP Standard SWM-CD Check Dams (2014) and VDOT Special Provision for Stormwater Miscellaneous (2014). Check dams are installed within wet swales to decrease the effective slope of the channel as necessary. Typical check dam spacing to achieve effective swale slopes may be found in Section 3, Table 3.3.

Check dams may also be used for velocity reduction. Velocities in wet swales should not exceed 3 fps to prevent erosion.

Keep in mind that the first cell created by a series of check dams will function, at least to some degree, as a pre-treatment forebay, allowing sediment to settle out of the stormwater prior to the runoff moving further down the swale. This first cell should be one thing checked during maintenance inspections, to ensure design capacity is being maintained so the cell performs properly in its pollution removal function.

9.3.6 Overflow

The wet swale shall be designed to convey the 10-year storm within the banks with a minimum of 3” of freeboard. The downstream end of the wet swale may discharge into an overflow structure (such as a VDOT Std DI-7), and overflow channel, or an overflow pipe.

9.3.7 Planting Plan

Plants selected for use in wet swales are required to be tolerant of both wet and dry periods. A list of suitable species is found in Virginia Stormwater Design Specification No. 13, Constructed Wetlands, Draft (DCR/DEQ, 2013). Salt tolerant species should be selected for use on VDOT projects.

9.3.8 Regenerative Conveyance Systems

Regenerative conveyance systems (RCS) are a more complex variation of wet swales that are designed, and primarily used with steep slopes. . RCS uses riff pools, engineered soil media, check dams and other features to detain and convey stormwater Due to installation cost, special design, and maintenance considerations, regenerative systems should not be considered without receiving permission from VDOT.

Design of these systems is beyond the scope of this document. The Ann Arundel County design specification can be found at:
http://www.aacounty.org/DPW/Watershed/StepPoolStormConveyance.cfm

9.4 Design Example

This section presents the design process applicable to wet swales serving as water quality BMPs. The pre- and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 11 of the *Virginia Stormwater Management Handbook, 2nd Edition*, Draft (DCR/DEQ, 2013) for details on hydrologic methodology.

A Level 1 wet swale design is being proposed to treat runoff from a 1,500' long section of a road improvement project along Route 620 in Isle of Wight County. The longitudinal slope along the proposed redesign of this section of Route 620 is very flat (approximately 0.8%). The proposed project includes removal of approximately 1,500 LF of road to grade a series of vertical curves (humps) that do not meet VDOT's current design standards. This will require complete removal of the current pavement cross-section, regrading of subgrade, and replacement of the pavement section with a width matching the existing pavement. Runoff from the centerline crown to each side of the road can be directed to wet swales on either side of the road. Calculations shown are for a single side (south side of road) only. The current lane (on BMP side of crown) and shoulder represent 0.40 acres of impervious area (all overlaying HSG C soils). In addition there is 1.20 acres of turf covered shoulder that drains to the BMP treatment area (0.80 acres in HSG C soils and 0.40 acres in HSG D soils). A summary of the data is found in **Table 9.3**. In the post-development condition, the time of concentration has been calculated to be 9 minutes.

Geotechnical investigations reveal a seasonally high ground water table adjacent to the site in several locations.

Table 9.3 - Hydrologic Characteristics of Example Project Site

		Impervious		Turf	
Pre	**Soil Classification**	**HSG C**	**HSG D**	**HSG C**	**HSG D**
	Area (acres)	0.40	0.00	0.80	0.40
Post	**Soil Classification**	**HSG C**	**HSG D**	**HSG C**	**HSG D**
	Area (acres)	0.40	0.00	0.80	0.40

Step 1 - Enter Data into VRRM Spreadsheet

The required site data from **Table 9.3** is input into the VRRM Spreadsheet for Redevelopment (2014) to compute load reductions for this linear project, resulting in site data summary information shown in **Table 9.4**. Note that using the redevelopment spreadsheet, the required reduction for linear projects is computed as the sum of the Post-Redevelopment Load and the Post-Development Load minus 80% of the Predevelopment Listed load.

Table 9.4 - Summary Data from VRRM Site Data Analysis

Site Rv	0.41
Post-development TP Load (lb/yr)	1.50
Total TP Load Reduction Required (lb/yr)	0.30

The entire disturbed area drains to the proposed location of the BMP. Due to the presence of high groundwater, a Level 1 wet swale is proposed as the treatment BMP. Appropriate data for post-development conditions is input into the VRRM Spreadsheet Drainage Area tab, yielding compliance results summarized in **Table 9.5.**

Table 9.5 - Summary Data from Level 1 Wet Swale Treatment

Total Impervious Cover Treated (acres)	0.40
Total Turf Area Treated (acres)	1.20
Total TP Load Reduction Achieved in D.A. A (lb/yr)	0.30

In this case, the total phosphorus reduction required is 0.30 lbs/yr. The estimated removal is 0.30 lbs/yr; therefore, the target has been met.

Step 2 - Compute the Required Treatment Volume

The treatment volume can be calculated using **Section 1, Equation 1** or taken directly from the VRRM Spreadsheet Drainage Area tabs. For this example, the reported treatment volume on the drainage area tab (treating the 1.60 acre area described by data in **Table 9.3**) is 2,381 ft^3.

Step 3 - Enter Data in Channel and Flood Protection Tab

Hydrologic computations for required design storms for flood and erosion compliance are not shown as part of this example. The user is directed to the VDOT Drainage Manual for appropriate levels of protection and design requirements related to erosion and flood protection. However, hydrologic computations are necessary to compute peaks to design components of the Wet Swale. In particular, the 10-year 24-hour design storm is used to size the rectangular notch is check dams.

Values for the 1-, 2-, and 10-year 24-hour rainfall depth should be determined from the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14. For this site (Lat 36.962591, Long 76.676985), those values are shown in **Table 9.6**.

Table 9.6 - Rainfall Totals from NOAA Atlas 14

	1-year storm	**2-year storm**	**10-year storm**
Rainfall (inches)	2.95	3.59	5.53

Because no runoff reduction is provided by a wet swale, there is no curve number adjustment (Virginia Runoff Reduction Spreadsheet for Linear Development, 2015). For this site, results from the runoff reduction spreadsheet yield an unadjusted curve number (**Table 9.7)** of 82 for all storms within the BMP drainage area.

Table 9.7 - Adjusted CN from Runoff Reduction Channel and Flood Protection Sheet [No Reduction for Wet Swale BMP]

	1-year Storm	**2-year Storm**	**10-year Storm**
$RV_{Developed}$ (in) with no Runoff Reduction	1.34	1.86	3.56
$RV_{Developed}$ (in) with Runoff Reduction	1.34	1.86	3.56
Adjusted CN	**82**	**82**	**82**

Input data is used in the Natural Resource Conservation Service Technical Release 55 (NRCS TR-55) Tabular method to calculate discharge hydrographs. (**Note that other hydrologic methodologies are suitable-see VDOT Drainage Manual, Hydrology for guidance**). Peaks of those hydrographs for the 1-, 2-, and 10-year storms are reported in **Table 9.8**.

Table 9.8 - Post-development Discharge Peaks to BMP

	1-year storm	2-year storm	10-year storm
Discharge (cfs)	2.5	3.5	6.8

Step 4 - Compute the Treatment Volume Peak Discharge

Sizing of wet swales follow similar procedures to those using to size grass channels (Section 3). The first step in the analysis is computation of discharge for the proposed treatment volume (q_{pT_v}). To do this, an adjusted CN must be computed that generates runoff equivalent to the treatment volume from a 1" rainfall. Note that this adjusted curve number is different than the adjusted curve numbers associated with runoff reduction.

$$CN = \frac{1000}{[10 + 5P + 10Q_a - 10(Q_a^2 + 1.25Q_aP)^{0.5}]} \qquad \textbf{(9.1)}$$

where,

CN = Adjusted curve number
P = Rainfall (inches), (1.0" in Virginia)
Q_a = Runoff volume (watershed inches), equal to $Tv \div drainage\ area$

$$Q_a = \frac{2{,}381\,ft^3}{1.6\ ac\left(\frac{43{,}560\ ft}{1\ ac}\right)}\left(\frac{12\,in}{1\ ft}\right) = 0.41\ in$$

$$CN = \frac{1000}{\left[10 + 5(1\ in) + 10(0.41\ in) - 10\left((0.41\ in)^2 + 1.25(0.41\ in)(1\ in)\right)^{0.5}\right]}$$

$$CN = 92$$

$$q_{pTv} = (q_u)(A)(Q_a) \qquad \textbf{(9.2)}$$

where,

q_{pTv} = Treatment Volume peak discharge (cfs)
q_u = unit peak discharge (cfs/mi^2/in)
A = drainage area (mi^2)
Q_a = runoff volume (watershed inches = Tv/A)

All of the variables are known in the above equation with the exception of q_u. To determine its value, first the initial abstraction must be computed using the equation:

$$I_a = \frac{200}{CN} - 2 \qquad \textbf{(9.3)}$$

$$I_a = \frac{200}{92} - 2 = 0.17\ inches$$

Compute I_a/P where P is the 1” rainfall (inches), which equates to 0.17.

Read the unit peak discharge, q_u, from Exhibit 4-II of the *SCS TR-55 Handbook (NRCS, 1986)*. Reading the chart yields a value of 855 cfs/mi^2/in.

$$q_{pTv} = \left(\frac{855\,\frac{\text{cfs}}{\text{mi}^2}}{\text{in}}\right)\left(\frac{1.6\ ac}{640\,{}^{ac}/_{mi^2}}\right)\left(\frac{2{,}381\ \text{ft}^3}{1.6\ ac \times \left(43{,}560\ {}^{ft}/_{1\ ac}\right)}\right)\left(\frac{12in}{1\ ft}\right)$$

$$q_{pTv} = 0.88\ cfs$$

Step 5 - Compute the Channel Bottom Width

The length of the project along Route 620 is approximately 1,500’. Since the proposed channel cross-section and longitudinal slope is consistent (0.8%) along the entire length, the channel will be evaluated for compliance at the most downstream end.

Based on the requirements set forth in Section 3, Grass Channels, the Manning ‘n’ coefficient is 0.2 for a depth of up to 4”. Based on geotechnical observations, it is estimated that a seasonally high groundwater table will intersect with the bottom 4” of the swale during a portion of the year. Because specifications allow treatment credit to be applied to both the permanent volume (as well as temporary storage, if any) the initial assumption will be that treatment can occur in the first 4” of depth. The estimated width may be calculated through modification of **Equation 3.3** (**Section 3**, **Grass Channels**), reproduced below for convenience.

$$W = (n)(q_{pT_v})/(1.49D^{5/3}S^{1/2})$$

$$W = (0.20)(0.88\ \text{cfs})/(1.49(0.33\ ft)^{5/3}(0.008\frac{\text{ft}}{\text{ft}})^{1/2})$$

$$W = 8.4\ ft$$

Step 6 - Compute the Required Minimum Treatment Length

Using the discharge (0.88 cfs), the flow depth (0.33’), and the channel width (8.4’), velocity can now be approximated using **Section 3, Equation 3.4** as:

$$V = \frac{0.88\ cfs}{(8.4\ ft\ \times 0.33\ ft)} = 0.32\frac{ft}{s}$$

This velocity is less than the maximum velocity of 1 fps required, and is therefore is an acceptable design.

The minimum swale length is calculated using **Section 3, Equation 3.5** as:

$$L = 540V = (540\ sec)\left(0.32\frac{ft}{s}\right) = 173\ ft$$

The total length of the swale will be a minimum of 1,673', which includes the length adjacent to the project (1,500') and the length downstream of the last inflow location (corresponding to the termination of the project). Note that this length can be reduced if check dams are used to increase the surface storage volume.

Step 7 - Design of Overflow and Conveyance Structures

Overflow and conveyance structures must be designed to pass the specified design storm based on functional classification of the road. This includes calculations for overtopping of the wet swale by storms of lower recurrence (i.e. 25-, 50-, and 100-year storms). These computations are beyond the scope of this design example. However, the user is directed to the VDOT Drainage Manual for guidance on flood and erosion compliance calculations.

Step 8 - Planting Selection

For maintenance purposes, VDOT prefers grass and other herbaceous varieties to be planted in wet swales, instead of trees and shrubs. See Stormwater Specification 13, Constructed Wetland, Draft (DCR/DEQ, 2013) for a list of acceptable plant species.

10.1 Overview of Practice

Stormwater filters are used to collect and treat runoff from small, highly impervious areas. These practices treat runoff by providing a pretreatment chamber that slows and settles larger particles from runoff, and then through a secondary treatment filter which provides an underdrain for discharging the treated stormwater into a downstream conveyance system. Although filters are moderately efficient at removal of pollutants, the practice affords no reduction in the computed stormwater volume leaving the site. Stormwater filters are best applied on sites where nearly 100% of the contributing drainage area is impervious to limit the potential of clogging due to sediment laden runoff from erosion on permeable surfaces. Linear stormwater filters are very suitable for highway projects and may be designed as a series of filters. In practice, on linear projects, the layout of filter practices will be very similar to dry swale configurations. Requirements shown herein are modifications to specifications found in Virginia Stormwater Design Specification No. 12, Filtering Practices, Draft (DCR/DEQ, 2013), for specific application to VDOT projects. **Table 10.1** describes a summary of stormwater functions provided by filtering practices.

Filtering practices can be an important part of the stormwater quality treatment train, but they require special design considerations to minimize maintenance. Otherwise, they can become a maintenance burden. Good design can eliminate or at least minimize such problems.

Table 10.1 - Summary of Stormwater Functions Provided by Filtering Practices
Virginia Stormwater Design Specification No. 12, Filtering Practices, Draft (DCR/DEQ, 2013)

Stormwater Function	Level 1 Design	Level 2 Design
Annual Runoff Volume Reduction (RR)	0%	0%
Total Phosphorus (TP) EMC Reduction[1] by BMP Treatment Process	60%	65%
Total Phosphorus (TP) Mass Load Removal	60%	65%
Total Nitrogen (TN) EMC Reduction[1] by BMP Treatment Process	30%	45%
Total Nitrogen (TN) Mass Load Removal	30%	45%
Channel Protection	Limited – Runoff diverted off-line into a storage facility for treatment can be supplemented with an outlet control to provide peak rate control.	
Flood Mitigation	None. Most filtering practices are off-line and do not materially change peak discharges.	
[1] Change in the event mean concentration (EMC) through the practice..		

Sources: CWP and CSN (2008) and CWP (2007)

10.1.1 Typical Configurations

Typical configurations of filters used for highway projects include surface filters and perimeter sand filters, shown in **Figures 10.1 and 10.2**, respectively. Surface filters, although similar in design to bioretention, several differences include: impermeable filter fabric lining bottom of facility, an underdrain is always present, surface cover is gravel, sand, or turf (no other plants), media is one hundred percent sand, and the filter includes an upstream dry or wet settling basin/chamber. Perimeter sand filters are typically precast systems that include inlet grates, a sedimentation chamber, and the media filter bed, with underdrain. Although very practical for highway projects due to the relatively small size and linear nature, the overall dimensions will limit the contributing area that may be treated by the device. Manufactured Treatment Devices (MTDs) may also be allowed by VDOT on a case by case basis. Information regarding specific MTD structures shall be submitted to VDOT Materials Division for acceptance during design.

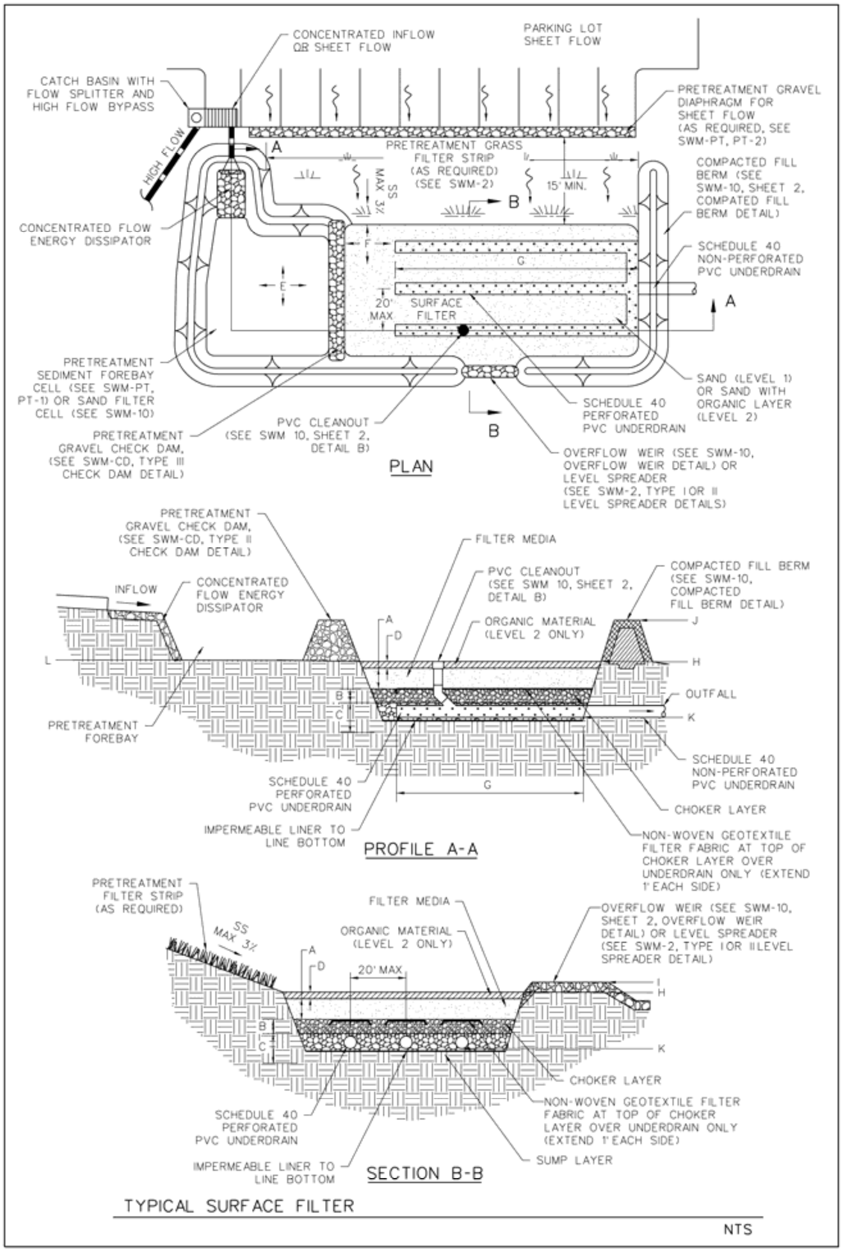


Figure 10.1 - Typical Surface Filter
VDOT SWM-10, Filtering Practices (2014)

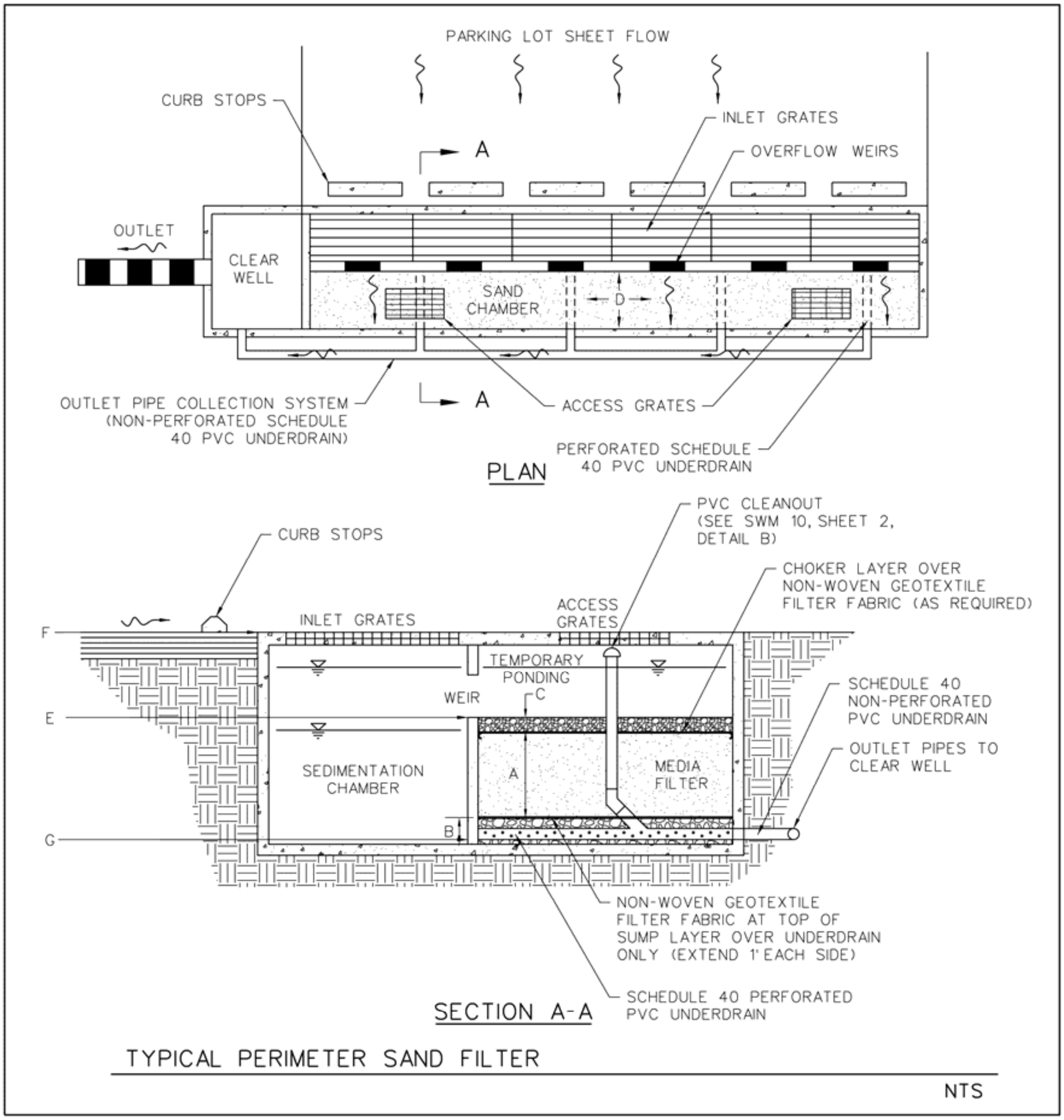


Figure 10.2 - Typical Perimeter Sand Filter
VDOT SWM-10, Filtering Practices (2014)

10.2 Site Constraints and Siting of the Facility

When a stormwater filter is proposed, the designer must consider a number of site constraints to ensure that the practice is applicable to the suggested use.

10.2.1 Hydraulic Head

Hydraulic head is the driving force which allows the filter to operate. Although the head required to most efficiently operate filters ranges between 2’ to 10’, perimeter sand filters (**Figure 10.2**) can function with minimal head of as little as 2’.

10.2.2 Maximum Contributing Drainage Area (CDA)

Stormwater filters are best applied on small sites where the contributing drainage area (CDA) is as close to 100% impervious as possible to minimize the sediment and organic solids load to the filter. A maximum CDA of 5 acres is recommended for surface sand filters, and a maximum CDA of 2 acres is recommended for perimeter or underground filters. Filters can be designed to treat runoff from larger areas; however, the increased hydraulic loading will contribute to greater frequency of media surface clogging and associated maintenance costs. It is important to design filtering BMPs within the limits established for CDAs. Too much or too little runoff can result in performance issues and the need for subsequent repairs or reconstruction. **Section 5** of Virginia Stormwater Design Specification No. 12, Filtering Practices, Draft (DCR/DEQ, 2013) provides additional information about the design variations that can allow sand filters to be used at challenging sites.

10.2.3 Space Required

The amount of space required for a Filter Practice depends on the design variant selected. Both sand and organic surface filters typically consume about 2% to 3% of the CDA, while perimeter sand filters typically consume less than 1%. Underground stormwater filters can be placed under parking or open space and generally allow the surface area to be used for other purposes.

Surface Sand Filters are normally designed to be off-line facilities in order to economize the size of the filter components and reduce maintenance costs. However, in some cases they can be installed as a treatment component within the bottom of a Dry Extended Detention (ED) Pond that has a shallow total ponding.

10.2.4 Depth to Water Table

Separation of at least 2' between the seasonally high groundwater table and the bottom of the filter is required. A minimum of one test location should be used at the existing low point in grade that lies within the footprint of the proposed filter locations for establishment of the water table elevation.

10.2.5 Depth to Bedrock

Separation of at least 2' between bedrock and the invert of the filter is required. A minimum of one soil boring is required at the existing low point in grade that lies within the footprint of the proposed filter location(s) for establishment of bedrock elevation.

10.2.6 Site Soils

Generally, due to the presence of the impermeable base of the filter and underdrain, filters may be constructed in any soil condition, including fill material. A minimum of one soil boring is required within the footprint of the proposed filter location(s) to evaluate soil suitability for the proposed structure.

10.2.7 Karst Areas

Because filters do not promote infiltration, they are an excellent option in karst areas. Inspection during construction should ensure that practices are watertight. See VDOT Special Provision for Filtering Practice (2014).

10.2.8 Linear Highway Sites

Linear stormwater filters are a preferred practice for constrained highway rights-of-way when designed as a series of individual on-line or off-line cells. In these situations, the final design closely resembles that of Dry Swales with vegetated filter strip pretreatment. Salt-tolerant grass species should be selected if the contributing roadway will be salted in the winter.

10.2.9 Existing Utilities

Although possible to construct filters over existing utilities, or easements, care should be taken to evaluate future maintenance/installation within the footprint of the filter. Installation of filters above existing utilities should be avoided, where possible, and should not be done without prior approval from VDOT.

10.2.10 Maintenance Reduction Features

The following maintenance issues should be addressed during filter design to reduce future maintenance problems:

- ***Access -*** Good maintenance access is needed to allow crews to perform regular inspections and maintenance activities. "Sufficient access" is operationally defined as the ability to get a vacuum truck or similar equipment close enough to the sedimentation chamber and filter to enable cleanouts.

- ***Visibility -*** Stormwater Filters should be clearly visible at the site so inspectors and maintenance crews can easily find them. Adequate signs or markings should be provided at manhole access points for Underground Filters.

- ***Confined Space Issues -*** Underground Filters are often classified as an *underground confined space*. Consequently, special OSHA rules and training are needed to protect the workers that access them. These procedures often involve training about confined space entry, venting, and the use of gas probes.

10.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing a filtering practice for improvement of water quality. Cross-section details for specific design features, including material specifications, can be found in the VDOT SWM-10, Filtering Practices (2014). General guidance for filtering practices can be found in Table 10.2.

Table 10.2 - Filtering Practice Design Guidance
Virginia Stormwater Design Specification No. 12, Filtering Practices, Draft (DCR/DEQ, 2013)

Level 1 Design (RR:0; TP:60; TN:30)	Level 2 Design (RR:0 [1]; TP:65; TN:45)
Tv = [(1.0)(Rv)(A)] / 12 – the volume reduced by an upstream BMP	Tv = [(1.25)(Rv)(A)] / 12 – the volume reduced by an upstream BMP
One cell design[2]	Two cell design[2]
Sand media	Sand media with an organic layer
Contributing Drainage Area (CDA) contains pervious area	CDA is nearly 100% impervious
[1] May be increased if the 2nd cell is utilized for infiltration in accordance with Stormwater Design Specification No. 8 (Infiltration) or Stormwater Design Specification No. 9 (Bioretention). The Runoff Reduction (RR) credit should be proportional to the fraction of the Tv designed to be infiltrated. [2] A pretreatment sedimentation chamber or forebay is not considered a separate cell	

10.3.1 Sizing

For preliminary sizing and space planning, a general rule of thumb is that surface filters will occupy an area ranging between 2%-3% of the contributing drainage area, while perimeter sand filters or MTDs may be 1% or less.

Actual dimensions are determined from **Equations 10.1 and 10.2 (below)**, from the Virginia Stormwater Design Specification No. 12, Filtering Practices, Draft (DCR/DEQ, 2013)

The required filter surface area size is determined by the following equation:

$$A_f = \frac{(T_v)(d_f)}{(K)(h_f + d_f)(t_f)} \qquad \textbf{(10.1)}$$

where:

A_f = area of the filter surface (ft^2)
T_v = Treatment volume *(storage volume in ft^3)
K = Coefficient of permeability—3.5 ft/day
h_f = Average height of water above bed (ft) [maximum of 5']

d_f = Filter media depth (thickness) [minimum 1']
t_f = Allowable drawdown time [1.67 days]

* Stormwater filters are typically the only practice in a drainage area, or in some cases used as pretreatment for another BMP; however, where runoff reduction practices are upstream of the filter (i.e., the filter is part of a treatment train), the design Tv must be reduced by the upstream runoff reduction, or $T_{v_{BMP}}$.

As described in Virginia Stormwater Design Specification No. 12, Filtering Practices, Draft (DCR/DEQ, 2013), the coefficient of permeability is chosen to assume a condition near the end of the sand media operational life (i.e., in a clogged condition). Although water begins filtering and exiting the system through the underdrain shortly after the beginning of a runoff event, fluctuations in filtration rates due to head conditions require storage to prevent bypass of the filter. The volume of storage required is estimated by **Equation 10.2**, as found in Virginia Stormwater Design Specification No. 12, Filtering Practices, Draft (DCR/DEQ, 2013).

$$V_s = 0.75(T_v) \qquad \textbf{(10.2)}$$

where:

V_s = Volume of storage (ft^3)
T_v = Treatment Volume (ft^3)

The computed treatment volume used in **Equation 10.2** is computed using **Equations 1.1 and 1.2**, with information found in **Table 1.1** (all from **Section 1**).

10.3.2 Pretreatment

Pretreatment is always required upstream of filters to remove trash, capture coarse sediment, and provide for even flow distribution into the filter bed at near zero velocity. The pretreatment volume is required to be a minimum of $0.25T_v$. For surface filters, the pretreatment (sediment forebay cell) shall conform to the PT-1 Detail in VDOT SWM-PT, Pretreatment (2014) [see **Figure 10.3**]. Flow entering surface cells directly from paved areas may require a pretreatment gravel diaphragm in accordance with PT-2 in VDOT SWM-PT, Pretreatment (2014) to insure that flow enters the cell as sheet flow. As required, a grass filter strip at least 15' long and meeting the requirements of VDOT SWM-2, Sheet Flow to Vegetated Filter Strip (2014), may be incorporated to further pretreat runoff into the filter bed.

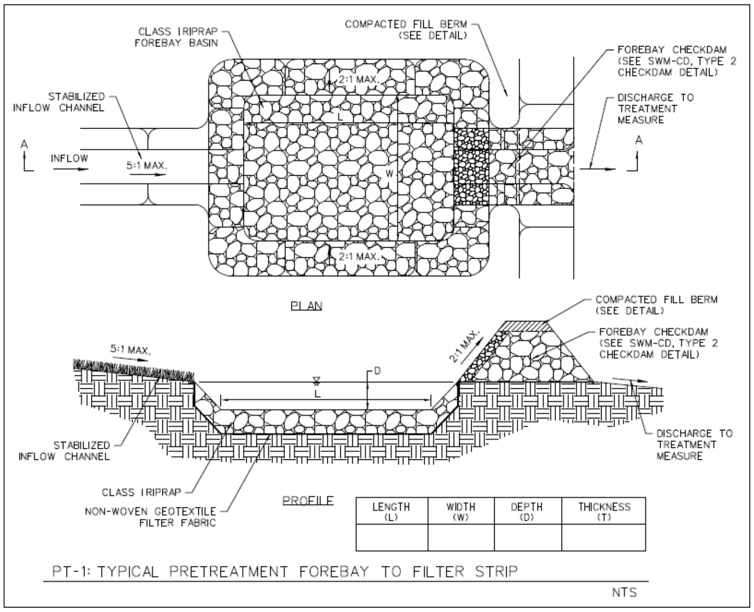


Figure 10.3 - Typical Pretreatment Forebay
VDOT SWM-PT, Pretreatment (2014)

The check dam used to create the pretreatment forebay through separation from the main filter bed shall be constructed in accordance with the VDOT SWM-CD, Type 2 (2014) [**Figure 10.4**].

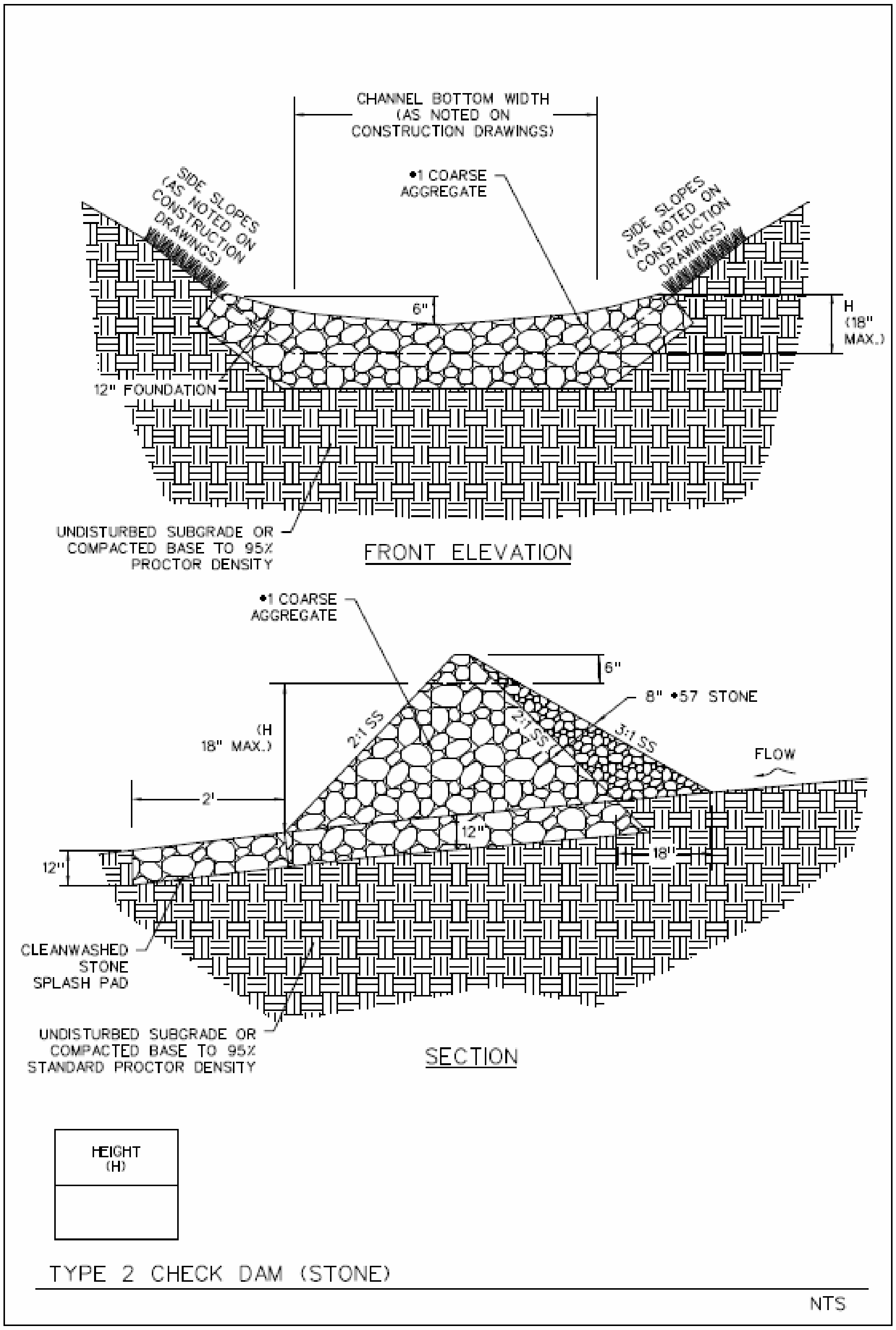


Figure 10.4 - Type 2 Check Dam Separating Pretreatment Cell from Filter Cell
VDOT SWM-CD, Check Dams (2014)

10.3.3 High Flow Bypass

Off-line systems must be designed with a bypass to divert larger storms around the filter cells. Flow splitting can be accomplished through precast flow splitters, weirs, bypass channels, or other similar methods. Calculations for all design events must be submitted for review to insure proper functioning over the entire range of storms.

10.3.4 Dewatering

Filters shall be designed to dewater in less than 40 hours after a runoff producing storm event.

10.3.5 Surface Cover

Surface cover for surface sand filters (Level 2 only) shall consist of a 3” layer of topsoil (organic material), conforming to the requirements found in the VDOT Special Provision for Soil Compost Amendment (2014). Organic material is not applied to Level 1 surface sand filters.

Surface cover for underground sand filters (Level 2 only) shall consist of a 4” choker layer meeting the requirements of Part II.(e) of the VDOT Special Provision for Filtering Practices (2014) placed over a non-woven geotextile filter fabric conforming to the requirements of the VDOT Special Provision for Stormwater Miscellaneous (2014).

10.3.6 Filter Media

Media shall conform to *VDOT Road and Bridge Specification* Section 202.02, Grade C Sand. When incorporating an organic layer in the design (primarily when attempting to remove metals and hydrocarbons), requirements for composition shall adhere to Engineered Soils Media, Type 3, found in VDOT Special Provision for Soil Compost Amendments (2014). Filter media shall consist of clean, washed sand with grain size between 0.02” and 0.04” in diameter, and conform with the requirements specified in Part II(h) of the VDOT Special Provision for Filtering Practices (2014).

10.3.7 Media Depth

Generally, bed depth should range between 12” and 18”. Depth may be increased for facilitation of future maintenance (e.g., removing 1”-3” of sand without having to replace it during each scheduled maintenance); however, this practice shall be approved by VDOT prior to implementation.

10.3.8 Underdrains, Inspection Ports, and Cleanouts

Underdrain, cleanouts, and inspection ports and all other components of the underdrain system shall conform to the VDOT Special Provision for Stormwater Miscellaneous (2014). Underdrain pipes shall be 4” to 6” and installed at no greater than a 20’ spacing between pipes. A minimum of one cleanout pipe will be required per 2000 ft^2 of filter surface area. A choker layer shall be installed above and below the perforated underdrain pipe consisting of VDOT #8, #78, or #8P aggregate, or as allowed per special approval outlined in Part II.(e) of the VDOT Special Provision for Filtering Practices (2014). Non-woven geotextile filter fabric shall be placed between the media (sand) layer and choker layer over top of the underdrains. The width of this fabric shall extend no greater than 1’ to either side of the pipe.

10.3.9 Manhole Access for Underground Filters

Access grates or covers will be required for all underground filters. Access is required into all chambers to facilitate inspection and maintenance. Although the access may be through rectangular or circular grates, or solid covers, the minimum opening shall be 30” in diameter and include steps to facilitate entry.

10.3.10 Installation in Coastal Plain

Slight modifications to the design requirements discussed herein may be necessary for installation in areas with very flat surface slopes and a high seasonal water table. As discussed in the Virginia DEQ Stormwater Design Specification No. 12, Filtering Practices (2013), modification may be made as follows:

- The combined depth of the underdrain and sand filter layer may be reduced to 18” total.
- The length of the cell may be maximized; or, treatment may be in multiple connected cells.
- The minimum depth from the bottom of the cell to the high groundwater table may be decrease to as little as 1’ proved that a 6” underdrain is designed and installed that is only partially efficient at dewatering the filter.
- Further decreases in distance to groundwater table may be allowed if the installation is watertight with respect to surrounding soil. Anchoring may be required to ensure that floatation is not a concern. A buoyancy analysis shall be required to be submitted, reviewed, and approved by VDOT prior to allowing these installations.
- A minimum slope of 0.5% is required on the underdrain to discharge to grade, or tie into the receiving channel or pipe.

10.3.11 Steep Terrain

Two celled terraced designs may be used in areas of steep terrain, provided that the drop between cells is limited to 1’ and the slope is armored. This allows the gradient of upstream slopes contributing runoff to the filter to be increased up to 15%.

10.3.12 Cold Climate and Winter Performance

Surface or perimeter filters may not always be effective during the winter months. The main problem is ice that forms over and within the filter bed. Ice formation may briefly cause nuisance flooding if the filter bed is still frozen when spring melt occurs. To avoid these problems, filters should be inspected before the onset of winter (prior to the first freeze) to dewater wet chambers and scarify the filter surface. Other measures to improve winter performance include the following:

- Provide a weir between the pre-treatment chamber and filter bed to reduce ice formation; the weir is a more effective substitute than a traditional standpipe orifice.
- Extend the filter bed below the frost line to prevent freezing within the filter bed.
- Oversize the underdrain to encourage more rapid drainage and to minimize freezing of the filter bed.
- Expand the sediment chamber to account for road sand. Pre-treatment chambers should be sized to accommodate up to 40% of the Tv.

10.3.13 Construction and Inspection

Construction and inspection shall be in conformance with the *VDOT Special Provision for Filtering Practices (2014).*

10.4 Design Example

This section presents the design process applicable to stormwater filters serving as water quality BMPs. The pre- and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 11 of the *Virginia Stormwater Management Handbook, 2nd Edition (DCR/DEQ, 2013)* for details on hydrologic methodology.

A Level 1 perimeter sand filter is being proposed to treat runoff from 1.05 acre park and ride lot near the U.S. 460 and Interstate 81 interchange in Christiansburg, VA. The hydrologic classification of on-site soils is HSG B. Post-development conditions within the disturbed area indicate 0.95 acres of impervious area, and 0.10 acres of managed turf. Summaries of these parameters are found in **Table 10.3**. The time of concentration to the filtering practice has been computed as 8 minutes. The project site does not exhibit a high or seasonally high groundwater table or indicate the presence of bedrock, based on geotechnical tests performed on site.

Table 10.3 - Hydrologic Characteristics of Example Project Site

		Impervious	Turf	Forest
Pre	**Soil Classification**	**HSG B**	**HSG B**	**HSG B**
	Area (acres)	0.00	0.15	0.90
Post	**Soil Classification**	**HSG B**	**HSG B**	**HSG B**
	Area (acres)	0.95	0.10	0.00

Step 1 - Enter Data into VRRM Spreadsheet

The required site data from **Table 10.3** is input into the VRRM Spreadsheet for New Development (2014), resulting in site data summary information shown in **Table 10.4**.

Table 10.4 - Summary of Output from VRRM Site Data Tab

Site Rv	0.88
Post-development Treatment Volume (ft^3)	3,349
Post-development TP Load (lb/yr)	2.10
Total TP Load Reduction Required (lb/yr)	1.67

Information should now be entered in the Drainage Area tab of the spreadsheet using the proposed treatment train with sheet flow to vegetated filter strip to a Level 1 filter. Appropriate data for post-development conditions is input into the VRRM Spreadsheet Drainage Area tab, yielding compliance results summarized in **Table 10.5.**

Table 10.5 - Summary Data from Treatment Train Treatment

Total Impervious Cover Treated (acres)	0.95
Total Turf Area Treated (acres)	0.10
Total TP Load Reduction Achieved in D.A. A (lb/yr)	1.68

In this case, the total phosphorus reduction required is 1.67 lbs/yr. The estimated removal is 1.68 lbs/yr; therefore, the target has been met.

Step 2 - Compute the Required Treatment Volume

The treatment volume can be calculated using **Section 1, Equation 1** or taken directly from the VRRM Spreadsheet Drainage Area tabs. For this example, the treatment volume is calculated using Equations 1.1 and 1.2. In order to meet pollutant removal requirement (0.41 lbs/acre/year), a vegetated filter strip pretreatment and Level 1 stormwater filter is proposed.

Information from **Table 10.3** is used in conjunction with **Equation 1.2** and **Table 1.1** (both from **Section 1**) to calculate $R_{v_{composite}}$ for the post-development condition.

$$R_{v_{composite}} = (R_{v_I} \times \%I) + (R_{v_T} \times \%T) + (R_{v_F} \times \%F)$$

$$R_{v_{composite}} = \left(0.95 \times \frac{0.95 \text{ acres}}{1.05 \text{ acres}}\right) + \left(0.20 \times \frac{0.10 \text{ acres}}{1.05 \text{ acres}}\right) = 0.88$$

Once the $R_{v_{composite}}$ has been calculated, the Treatment Volume for the 1.0" runoff through the facility can be directly computed using **Equation 1.1** (from **Section 1**) for a Level 1 facility.

$$T_v = \left[\frac{(1.00)(1.0 \text{ in.})\left(R_{v_{composite}}\right)(A)}{12}\right]$$

$$T_v = \left[\frac{(1.00)(1.0 \text{ in.})(0.88)(1.05 \text{ acres})}{12}\right] = 0.077 \text{ acre-ft} = 3{,}354\ ft^3$$

Because the filter is part of a treatment train, and the vegetated filter strip results in a runoff reduction of 1,638 ft^3 of runoff as calculated by the VRRM spreadsheet, the total treatment volume above can be reduced by that amount:

$$T_v = 3{,}354\ ft^3 - 1{,}638 ft^3 = 1{,}716 ft^3$$

Step 3 - Enter Data in Channel and Flood Protection Tab

Values for the 1-, 2-, and 10-year 24- hour rainfall depth should be determined from the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 and entered into the "Channel and Flood Protection" tab of the spreadsheet. For this site (Lat 37.1342, Long -80.3722), those values are shown in **Table 10.6**. Curve numbers used for computations should be those calculated as part of the runoff reduction spreadsheet Virginia Runoff Reduction Spreadsheet for New Development (2014). For this site, results from the runoff reduction spreadsheet are shown in **Table 10.7**, and result in adjusted curve numbers of 89, 89 and 90 for the 1-, 2- and 10-year storms, respectively. Note that the volume reduction achieved is from the vegetated filter pretreatment, and that no volume reduction is achieved through use of the filtering practice.

Table 10.6 - Rainfall Totals from NOAA Atlas 14

	1-year storm	**2-year storm**	**10-year storm**
Rainfall (inches)	2.31	2.80	4.19

Table 10.7 - Adjusted CN from Runoff Reduction Channel and Flood Protection

	1-year Storm	2-year Storm	10-year Storm
$RV_{Developed}$ (in) with no Runoff Reduction	1.69	2.16	3.51
$RV_{Developed}$ (in) with Runoff Reduction	1.25	1.72	3.07
Adjusted CN	**88**	**89**	**90**

Input data is used in the Natural Resource Conservation Service Technical Release 55 (NRCS TR-55) Tabular method to calculate discharge hydrographs. (**Note that other hydrologic methodologies are suitable-see VDOT Drainage Manual, Hydrology for guidance**). Peaks of those hydrographs for the 1-, 2-, and 10-year storms are reported in **Table 10.8**. These values will be used to size the conveyance downstream of the filtering practice.

Table 10.8 - Post-development Discharge Peaks Exiting BMP

	1-year storm	2-year storm	10-year storm
Discharge (cfs)	1.68	2.39	4.31

Step 4 - Compute Minimum Filter Area

Using a media depth of 12” (1.0’), and a surface ponding depth average of 1’ (note that the maximum head is 2’), the required filter area is calculated using **Equation 10.1** as:

$$A_f = \frac{(1{,}716\ ft^3)(1.0\ ft)}{\left(3.5\ \frac{ft}{day}\right)(1.0\ ft + 1.0\ ft)(1.67\ days)} = 147\ \text{ft}^2$$

Step 5 - Pretreatment

The parking lot runoff drains directly to a gravel diaphragm that runs along the edge of the proposed pavement to introduce stormwater runoff to the vegetated filter strip as sheet flow. The diaphragm is installed according to detail SWM-PT, PT2, and the vegetated filter strips according to guidelines set forth in Section 2 of this manual. Runoff then is concentrated into a small perimeter grass channel, where it is conveyed into the pretreatment sediment forebay. The minimum forebay size is calculated as $0.25T_v$, which is 429 ft^3. However, due to limitations in storage above the sand bed for this particular facility, the sediment forebay is increased in size to allow ponding of approximately 1,400 ft^3 of surface runoff upstream of the rock check dam separating the pretreatment cell from the treatment filter.

Step 6 - Specify Media Depth

The depth of the facility's filtering media should be a minimum of 12" and typically a maximum of 18". As stated above, a depth of 12" is used for this design example. Limitations in available surface storage for head, limitations in discharge elevation, and long-term maintenance needs and costs will be typical driving factors that must be weighed in determining depth of media.

Step 7 - Design Overflow Structure

An overflow structure must be provided for large runoff producing events to bypass excess runoff when the sand filter T_v is exceeded. This filter bed has been designed with an overflow DI-7 grate that corresponds with the maximum elevation of the treatment volume over the bed. Because there is no runoff reduction associated with the stormwater filter (runoff reduction in this case was integral to the first step in the treatment train—the vegetated filter), the T_v of the filter will in essence be subtracted from the hydrograph prior to activation of the overflow spillway into the grass channel. One method of determining the peak overflow after removal of the treatment volume from the inflow hydrograph is shown below.

First, the hydrograph ordinates should be used to compute the cumulative volume for preceding flow at each discrete hydrograph time interval. For the 2-year storm used in this example, the resulting chart is shown in **Figure 10.5**. Since the treatment volume of the filter is known to be 1,716 ft^3, the peak discharge associated with this value is found to be approximately 2.34 cfs from the generated curve (**Figure 10.5**). Note that this occurs prior to the peak of 2.39 cfs; therefore, the peak of 2.39 cfs should still be used in overflow calculations to determine that no erosion to the system occurs when discharging to a manmade conveyance. Although this method does not yield an exact solution due to fluctuating outflow rates through the filter, depending on head conditions, it is expected that in most cases the resulting volume intersection will occur on the rising limb of the hydrograph and result in use of the computed hydrograph peak (in this case, 2.39 cfs).

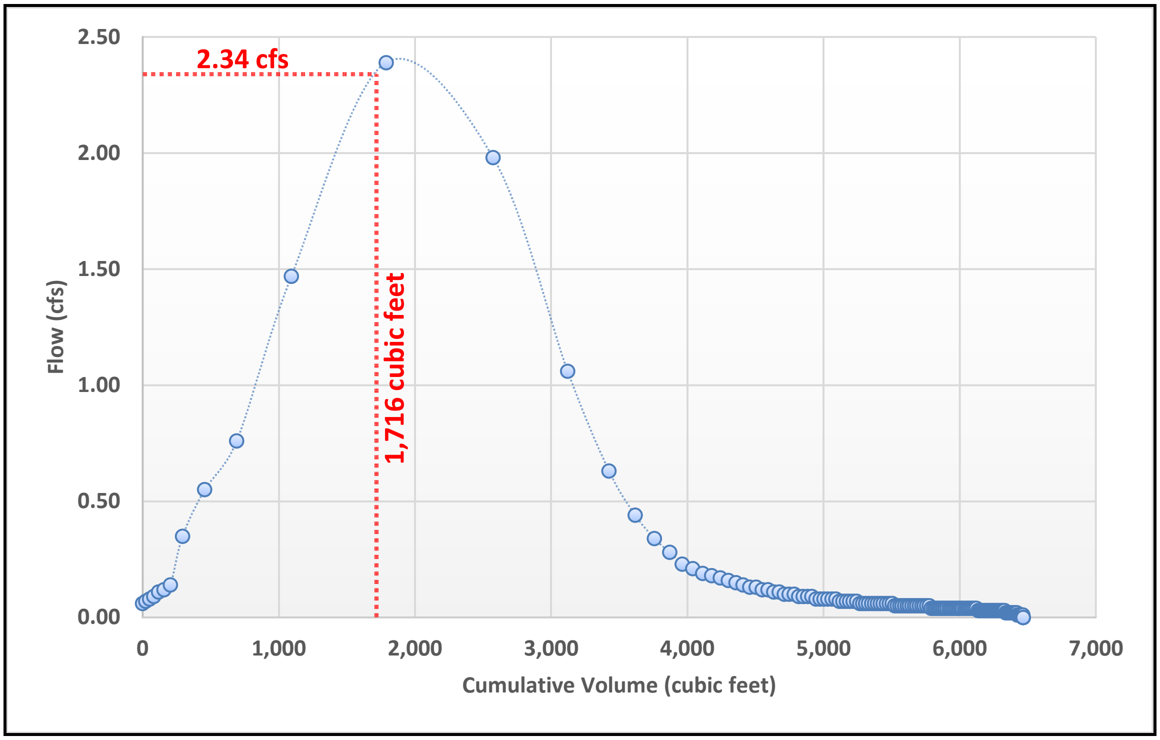


Figure 10.5. Discharge-Volume Curve for the 2-year 24-hour storm

For the 10-year storm, if a similar curve is plotted (not shown), the treatment volume will intersect the curve prior to the hydrograph peak of 4.31 cfs. Therefore, the 10-year peak of 4.31 cfs should be used to determine the adequacy of the downstream manmade conveyance system. Discharges to natural systems will require additional analysis for the 1-year storm to meet the requirements of 9VAC25-870-66 of the Virginia Administrative Code. Although, there is an additional discharge related to the filter underdrains, based on the filter surface area of 147 ft^2 and the assumed drawdown rate of 3.5 ft/day (see **Equation 10.1**), the average bed discharge is negligible, calculating to less than 0.1 cfs.

Adequacy of the DI-7 to convey the peak discharge with the available head should be verified using applicable nomographs in the VDOT Drainage Manual (latest edition). VDOT Figure 9C-14 is used to determine flow capacity of a DI-7 in a sump. For this example, the DI-7 crest is set at 1’ above the filter bed surface (maximum ponding depth), with a height of 1.0’ between DI-7 crest and top of berm. Flow is computed through use of the VDOT 9C-14 nomograph, as shown in **Figure 10.6**. Evaluation of the 10-year peak (**Figure 10.6**) shows that a head of approximately 0.46’ above the DI-7 crest is needed to convey the 10-year storm. Since this is less than the 1.0’ height to top of berm, the system is adequate for the 10-year storm, even if partially clogged.

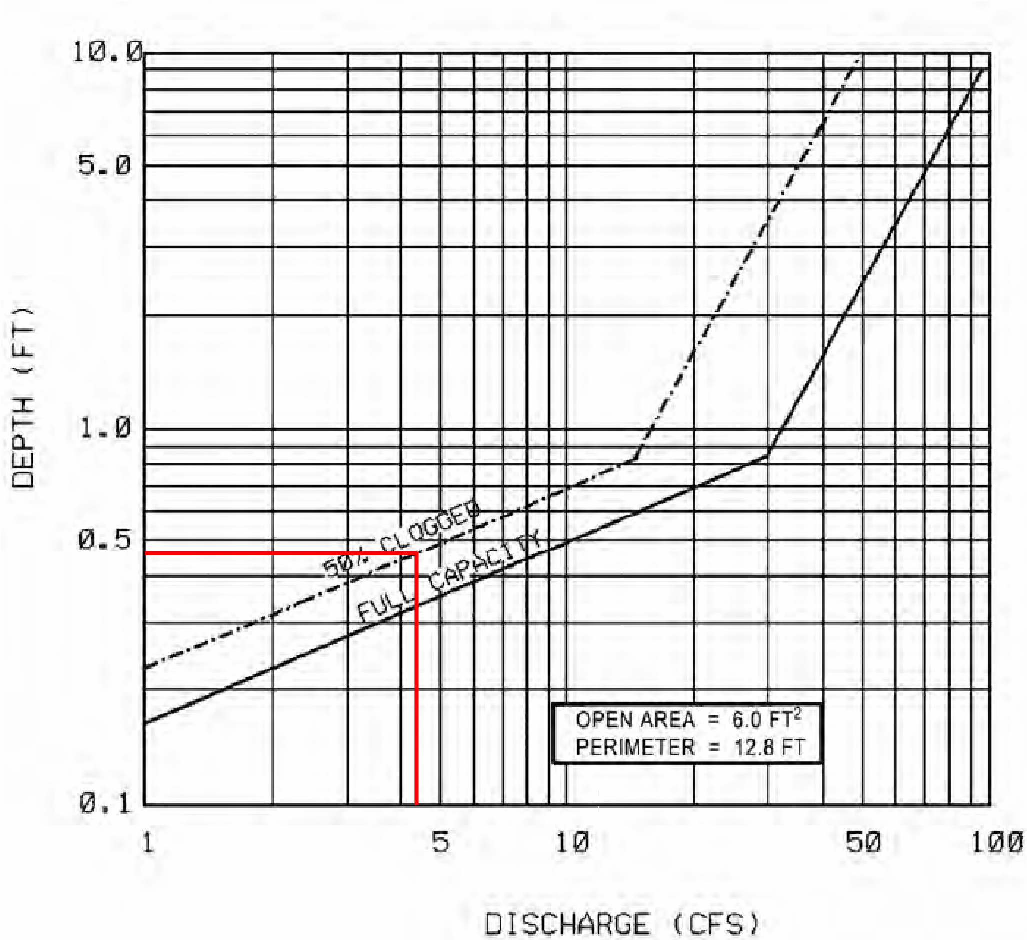


Figure 10.6 - VDOT Performance Curve for DI-7 in Sump
VDOT Drainage Manual, Appendix 9C-14 (2014)

The discharge pipe from the DI-7 manhole will be sized to convey the 10-year discharge without surcharge. The designer should use nomographs in the VDOT Drainage Manual or hydraulic design software with the capability of solving the Manning equation for flow in partially full pipes, to determine preliminary pipe size. Assuming that discharge will be to a reinforced concrete pipe on a 1% slope, a 15" minimum pipe size is required to discharge the 10-year flow of 4.31 cfs. Specific elevations and pipe slopes are dependent on site elevations at the filter bed, and in the receiving channel.

Step 8 - Underdrains

Underdrains are required to be installed on all stormwater filters. Typically, on small beds, 4" perforated pipes are sufficient to convey filtered flow. Spacing shall be no greater than 20' between pipes. The 147 ft^2 filter bed proposed in this example will be installed as a square bed that has approximate dimensions of 12.2' x 12.2' (rounded up to the nearest 0.1'). Due to the minimal width, a single run of 4” perforated pipe will be sufficient to provide drainage for the stone layer of the filter.

Step 9 - Seeding

Because the proposed facility is a Level 1 facility, no organic layer or seeding on top of the filter bed is required. However, the pretreatment area and vegetated filter should be seeded with salt-resistant species as specified in the Virginia Erosion and Sediment Control Handbook or the VDOT SWM-2 Vegetated Filter Strip guidance.

11.1 Constructed Wetlands - Overview of Practice

Constructed stormwater wetlands fall into a structural BMP category having the capacity to improve the quality of stormwater runoff in much the same manner as retention and enhanced extended detention basins. Like these impounding facilities, stormwater wetlands are seeded with a diverse mix of aquatic and emergent vegetation, which plays an integral role in the pollutant removal efficiency of the practice. Wetland BMPs improve the quality of runoff by physical, chemical, and biological means. The physical treatment of runoff occurs as a result of decreased flow velocities in the wetland, thus leading to evaporation, sedimentation, adsorption, and/or filtration. Chemical treatment arises in the form of chelation (bonding of heavy metal ions), precipitation, chemical adsorption, and microbial activity. Biological treatment occurs via uptake of nutrients and other constituents into plant tissue.

Constructed stormwater wetlands are typically the final element in treatment trains, provide no volume reduction credit, and should, generally, be used *only if* there is remaining pollutant removal to manage after all other upland runoff reduction options have been considered and properly credited. Although constructed wetlands can be designed to safely pass flood-level design storms, when a BMP is employed as a quantity control practice, there is an inherent expectation of rapidly fluctuating water levels in the practice following runoff producing events. Rapid fluctuations in water level subject emergent wetland and upland vegetation to enormous stress, and many wetland species cannot survive such conditions. In addition to producing large surges of stormwater runoff, land use conversion resulting in a loss of pervious cover will often result in a decrease of perennial baseflow from a watershed. The decrease or absence of such baseflow is problematic for the establishment of a diverse and healthy mix of wetland vegetation. Requirements shown herein are modifications to specifications found in Virginia Stormwater Design Specification No. 13, Constructed Wetland, Draft (DCR/DEQ, 2013), for specific application to VDOT projects.

Constructed Stormwater Wetlands can be an important part of the stormwater quality treatment train, but they require special design considerations to minimize maintenance. Otherwise, they can become a maintenance burden, particularly if sediment accumulates or if flows cause erosion. Good design can eliminate or at least minimize such problems.

Table 11.1 - Summary of Stormwater Functions Provided by Constructed Wetland
Virginia Stormwater Design Specification 13, Constructed Wetland, Draft (DCR/DEQ, 2013)

Stormwater Function	Level 1 Design	Level 2 Design
Annual Runoff Volume Reduction (RR)	0%	0%
Total Phosphorus (TP) EMC Reduction[1] by BMP Treatment Process	50%	75%
Total Phosphorus (TP) Mass Load Removal	50%	75%
Total Nitrogen (TN) EMC Reduction[1] by BMP Treatment Process	25%	55%
Total Nitrogen (TN) Mass Load Removal	25%	55%
Channel Protection	Yes. Up to 1' of detention storage volume can be provided above the normal pool.	
Flood Mitigation	Yes. Flood control storage can be provided above the normal pool.	
1 Change in event mean concentration (EMC) through the practice.		

11.1.1 Typical Configurations

Typical details for plan and profile views of constructed wetlands are shown in **Figures 11.1 and 11.2**. Due to the water balance requirements for maintaining vegetative species found in wetlands, applications on linear sites can be challenging. Therefore, their used by VDOT will typical be limited to non-linear sites or interchanges. However, linear wetland cells and regenerative conveyance systems are well suited to treat runoff within swales located along roads.

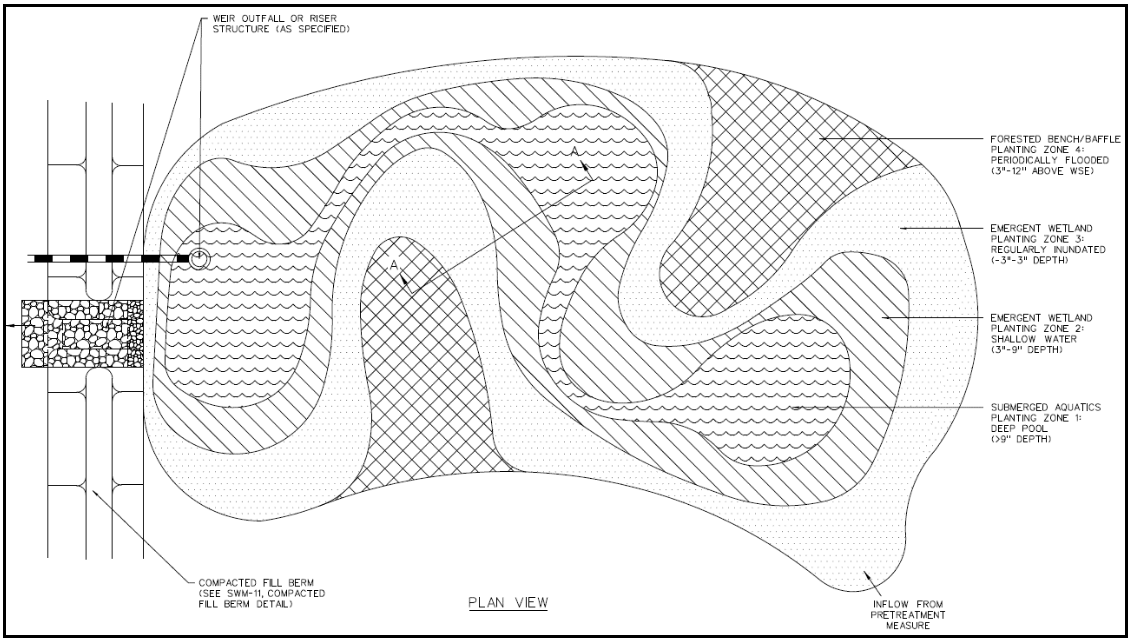


Figure 11.1 - ***Typical Level I Constructed Wetland***
VDOT SWM-11 Constructed Wetlands (2014)

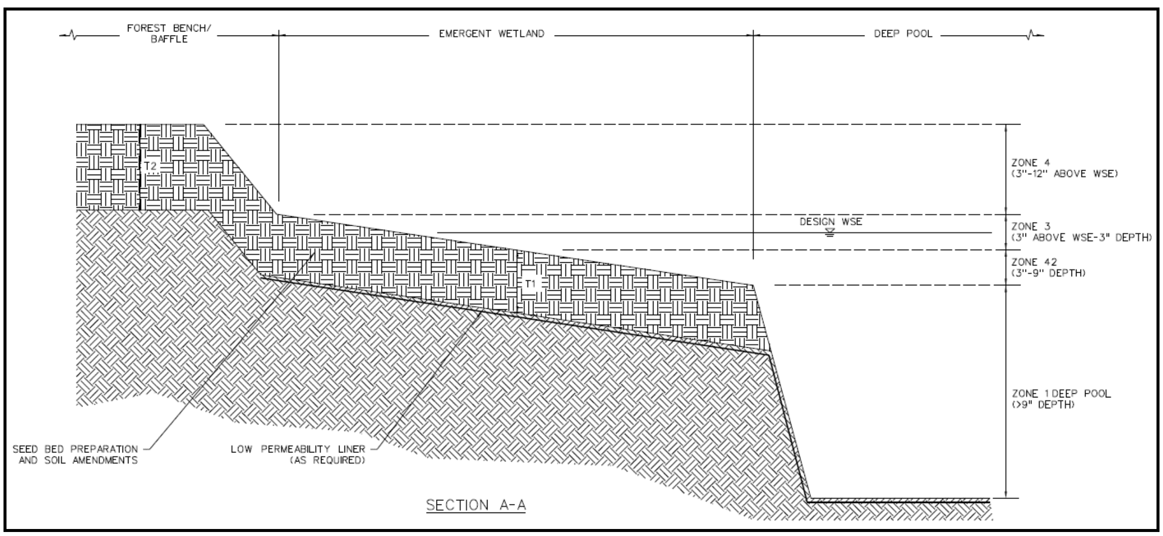


Figure 11.2 - ***Varying Wetland Depth Zones (Profile)***
VDOT SWM-11 Constructed Wetlands (2014)

Table 11.2 - Constructed Wetland Design Criteria
Virginia Stormwater Design Specification 13, Constructed Wetland, Draft (DCR/DEQ, 2013)

Level 1 Design (RR:0; TP:50; TN:25)	Level 2 Design (RR:0; TP:75; TN:55)
Tv = [(R_v)(A)] / 12 – the volume reduced by an upstream BMP	Tv = [1.5(R_v)(A)] / 12 – the volume reduced by an upstream BMP
Single cell (with a forebay and micro-pool outlet)[1,2]	Multiple cells or a multi-cell pond/wetland combination[1,2]
Extended Detention (ED) for 50% of Tv (24 hr)[3] or Detention storage (up to 12”) above the wetland pool for channel protection (1-year storm event)	No ED or detention storage. (limited water surface fluctuations allowed during the 1” and 1-year storm events**)**
Uniform wetland depth. [2] Allowable mean wetland depth is > 1’	Diverse micro-topography with varying depths [2]**;** Allowable mean wetland depth ≤1’
The surface area of the wetland is ≤ 3% of the contributing drainage area (CDA)	The surface area of the wetland is > 3% of the CDA
Length/Width ratio *OR* Flow path = 2:1 or more Length of shortest flow path/overall length = 0.5 or more [3]	Length/Width ratio *OR* Flow path = 3:1 or more Length of shortest flow path/overall length = 0.8 or more [4]
Emergent wetland design	Emergent and Upland wetland design

[1] Pre-treatment Forebay required
[2] Internal T_v storage volume geometry – refer to **Section 11.3**
[3] Extended Detention may be provided to meet a maximum of 50% of the T_v; Refer to Stormwater Design Specification 15 for ED design
[4] In the case of multiple inlets, the flow path is measured from the dominant inlets (that comprise 80% or more of the total pond inflow)

11.2 Site Constraints and Siting of the Facility

Constructed wetlands normally require a footprint that takes up about 3% of the contributing drainage area, depending on the contributing drainage area’s impervious cover and the constructed wetland’s pool average depth. When a constructed wetland is proposed the designer must consider a number of site constraints to ensure that the practice is applicable to the suggested use.

11.2.1Adequate Water Balance

A water balance analysis must be performed to ensure that adequate water in the form of stormwater runoff, groundwater inflow, or base flow is present to prevent the micro-pools from completely drying out after a 30-day drought.

11.2.2 Maximum Contributing Drainage Area (CDA)

The contributing drainage area must be large enough to sustain a permanent water level within the stormwater wetland. If the only source of wetland hydrology is stormwater runoff, then a minimum of 10 to 25 acres of drainage area are typically needed to maintain adequate water elevations. Of critical concern is the presence of adequate baseflow to the facility. Many species of wetland vegetation cannot survive extreme drought conditions. Additionally, insufficient baseflow and the subsequent stagnation of wetland marsh areas can lead to the emergence of undesirable odors from the wetland. Regardless of drainage area, all proposed wetlands should be subjected to a low flow analysis to ensure that an adequate marsh volume is retained even during periods of dry weather when evaporation and/or infiltration are occurring at a high rate. The anticipated baseflow from a fixed drainage area can exhibit great variability, and insufficient baseflow may require consideration of alternate BMP measures. When infiltration losses from the wetland are excessive, a clay liner or geosynthetic membrane may be considered. Such a liner should meet the approval and specifications of the Materials Division.

The presence of a shallow groundwater table, as common in the Tidewater region of the state, may allow for the implementation of a constructed wetland whose contributing drainage area is very small. These circumstances are site-specific, and the groundwater elevation must be monitored closely to establish the design elevation of the permanent pool.

It is important to design constructed stormwater wetlands within the limits established for CDAs. Too much or too little runoff can result in performance issues and the need for subsequent repairs or reconstruction.

11.2.3 Hydraulic Head

Available hydraulic head is usually constrained based on the discharge elevation at the downstream end of the practice. Hydraulic head necessary to drive the system is typically a minimum of 2’ to 4’.

11.2.4 Site Slopes

Stormwater wetlands should, generally, not be constructed within 50’ of any slope steeper than 10%. When this is unavoidable, or when the facility is located at the toe of a slope greater than 10%, a geotechnical report should be performed to address the potential impact of the facility in the vicinity of such a slope. When flow must be conveyed down steeper slopes and constructed wetlands must be integrated in the stormwater management design, Regenerative Conveyance Systems (RCS) may be considered with the permission of the District Office.

11.2.5 Depth to Water Table

Due to the desired interaction of the water table with maintenance of the minimum pool elevation, depth to water table is not typically a constraint in implementing constructed wetlands. However, high groundwater inflows may inhibit the proper water quality treatment function of the wetland and, thus, affect the allowed pollutant reduction credits assigned to the facility. High groundwater may also increase excavation costs. Furthermore, in Coastal areas there is the possibility that salt water may have intruded into the groundwater table, and this will have implications for the selection of wetland plants to use.

11.2.6 Setbacks

Generally, edges of wetlands should be 20’ from the right of way line, 25’ from foundations, 50’ from septic drainfields, and 100’ from wells. Variations from these requirements shall be requested and approved through the District Office, prior to integration in Contract Documents.

11.2.7 Karst

Typically, constructed wetlands should not be implemented in karst areas due to the risk of sinkhole formation and groundwater contamination. However, if a geotechnical investigation shows at least a 3’ separation between the bottom of the wetlands and bedrock, the practice can be implemented with approval from the District Office, and with the installation of an impermeable liner (clay or, preferably, geosynthetic) meeting the specifications shown in **Table 11.3**. If a constructed wetland are used in karst terrain, then shallow, linear and multiple-cell wetland configurations are preferred. Deeper wetland configurations, such as a pond/wetland system and the ED wetland have limited application in karst terrain.

11.2.8 Existing Utilities

Basins should not be constructed over existing utility rights-of-way or easements. This can have significant repercussions for long-term maintenance of the basin. When this situation is unavoidable, permission to impound water over these easements must be obtained from the utility owner *prior* to design of the basin. When it is proposed to relocate existing utility lines, the costs associated with their relocation should be considered in the estimated overall basin construction cost.

Table 11.3 - Required Liners for Constructed Wetlands in Karst Terrain
Virginia Stormwater Design Specification 13, Constructed Wetland, Draft (DCR/DEQ, 2013)

Situation	Criteria
Not Excavated to Bedrock	24" of soil with a maximum hydraulic conductivity of 1 x 10^{-5} cm/sec
Excavated to or near Bedrock	24" of clay[1] with maximum hydraulic conductivity of 1 x 10^{-6} cm/sec
Excavated to Bedrock within wellhead protection area, in recharge are for domestic well or spring, or in known faulted or folded area	24" of clay[1] with maximum hydraulic conductivity of 1 x 10^{-7} cm/sec and a synthetic liner with a minimum thickness of 60 mil.
Plasticity Index of Clay: Not less than 15% (ASTM D-423/424) Liquid Limit of Clay: Not less than 30% (ASTM D-2216) Clay Particles Passing: Not less than 30% (ASTM D-422) Clay Compaction: 95% of standard proctor density (ASTM D-2216)	

Source: WVDEP, 2006 and VA DCR/DEQ, 1999

11.2.9 Soils

The implementation of constructed stormwater wetlands can be successfully accomplished in the presence of a variety of soil types. However, when such a facility is proposed, a subsurface analysis and permeability test is required. The required subsurface analysis should investigate soil characteristics to a depth of no less than 3' below the proposed bottom of the wetland. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils.

To ensure the long-term success of a constructed wetland, it is essential that water inflows (baseflow, surface runoff, and groundwater) be greater than losses to evaporation and infiltration. This requires the designer to calculate a monthly water budget. Due to excessive infiltration losses, soils exhibiting high infiltration rates (Hydrologic Soil Groups A and B) are not typically suited for the construction of stormwater wetlands whose lone source of inflow is from surface runoff. Often, soils of moderate permeability (on the order of $1x10^{-6}$ cm/sec), as well as those of Hydrologic Soil Group C and D, are capable of supporting the shallow marsh areas of a stormwater wetland. However, the hydraulic head (pressure) generated from deeper regions, such as the wetland micro-pool, may increase the effective infiltration rate rendering similar soils unsuitable for wetland construction.

Mechanical compaction of existing subsoils, a clay liner, geosynthetic membrane, or other material (as approved by the Materials Division) may be employed to combat excessively high infiltration rates. The wetland embankment material must meet the specifications detailed later in this section and/or be approved by the Materials Division and be installed in accordance with specifications found in the VDOT Special Provision for Constructed Wetland (2014).

11.2.10 Discharge to Sensitive Aquatic Habitats

Construction of the practice is watersheds containing trout streams is discouraged due to the potential of temperature impairment caused by the long term impoundment of water. District approval will be required prior to the installation of constructed wetlands in watersheds containing trout streams.

Installation within existing wetlands and jurisdictional waters is not allowed. VDOT Environmental shall be contacted to determine if waters are jurisdictional prior to design.

11.2.11 Coastal Plain Settings

Constructed wetlands are an ideal practice for the flat terrain, low hydraulic head and high water table conditions found at many coastal plain development sites. The following design adaptations can make them work more effectively in coastal plain settings:

- Shallow, linear and multiple-cell wetland configurations are preferred.
- It is acceptable to excavate up to 6” below the seasonally high groundwater table to provide the requisite hydrology for wetland planting zones, and up to 3’ below for micro-pools, forebays and other deep pool features.
- The volume below the seasonably high groundwater table is acceptable for the T_v, as long as the other primary geometric and design requirements for the wetland are met (e.g., flow path and micro-topography).
- Plant selection should focus on species that are wet-footed and can tolerate some salinity.
- A greater range of coastal plain tree species can tolerate periodic inundation, so designers should consider creating forested wetlands, using species such as Atlantic White Cedar, Bald Cypress and Swamp Tupelo.
- The use of flashboard risers is recommended to control or adjust water elevations in wetlands constructed on flat terrain.

The regenerative conveyance system is particularly suited for coastal plain situations where there is a significant drop in elevation from the channel to the outfall location.

11.2.12 Maintenance Reduction Features

The following design criteria will help to avoid significant maintenance problems pertaining to constructed wetlands:

Maintenance Access. Good access is needed so crews can remove sediments, make repairs and preserve wetland treatment capacity).

- Maintenance access must be provided to the forebay, safety benches, and outlet riser area.

- Access roads must (1) be constructed of load bearing materials, (2) have a minimum width of 12', and (3) possess a maximum profile grade of 15%.
- Turnaround areas may also be needed, depending on the size and configuration of the wetland.

Clogging Reduction. If the low flow orifice clogs, it can result in a rapid change in wetland water elevations that can potentially kill wetland vegetation. Therefore, designers should carefully design the flow control structure to minimize clogging, as follows:

- A minimum 3" diameter orifice is recommended in order to minimize clogging of an outlet or extended detention pipe when it is surface fed. It should be noted, however, that even a 3" orifice will be very susceptible to clogging from floating vegetation and debris.
- Smaller openings (down to 1" in diameter) are permissible, using internal orifice plates.
- All outlet pipes should be adequately protected by trash racks, half-round CMP, other anti-clogging measures, or reverse-sloped pipes extending to mid-depth of the micro-pool.

11.3 General Design Guidelines

Constructed wetlands are designed based on three major factors: (1) **the desired plant community** (an emergent wetland – Level 1 design; a mixed wetland – emergent and forest; or an emergent/pond combination – Level 2 design); (2) **the contributing hydrology** (groundwater, surface runoff or dry weather flow); and (3) the **landscape position** (linear or basin).

Constructed wetlands shall typically fall within one of three categories, as defined in the Virginia Stormwater Design Specification No. 13, Constructed Wetland, Draft (DCR/DEQ, 2013). These are:

- Constructed Wetland Basin – Level 1 (1.0 x Treatment Volume [T_v])
- Constructed Multi-Cell Wetland – Level 2 (1.5 T_v)
- Constructed Multi-Cell Pond/Wetland Combination – Level 2 (1.5 T_v)

Details found in VDOT Detail SWM-11: Constructed Wetlands (2014) should be incorporated in the design. More specific requirements for each of the three constructed wetland types are found below.

To avoid performance issues, the facility must be sized properly for the target Treatment Volume. However, oversizing the storage provided in the BMP, as compared to what is required to achieve the BMP's performance target, can decrease the frequency of maintenance needed and, thus, potential life-cycle costs. Oversizing, where feasible, can also help VDOT achieve its broader pollution reduction requirements associated with its DEQ MS4 Permit and the

Chesapeake Bay TMDL. Oversizing options are likely to involve the adjustment of detention times and may require prior approval by DEQ.

11.3.1 Constructed Wetland Basin – Level 1

Several configuration options exist that allow for implementation of the T_{vBMP} credit from the Virginia Runoff Reduction Method. Allowable storage components for a Level 1 constructed wetland that may be used to demonstrate treatment volume retention requirements are:

1. Water volume stored below the normal pool (including deep pools).
2. A 24-hour extended detention storage, including a maximum of 0.5T_v above the normal pool elevation.
3. Void storage in submerged rock, sand, or stone layers may be used in the computation of required T_v storage.

Note that the one year channel protection detention and detention volume depth above normal pool shall not exceed 1'. Typically, the 1-year storm volume will drive this requirement since it is likely that the T_v will be less than the 1-year storm volume. The maximum water level fluctuation during routing of the treatment volume and/or the 1-year storm is limited to 12". A weir or other outlet control structure may be used to ensure that this maximum is not violated (see VDOT Detail SWM-11: Constructed Wetlands (2014)).

11.3.2 Constructed Multi-Cell Wetland – Level 2

Similar to the Level 1 facility described above, the components of a multi-cell wetland that can be used to meet the treatment volume requirements are:

1. Entire water volume stored below the normal pool of each cell (including deep pools).
2. Void storage in submerged rock, sand, or stone layers may be used in the computation of required T_v storage.

Routing of the treatment volume requires that the water level fluctuations not exceed 8". Further, a maximum of 12" fluctuation in water level is allowed during routing of the 1-year storm volume through the constructed wetland.

11.3.3 Constructed Multi-Cell Pond/Wetland Combination – Level 2

Due to the presence of the pond component, demonstration of the treatment volume storage is slightly different for the constructed multi-cell pond/wetland system. Additional details for wet pond design can be found in Section 12, Wet Ponds. The components that may be used are as follows:

1. Water volume stored below the normal pool (including deep pools). This is required to include a minimum of 50% of the total Level 2 treatment volume (i.e. 0.75T_v). The remaining 50% can be in a pond with the following requirements for volume storage credit:
   a. Permanent pool volume, containing a minimum of 50% of the pond cell design volume.
   b. Extended detention storage above the permanent pool, containing a maximum of 50% of the pond cell design volume.
2. Void storage in submerged rock, sand, or stone layers may be used in the computation of required T_v storage.

11.3.4 Water Balance Analysis

An analysis of the system water balance for the contributing drainage area is required as part of the design. This is used to ensure the long term viability of the system when taxed by environmental stressors such as infiltration, evapotranspiration, and drought. Water balance guidance for the wet pond component of a Level 2 facility is found in Section 12, Wet Ponds. For constructed wetlands, designers should use the Hunt Water Balance Equation as adapted from the Virginia DEQ Stormwater Design Specification No. 13, Constructed Wetland (2013).

$$DP = RF_m \times \frac{(CDA \times R_{v_{composite}})}{A_{wetland}} - ET - INF - RES \qquad (\textbf{11.1})$$

where:

DP = Depth of pool (inches)
RF_m = Monthly rainfall during drought (inches)
CDA = Contributing drainage area (acres)
$R_{v_{composite}}$ = Composite runoff volume coefficient for constructed wetland CDA
$A_{wetland}$ = Area of the wetland footprint (acres)
ET = Summer evapotranspiration rate (assumed to be 8”)
INF = Monthly infiltration loss (assume 7.2” @ 0.1 in/hr)
RES = Reservoir of water for a factor of safety (assume 6”)

Based on the assumption of zero rainfall (drought), a minimum depth of pool of 21.2” is calculated from **Equation 11.1**. Therefore, without other known sources of inflow such as baseflow or groundwater inflow, the minimum pool depth **should be at least 22”.**

11.3.5 Integrated Design Components and Geometry

Research and experience have shown that the internal design geometry and depth zones are critical in maintaining the pollutant removal capability and plant diversity of the stormwater wetland. Wetland performance is enhanced when the wetland has multiple cells, longer flow paths, and a high ratio of surface area to volume. Whenever possible, constructed wetlands should be irregularly shaped with long, sinuous flow paths. The following design elements are *required* for Constructed Wetlands:

11.3.6 Pool Depths

Level 1 designs may have a pool depth exceeding 1'. Level 2 wetland cells are restricted to a mean pool depth of 1' or less. Variable pool depths should be integrated in the design in order to promote both open water and diverse vegetative cover. Specific design parameters for depth zones are as follows:

1. **Deep Pools:** A forebay (distinct from pretreatment forebays), center, and micro-pool, each ranging in depth from between 18" and 48", should be provided which cumulatively hold approximately 25% of the design treatment volume. See **Section 11.3.4** for further guidance on minimum deep pool depth.
2. **High Marsh:** Approximately 70% of the cell **surface area** should have elevations ranging between -6" to +6" relative to the normal pool elevation).
3. **Low Marsh:** This zone contains storage at -6" to -18" below the normal pool elevation. This zone is not considered to be an effective wetland zone and should provide a short transition between high marsh and deep pools. Maximum slopes in this transition zone from the deep pool to the high marsh should be 5H:1V (or preferably flatter). Biodegradable erosion control fabric should be used to prevent erosion of this zone during construction, to prevent erosion or slumping due to difficulty in quickly establishing vegetative cover.

11.3.7 Multiple-Cell Wetlands (Level 2)

In addition to the forebay and micro-pool discussed above, the Level 2 design is required to have at least two additional deep pool cells. Typically, cells will be installed at successively lower elevations. The ultimate goal is to provide a 50%-50% mix of emergent and forested wetland vegetation across all cells. Cells can be formed using a variety of berming techniques (see VDOT Detail SWM-11 Constructed Wetland (2014)). The pretreatment forebay is typically at a higher elevation than the secondary cell, which is the normal pool elevation. The third cell is typically 3" to 6" lower than the second cell (normal pool). The final cell (micro-pool) is located at the point of discharge from the system (through an outlet structure or weir).

11.3.8 Micro-topography

Variations in topography resulting in small variations in elevation are used to create the various regions described above. At least two of the following design features must be integrated into a Level 2 design:

1. Tree peninsulas, high marsh wedges, or rock filter cells installed perpendicular to primary flow path.
2. Tree islands above both the normal pool and maximum extended detention zone, formed by coir fiber logs.

3. Inverted root wads or large wood-based debris.
4. Gravel diaphragms within high marsh zone(s).
5. Internal weirs/baffles made of cobble with sand backfill, gabion baskets, or stabilized earthen berms.

11.3.9 Side Slopes

Side slopes for the wetland should generally have gradients of 4H:1V to 5H:1V. Such mild slopes promote better establishment and growth of the wetland vegetation. They also contribute to easier maintenance and a more natural appearance.

11.3.10 Flow Path

The overall flow path through the wetland shall have a 2:1 length-to-width ratio for Level 1 designs and a 3:1 ratio for Level 2 designs. One modification that may achieve these ratios is the design of sinuosity within the system, as shown in **Figure 11.1**. The ratio of the shortest flow path (shortest distance from closest inlet into the system to the outlet) to the overall length must be at least 0.5 for Level 1 designs and 0.8 for Level 2 designs.

11.3.11 Pretreatment Forebay

Proper pre-treatment preserves a greater fraction of the Treatment Volume over time and prevents large particles from clogging orifices and filter media. Selecting an improper type of pre-treatment or designing and constructing the pre-treatment feature incorrectly can result in performance and maintenance issues.

Sediment forebays shall be installed to maintain the long-term viability of the wetland system. These forebays allow settling of a portion of the suspended sediment and reduce velocity of flow entering the system. A forebay is required at each major inlet (defined as any location contributing at least 10% of the overall drainage area) to the wetland system and must meet the following requirements:

1. Forebays consist of separate cells (beyond those discussed in previous sections) in both Level 1 and Level 2 designs and are formed by acceptable barriers (see VDOT Detail SWM-11: Constructed Wetland (2013)).
2. Forebay shall be a maximum depth of 4’ at the inlet, or as determined by **Equation 11.1**, but transition to a 1’ depth at the entrance into the first wetland cell.
3. For safety, an aquatic bench (1’ to 2’ in depth) shall be installed around perimeter (4’ to 6’ in width). This bench shall transition to 0’ in width at the entrance into the first wetland cell.
4. Total volume stored in all forebays shall be at least 15% of the total treatment volume. If multiple forebays are include, relative volumetric

sizing should be related to the percentage of total volumetric inflow into the wetland at each location.

5. The bottom of the forebay may include a concrete surface for easier maintenance. This item should be discussed with the VDOT District Office prior to integration into the design.
6. A metered rod should be installed within the forebay to monitor long term sediment accumulation and to aid in scheduling maintenance.

For forebay design design information, refer to **Appendix D: Sediment Forebays** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site, at the following web address:
http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

Other forms of pre-treatment for sheet flow and concentrated flow at minor inflow points should be designed consistent with pre-treatment criteria found in **Section 6.4** of **Stormwater Design Specification No. 9: Bioretention**.

11.3.12 Geotechnical Testing

Soil borings shall be provided at the following (minimum) locations:

1. Within the footprint of the proposed embankment(s).
2. At the location of the proposed outlet structure.
3. A minimum of two locations within the proposed treatment area.

Data from the borings will be used to:

1. Determine the adequacy of excavated material for structural fill.
2. Determine the need for and design requirements related to an embankment cut-off trench.
3. Provide data regarding bearing capacity and buoyancy analysis for use in designing outlet works.
4. Determine design depth to seasonal high groundwater and bedrock.
5. Determine potential infiltration losses (and the potential need for a liner).

11.3.13 Embankment

The top width of the embankment should be a minimum of 10' in width to provide ease of construction and maintenance. The design of the dam should be in accordance with **Appendix A: Earthen Embankments** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site:
http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

To permit mowing and other maintenance, the embankment slopes should be no steeper than 4H:1V., or 3H:1V if a safety bench is employed.

11.3.14 Inlet Protection

Inlet areas should be stabilized to ensure that non-erosive conditions exist during storm events up to the overbank flood event (i.e., the 10-year storm event). Inlet pipe inverts should generally be located at or slightly below the permanent pool elevation.

11.3.15 Principal Spillway

Weir spillways have a large cross-sectional area that can pass a considerable flow rate at low head conditions. Since reducing the depth of ponding in a constructed wetland helps to avoid stressing plant communities, an armored, weir-type spillway may be the most desirable overflow device for a constructed stormwater wetland. Further, the use of an adjustable weir will help maintain the proper water surface elevation during seasonal extremes.

Design the principal spillway with acceptable anti-flotation, anti-vortex and trash rack devices. The spillway must generally be accessible from dry land. Refer to **Appendix B: Principal Spillways** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site: http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

11.3.16 Conveyance and Overflow

Several conveyance and overflow conditions should exist that will effectively introduce and transfer the treatment volume through the constructed wetland cells. These include:

1. The slope profile within individual wetland cells should be generally flat from inlet to outlet (adjusting for micro-topgraphy), and the maximum elevation change between adjacent wetland cells shall be 12” or less.
2. A maximum depth of 4’ over the normal pool elevation is recommended during 10-year and 100-year flooding events for on-line facilities.
3. The designer should consider using flashboard risers to allow modification of normal pool elevation after construction.
4. The discharge from the pond cell to the wetland cells in a Level 2 Pond/Wetland should be through reverse-slope pipes. The invert from the pond should be situated at least 2’ below the normal pool elevation to prevent clogging by floating organic matter (leaves, grass clippings, etc.). A gate valve may be included in the design to provide the ability to adjust outflow for fluctuating inflows throughout the year.
5. A minimum 3” diameter orifice is recommended to prevent clogging; however, in certain situations, this may be reduced to 1” when using internal orifice plates within the pipe. Approval for reduction to sizes less than 3” shall be requested through VDOT prior to integration in final design.

6. All outlet controls shall be protected by trash control measures (i.e. trash racks, inverted pipes, etc.)

11.3.17 Emergency Spillway

Wet Ponds must be constructed with overflow capacity to pass the 100-year design storm event through either the Primary Spillway (with 2' of freeboard to the settled top of embankment) or a vegetated or armored Emergency Spillway (with at least 1' of freeboard to the settled top of embankment). The emergency spillway shall be stabilized with rip rap, concrete, or any other non-erodible material approved by the VDOT Material Division. Refer to **Appendix C: Emergency Spillways** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site at the following URL:
http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

11.3.18 Landscaping Plan

The landscaping plan shall be developed by a wetlands expert or a certified landscape architect with input from the design engineer regarding the aerial extent of various zones. Planting shall be in accordance with standards specified in the VDOT Special Provision for Constructed Wetland (2014). The plan should contain native species that exist in surrounding native wetlands to the extent possible. For extensive information regarding plant selections for various wetland zones, the design professional is referred to the Virginia Stormwater Design Specification No. 13, Constructed Wetland, Draft (DCR/DEQ, 2013). Additional recommendations regarding pond landscaping can be found in *Appendix E: Landscaping* of the Introduction to the New Virginia Stormwater Design Specifications , as posted on the Virginia Stormwater BMP Clearinghouse web site: http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

11.3.19 Construction and Maintenance

Construction guidelines and maintenance access requirements are found in the VDOT Special Provision for Constructed Wetland (2014).

11.3.20 Winter Performance

Due to the likelihood of influx of salt and/or sand during winter months because of treatment operations, several modifications should be implemented in constructed wetland systems related to VDOT projects. Note that items 2-4 below are only require in colder regions of the Commonwealth. Consult the District Office to determine if these modifications are required on individual projects. Modifications may include:

1. Plant salt-tolerant vegetation.

2. Restrict submergence of inlet pipes and provide slopes of 1% minimum on pipes to discourage ice formation (Note: this is only required in the colder regions).
3. Angle trash racks to prevent build-up of ice.
4. Over-size riser and/or weirs to compensate for ice build-up.
5. Increase the pretreatment forebay size to accommodate increased loading.

11.4 Design Example

This section presents the design process applicable to constructed wetlands serving as water quality BMPs. The pre- and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 11 of the *Virginia Stormwater Management Handbook, 2nd Edition (DCR/DEQ, 2013)* for details on hydrologic methodology.

The proposed project includes the installation of an additional lane along a section of I-295 adjacent to Fort Lee near Hopewell, Virginia. The hydrologic classification of on-site soils over the entire site is HSG B/D. The B/D designation indicates that for un-drained soils (in their native condition), a designation of D is used. For this site, due to the proximity of Fort Lee, the site is assumed to be in its natural condition for undisturbed areas. Portions of existing lanes draining to the wetland area are assumed to be drained, and therefore are HSG B. The disturbed area of the project within this drainage area is approximately 3.15 acres; however, a contributing drainage area of 15.3 acres (total including existing lanes and adjacent R/W) drains to the proposed site of the constructed wetland. Pre-development and post-development conditions within the contributing drainage area are described in **Table 11.4**. The time of concentration to the constructed wetlands as determined by standard methodology (see *VDOT Drainage Manual (2014),* Chapter 6, Hydrology) is 27.5 minutes. The project site does exhibit the presence of a high groundwater table that must be incorporated with the design. Geotechnical borings do not indicate the presence of significant bedrock within 5' vertically below the proposed maximum depth of deep pools.

Table 11.4 - Hydrologic Characteristics of Total Example Project Site

		Impervious	Impervious	Turf	Forest
Pre	Soil Classification	HSG B	HSG D	HSG D	HSG D
	Area (acres)	4.36	0.00	1.70	9.24
Post	Soil Classification	HSG B	HSG D	HSG D	HSG D
	Area (acres)	4.36	1.45	1.70	7.79

Step 1 - Enter Data into VRRM Spreadsheet

The required site data must be input into the VRRM Spreadsheet for Redevelopment (2014), to determine the required load reduction of phosphorus for this linear site. Note that using the redevelopment spreadsheet, the required reduction for linear projects is computed as the sum of the Post-Redevelopment Load and the Post-Development Load minus 80% of the Predevelopment Listed load. Although the total contributing drainage area is defined by components listed in **Table 11.4**, the area that is used to calculate water quality improvements is tied to the actual disturbed area of 3.15 acres. It is these components that should be first entered into the Runoff Reduction Spreadsheet to determine required removal. The breakdown of the 3.15 acres, which is used in the 'Site Data' tab of the spreadsheet for the disturbed area in pre- and post-development conditions, is shown in **Table 11.5**. The resulting summary output from the spreadsheet is then shown in **Table 11.6**.

Table 11.5 - Hydrologic Characteristics of Disturbed Area

		Impervious	Turf	Forest
Pre	Soil Classification	HSG D	HSG D	HSG D
	Area (acres)	0.00	1.70	1.45
Post	Soil Classification	HSG D	HSG D	HSG D
	Area (acres)	1.45	1.70	0.00

Table 11.6 - Summary of Output from VRRM Site Data Tab

Site Rv	**0.57**
Post-development Treatment Volume (ft^3)	**6543**
Post-development TP Load (lb/yr)	**4.11**
Total TP Load Reduction Required (lb/yr)	**3.20**

Output from the RRM Summary Spreadsheet is shown in **Table 11.6**, and indicates that the required removal load is 3.20 lbs/yr. Although a Level 1 constructed wetland treating only the disturbed area does not meet the requirement (only resulting in a net removal of 2.05 lbs/yr), an analysis performed by inputting and treating the full drainage area to the constructed wetland, including treatment of the two additional (undisturbed) lanes and the remaining upstream drainage area, results in a load reduction of 6.77 lbs/year as indicated in **Table 11.7**. This is achieved by input of the post-development land use given in **Table 11.4** in to the Drainage Area tab of the spreadsheet, and treating the area with a Level 1 constructed wetland.

Table 11.7 - Summary of Output from VRRM Site Data Tab for Full Treatment Area

Total Impervious Cover Treated (acres)	**5.81**
Total Turf Area Treated (acres)	**1.70**
Total TP Load Reduction Achieved in D.A. A (lb/yr)	**6.77**

Step 2 - Compute the Required Treatment Volume

The treatment volume can be calculated using **Section 1, Equation 1** or taken directly from the VRRM Spreadsheet Drainage Area tabs. For this example, the reported treatment volume on the drainage area tab (treating the 15.30 acre area described by post-development data in **Table 11.4**) is 22,992 ft^3.

Step 3 - Enter Data in Channel and Flood Protection Tab

Hydrologic computations for required design storms for flood and erosion compliance must be computed to verify that design components meet guidelines.

Values for the 1-, 2-, and 10-year 24- hour rainfall depth should be determined from the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 and entered into the “Channel and Flood Protection” tab of the spreadsheet. For this site (Lat 37.2751, Long -77.3311), those values are shown in **Table 11.8**. Curve numbers used for computations should be those calculated as part of the runoff reduction spreadsheet (Virginia Runoff Reduction Spreadsheet for Linear Development, 2015). For runoff draining to the constructed wetlands, adjusted curve numbers from the runoff reduction spreadsheet are shown in **Table 11.9.** Note that constructed stormwater wetlands receive no volume reduction credit.

Table 11.8 - Rainfall Totals from NOAA Atlas 14

	1-year storm	2-year storm	10-year storm
Rainfall (inches)	2.79	3.38	5.14

Table 11.9 - Unadjusted CN from Runoff Reduction Channel and Flood Protection

	1-year Storm	2-year Storm	10-year Storm
$RV_{Developed}$ (in) with no Runoff Reduction	1.41	1.91	3.50
$RV_{Developed}$ (in) with Runoff Reduction	1.41	1.91	3.50
Adjusted CN	85	85	85

Input data is used in the Natural Resource Conservation Service Technical Release 55 (NRCS TR-55, 1986) Tabular method to calculate discharge hydrographs. (**Note that other hydrologic methodologies are suitable-see VDOT Drainage Manual, Hydrology for guidance**). Peaks of those hydrographs for the 1-, 2-, and 10-year storms are reported in **Table 11.10**. These values will be used to size the conveyance downstream of the constructed wetland.

Table 11.10 – Post-development Discharge Peaks Exiting BMP

	1-year Storm	2-year Storm	10-year Storm
Discharge (cfs)	17.45	24.09	44.23

Step 4 - Sizing the Sediment Forebays

Volume of sediment forebays shall be designed to be a minimum of 15% of the treatment volume, or:

$$Volume\ Forebay(s) = 0.15 \times 22{,}992\ ft^3 = 3{,}449 ft^3$$

This volume should be distributed proportionally to total volume for each inlet location based on runoff generated from a 1” rainfall. For this design, a single inlet will introduce flow from the impervious and turf portions of the project (from the I-295 lanes and shoulder), and a secondary inlet will introduce flow from the west (undisturbed forested portions of the drainage area). Runoff volume from the impervious and turf areas can be calculated using the runoff equation found in the NRCS TR-55 Manual (1986):

$$Q = \frac{[\mathrm{P} - I_a]^2}{(\mathrm{P} - I_a) + S} \qquad (\mathbf{11.2})$$

where:

Q = runoff (inches)
P = rainfall (inches)
S = potential maximum retention after runoff begins (inches)
I_a = initial abstraction (inches) = 0.2 S

Storage, S is related to curve number (CN) by the following equation:

$$S = \frac{1000}{CN} - 10 \qquad (\mathbf{11.3})$$

Substituting **Equation 11.3** into **Equation 11.2** yields:

$$Q = \frac{\left[\mathrm{P} - 0.2(\frac{1000}{CN} - 10)\right]^2}{P + 0.8(\frac{1000}{CN} - 10)} \qquad (\mathbf{11.4})$$

Equation 11.4 can now be used with computed curve numbers for the road and forest components using information from the channel and flood protection tab of the Runoff Reduction Spreadsheet to compute runoff for each from a 1” rainfall.

$$Q_{Road} = \frac{\left[1\ in - 0.2(\frac{1000\ in}{85} - 10\ in)\right]^2}{1\ in + 0.8(\frac{1000\ in}{85} - 10\ in)} = 0.17\ \text{inches}$$

$$Q_{Forest} = \frac{\left[1\,in - 0.2(\frac{1000\,in}{77} - 10\,in)\right]^2}{1\,in + 0.8(\frac{1000\,in}{77} - 10\,in)} = 0.05 \text{ inches}$$

Using the drainage area ratios of each, the proportion of the total sediment forebay volume that should be used for each inlet area is calculated as:

$$\frac{SF_{Road}}{SF_{Forest}} = \frac{0.17\,in \times \frac{7.51\,acres}{15.3\,acres}}{0.05\,in \times \frac{7.79\,acres}{15.3\,acres}} = \frac{0.083}{0.025} \cong \frac{3.3}{1}$$

Therefore, of the total forebay volume, ~11.5% should be used to treat runoff from the road components, and ~3.5% from the forested inflow components. Therefore, the sediment forebay for the forested area will be 22,992 ft^3 x 0.035, or 805 ft^3, and that of the road area would be the remaining 2,644 ft^3. The sediment forebay will not need to be increased in size since sand is not used for road treatment in this area.

Step 5 - Sizing the Various Pool Volumes

The deep pool should have a volume of approximately 25% of the design treatment volume. Therefore, the deep pool volume (V_{deep}) is calculated as:

$$V_{deep} = 0.25(T_V) = 0.25(22{,}992\,ft^3) = 5{,}748\,ft^3$$

The deep pool volume listed above includes the volume of the sediment forebays, calculated above as 3,449 ft^3. The remaining 2,299 ft^3 will be split evenly between the central pool and the micro-pool at the outlet/overflow location.

This Level 1 BMP will be designed to hold 50% of the treatment volume above the wet pool elevation for an extended drawdown of at least 24 hours. Therefore, the remainder of the treatment volume not in the storage area above the wet pool (50%) and in the deep pools (25%) is 25%.

Assuming average deep pool depths of 48”, the surface area of the deep pools are estimated as:

$$\frac{5{,}748\,ft^3}{4\,ft} = 1{,}437\,ft^2$$

Initially, the designer must assume surface area ratios (or percentages) corresponding to each component of the constructed wetland (deep pools, high marsh, and low marsh). Initially, assuming that the deep pools contain approximately 8% of the total surface area each, enables computation of an initial estimate of total surface area.

$$\frac{1{,}437\ ft^2}{0.08} = 17{,}963\ ft^2$$

Approximately 70% of the cell surface area should have elevations ranging between -6" and +6" (measures relative to the normal pool) as high marsh areas.

$$17{,}963\ ft^2 \times 0.70 = 12{,}574\ ft^2$$

The low marsh area is initially estimated as the remaining 22% of the total area, or 3,952 ft^2.

Now, the assumed surface areas must be used to estimate volumes and verify that the treatment volume has been successfully integrated in the design. Due to the use of the extended drawdown volume above the wet pool, the remaining volume to be treated in the high marsh area that is between the submerged portion of the high marsh area and the low marsh is calculated as:

$$0.50(T_V) - V_{deep} = 0.50(22{,}992\ ft^3) - 5{,}748\ ft^3 = 5{,}748\ ft^3$$

Assuming an average low marsh depth of 1', and an average submerged high marsh depth of 0.25', with the surface areas of the high marsh and low marsh components as computed above, the estimated volume of submerged storage in these areas can be calculated as:

$$0.5 \times 12{,}574\ ft^2 \times 0.25\ ft + 3{,}952\ ft^2 \times 1\ ft = 5{,}524\ ft^3$$

Because this estimate is slightly less than the required 5,748 ft^3, minor adjustments to yield the necessary volume must be made to the grading plan. It is assumed that the adjustment will be made in the low marsh area, with an adjusted volume during final design of 4,176 ft^3, but that the assumed surface area of 3,952 ft^2 can be carried forward in computations.

Summaries of the surface area and volume components of the various zones are found in **Tables 11.11 and 11.12**, respectively. Note that only 50% of the volume is shown in **Table 11.12** since the 24-hour extended drawdown volume that is temporarily stored above the permanent pool comprised 50% of the treatment volume.

Table 11.11 - Surface Area Summary of Varying Depth Zones

Zone / Depth	Surface Area (ft^2)	Percentage of Total Surface Area (%)
High Marsh (+6" to -6")	12,574	70
Low Marsh (-6 to -18")	3,952	22
Deep Pools* (0 to -48")	1,437	8
Total	***17,963***	***100***

*Includes sediment forebay and micro pool volumes

Table 11.12 - Volume Summary of Varying Depth Zones

Zone / Depth	Approximate Volume (ft^3)	Percentage of Total Treatment Volume (%)
High Marsh (0" to -6")	1,572	7
Low Marsh (-6 to -18")	4,176	18
Deep Pools* (0 to -48")	5,748*	25
Total	***11,496***	***50***

*Includes sediment forebay and micro pool volumes

Step 6 - Create Storage-Elevation Curve

After determining the required surface areas and storage volumes, the stage-storage relationship can be created. This curve is necessary for routing design storm hydrographs through the BMP to determine adequacy. **Table 11.13** presents the stage-storage relationship for this constructed wetland. The floor elevation of the wet pools has been measured to be at approximately elevation 48', above mean sea level, with the permanent pool set at 52'.

Table 11.13 - Stage – Storage Relationship

Elevation	Incremental Volume (ft^3)	Total Volume (ft^3)
48	0	0
48.5	696.96	696.96
49	696.96	1393.92
49.5	696.96	2090.88
50	696.96	2787.84
50.5	696.96	3484.8
51	1350.36	4835.16
51.5	2352.24	7187.4
52	4225.32	11412.72
52.5	7361.64	18774.36
53	9365.4	28139.76
53.5	10367.28	38507.04
54	11761.2	50268.24

Step 7 - Design of 24-hour Water Quality Drawdown Structure

The proposed facility is designed to store 50% of the treatment volume above the permanent pool. The elevation corresponding to the treatment volume of 22,992 ft^3 is approximately 52.72' (see Table **11.12**). The volume above the permanent pool elevation (52.00') is required to have a drawdown of at least 24 hours. In addition, the 1-year 24-hour storm should have a maximum ponding depth of less than 1', or a maximum elevation of 53.0'. It is recommended that the designer use hydraulic design software that has the ability to model a multi-stage structure. It is typical that many iterations may be necessary to meet multiple criteria related to the design.

For this particular installation, a combination 6" wide rectangular weir and 48"x48" riser crest conforming to the VDOT SWM-1 Standard Detail, with crest elevation at 52.6' achieves the required extended detention and impoundment goals. Note that for smaller installations, it is recommended that the drawdown and baseflow structure be a submerged inverted pipe to prevent clogging. However, due to the design volumes treated by this facility, the 6" rectangular weir is less prone to clogging by organic matter. Drawdown calculations using the designed control structure are shown in **Table 11.14**., showing that the required 24-hour drawdown is met.

Routing calculations showing the maximum depth of the 1-year 24-hour storm are shown in **Table 11.15**. Note that during routing calculations it is assumed that the starting pool elevation is at the permanent pool elevation of the facility (52'). The maximum elevation of the 1-year 24-hour storm as shown in **Table 11.14** is 52.97', which is lower than the maximum allowed elevation of 53.00'.

The conveyance pipe providing outfall from the riser structure is a 30" RCP pipe at 2.0% slope. The discharge pipe has been designed to convey the 10-year outflow to a point of adequate discharge (calculations for adequacy not shown). Modified puls routing calculations of the 10-year 24-hour post-development storm using the outlet structure and rating curves developed above result in a peak elevation of 53.36' and a peak outflow of 42.31 cfs. See abbreviated set of routing calculations for 10-year storm in **Table 11.16**. An emergency spillway for conveyance of the 100-year storm should be designed with a crest elevation of approximately 53.40'. The 100-year storm elevation is required to have a maximum elevation less than 4' above the maximum pool elevation, or 56.00'. Calculations for the 100-year storm yield a peak elevation of 53.99' if a 20' wide emergency spillway is installed at 53.40' (calculations not shown).

Table 11.14 - Extended Drawdown Calculations for 0.5T_v

Elevation (ft)	Storage (acre-ft)	Outflow (cfs)	Time (hours)
52.72	0.526	3.65	
52.63	0.488	1.23	0.1885
52.53	0.442	0.61	0.598
52.42	0.404	0.44	0.882
52.32	0.369	0.28	1.1973
52.21	0.333	0.16	1.9527
52.11	0.298	0.06	4.0597
52.00	0.262	0.00	15.2692
		Total	**24.1474**

Table 11.15 - Portion of Modified Puls Routing Analysis of 1-Year Storm

Runoff Time (hrs)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
0.00	0.50	0.50	0.26	52.00	0.00	0.00
0.10	0.56	0.56	0.27	52.01	0.01	0.01
0.20	0.62	0.62	0.27	52.03	0.01	0.01
0.30	0.67	0.67	0.28	52.04	0.02	0.02
0.40	0.76	0.76	0.28	52.06	0.03	0.03
0.50	0.85	0.85	0.29	52.08	0.04	0.04
0.60	0.94	0.94	0.30	52.10	0.05	0.05
0.70	1.18	1.18	0.30	52.12	0.07	0.07
0.80	1.43	1.43	0.31	52.15	0.10	0.10
0.90	1.68	1.68	0.33	52.19	0.13	0.13
1.00	2.80	2.80	0.34	52.24	0.19	0.19
1.10	5.22	5.22	0.37	52.33	0.31	0.31
1.20	9.73	9.73	0.43	52.50	0.57	0.57
1.30	15.10	15.10	0.52	52.71	3.07	3.07
1.40	17.45	17.45	0.60	52.89	10.69	10.69
1.50	16.83	16.83	0.63	**52.97**	15.12	15.12
1.60	13.48	13.48	0.63	**52.97**	15.15	15.15
1.70	10.05	10.05	0.61	52.93	12.78	12.78
1.80	7.69	7.69	0.59	52.88	10.17	10.17
1.90	6.25	6.25	0.57	52.83	8.14	8.14
2.00	4.82	4.82	0.56	52.80	6.56	6.56

Table 11.16 - Portion of Modified Puls Routing Analysis of 10-Year Storm

Runoff Time (hrs)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
1.00	7.86	7.86	0.4839	52.62	1.09	1.09
1.10	14.21	14.21	0.548	52.77	5.45	5.45
1.20	25.75	25.75	0.6297	52.96	14.75	14.75
1.30	39.05	39.05	0.7227	53.16	27.54	27.54
1.40	**44.23**	44.23	0.7933	53.31	38.65	38.65
1.50	42.39	42.39	0.8166	**53.36**	42.31	**42.31**
1.60	33.61	33.61	0.795	53.31	38.92	38.92
1.70	24.83	24.83	0.7467	53.21	31.2	31.2
1.80	18.9	18.9	0.6989	53.11	24.09	24.09
1.90	15.3	15.3	0.662	53.03	19.04	19.04
2.00	11.71	11.71	0.6324	52.97	15.13	15.13

Step 8 - Water Balance Calculation

To ensure that the wetland permanent marsh does not become dry during extended periods of low or absent inflow, the designer must perform a water balance calculation. **Equation 11.1**, discussed previously, includes a brief analysis of minimum pool depths related to drought conditions. The minimum deep pool depth recommended is 22". The deep pools in this analysis are proposed at 48", which exceeds the minimum depth for drought conditions.

A secondary analysis is performed for the anticipated low flow conditions. For Hopewell, Virginia, the month with the lowest average precipitation is February, at 3.19". Using this average rainfall, **Equation 11.1** is evaluated as:

$$DP = 3.19\ in \times \frac{(15.3\ ac \times 0.41)}{0.41\ ac} - 8\ in - 7.2\ in - 6\ in = 28\ inches$$

This exceeds the recommended minimum deep pool depth (22") during drought conditions.

Step 9 - Landscaping

As discussed previously, landscaping plans should be designed by a wetlands expert or a certified landscape architect with input from the design engineer regarding the aerial extent of various zones. The four inundation zones that must be evaluated for planting are:

- **Zone 1**: -6" to -12" below normal pool
- **Zone 2**: -6" to normal pool
- **Zone 3**: Normal pool to +12"
- **Zone 4**: +12" to +36"

Specific guidance on plant species suitable for each zone can be found in the Virginia Stormwater Design Specification No. 13, Constructed Wetland, Draft (DCR/DEQ, 2013). Invasive species such as cattails, Phragmites, and purple loosestrife should be avoided.

12.1 Wetlands - Overview of Practice

A wet pond is a basin that retains a portion of its inflow in a permanent pool so the basin is typically wet, even during non-runoff producing periods. Generally, stormwater runoff is stored above the permanent pool, as necessary, to provide flood control and/or downstream channel protection. Wet ponds are capable of providing downstream flood control, water quality improvement, channel erosion control, and the reduction of post-development runoff rates to pre-development levels.

Typically, wet ponds are difficult to incorporate on VDOT projects due to the area required for the footprint of the facility. Also, because wet ponds provide no runoff reduction credit, they should be used only if additional water quality improvement credit is required after all other options are exhausted. Requirements shown herein are modifications to specifications found in Virginia Stormwater Design Specification No. 14, Wet Pond (DCR/DEQ, 2013), for specific application to VDOT projects.

Wet ponds can be an important part of the stormwater quality treatment train, but they require special design considerations to minimize maintenance. Otherwise, they can become a maintenance burden, particularly if sediment accumulate or if flows cause erosion. Good design can eliminate or at least minimize such problems.

Table 12.1 - Summary of Stormwater Functions Provided by Wet Ponds
Virginia Stormwater Design Specification No. 14, Wet Pond (DCR/DEQ, 2013)

Stormwater Function	Level 1 Design	Level 2 Design
Annual Runoff Volume Reduction (RR) [1]	0%	0%
Total Phosphorus (TP) EMC Reduction[2] by BMP Treatment Process	50% (45%) [3]	75% (65%) [3]
Total Phosphorus (TP) Mass Load Removal	50% (45%) [3]	75% (65%) [3]
Total Nitrogen (TN) EMC Reduction[2] by BMP Treatment Process	30% (20%) [3]	40% (30%) [3]
Total Nitrogen (TN) Mass Load Removal	30% (20%) [3]	40% (30%) [3]
Channel Protection	Yes; detention storage can be provided above the permanent pool.	
Flood Mitigation	Yes; flood control storage can be provided above the permanent pool.	

[1] Runoff Reduction rates for ponds used for year round irrigation can be determined through a water budget computation.
[2] Change in event mean concentration (EMC) through the practice.
[3] Number in parentheses is slightly lower EMC removal rate in the coastal plain (or any location) if the wet pond is influenced by groundwater.

Sources: CWP and CSN (2008), CWP (2007)

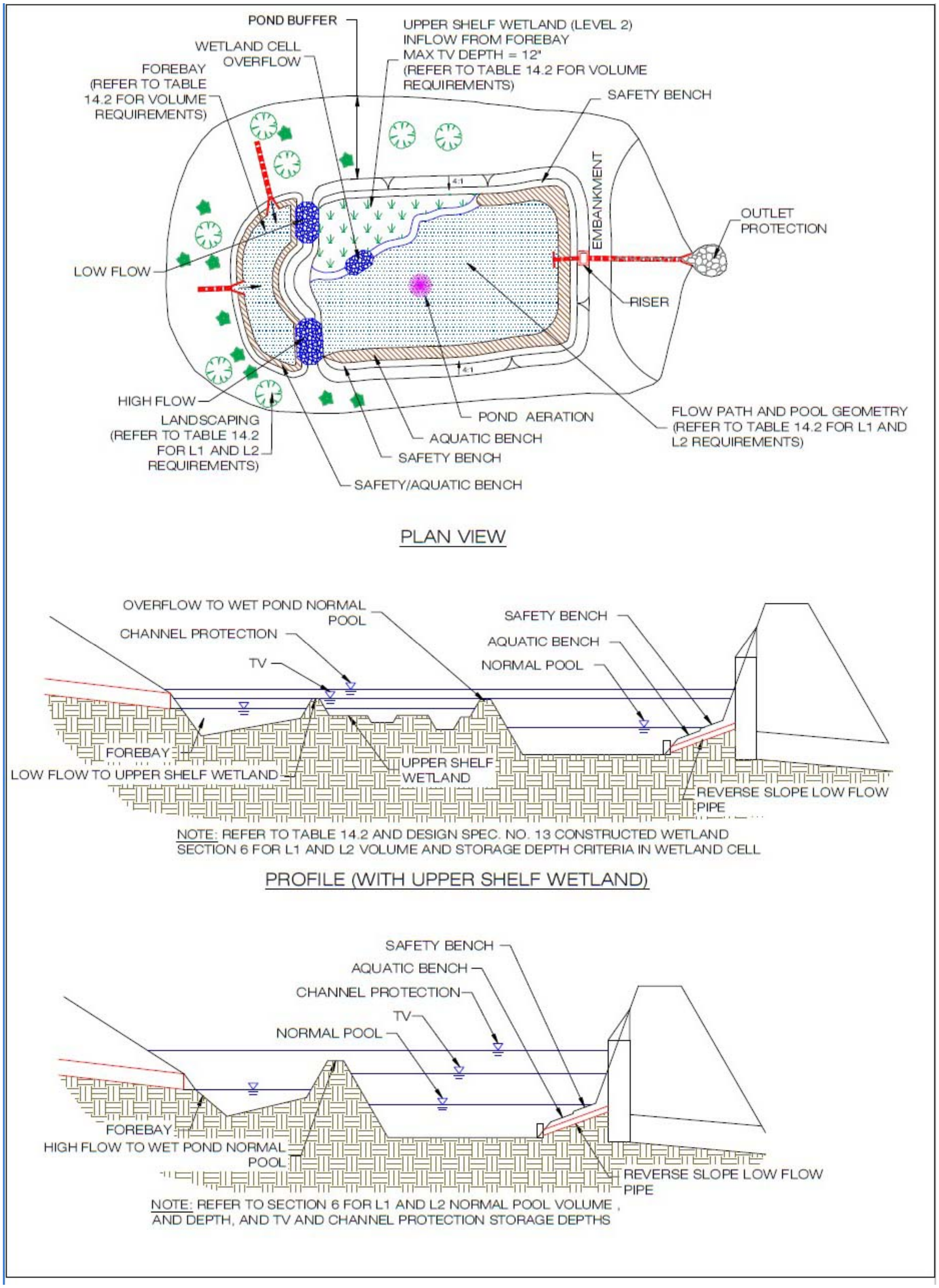


Figure 12.1 - Schematic of Wet Pond Facility
Virginia Stormwater Design Specification No. 14, Wet Pond (DCR/DEQ, 2013)

12.2 Site Constraints and Siting of the Facility

The surface area of a wet pond will normally be at least 1% to 3% of the contributing drainage area, depending on the impervious cover, pond geometry, etc. In addition to the new impervious cover in the contributing drainage area, the designer must consider additional site constraints when the implementation of wet pond is proposed. These constraints are discussed as follows.

12.2.1 Minimum Contributing Drainage Area (CDA)

The minimum contributing drainage area (CDA) to a wet pond is recommended to be 10 to 25 acres or greater in order to maintain the hydrologic and ecologic functioning of the facility. Although a smaller CDA is possible, extreme fluctuations in the permanent pool elevation can result, cause nuisances and clogging. In these cases, designers should look at implementing constructed wetlands instead of wet ponds.

It is important to design wet ponds within the limits established for CDAs. Too much or too little runoff can result in performance issues and the need for subsequent repairs or reconstruction.

12.2.2 Hydraulic Head

Typically, wet pond requires at least 6 to 8’ of head to drive the system.

12.2.3 Minimum Setbacks

Typically, the temporary pool impoundment should be no closer than 20’ to property/right-of-way lines, 25’ from foundations, 50’ to septic drain fields, and 100’ from private water supply wells. Variances to these typical setback requirements may be considered, but must be approved by the District Office.

12.2.4 Site Slopes

Generally, wet ponds should not be constructed within 50’ of any slope steeper than 15%. When this is unavoidable, a geotechnical report is required to address the potential impact of the facility in the vicinity of such a slope. Some adjustment can be made by terracing pond cells in a linear manner, using a 1’ to 2’ armored elevation drop between individual cells. Terracing may work well on longitudinal slopes with gradients up to approximately 10%.

12.2.5 Site Soils

A wet pond can be built and can function successfully on a variety of soil types. However, when such a facility is proposed, *a subsurface analysis and permeability test is required*. Soils exhibiting excessively high infiltration rates are not suited for the construction of a wet pond, as they will behave as an infiltration facility until clogging occurs. The designer should also keep in mind that as the ponded depth within the basin increases, so does the hydraulic head. This increase in hydraulic head results in increased pressure, which leads to an increase in the observed rate of infiltration. To combat excessive infiltration rates, a clay liner, geosynthetic membrane, or other material (as approved by the Materials Division) may be employed. The basin's embankment material must meet the specifications detailed later in this section and/or be approved by the Materials Division. Embankment design shall be in accordance with DCR dam safety regulations.

12.2.6 Depth to Water Table

Construction in areas with high water table is possible; however, excavation is difficult and more costly in areas with high groundwater, and pollutant removal efficiencies are typically diminished, as described in **Table 12.1**.

12.2.7 Depth to Bedrock

Typically, wet ponds are not recommended in areas with high bedrock due to the danger that fractures in the rock will allow rapid exfiltration of the wet pool. If sufficient separation between the bottom of pond and bedrock (typically 3') is employed, in addition to a pond liner, exceptions to locations may be granted by VDOT.

12.2.8 Existing Utilities

Basins should not be constructed over existing utility rights-of-way or easements. This can have significant repercussions for long-term maintenance of the basin. When this situation is unavoidable, permission to impound water over these easements must be obtained from the utility owner *prior* to design of the basin. When it is proposed to relocate existing utility lines, the costs associated with their relocation should be considered in the estimated overall basin construction cost.

12.2.9 Karst

Wet ponds are not recommended for installation in or near karst areas. If the geotechnical report indicates that less than 3' of vertical separation exists between the bottom of the pond and the underlying soil/bedrock interface, a wet pond should not be used due to the risk of sinkhole formation.. Exceptions may be granted by VDOT. If ponds are employed in karst areas, the following criteria must apply:

- A minimum of 6' of unconsolidated soil material exists between the bottom of the basin and the top of the karst layer.
- Maximum temporary or permanent water elevations within the basin do not exceed 6'.
- Annual maintenance inspections must be conducted to detect sinkhole formation. Sinkholes that develop should be reported immediately after they

have been observed, and should be repaired, abandoned, adapted or observed over time following the guidance prescribed by the appropriate local or state groundwater protection authority

- A liner is installed that meets the requirements outlined in **Table 12.2** below.

Table 12.2 - Required Groundwater Protection Liners for Ponds in Karst Terrain
Virginia Stormwater Design Specification No. 14, Wet Pond (DCR/DEQ, 2013)

Situation	Criteria
Pond *not* excavated to bedrock	24" of soil with a maximum hydraulic conductivity of 1 x 10^{-5} cm/sec.
Pond excavated to or near bedrock	24" of clay[1] with a maximum hydraulic conductivity of 1 x 10^{-6} cm/sec.
Pond excavated to bedrock within a wellhead protection area, in a recharge area for a domestic well or spring, or in a known faulted or folded area	Synthetic liner with a minimum thickness of 60 mil.
[1] Clay properties as follows: Plasticity Index of Clay = Not less than 15% (ASTM D-423/424) Liquid Limit of Clay = Not less than 30% (ASTM D-2216) Clay Particles Passing = Not less than 30% (ASTM D-422) Clay Compaction = 95% of standard proctor density (ASTM D-2216)	

Source: WVDEP (2006) and VA DCR/DEQ (1999)

12.2.10 Wetlands and Perennial Streams

Wet ponds cannot be located in jurisdictional waters without obtaining necessary permits as determined after discussing with VDOT Environmental; however, the practice is typically discouraged. The presence of wet ponds in the vicinity of natural wetlands or streams can alter the hydraulics of the area and have unintended long term consequences for the ecosystem.

12.2.11 Upstream Sediment Considerations

Close examination should be given to the flow velocity at all basin inflow points. When entering flows exhibit erosive velocities, they have the potential to greatly increase the basin's maintenance demands by transporting large amounts of sediment. Additionally, when a basin's contributing drainage area is highly pervious, there is also a risk that inflow will contain excessive sediment.

12.2.12 Floodplains

The construction of a wet pond within floodplains is strongly discouraged. When this situation is deemed unavoidable, critical examination must be given to ensure that the proposed basin remains functioning *effectively* during the 10-year flood event. The structural integrity and safety of the basin must also be evaluated thoroughly for 100-year flood conditions as well as the basin's impact on the characteristics of the 100-year floodplain. When basin construction is proposed within a floodplain, construction and permitting must comply with all applicable regulations under FEMA's National Flood Insurance Program.

12.2.13 Basin Location

When possible, wet ponds should be placed in low profile areas. When such a basin must be situated in a high profile area, care must be given to ensure that the presence of the facility does not result in nuisance conditions or have negative impacts from a wildlife management perspective (e.g., attracts abundant geese and ducks, beavers and muskrats, etc.).

"Design of any stormwater management facilities 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."

12.2.14 Discharge to Trout Streams

Impoundment of water causes increased discharge temperature due to radiant heating of the water volume. Use of wet ponds in trout stream drainage sheds is prohibited unless special permission is acquired through conversations with VDOT Hydraulics.

12.3 General Design Guidelines

The following presents a collection of broad design issues to be considered when designing a wet pond. Many of these items are expanded upon later in this document within the context of a full design scenario. A summary of general sizing requirements are found in **Table 12.3**. To avoid performance issues, the facility must be sized properly for the target Treatment Volume. However, oversizing the storage provided in the BMP, as compared to what is required to achieve the BMP's performance target, can decrease the frequency of maintenance needed and, thus, potential life-cycle costs. Oversizing, where feasible, can also help VDOT achieve its broader pollution reduction requirements associated with its DEQ MS4 Permit and the Chesapeake Bay TMDL. Oversizing options are likely to involve the adjustment of detention times and may require prior approval by DEQ.

The wet pond is designed to manage the design treatment volume within a permanent pool, multiple pool cells, or a combination of the permanent pool and extended detention storage. The design shall be based on the treatment volume of the contributing drainage area, less any volume treated (and reduced) by upstream BMPs to determine the permanent pool volume, as well as any other pond features (forebays, etc.).

12.3.1 Treatment Volume

As shown in **Table 12.3**, Level 1 facilities are designed based on the total contributing area treatment volume, while a Level 2 facility requires an additional 50% treatment volume, or $1.50(R_v)(A)$. For Level 1 facilities, the entire treatment volume should be below the permanent pool elevation, while several volume distribution options exist for Level 2 facilities. For Level 2 facilities, the treatment volume shall either:

- Be treated below the permanent pool in a minimum of 3 internal cells (one of which may be a sediment forebay)

Up to 50% may be treated in extended detention above the permanent pool elevation when using one or multiple cells.

Table 12.3 - *Wet Pond Design Criteria*
Virginia Stormwater Design Specification No. 14, Wet Pond (DCR/DEQ, 2013)

Level 1 Design (RR:0 [1]; TP: 50 [5]; TN:30 [5])	Level 2 Design (RR:0 [1]; TP: 75 [5]; TN:40 [5])
Tv = [(1.0)(Rv)(A)/12] – volume reduced by upstream BMP	Tv = [1.5 (Rv) (A) /12] – volume reduced by upstream BMP
Single Pond Cell (with forebay)	Wet ED [2] (24 hr) and/or a Multiple Cell Design [3]
Length/Width ratio OR Flow path = 2:1 or more; Length of shortest flow path / overall length [4] = 0.5 or more	Length/Width ratio OR Flow path = 3:1 or more; Length of shortest flow path/overall length[4] = 0.8 or more
Standard aquatic benches	Wetlands more than 10% of pond area
Turf in pond buffers	Trees, shrubs, and herbaceous plants in pond buffers; Shoreline landscaping to discourage geese
No Internal Pond Mechanisms	Aeration (preferably bubblers that extend to or near the bottom or floating islands

[1] Runoff volume reduction can be computed for wet ponds designed for water reuse and upland irrigation.
[2] Extended Detention may be provided to meet a maximum of 50% of the Level 2 Treatment Volume; Refer to Design Specification 13 for ED design
[3] At least three internal cells must be included, including the forebay
[4] In the case of multiple inflows, the flow path is measured from the dominant inflows (that comprise 80% or more of the total pond inflow)
[5] Due to groundwater influence, slightly lower TP and TN removal rates in coastal plain, CSN Technical Bulletin No. 2. (2009)

Sources: CSN (2009), CWP and CSN (2008), CWP (2007)

12.3.2 Storage Volume

For a Level 1 design, the vertical depth of the permanent pool (volume equal to BMP treatment volume) should be between 4' and 6'. Depths for flood control (e.g. 2-, 10-, and 100-year events) may exceed this limitation when a multistage outlet control is employed.

For Level 2 designs, the storage volume is divided into multiple cells (**Table 12.3**), of which one cell can be a sediment forebay. Typically, other cells related to a Level 2 design consist of deep pools and wetland cells (see **Section 11**, Constructed Wetlands). Typically, the pool configuration is designed to maximize hydraulic residence time in order to boost the sediment and pollutant removal functioning of the facility. This includes design elements such as incorporation of long flow paths and relatively shallow depths through a portion of the facility. In Level 2 facilities, the allowed extended detention volume cannot exceed a depth of 12" above the permanent pool elevation; however, additional storage volume can extend to 5' above the permanent pool when providing storage for downstream channel and flood protection. Non-erodible berms or simple weirs should be used instead of pipes to separate multiple pond cells.

12.3.3 Water Balance Testing

Water balance computations must be performed in order to verify that sufficient inflows compensate for combined infiltrative and evaporative losses during extended dry periods such as a 30 day drought. **Equation 12.1** is as recommended in Virginia Stormwater Design Specification No. 14, Wet Pond (DCR/DEQ, 2013).

$$DP > ET + INF + RES - MB \qquad (\mathbf{12.1})$$

where:

DP	=	Average design depth of the permanent pool, inches
ET	=	Summer evapotranspiration rate, inches (assume 8")
INF	=	Monthly infiltration loss (assume 7.2 @ 0.01 in/hr)
RES	=	Reservoir of water for a factor of safety (assume 24")
MB	=	Measured baseflow rate to the pond, if any (convert to inches)

Translating the baseflow to inches refers to the depth within the pond. Therefore, the following equation can be used to convert the baseflow, measured in cubic feet per second (ft^3/s), to pond-inches:

$$\text{Pond inches} = ft^3/s * (2.592E6) * (12''/ft) / SA \text{ of Pond } (ft^2) \qquad (\mathbf{12.2})$$

where:

2.592E6	=	Conversion factor: cfs to ft^3/month.
SA	=	surface area of pond in ft^2

12.3.4 Internal slopes

Side slopes within the facility should typically be kept from 4H:1V to 5H:1V in exposed planting areas to facilitate vegetative growth and maintenance, and to prevent excessive erosion. Internal submerged slopes of deep pools and forebays can typically be steeper, but generally should not exceed a maximum slope of 3H:1V.

The internal slope of the pond bottom should be at least 0.5% to 1% to ensure flow proceeds within the facility toward the outlet structure.

12.3.5 Pretreatment Forebay

Proper pre-treatment preserves a greater fraction of the Treatment Volume over time and prevents large particles from clogging orifices and filter media. Selecting an improper type of pre-treatment or designing and constructing the pre-treatment feature incorrectly can result in performance and maintenance issues. For wet ponds, a forebay shall be located at all major inlet locations to trap sediment for settling prior to entering the main treatment area of the wet pond facility.

12.3.6 Internal Flow Path

Flow paths within the facility should be long and have significant sinuosity in order to promote increased hydraulic residence time. The overall flow path through the main portion of the pond should have a minimum length to width ratio of 2L:1W for Level 1 designs, and 3L:1W for Level 2 designs. This can be accomplished through incorporations of islands, berms, peninsulas and the effective placement of multiple wetland cells.

The ratio of the shortest flow path (from closest inlet to the outlet structure) should be a minimum of 0.5 for Level 1 designs and 0.8 for Level 2 designs. If these requirements cannot be met, the drainage area contributing to the closest inlet may not constitute more than 20% of the total contributing drainage area to the wet pond.

12.3.7 Benching

All pools with a depth of 4’ or greater shall employ safety and aquatic benches.

A safety bench (intended to reduce the risk of someone falling into the pond) with a minimum width of 10’ should be employed just above the permanent pool elevation. The cross slope shall be approximately 2%. Slopes below the bench should not exceed 3H:1V. If pond side slopes above the permanent pool are less than 5H:1V, benching is not required.

Aquatic benches (shallow areas just inside the perimeter of the normal pool that promote growth of aquatic and wetland plants and also provide a safety feature) shall be employed around the perimeters of forebays, micropools, and wetland pools. Depth shall range between 0 and 18”. A 10’ minimum width is required for forebays, micropools and deep pools.

Landscaping (thick shoreline vegetation) should be included in both bench types to reduce access to the water’s edge by humans or geese.

12.3.8 Inlet Protection

Inlet areas should be stabilized to ensure that non-erosive conditions exist during storm events up to the overbank flood event (i.e., the 10-year storm event). Inlet pipe inverts should generally be located at or slightly below the permanent pool elevation.

12.3.9 Principal Spillway

Design the principal spillway with acceptable anti-flotation, anti-vortex and trash rack devices. The spillway must generally be accessible from dry land. Refer to **Appendix B: Principal Spillways** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site:
http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

12.3.10 Low Flow Orifice

Typically, orifice sizes should be a minimum of 3" in diameter to prevent clogging. Risks to clogging of small orifices can be minimized by:

- Providing a 4' deep micropool at the outlet structure, using a reverse slope pipe (for discharge) that extends downward from the riser to an inflow point 1' below the pool elevation
- Maximizing the size of the sediment forebay to reduce the likelihood of trash reaching outlet location
- Implementation of trash racks to protect low flow orifice
- Employ a broad crested rectangular or V-notch weir, protected by half (semicircular) CMP extending 12" below the pool elevation

12.3.11 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam.

"The foundation material under the dam and the material used for the embankment of the dam should be an AASHTO Type A-4 or finer and/or meet the approval of the Materials Division. If the native material is not adequate, the foundation of the dam is to be excavated and backfilled a minimum of 4' or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."

The design of the dam should employ a homogenous embankment with seepage controls or zoned embankments, or similar design in accordance with recommendations of the VDOT Materials Division.

Soil borings should be conducted within the footprint of the proposed embankment, in the vicinity of the proposed outlet structure, and in at least two locations within the proposed Wet Pond treatment area. Soil boring data is needed to (1) determine the physical characteristics of the excavated material to determine its adequacy as structural fill or for other uses, (2) determine the need and appropriate design depth of the embankment cut-off trench; (3) provide data for structural designs of the outlet works (e.g., bearing capacity and buoyancy), (4) determine the depth to groundwater and bedrock and (5) evaluate potential infiltration losses (and the potential need for a liner).

During the initial subsurface investigation, additional borings should be made near the center of the proposed basin when:

- Excavation from the basin will be used to construct the embankment
- There is a potential of encountering rock during excavation
- A high or seasonally high water table, generally 2' or less, is suspected

12.3.12 Embankment

The top width of the embankment should be a minimum of 10' in width to provide ease of construction and maintenance. The design of the dam should be in accordance with **Appendix A: Earthen Embankments** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site:

http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

To permit mowing and other maintenance, the embankment slopes should be no steeper than 4H:1V., or 3H:1V if a safety bench is employed.

12.3.13 Embankment Height

A detention basin embankment may be regulated under the Virginia Dam Safety Act, Article 2, Chapter 6, Title 10.1 (10.1-604 et seq.) of the Code of Virginia and Dam Safety Regulations (4 VAC 50-20 et seq.) established by the Virginia Soil and Water Conservation Board (VS&WCB). A detention basin embankment may be excluded from regulation if it meets any of the following criteria:

- is less than 6' in height
- has a capacity of less than 50 acre-ft and is less than 25' in height
- has a capacity of less than 15 acre-ft and is more than 25' in height
- will be owned or licensed by the Federal Government

When an embankment is not regulated by the Virginia Dam Regulations, it must still be evaluated for structural integrity when subjected to the 100-year flood event.

12.3.14 Outfall Piping

The pipe culvert under or through the basin's embankment shall be reinforced concrete equipped with rubber gaskets. Pipe: Specifications Section 232 (AASHTO M170), Gasket: Specification Section 212 (ASTM C443).

A concrete cradle shall be used under the pipe to prevent seepage through the dam. The cradle shall begin at the riser or inlet end of the pipe, and run the full length of the pipe.

The design must specify an outfall that will be stable for the maximum (pipe-full) design discharge (the 10-year design storm event or the maximum flow when surcharged during the emergency spillway design event, whichever is greater). The channel immediately below the pond outfall must be modified to prevent erosion and conform to natural dimensions in the shortest possible distance. Outlet protection should be provided consistent with guidelines provided in the *VDOT Drainage Manual (2014).*

12.3.15 Emergency Spillway

Wet Ponds must be constructed with overflow capacity to pass the 100-year design storm event through either the Primary Spillway (with 2' of freeboard to the settled top of embankment) or a vegetated or armored Emergency Spillway (with at least 1' of freeboard to the settled top of embankment). The emergency spillway shall be stabilized with rip rap, concrete, or any other non-erodible material approved by the VDOT Material Division. Refer to **Appendix C: Emergency Spillways** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site at the following URL:
http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

12.3.16 Safety and Fencing

Although most projects will be in limited access areas, safety measures shall be employed on all pond components and outfall structures to ensure public safety. Trash racks and/or fencing shall be used on principle outlet structures and pipe outfalls to prevent access.

- o Ponded depths greater than 3' and/or excessively steep embankment slopes
- o The basin is situated in close proximity to schools or playgrounds, or other areas where children are expected to frequent
- o It is recommended by the VDOT Field Inspection Review Team, the VDOT Residency Administrator, or a representative of the City or County who will take over maintenance of the facility

"No Trespassing" signs should be considered for inclusion on all detention facilities, whether fenced or unfenced.

12.3.17 Discharge Protection

All basin outfalls must discharge into an adequate receiving channel per the most or meet the channel protection requirements of the Virginia Stormwater Management Regulations. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year discharge event, or the design discharge through the emergency spillway, whichever is greater.

12.3.18 Drawdown System

Wet ponds shall be designed with a system for drawdown in order to perform maintenance and remove accumulated sediment. The draw down pipe should have a gate valve, or similar device, installed to allow manual operation during drawdown activities. Note that the design of valve system should take into account expected debris buildup in the draw down piping, which may affect the operation of the valve.

If a gravity based drawdown system is not feasible, such as in areas with high groundwater conditions, a pump wet well shall be provided for incorporation of temporary pumps required to draw down the permanent pool for maintenance activities.

12.3.19 Pond Liners

When a wet pond is located over permeable soils (greater than $1x10^{-6}$ cm/sec) or fractured bedrock, a liner may be needed to sustain a permanent pool of water. Suitable options for liners may include:

- A clay liner following the specifications outlined in Table **12.3**
- A 30 mil poly-liner
- Bentonite
- Chemical additives
- Alternative engineering design, as approved on a case-by-case basis by VDOT.

A clay liner meeting the specifications shown in **Table 12.3** should have a minimum thickness of 12” with an additional 12” layer of compacted soil above. If the pond is being constructed in Karst terrain, the liner must conform to criteria in **Table 12.2**.

Table 12.4 - Clay Liner Specifications

Virginia Stormwater Design Specification No. 14, Wet Pond (DCR/DEQ, 2013)

Property	Test Method	Unit	Specification
Permeability	ASTM D-2434	Cm/sec	1×10^{-6}
Plasticity Index of Clay	ASTM D-423/424	%	Not less than 15
Liquid Limit of Clay	ASTM D-2216	%	Not less than 30
Clay Particles Passing	ASTM D-422	%	Not less than 30
Clay Compaction	ASTM D-2216	%	95% of standard proctor density

Source: DCR/DEQ (1999)

12.3.20 Landscaping

A landscaping plan must be provided that indicates the methods used to establish and maintain vegetative coverage in the pond and its buffer. Minimum elements of a plan include the following:

- Delineation of pondscaping zones within both the pond and buffer
- Selection of corresponding plant species
- The planting plan
- The sequence for preparing the aquatic and safety benches (including soil amendments, if needed)
- Sources of native plant material
- The landscaping plan should provide elements that promote diverse wildlife and waterfowl use within the stormwater pond and buffers. **However to the extent possible, the aquatic and safety benches should be planted with dense shoreline vegetation to help establish a safety barrier, as well as discourage resident geese.**
- Woody vegetation may not be planted or allowed to grow within 15’ of the toe of the embankment nor within 25’ outward from the maximum water surface elevation of the wet pond. Permanent structures (e.g., buildings) should not be constructed within the buffer area. Existing trees should be preserved in the buffer area during construction.

- The soils in the stormwater buffer area are often severely compacted during the construction process, to ensure stability. The density of these compacted soils can be so great that if effectively prevents root penetration and, therefore, may lead to premature mortality or loss of vegetative vigor. As a rule of thumb, planting holes should be three times deeper and wider than the diameter of the root ball for ball-and-burlap stock, and five times deeper and wider for container-grown stock.
- Avoid species that require full shade, or are prone to wind damage. Extra mulching around the base of trees and shrubs is strongly recommended as a means of conserving moisture and suppressing weeds.

For more guidance on planting trees and shrubs in Wet Pond buffers, consult the following:

- Cappiella et al (2006)
- DCR/DEQ's Riparian Buffer Modification & Mitigation Guidance Manual, available online at:
  http://www.deq.virginia.gov/Portals/0/DEQ/Water/Publications/RiparianBufferManual.pdf

- Appendix E: Landscaping of the Introduction to the New Virginia Stormwater Design Specifications , as posted on the Virginia Stormwater BMP Clearinghouse web site: http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

The landscaping plan shall be developed by a wetlands expert or a certified landscape architect with input from the design engineer regarding the aerial extent of various zones. Planting, when incorporating constructed wetland components, shall be in accordance with standards specified in the VDOT Special Provision for Constructed Wetland (2014). The plan should contain native species that exist in surrounding native wetlands to the extent possible. For extensive information regarding plant selections for various wetland zones, the design professional is referred to the Virginia Stormwater Design Specification No. 13, Constructed Wetland (DCR/DEQ, 2013).

12.3.21 Maintenance Access

Good access to the facility is needed so maintenance crews can remove sediments, make repairs and preserve pond treatment capacity.

- Adequate maintenance access must extend to the forebay, safety bench, riser, and outlet structure and must have sufficient area to allow vehicles to turn around.
- The riser should be located within the embankment for maintenance access, safety and aesthetics. Access to the riser should be provided by lockable manhole covers and manhole steps within easy reach of valves and other controls.
- Access roads must (1) be constructed of materials that can withstand the expected frequency of use, (2) have a minimum width of 12’, and (3) have

a profile grade that does not exceed 15%. Steeper grades are allowable if appropriate stabilization techniques are used, such as a gravel surface.
- A maintenance right-of-way or easement must extend to the stormwater pond from a public or private road.

12.3.22 Pond Aeration

Level 2 designs are required to have internal aeration systems. Specific types of mechanical or electrical aerators must be approved by the VDOT Materials division prior to incorporation in design documents. Typically, an electrical connection is necessary for operation of aeration systems. Aerators can be used on a continuous, seasonal, or temporary basis as needed to maintain minimum oxygen levels.

12.3.23 Application in Coastal Plains

Due to flat terrain, low hydraulic head, and high water table, application of wet ponds in coastal plains areas is difficult. Although allowed, adjustments to nutrient removal credits are applied in these situations, as outlined in **Table 12.1**. Typically, constructed wetlands would be a preferred alternative in coastal plains areas.

12.3.24 Design Adjustments for Cold Climates and High Elevations

Wet pond performance is negatively affected in areas subject to extended cold temperatures due to ice formation and accumulation. In addition, ponds in these areas are typically subject to runoff with higher salt loading due to winter road maintenance. The following adjustments are recommended for application in these areas, as found in Virginia Stormwater Design Specification No. 14, Wet Pond (DCR/DEQ, 2013):

- Treat larger runoff volumes in the spring by adopting seasonal operation of the permanent pool (see MSSC, 2005).
- Plant salt-tolerant vegetation in pond benches.
- Do not submerge inlet pipes, and provide a minimum 1% pipe slope to discourage ice formation.
- Locate low flow orifices so they withdraw at least 6” below the typical ice layer.
- Place trash racks at a shallow angle to prevent ice formation.
- Oversize riser and weir structures to avoid ice formation and pipe freezing.
- If winter road sanding is prevalent in the contributing drainage area, increase the forebay size to accommodate additional sediment loading.

12.4 Design Example

This section presents the design process applicable to wet ponds serving as water quality BMP. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 11 of the *Virginia Stormwater Management Handbook, 2nd Edition, Draft (DCR/DEQ, 2013)* for details on hydrologic methodology.

The proposed project includes the installation of a new interchange on US 460 in Blacksburg, Virginia. The proposed intersection is to serve as a relocation and improvement in level of service classification to the existing Southgate Drive signalized intersection. The hydrologic classification of on-site soils within the contributing drainage area is a mixture of approximately 55% HSG B and 45% HSG C. Although part of a larger hydrologic analysis, the portion of the project in the contributing drainage area covered by this example drains to the proposed location of the facility on the south side of Southgate Drive, located at Lat 37.213620, and Long -80.429768. The disturbed area of the project within this drainage area is approximately 78.0 acres; however, a contributing drainage area of approximately 105.0 acres drains to the proposed site of the wet pond. Pre-development and post-development conditions within the contributing drainage area are described in **Table 12.5**, and those of the disturbed area are shown in **Table 12.6**. The time of concentration to the wet pond as determined by standard methodology (see VDOT Drainage Manual) is 42.0 minutes. The project site does not exhibit the presence of a high groundwater table. Geotechnical borings do not indicate the presence of significant bedrock within 5' vertically below the proposed basin bottom.

Table 12.5 - Hydrologic Characteristics of Disturbed Area of Example Project Site

		Imp.	Turf	Forest	Imp.	Turf	Forest
Pre	Soil Classification	HSG B			HSG C		
	Area (acres)	2.6	47.2	8.0	2.2	42.0	3.0
Post	Soil Classification	HSG B			HSG C		
	Area (acres)	30.3	19.5	8.0	29.5	14.7	3.0

The Virginia Runoff Reduction Method (VRRM) is used to compute the acceptable phosphorus load for the site and the required post-construction phosphorus removal. Use of the VRRM spreadsheet will not result in adjusted curve numbers since wet ponds do not receive any volume reduction credit. In the case of this project, conversations with Virginia Tech have resulted in plans to use this facility as a regional stormwater management facility for future development. Therefore, the numbers in **Table 12.5** are further refined to include expected build-out, as shown in **Table 12.6**, prior to entering data into the VRRM spreadsheet.

Table 12.6 - Hydrologic Characteristics of Disturbed Area of Example Project Site

		Imp.	Turf	Forest	Imp.	Turf	Forest

Pre	Soil Classification	HSG B			HSG C		
	Area (acres)	0.0	34.7	8.0	0.0	32.3	3.0
Post	Soil Classification	HSG B			HSG C		
	Area (acres)	27.7	7.0	8.0	27.3	5.0	3.0

Step 1 - Enter Data into VRRM Spreadsheet

The required disturbed area data from **Table 12.8** is input into the VRRM Spreadsheet for Redevelopment (2015), resulting in site data summary information shown in **Table 12.7**.

Table 12.7 - Summary of Output from VRRM Site Data Summary Tab

Site Rv	**0.71**
Post-development TP Load (lb/yr)	**125.69**
Total TP Load Reduction Required (lb/yr)	**97.92**

It is important to note again that the values entered in the VRRM spreadsheet (**Table 12.6**) are only the values for the disturbed area of the project. Although other areas (105.00 acres total) drain to the proposed facility as described in the problem statement, they are not part of the disturbed area, and should not be entered as such in the VRRM Spreadsheet to compute required reductions.

Information for the full drainage area (**Table 12.5**) is then entered into the Drainage Area tab of the VRRM Spreadsheet. A Level 2 wet pond is chosen for the treatment BMP, and information is entered in the appropriate cells of the spreadsheet, resulting in summary output shown in **Table 12.8**.

Table 12.8 - Summary of Output from VRRM for Level 2 Wet Pond

Total Impervious Cover Treated (acres)	**59.80**
Total Turf Area Treated (acres)	**34.20**
Total TP Load Reduction Achieved in D.A. A (lb/yr)	**109.25**

In this case, the total phosphorus reduction required is 97.92 lbs/yr. The estimated removal is 109.25 lbs/yr; therefore, the target has been met.

Step 2 - Compute the Required Treatment Volume

The treatment volume can be calculated using **Section 1, Equation 1** or taken directly from the VRRM Spreadsheet Drainage Area tabs. For this example, the treatment volume is computed using the **Section 1** equations.

$$R_{v_F} = \frac{(8.0\ \text{acres})(0.03)}{11.0\ \text{acres}} + \frac{(3.0\ \text{acres})(0.04)}{11.0\ \text{acres}} = 0.03$$

$$R_{v_T} = \frac{(19.50\ \text{acres})(0.20)}{34.20\ \text{acres}} + \frac{(14.7\ \text{acres})(0.22)}{34.20\ \text{acres}} = 0.21$$

$$R_{v_I} = \frac{(30.30\ \text{acres})(0.95)}{59.80\ \text{acres}} + \frac{(29.50\ \text{acres})(0.95)}{59.80\ \text{acres}} = 0.95$$

$$R_{v_{composite}} = (R_{v_I} \times \%I) + (R_{v_T} \times \%T)$$

$$R_{v_{composite}} = \left(0.03 \times \frac{11.0\ \text{acres}}{105.0\ \text{acres}}\right) + \left(0.21 \times \frac{34.2\ \text{acres}}{105.0\ \text{acres}}\right) + \left(0.95 \times \frac{59.8\ \text{acres}}{105.0\ \text{acres}}\right) = 0.61$$

Once the $R_{v_{composite}}$ has been calculated, the Treatment Volume for the 1.0" runoff through the facility can be directly computed using **Equation 1.1** for a Level 2 facility.

$$T_v = \left[\frac{(1.50)(1.0\ \text{in.})(0.61)(105.0\ \text{acres})}{12}\right] = 8.0\ \text{acre-ft} = 348{,}480\ ft^3$$

Step 3 - Enter Data in Channel and Flood Protection Tab

Hydrologic computations for required design storms for flood and erosion compliance are not shown as part of this example. The user is directed to the VDOT Drainage Manual for appropriate levels of protection and design requirements related to erosion and flood protection. However, hydrologic computations are necessary to compute peaks to design components of the Wet Pond.

Values for the 1-, 2-, and 10-year 24-hour rainfall depth should be determined from the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 and entered into the "Channel and Flood Protection" tab of the spreadsheet. For this site, those values are shown in **Table 12.9**. Curve numbers used for computations should be those calculated as part of the Runoff Reduction Spreadsheet (Virginia Runoff Reduction Spreadsheet for Redevelopment, 2014), which in this case are unadjusted. The resulting unadjusted curve numbers for all return periods are reported in the channel and flood protection tab of the VRRM spreadsheet, with a value of 84, as shown in **Table 12.10**.

Table 12.9 - Rainfall Totals from NOAA Atlas 14

	1-year storm	2-year storm	10-year storm	100-year storm
Rainfall (inches)	2.26	2.74	4.08	6.49

Table 12.10 - UnAdjusted CN from Runoff Reduction Channel and Flood Protection

	1-year Storm	2-year Storm	10-year Storm
$RV_{Developed}$ (in) with no Runoff Reduction	0.93	1.31	2.44
$RV_{Developed}$ (in) with Runoff Reduction	0.93	1.31	2.44
Adjusted CN	84	84	84

Input data is used in the Natural Resource Conservation Service Technical Release 55 (NRCS TR-55) Tabular method to calculate discharge hydrographs. (**Note that other hydrologic methodologies are suitable-see VDOT Drainage Manual, Hydrology for guidance**). Peaks of those hydrographs for the 1-, 2-, 10-, and 100-year storms are reported in **Table 12.11**. These values will be used to size the conveyance downstream of the wet pond. Although full hydraulic calculations for flood and channel protection are not fully explored in this design example, the pre-development peak flows to this location are shown in **Table 12.12** for comparison.

Table 12.11 - *Post-development Discharge Peaks to Wet Pond*

	1-year storm	2-year storm	10-year storm	100-year storm
Discharge (cfs)	61	88	172	324

Table 12.12 - Pre-development Discharge Peaks to Wet Pond

	1-year storm	2-year storm	10-year storm	100-year storm
Discharge (cfs)	12	26	77	198

Step 4 - Sizing the Sediment Forebays

A sediment forebay will be included on the inflow side of the project. The majority of runoff (>90%) will enter the facility from a single direction. Volume of the sediment forebay is required to be designed to be a minimum of 0.25" of runoff per impervious acre of contributing drainage area, or:

$$Volume\ Forebay = 0.25\ \text{in}\left(\frac{1\ ft}{12\ in}\right) \times 59.8\ \text{ac}\left(\frac{43{,}560\ ft^2}{1\ ac}\right) = 54{,}268\ ft^3$$

Step 5 - Sizing the Main Pond and Extended Detention Volumes

As a Level 2 wet pond, the facility is required to either use multiple pools to store the treatment volume below the permanent pool elevation, or provide extended detention for up to 50% of the treatment volume (24-hour minimum drawdown) within 1' above the permanent pool. For this site, the second option will be used to meet the requirements. In order to have some indication of elevations and storage, the first step is to create a storage elevation table (**Table 12.13**) from topographic data. The desire is to use existing site grades for the facility, to the extent possible, in order to limit disturbance and earthwork required at the wet pond site.

Table 12.13 - Stage – Storage Relationship

Elevation	Total Volume (acre - ft)
2024.1	0.000
2026	2.027
2028	4.967
2030	10.867
2032	20.117
2034	32.137

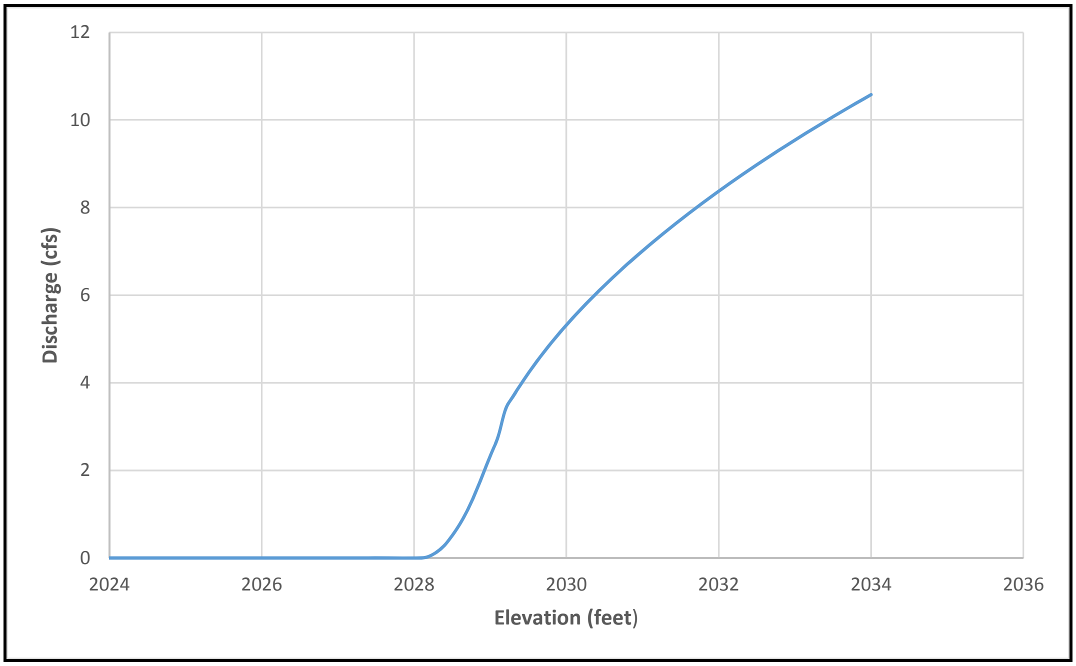
Initially, there are two particular elevations that need to be derived from the stage-storage relationship. They are the elevation corresponding to the treatment volume (8.0 acre-ft), and that corresponding to 50% of the treatment volume (4.0 acre-ft). Based on linear interpolation, the elevation corresponding to 50% of the treatment volume is 2027.34’, and that corresponding to the treatment volume is 2029.03. Because the extended detention storage volume cannot exceed 1’ (12”) above the permanent pool, a 50%-50% split is not possible since 2029.03’ – 2027.34’ is 1.69’. Therefore, the permanent pool elevation will be set at 2028.1’, which corresponds to a volume of 5.26 acre-ft. Therefore, the permanent pool will store 66% of the volume, and the extended detention portion will temporarily store the remaining 34%.

Step 6 - Design of 24-hour Water Quality Drawdown Structure

The proposed facility will be designed to store 34% of the treatment volume above the permanent pool. The volume above the permanent pool elevation is required to have a drawdown of at least 24 hours. It is recommended that the designer use hydraulic design software that has the ability to model a multi-stage structure. It is typical that many iterations may be necessary to meet multiple criteria related to the design. Because these computations are not normally done by hand, detailed orifice and grate sizing computations are not shown in this document. If hand calculations are performed, the user is directed to the *VDOT Drainage Manual* for detailed guidance on orifice and grate sizing calculations.

For this particular installation, a 1’ circular orifice at elevation 2028’.1 is proposed as the water quality orifice. Using a 0.6 orifice coefficient, the discharge elevation curve for the orifice is shown in **Figure 12.2**.

Figure 12.2 - Discharge-Elevation Curve for Circular Orifice



Next, drawdown (or empty time) calculations must be performed to ensure that the selected orifice size meets the minimum drawdown of 24 hours for the extended detention volume. Drawdown calculations using pond routing software (employing the Modified Puls routing technique) are shown in **Table 12.14**. Based on these calculations, the extended drawdown requirement is met. At this point, the designer may wish to increase the orifice size in order to decrease the drawdown time to a point closer to the 24-hour minimum; however, additional channel protection requirements requires that discharge limitations must be determined prior to increasing the orifice size.

Table 12.14 - *Extended Drawdown Calculations*

Elevation (ft)	Storage (acre-ft)	Outflow (cfs)	Time (hours)
2029.03	8.00	2.32	
2028.53	6.52	0.54	12.56
2028.10	5.41	0.21	35.91
		Total	**48.47**

Step 7 - Water Balance Calculation

To ensure that the wet pond does not become dry during extended periods of low or absent inflow, the designer must perform a water balance calculation. **Equation 12.1** calculates a recommended minimum pool depth to ensure that adequate pool volume will remain during drought conditions. The minimum deep pool depth (prior to calculation) as recommended is 22”. The deep pools in this example are proposed at 48”, which exceed the minimum depth for drought conditions.

DP > 8"+ 7.2" + 24" – 0 (**12.1**)

Although there is minimal base flow into the wet pond area, it is negligible for most of the year, and assumed to be 0, which is conservative. The equation above evaluates to a minimum deep pool depth of 39.2".

Step 8 - Permanent Pool Length to Width Ratio

The total length of the facility along the flow path from the inflow to the outflow point is 515'. The maximum width is 165'. Both of these measurements are taken at the elevation of the permanent pool. The ratio evaluates to 515:165, or 3.12:1. Therefore, the Level 2 requirement of 3:1 or higher ratio has been achieved.

Due to the direction of flow, the short circuiting ratio is not an issue for this wet pond implementation since a very small percentage of the flow enters the pond near the outlet.

Step 9 - Wetland Area Requirement

Two requirements must be achieved for the Level 2 design. First, a minimum 10' aquatic bench must be provided around the perimeter of the facility. Second, a minimum of 10% of the pond surface area (at the permanent pool elevation) must be wetland. The perimeter of the contour at the permanent pool elevation (2028.1') is 1,378', and the area is 83,372 ft^2. If the 10' aquatic bench is employed, the area will be approximately 13,207 ft^2 (note that this is slightly less than 1,378' x 10' since the 10' offset is into the wet pool area), which should be evaluated using CAD software. This area can be used to compute the wetland areal coverage and determine if additional wetland area is required. The percentage evaluates as:

$$\frac{13{,}207\,ft^2}{83{,}372\,ft^2} \times 100 = 15.8\%$$

Therefore, additional wetland area is not required above that required for the aquatic bench.

Step 10 - Buoyancy Calculation

Many wet ponds and extended detention facilities have control structures that are within the zone of saturation, which requires a full buoyancy analysis. In this case, control structures are designed to be away from the main pool, and embedded in the embankment outside the zone of saturation; therefore a buoyancy analysis is not warranted for this specific installation. For more details on buoyancy calculations, see the design example in **Section 13**, Extended Detention.

Step 11 - Design of Overflow and Conveyance Structures

Overflow and conveyance structures must be designed to pass the specified design storm based on functional classification of the road. This includes calculations for overtopping of storms of lower recurrence (i.e. 25-, 50-, and 100-year storms). These computations are beyond the scope of this design example. However, the user is directed to the VDOT Drainage Manual for guidance on flood and erosion compliance calculations, or for Section 13 for an example routing through and Extended Detention Facility.

Step 12 - Landscaping

As discussed previously, landscaping plans should be designed by a wetlands expert or a certified landscape architect with input from the design engineer regarding the aerial extent of various zones. The four inundation zones that must be evaluated for planting are:

- **Zone 1**: -6” to -12” below normal pool
- **Zone 2**: -6” to normal pool
- **Zone 3**: Normal pool to +12”
- **Zone 4**: +12” to +36”

Specific guidance on plant species suitable for each zone can be found in the Virginia DEQ Stormwater Design Specification No. 13, Constructed Wetland (2013). Invasive species such as cattails, Phragmites, and purple loosestrife should be avoided.

Step 13 - Downstream Channel Protection

Discharge locations should be evaluated using requirements set forth in the Virginia Erosion and Sediment Control Handbook, and specifically MS-19, to prevent erosion at discharge locations. The reader is directed to that reference to determine minimum sizing of outlet protection for this application. Although channel protection and flood protection was evaluated above to be adequate, these are still locations of concentrated discharge, and must be protected.

Step 14 - Pond Aeration

Pond aeration is required and will be implemented using aerators as approved by VDOT’s Materials Division. Plans shall indicate the source of power for the aerators and the type and number to be installed throughout facility.

13.1 Extend Detention Basin - Overview of Practice

An extended detention basin is defined as an impoundment which temporarily detains runoff and releases that runoff at a controlled rate over a specified period of time. Extended detention (ED) facilities are particularly effective at reducing the peak discharge for storms with lower recurrence intervals and consequently, may be effective measures for reducing downstream erosion caused by increased runoff peaks. Due to space requirements, ED facilities are difficult to implement on highway projects.

Extended detention ponds can be an important part of the stormwater quality treatment train, but they require special design considerations to minimize maintenance. Otherwise, they can become a maintenance burden, particularly if sediment accumulates if flows cause erosion. Good design can eliminate or at least minimize such problems.

An extended detention pond should be the last element in a treatment sequence and **"should be considered only if there is remaining Treatment Volume or Channel Protection Volume to manage after all other upland runoff reduction practices have been considered and properly credited"** (Virginia Stormwater Design Specification 15, Extended Detention Pond, (DCR/DEQ,2013)). Additionally, extended detention facilities should be designed to provide a 24-hour (Level 1) to 36-hour (Level 2) drawdown storage for the required treatment volume, which is dependent on the level of design. Performance credits related to the use of extended detention ponds are found in **Table 13.1**. Requirements shown herein are modifications to specifications found in Virginia Stormwater Design Specification 15, Extended Detention Pond, (DCR/DEQ,2013), for specific application to VDOT projects.

Table 13.1 - Summary of Stormwater Functions Provided by ED Ponds

Virginia Stormwater Design Specification 15, Extended Detention Pond, (DCR/DEQ,2013)

Stormwater Function	Level 1 Design	Level 2 Design
Annual Runoff Volume Reduction (RR)	0%	15%
Total Phosphorus (TP) EMC Reduction[1] by BMP Treatment Process	15%	15%
Total Phosphorus (TP) Mass Load Removal	15%	31%
Total Nitrogen (TN) EMC[1] Reduction by BMP Treatment Process	10%	10%
Total Nitrogen (TN) Mass Load Removal	10%	24%
Channel Protection	Yes; storage volume can be provided to accommodate the full Channel Protection Volume (CP_v)	
Flood Mitigation	Yes; flood control storage can be provided above the maximum extended detention volume	
[1] Change in event mean concentration (EMC) through the practice. The actual nutrient mass load removed is the product of the removal rate and the runoff reduction rate (see Table 1 in the *Introduction to the New Virginia Stormwater Design Specifications*)		

Sources: CWP and CSN (2008), CWP (2007)

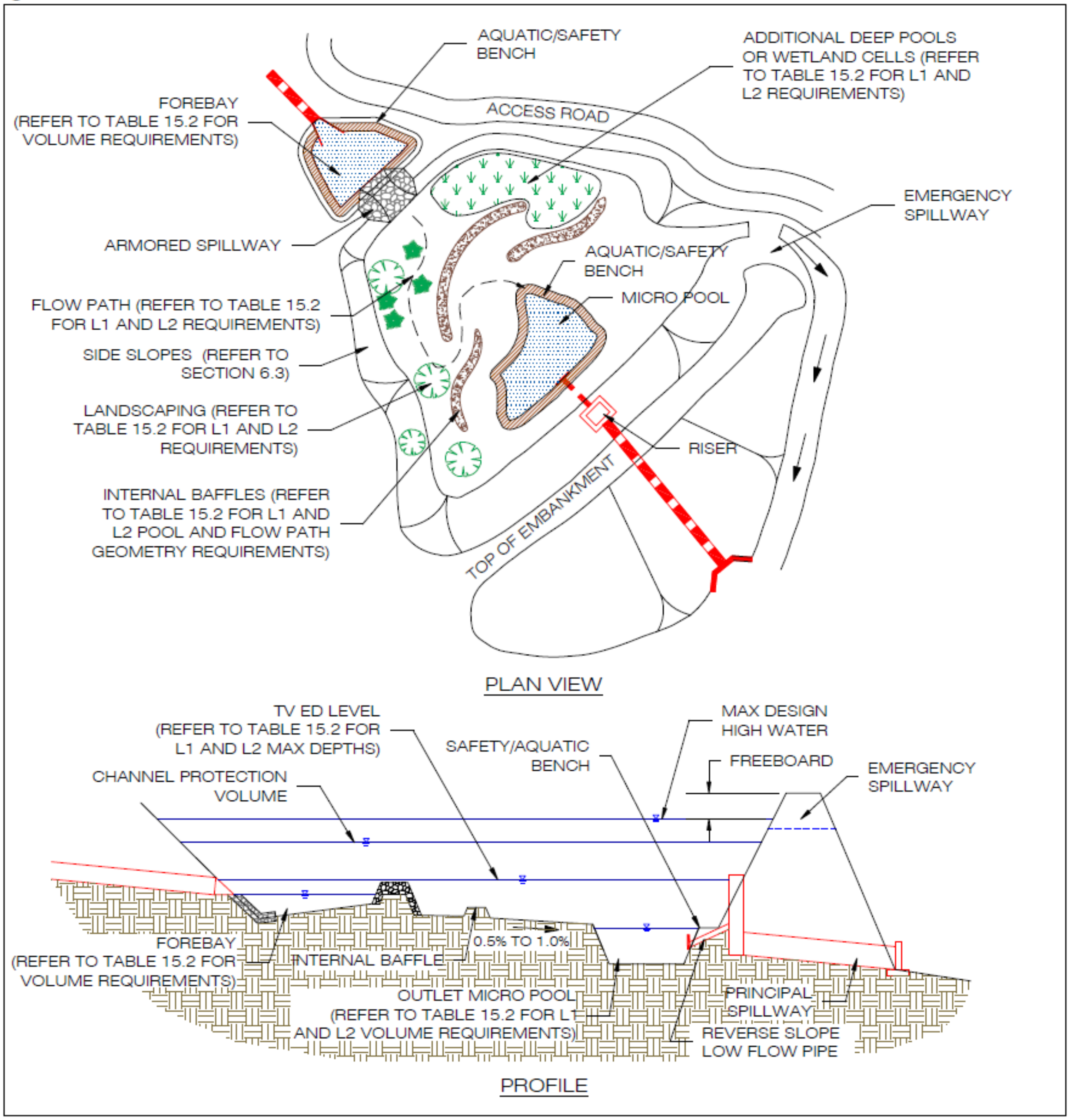


Figure 13.1 - Schematic Extended Detention Basin Plan View
Virginia Stormwater Design Specification 15, Extended Detention Pond, (DCR/DEQ, 2013)

13.2 Site Constraints and Siting of the Facility

A typical ED Pond requires a footprint of 1% to 3% of its contributing drainage area, depending on the impervious cover, pond geometry, etc. In addition to the new impervious cover in the contributing drainage area, the designer must consider additional site constraints when the implementation of an extended detention basin is proposed. These constraints are discussed below.

13.2.1 Minimum Contributing Drainage Area (CDA)

The minimum contributing drainage area (CDA) to an extended detention facility is recommended to be 10 acres or greater in order to maintain the hydrologic and ecologic functioning of the facility. Although a smaller CDA is possible, the small orifice sizes required to meet the minimum drawdowns are likely to cause clogging, increasing maintenance demands. It is important to design wet ponds within the limits established for CDAs. Too much or too little runoff can result in performance issues and the need for subsequent repairs or reconstruction.

13.2.2 Hydraulic Head

Typically, an extended detention (ED) facility requires at least 4' to 6' of head to drive the system, but the necessary head may exceed 10' if the facility is used to meet channel and flood protection requirements.

13.2.3 Minimum Setbacks

Typically, the temporary pool impoundment should be no closer than 10' to property/right-of-way lines, 25' from foundations, 35' to septic drain fields, and 50' from private water supply wells. Variations to these typical setback requirements may be considered, but must be approved by the District Office.

13.2.4 Site Slopes

Generally, extended detention basins should not be constructed within 50' of any slope steeper than 15% and, generally, not in steep terrain at all. When this is unavoidable, a geotechnical report is required to address the potential impact of the facility in the vicinity of such a slope.

13.2.5 Site Soils

The implementation of an extended detention basin can be successfully accomplished in the presence of a variety of soil types. However, when such a facility is proposed, *a subsurface analysis and permeability test is required*. Soils exhibiting excessively high infiltration rates are not suited for the construction of an extended detention facility, as they will behave as an infiltration facility until clogging occurs. The designer should also keep in mind that as the ponded depth within the basin increases, so does the hydraulic head. This increase in hydraulic head results in increased pressure, which leads to an increase in the observed rate of infiltration. To combat excessively high infiltration rates, a clay liner, geosynthetic membrane, or other material (as approved by the Materials Division) may be employed. The basin's embankment material must meet the specifications detailed later in this section and/or be approved by the Materials Division. Embankment design shall be in accordance with DCR dam safety regulations.

13.2.6 Depth to Water Table

If the depth to water table is within 2' of the basin bottom, ED basins should not be employed. Instead, shallow constructed wetlands should be considered.

13.2.7 Depth to Bedrock

The presence of bedrock within the proposed construction envelope of an extended detention basin should be investigated during the subsurface investigation. When blasting of rock is necessary to obtain the desired basin volume, a liner should be used to eliminate unwanted losses through seams in the underlying rock.

13.2.8 Existing Utilities

Basins should not be constructed over existing utility rights-of-way or easements. This can have significant repercussions for long-term maintenance of the basin. When this situation is unavoidable, permission to impound water over these easements must be obtained from the utility owner *prior* to design of the basin. When it is proposed to relocate existing utility lines, the costs associated with their relocation should be considered in the estimated overall basin construction cost.

13.2.9 Karst

The presence of Karst topography places even greater importance on the subsurface investigation. Implementation of extended detention facilities in Karst regions may greatly impact the design and cost of the facility, and must be evaluated early in the planning phases of a project. *Construction of stormwater management facilities within a sinkhole is prohibited.* When the construction of such facilities is planned along the periphery of a sinkhole, the facility design must comply with the guidelines found in Chapter 5 of this Manual and DCR/DEQ's Technical Bulletin #2 *"Hydrologic Modeling and Design in Karst."*

13.2.10 Wetlands

When the construction of an extended detention facility is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify the wetlands' boundaries, their protected status, and the feasibility of BMP implementation in their vicinity. In Virginia, the Department of Environmental Quality (DEQ) and the U.S. Army Corps of Engineers (USACOE) should be contacted when such a facility is proposed in the vicinity of known wetlands.

13.2.11 Upstream Sediment Considerations

Close examination should be given to the flow velocity at all basin inflow points. When entering flows exhibit erosive velocities, they have the potential to greatly increase the basin's maintenance demands by transporting large amounts of sediment. Additionally, when a basin's contributing drainage area is highly pervious, there is also a risk that inflow will contain excessive sediment.

13.2.12 Floodplains

The construction of extended detention facilities within floodplains is strongly discouraged. When this situation is deemed unavoidable, critical examination must be given to ensure that the proposed basin remains functioning *effectively* during the 10-year flood event. The structural integrity and safety of the basin must also be evaluated thoroughly under 100-year flood conditions as well as the basin's impact on the characteristics of the 100-year floodplain. When basin construction is proposed within a floodplain, construction and permitting must comply with all applicable regulations under FEMA's National Flood Insurance Program.

13.2.13 Basin Location

When possible, extended detention facilities should be placed in low profile areas. The location of an extended detention basin in a high profile area places a great emphasis on facility maintenance.

"Design of any stormwater management facilities 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."

Generally, installation of facilities in perennial streams or jurisdictional waters is not allowed. If no other options exist, the District office may consider allowing installation on perennial streams if the necessary state and federal permits can be obtained.

13.2.14 Discharge to Trout Streams

Impoundment of water causes increased discharge temperature due to heating of the water volume. Use of ED ponds in trout stream drainage sheds is prohibited unless upland practices meet the channel protection requirements, drawdown occurs in less than 12 hours, the outlet pool is minimized to prevent clogging and heating, the facility perimeter is planted with trees to provide full shading, and the facility is located outside of any required stream buffers.

13.3 General Design Guidelines

The following presents a collection of broad design issues to be considered when designing an extended detention basin. Many of these items are expanded upon later in this document within the context of a full design scenario. A summary of general sizing requirements is found in **Table 13.2**.

To avoid performance issues, the facility must be sized properly for the target Treatment Volume. However, oversizing the storage provided in the BMP, as compared to what is required to achieve the BMP's performance target, can decrease the frequency of maintenance needed and, thus, potential life-cycle costs. Oversizing, where feasible, can also help VDOT achieve its broader pollution reduction requirements associated with its DEQ MS4 Permit and the Chesapeake Bay TMDL. Oversizing options are likely to involve the adjustment of detention times and may require prior approval by DEQ.

Table 13.2 - Extended Detention (ED) Pond Criteria
Virginia Stormwater Design Specification 15, Extended Detention Pond, (DCR/DEQ,2013)

Level 1 Design (RR:0; TP:15; TN:10)	Level 2 Design (RR:15; TP:15; TN:10)
Tv = [(1.0) (Rv) (A)] / 12 – the volume reduced by an upstream BMP	Tv = [(1.25) (R_v) (A)] / 12 – the volume reduced by an upstream BMP
A minimum of 15% of the Tv in the permanent pool (forebay, micropool)	A minimum of 40% of Tv in the permanent pool (15% in forebays and micropool, and 25% in constructed wetlands)
Length/Width ratio *OR* flow path = 2:1 or more; Length of the shortest flow path / overall length = 0.4 or more.	Length/Width ratio *OR* flow path = 3:1 or more; Length of the shortest flow path / overall length = 0.7 or more.
Average Tv ED time = 24 hours or less.	Average Tv ED time = 36 hours.
Vertical Tv ED fluctuation may exceed 4'.	Maximum vertical Tv ED limit of 4'.
Turf cover on floor	Trees, shrubs, and herbaceous plants in upper elevations, and emergent plants in wet features
Forebay and micropool	Incudes additional cells or features (deep pools, wetlands, etc.)

13.3.1 Treatment Volume

The ED Pond is designed to hold the design T_v within the water volume below the normal pool elevation of any micro-pools, forebays and wetland areas (minimum of 15% for ED Level 1, and 40% for Level 2), as well as the temporary extended detention storage volume above the normal pool. To qualify for the higher nutrient reduction rates associated with the Level 2 design, the ED Pond must be designed with a T_v that is 25% greater [i.e., 1.25(R_v)(A)] than the T_v for the Level 1 design (additional channel protection volume is not required.

Designers should use the BMP design treatment volume, Tv_{BMP} (defined as the treatment volume based on the contributing drainage area, Tv_{DA}, minus any volume reduced by upstream runoff reduction practices) to size and design the wet features and extended detention volume. If additional detention storage is proposed for channel protection and/or flood control, designers should use the adjusted curve number reflective of the volume reduction provided by the upstream practices as well as the ED Pond (Level 2) to calculate the developed condition energy balance detention requirements. (Refer to **Chapter 11** of the Virginia Stormwater Handbook, 2nd Edition, Draft (DCR/DEQ *2013)).*

13.3.2 Depth Limitations

For a Level 1 design, the vertical depth of the treatment volume cannot exceed 5' above the basin floor or normal pool elevation. For a Level 2 design, this depth limitation is decreased to a maximum of 4'. Depths for flood control (e.g. 2-, 10-, and 100-year events) may exceed this limitation when a multistage outlet control is employed.

13.3.3 Inlet Protection

Inlet areas should be stabilized to ensure that non-erosive conditions exist during storm events up to the overbank flood event (i.e., the 10-year storm event). Inlet pipe inverts should generally be located at or slightly below the permanent pool elevation.

13.3.4 Principal Spillway

Design the principal spillway with acceptable anti-flotation, anti-vortex and trash rack devices. The spillway must generally be accessible from dry land. Refer to **Appendix B: Principal Spillways** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site:
http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

13.3.5 Internal Flow Path

ED Pond designs should have an irregular shape and a long flow path from inlet to outlet to increase water residence time, treatment pathways, and pond performance. In terms of flow path geometry, there are two design considerations: (1) the overall flow path through the pond, and (2) the length of the shortest flow path (Hirschman et al., 2009):

- The overall flow path can be represented as the length-to-width ratio *OR* the flow path ratio (refer to Figure 2 of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater

BMP Clearinghouse web site, for additional information). These ratios must be at least 2L:1W for Level 1 designs and 3L:1W for Level 2 designs. Internal berms, baffles, or topography can be used to extend flow paths and/or create multiple pond cells.

- The shortest flow path represents the distance from the closest inlet to the outlet (the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site).
  http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html
- The ratio of the shortest flow to the overall length must be at least 0.4 for Level 1 designs and 0.7 for Level 2 designs. In some cases – due to site geometry, storm sewer infrastructure, or other factors – some inlets may not be able to meet these ratios. However, the drainage area served by these "closer" inlets should constitute no more than 20% of the total contributing drainage area.
- Micro-pool ED Ponds shall not have a low flow pilot channel, but instead must be constructed in a manner whereby flows are evenly distributed across the pond bottom, to promote the maximum infiltration possible.

13.3.6 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam.

"The foundation material under the dam and the material used for the embankment of the dam should be an AASHTO Type A-4 or finer and/or meet the approval of the Materials Division. If the native material is not adequate, the foundation of the dam is to be excavated and backfilled a minimum of 4' or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."

If the basin embankment height exceeds 15', or if the basin includes a permanent pool, the design of the dam should employ a homogenous embankment with seepage controls or zoned embankments, or similar design in accordance with the recommendations of the VDOT Materials Division.

During the initial subsurface investigation, additional borings should be made near the center of the proposed basin when:

- Excavation from the basin will be used to construct the embankment
- There is a potential of encountering rock during excavation
- A high or seasonally high water table, generally 2' or less, is suspected

13.3.7 Embankment

The top width of the embankment should be a minimum of 10' in width to provide ease of construction and maintenance.The design of the dam should be in accordance with **Appendix A: Earthen Embankments** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site:

http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

To permit mowing and other maintenance, the embankment slopes should be no steeper than 4H:1V., or 3H:1V if a safety bench is employed.

13.3.8 Embankment Height

A detention basin embankment may be regulated under the Virginia Dam Safety Act, Article 2, Chapter 6, Title 10.1 (10.1-604 et seq.) of the Code of Virginia and Dam Safety Regulations (4 VAC 50-20 et seq.) established by the Virginia Soil and Water Conservation Board (VS&WCB). A detention basin embankment may be excluded from regulation if it meets any of the following criteria:

- Is less than 6' in height
- Impounds a volume of less than 50 acre-ft and is less than 25' in height
- Impounds a volume of less than 15 acre-ft and is more than 25' in height
- Will be owned or licensed by the Federal Government

When an embankment is not regulated by the Virginia Dam Regulations, it must still be evaluated for structural integrity when subjected to the 100-year flood event.

13.3.9 Benching

A safety bench (intended to reduce the risk of someone falling into the pond) with a minimum width of 10' should be employed just above the high water elevation. The cross slope shall be approximately 2%. Sloped below the bench should not exceed 3H:1V.

Aquatic benches (shallow areas just inside the perimeter of the normal pool that promote growth of aquatic and wetland plants and also provide a safety feature) shall be employed around the perimeters of forebays, micropools, and wetlands pools. Depth shall range between 0 and 18". A 4' minimum width is required for forebays, and 6' for micropools.

Landscaping (thick shoreline vegetation) should be included in both bench types to reduce access to the water's edge by humans or geese.

13.3.10 Side and Internal Slopes

Side slopes leading to the ED Pond should generally have a gradient no steeper than 4H:1V; or 3H:1V with a safety bench. The mild slopes promote better establishment and growth of vegetation and provide for easier maintenance and a more natural appearance.

The internal slope of the pond bottom should be at least 0.5% to 1% to ensure flow proceeds within the facility toward the outlet structure.

13.3.11 Prevention of Short-Circuiting

Short circuiting of inflow occurs when the basin floor slope is excessive and/or the pond's length to width ratio is not large enough. Short circuiting of flow can greatly reduce the hydraulic residence time within the basin, thus negatively impacting the desired water quality benefit.

To combat short-circuiting, and reduce erosion, the maximum longitudinal slope of the basin floor shall be no more than 2%. To maintain minimal drainage within the facility, the floor shall be no less than 0.5% slope from entrance to discharge point.

For a Level 2 facility the basis is required to have a length to width ratio of 3:1 or greater, with the widest point typically observed at the outlet end. For a Level one facility, this is reduced to a minimum 2:1 length to width ratio. When this minimum ratio is not possible, consideration should be given to pervious baffles, berms, or multiple ponding cells.

The shortest flow path (distance from closest inflow point to outlet structure) must be used to calculate the ratio of this distance to the overall flow (maximum) length in the facility. For a Level 1 facility, this ratio must be 0.4 or higher. This ratio is increased to a minimum value of 0.7 for a Level 2 design. If these ratios cannot be met, the inflow locations violating these ratios should not contain more than 20% of the contributing drainage area.

13.3.12 Low Flow Orifice

Typically, orifice sizes should be a minimum of 3" in diameter to prevent clogging. Risks to clogging of small orifices can be minimized by:

- Providing a 4' deep micropool at the outlet structure, using a reverse slope pipe (for discharge) that extends downward from the riser to an inflow point 1' below the pool elevation
- Maximizing the size of the sediment forebay to reduce likelihood of trash reaching outlet location
- Implementation of trash racks to protect low flow orifice

13.3.13 Pond Liners

When a wet pond is located over highly permeable soils or fractured bedrock, a liner may be needed to sustain a permanent pool of water. Suitable options for liners may include:

- A clay liner following the specifications outlined in Table **13.3**
- A 30 mil poly-liner
- Bentonite
- Chemical additives
- Alternative engineering design, as approved on a case-by-case basis by VDOT.

A clay liner meeting the specifications shown in **Table 13.3** should have a minimum thickness of 12” with an additional 12” layer of compacted soil above. If the pond is being constructed in Karst terrain, the liner must conform to criteria in **Table 13.4**.

Table 13.3 - *Clay Liner Specifications*
Virginia Stormwater Design Specification 14, Wet Pond, (DCR/DEQ,2013)

Property	Test Method	Unit	Specification
Permeability	ASTM D-2434	Cm/sec	1×10^{-6}
Plasticity Index of Clay	ASTM D-423/424	%	Not less than 15
Liquid Limit of Clay	ASTM D-2216	%	Not less than 30
Clay Particles Passing	ASTM D-422	%	Not less than 30
Clay Compaction	ASTM D-2216	%	95% of standard proctor density

Source: DCR/DEQ (1999)

Table 13.4 - *Liner for Karst Areas Specifications*
Virginia Stormwater Design Specification 14, Wet Pond, (DCR/DEQ,2013)

Situation	Criteria
Pond *not* excavated to bedrock	24” of soil with a maximum hydraulic conductivity of 1×10^{-5} cm/sec.
Pond excavated to or near bedrock	24” of clay[1] with a maximum hydraulic conductivity of 1×10^{-6} cm/sec.
Pond excavated to bedrock within a wellhead protection area, in a recharge area for a domestic well or spring, or in a known faulted or folded area	Synthetic liner with a minimum thickness of 60 mil.
[1] Clay properties meeting those specified in Table 12.3, with exception of hydraulic conductivity, which shall be as specified above	

Source: WVDEP (2006) and VA DCR/DEQ (1999)

13.3.14 Outfall Piping

The pipe culvert under or through the basin's embankment shall be reinforced concrete equipped with rubber gaskets. Pipe: Specifications Section 232 (AASHTO M170), Gasket: Specification Section 212 (ASTM C443).

A concrete cradle shall be used under the pipe to prevent seepage through the dam. The cradle shall begin at the riser or inlet end of the pipe, and run the full length of the pipe.

The design must specify an outfall that will be stable for the maximum (pipe-full) design discharge (the 10-year design storm event or the maximum flow when surcharged during the emergency spillway design event, whichever is greater). The channel immediately below the pond outfall must be modified to prevent erosion and conform to natural dimensions in the shortest possible distance. Outlet protection should be provided consistent with guidelines established in the VDOT Drainage Manual (2014).

13.3.15 Emergency Spillway

Wet Ponds must be constructed with overflow capacity to pass the 100-year design storm event through either the Primary Spillway (with 2' of freeboard to the settled top of embankment) or a vegetated or armored Emergency Spillway (with at least 1' of freeboard to the settled top of embankment). The emergency spillway shall be stabilized with rip rap, concrete, or any other non-erodible material approved by the VDOT Material Division. Refer to **Appendix C: Emergency Spillways** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site at the following URL:
http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

13.3.16 Safety and Fencing

Although most projects will be in limited access areas, safety measures shall be employed on all pond components and outfall structures to ensure public safety. Trash racks and/or fencing shall be used on principle outlet structures and pipe outfalls to prevent access.

Fencing is typically *not required or recommended* on most VDOT detention facilities. However, exceptions do arise, and the fencing of an extended detention facility may be needed. Such situations include:

- Ponded depths greater than 3' and/or excessively steep embankment slopes
- The basin is situated in close proximity to schools or playgrounds, or other areas where children are expected to frequent
- It is recommended by the VDOT Field Inspection Review Team, the VDOT Residency Administrator, or a representative of the City or County who will take over maintenance of the facility

"No Trespassing" signs should be considered for inclusion on all detention facilities, whether fenced or unfenced.

13.3.17 Sediment Forebays

Proper pre-treatment preserves a greater fraction of the Treatment Volume over time and prevents large particles from clogging orifices and filter media. Selecting an improper type of pre-treatment or designing and constructing the pre-treatment feature incorrectly can result in performance and maintenance issues. Each basin inflow point should be equipped with a sediment forebay. The forebay volume is dependent on design level (**Table 13.2**).

For forebay design design information, refer to **Appendix D: Sediment Forebays** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site, at the following web address:

http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

Other forms of pre-treatment for sheet flow and concentrated flow at minor inflow points should be designed consistent with pre-treatment criteria found in **Section 6.4** of Virginia Stormwater Design Specification No. 9: Bioretention, Draft (DCR/DEQ, 2013).

13.3.18 Discharge Protection

All basin outfalls must discharge into an adequate receiving channel per the most or meet the channel protection requirements of the Virginia Stormwater Management Regulations. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year discharge event, or the design discharge through the emergency spillway, whichever is greater.

13.3.19 Landscaping

A landscaping plan must be provided that indicates the methods used to establish and maintain vegetative coverage within the ED Pond and its buffer. Minimum elements of a plan include the following:

- ***Delineation of pond-scaping zones within both the pond and buffer***
- ***Selection of corresponding plant species***
- ***The planting plan***
- ***The sequence for preparing the wetland bed, if one is incorporated with the ED Pond (including soil amendments, if needed)***
- ***Sources of native plant material***
- The landscaping plan should provide elements that promote diverse wildlife and waterfowl use within the stormwater wetland and buffers.

- The planting plan should allow the pond to mature into a native forest in the right places, but yet keep mowable turf along the embankment and all access areas. The wooded wetland concept proposed by Cappiella *et al*., (2005) may be a good option for many ED Ponds.
- Woody vegetation may not be planted or allowed to grow within 15' of the toe of the embankment nor within 25' from the principal spillway structure.
- A vegetated buffer of native plants that requires minimal maintenance should be provided that extends at least 25' outward from the maximum water surface elevation of the ED Pond. Permanent structures (e.g., buildings) should not be constructed within the buffer area. Existing trees should be preserved in the buffer area during construction.
- The soils in the stormwater buffer area are often severely compacted during the construction process. The density of these compacted soils can be so great that it effectively prevents root penetration and, therefore, may lead to premature mortality or loss of vigor. As a rule of thumb, planting holes should be three times deeper and wider than the diameter of the root ball for ball-and-burlap stock, and five times deeper and wider for container-grown stock.
- Avoid species that require full shade, or are prone to wind damage. Extra mulching around the base of trees and shrubs is strongly recommended as a means of conserving moisture and suppressing weeds.

For more guidance on planting trees and shrubs in ED Pond buffers, consult Cappiella et al (2006) and **Appendix E: Landscaping** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site:
http://www.vwrrc.vt.edu/swc/NonProprietaryBMPs.html

The landscaping plan shall be developed by a wetlands expert or a certified landscape architect with input from the design engineer regarding the aerial extent of various zones. Planting, when incorporating constructed wetland components, shall be in accordance with standards specified in the VDOT Special Provision for Constructed Wetland (2014). The plan should contain native species that exist in surrounding native wetlands to the extent possible. For extensive information regarding plant selections for various wetland zones, the design professional is referred to the Virginia Stormwater Design Specification No. 13, Constructed Wetland (DCR/DEQ, 2013).

13.3.20 Maintenance Access

Good access to the facility is needed so maintenance crews can remove sediments, make repairs and preserve pond treatment capacity.

- Adequate maintenance access must extend to the forebay, safety bench, riser, and outlet structure and must have sufficient area to allow vehicles to turn around.

- The riser should be located within the embankment for maintenance access, safety and aesthetics. Access to the riser should be provided by lockable manhole covers and manhole steps within easy reach of valves and other controls.
- Access roads must (1) be constructed of materials that can withstand the expected frequency of use, (2) have a minimum width of 12', and (3) have a profile grade that does not exceed 15%. Steeper grades are allowable if appropriate stabilization techniques are used, such as a gravel surface.
- A maintenance right-of-way or easement must extend to the stormwater pond from a public or private road.

13.3.21 Application in Coastal Plains

The lack of sufficient hydraulic head and the presence of a high water table of many coastal plain sites significantly constrain the application of ED Ponds. Excavating ponds below the water table creates what are known as dugout ponds where the water quality volume is displaced by groundwater, reducing the pond's mixing and treatment efficiency and creating nuisance conditions. In general, ***shallow Constructed Wetlands are a superior alternative to ED Ponds in coastal plain settings***.

13.3.22 Design Adjustments for Cold Climates and High Elevations

Wet pond performance is negatively affected in areas subject to extended cold temperatures due to ice formation and accumulation. In addition, ponds in these areas are typically subject to runoff with higher salt loading due to winter road maintenance. The following adjustments are recommended for application in these areas, as found in Virginia Stormwater Design Specification No. 14, Wet Pond, Draft (DCR/DEQ, 2013):

- Do not submerge inlet pipes.
- Provide a minimum 1% slope for inlet pipes to discourage standing water and potential ice formation in upstream pipes.
- Place all pipes below the frost line to prevent frost heave and pipe freezing.
- Locate low flow orifices in the micro-pool so they withdraw at least 6" below the typical ice layer.
- Place trash racks at a shallow angle to prevent ice formation.
- If winter road sanding is prevalent in the contributing drainage area, increase the forebay size to 25% of the total T_v to accommodate additional sediment loadings.

13.4 Design Example

This section presents the design process applicable to extended detention serving as water quality BMP. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT projects. The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 11 of the *Virginia Stormwater Management Handbook, 2nd Edition, Draft (DCR/DEQ, 2013)* for details on hydrologic methodology.

The proposed project includes the installation of a new interchange on I-581 in Roanoke, Virginia. The hydrologic classification of on-site soils over the entire site is HSG B. Portions of existing mall parking lots and existing travel lanes, in addition to the new interchanges, ramps flow to the proposed extended detention locations. The disturbed area of the project within this drainage area is approximately 9.60 acres; however, a contributing drainage area of 23.6 acres drains to the proposed site of the extended detention. Pre-development and post-development conditions within the contributing drainage area are described in **Table 13.5**. The time of concentration to the detention facility as determined by standard methodology (see VDOT Drainage Manual) is 28.0 minutes. The project site does not exhibit the presence of a high groundwater table. Geotechnical borings do not indicate the presence of significant bedrock within 5' vertically below the proposed basin bottom.

Table 13.5 - Hydrologic Characteristics of Disturbed Area of Example Project Site

		Impervious	Turf	Forest
Pre	Soil Classification	HSG B	HSG B	HSG B
	Area (acres)	2.40	7.20	0.00
Post	Soil Classification	HSG B	HSG B	HSG B
	Area (acres)	3.60	6.00	0.00

Table 13.6 - *Remainder of Drainage Area to Extended Detention Facility (Undisturbed)*

	Impervious	Turf	Forest
Soil Classification	HSG B	HSG B	HSG B
Area (acres)	1.00	13.00	0.00

Step 1 - Enter Data into VRRM Spreadsheet

The required site data from **Table 13.5** is input into the VRRM Spreadsheet for Redevelopment (2014), resulting in site data summary information shown in **Table 13.7**.

Table 13.7 - Summary of Output from VRRM Site Data Tab

Site Rv	**0.48**

Post-development Treatment Volume (ft^3)	**16771**
Post-development TP Load (lb/yr)	**10.54**
Total TP Load Reduction Required (lb/yr)	**3.70**

It is important to note that the values in **Table 13.5** are only the values for the disturbed area of the project. Although other areas (combining to 23.60 acres total) were described in the problem statement (**Table 13.6**), they are not part of the disturbed area, and should not be entered as such in the VRRM Spreadsheet to compute required reductions.

The required removal rate is 3.70 lbs/year of phosphorous, as shown in **Table 13.7**. Although a Level 2 extended detention facility treating only the disturbed area does not meet the requirement, an analysis performed by inputting the actual drainage area to the ED facility including treatment of a portion of I-581 and existing ramps (to remain) and the remaining upstream drainage area. Appropriate data for post-development conditions is input into the VRRM Spreadsheet Drainage Area tab for a Level 2 ED facility, yielding compliance results summarized in **Table 13.8.**

Table 13.8 - Summary of Output from VRRM Site Data Tab for Full Treatment Area

Total Impervious Cover Treated (acres)	**4.60**
Total Turf Area Treated (acres)	**19.00**
Total TP Load Reduction Achieved in D.A. A (lb/yr)	**5.16**

In this case, the total phosphorus reduction required is 3.70 lbs/yr. The estimated removal is 5.16 lbs/yr; therefore, the target has been met.

Step 2 - Compute the Required Treatment Volume

The treatment volume can be calculated using **Section 1, Equation 1** or taken directly from the VRRM Spreadsheet Drainage Area tabs. For this example, the treatment volume is calculated using **Equations 1.1 and 1.2** in conjunction with information from **Table 1.1** (all found in **Section 1**). Note that the treatment volume will be computed using the disturbed area plus the “undisturbed” area which is necessary to provide adequate phosphorus load reduction.

$$R_{v_I} = \frac{(4.60\ \text{acres})(0.95)}{4.60\ \text{acres}} = 0.95$$

$$R_{v_T} = \frac{(19.00\ \text{acres})(0.20)}{19.00\ \text{acres}} = 0.20$$

$$R_{v_{composite}} = (R_{v_I} \times \%I) + (R_{v_T} \times \%T)$$

$$R_{v_{composite}} = \left(0.95 \times \frac{4.60\ \text{acres}}{23.60\ \text{acres}}\right) + \left(0.20 \times \frac{19.00\ \text{acres}}{23.60\ \text{acres}}\right) = 0.35$$

Once the $R_{v_{composite}}$ has been calculated, the Treatment Volume for the 1.0" runoff through the facility can be directly computed using **Equation 1.1** for a Level 2 facility.

$$T_v = \left[\frac{(1.25)(1.0\text{ in.})(0.35)(23.6\text{ acres})}{12}\right] = 0.860\text{ acre-ft} = 37{,}462\ ft^3$$

Step 3 - Enter Data in Channel and Flood Protection Tab

Hydrologic computations for required design storms for flood and erosion compliance are not shown as part of this example. The user is directed to the VDOT Drainage Manual for appropriate levels of protection and design requirements related to erosion and flood protection.

Values for the 1-, 2-, and 10-year 24- hour rainfall depth should be determined from the National Oceanographic and Atmospheric Administration (NOAA) Atlas 14 and entered into the "Channel and Flood Protection" tab of the spreadsheet. For this site (Lat 37.2978, Long -79.9586), those values are shown in **Table 13.9**. Curve numbers used for computations should be those calculated as part of the runoff reduction spreadsheet (Virginia Runoff Reduction Spreadsheet for Redevelopment, 2014). For runoff draining to the ED facility, results from the runoff reduction spreadsheet are shown in **Table 13.10**, and result in adjusted curve numbers of 66, 67 and 67 for the 1-, 2- and 10-year storms, respectively.

Table 13.9 - Rainfall Totals from NOAA Atlas 14

	1-year storm	2-year storm	10-year storm
Rainfall (inches)	2.60	3.14	4.70

Table 13.10 - Adjusted CN from Runoff Reduction Channel and Flood Protection

	1-year Storm	2-year Storm	10-year Storm
$RV_{Developed}$ (in) with no Runoff Reduction	0.43	0.70	1.67
$RV_{Developed}$ (in) with Runoff Reduction	0.38	0.65	1.62
Adjusted CN	66	67	67

Input data is used in the Natural Resource Conservation Service Technical Release 55 (NRCS TR-55) Tabular method to calculate discharge hydrographs. (**Note that other hydrologic methodologies are suitable-see VDOT Drainage Manual, Hydrology for guidance**). Peaks of those hydrographs for the 1-, 2-, and 10-year storms are reported in **Table 13.11**. These values will be used to size the conveyance downstream of the ED facility.

Table 13.11 – Post-development Discharge Peaks to BMP

	1-year Storm	2-year Storm	10-year Storm

Discharge (cfs)	4.49	10.25	28.06

Step 4 - Sizing the Sediment Forebays

Volume of sediment forebays and the micropool combined shall be designed to be a minimum of 15% of the treatment volume, or:

$$Volume(s) = 0.15 \times 37{,}462\ ft^3 = 5{,}619\ ft^3$$

Due to the location of this ED facility, all runoff enters the facility at a single forebay location. If multiple inlets were used along the perimeter, then the various sediment forebays would be sized proportional to the runoff volume entering each. For sizing methodology, see design problem in Section 11, Constructed Wetlands.

Of the total 5,619 ft^3 of required forebay storage, 80% (4,495 ft^3) will be in a sediment forebay, and 20% (1,124 ft^3) will be in the micropool at the outlet structure location.

Step 5 - Sizing the Various Pool Volumes

Because this is a Level 2 facility, constructed wetlands will be contained within a portion of the facility to treat a total of 25% (9,366 ft^3) of the total treatment volume (below the permanent pool elevation).

The deep pools have been sized volumetrically as part of Step 4 above, since deep pools include the sediment forebays and micropool. The extended detention facility (Level 2) is designed to hold 60% of the treatment volume above the wet pool elevation for an extended drawdown of at least 36 hours.

Approximately 70% of the cell surface area should have elevations ranging between -6” and +6” (measured relative to the normal pool) as high marsh areas. The remaining 30% of the constructed wetlands area should have depths ranging from -6” to -18” below the permanent pool. Since the total volume of the constructed wetlands is known, the surface area may be approximated as:

$$Total\ Volume = (0.70 \times Average\ Depth\ High + 0.30 \times Average\ Depth\ Low) * Area$$

Solving for Area:

$$9{,}366\ ft^3 = (0.70 \times 0.25\ ft + 0.30 \times 1\ ft) * Area$$

$$Area = 19{,}717 ft^2$$

Note that this is only an approximation and should be verified through creation of a storage elevation curve. The average depth for the high marsh area is taken as the average between the normal pool and 6” in depth (0.25’), while that of the low marsh is taken as the mean of the low marsh depth range, or 1’.

Surface areas of the deep pools (sediment forebays and micropool), assumed to have an average depth of 4', is approximated from the volume computed in Step 2 as:

$$\frac{5{,}619\ ft^3}{4\ ft} = 1{,}405\ ft^2$$

Therefore, the total estimated surface area of the facility permanent pool is the sum of 19,717 ft^2 and 1,405 ft^2, or 21,122 ft^2 (0.48 acres). Note that the VRRM process requires the wet pond areas to be calculated as impervious areas in the VRRM spreadsheet. This likely means that design is an iterative process—unless the area for the detention facility is known at the beginning of design. For purposes of this example, this impervious area of the wet pool is assumed to be included in the impervious area shown in **Table 13.5**.

Summaries of the surface area and volume components of the various zones are found in **Tables 13.12 and 13.13**, respectively. Note that only 40% of the volume is shown in Table 13.13 since the 24-hour extended drawdown volume that is temporarily stored above the permanent pool comprised 60% of the treatment volume.

Table 13.12 - ***Surface Area Summary of Varying Depth Zones***

Zone / Depth	Surface Area (ft^2)	Percentage of Total Surface Area (%)
High Marsh (+6" to -6")	13,802	65.3
Low Marsh (-6 to -18")	5,915	28.0
Deep Pools* (0 to -48")	1,405	6.7
Total	***21,122***	***100***

*Includes sediment forebay and micro pool volumes

Table 13.13 - ***Volume Summary of Varying Depth Zones***

Zone / Depth	Approximate Volume (ft^3)	Percentage of Total Treatment Volume (%)
High Marsh (0" to -6")	3,450	7
Low Marsh (-6 to -18")	5,915	18
Deep Pools* (0 to -48")	5,619*	25
Total	***14,984***	***40***

*Includes sediment forebay and micro pool volumes

Step 6 - Create Storage-Elevation Curve

After determined the required surface areas and storage volumes, the stage-storage relationship can be determined. This curve is necessary for routing design storm hydrographs through the BMP to determine adequacy. **Table**

13.14 presents the stage-storage relationship for this ED facility. The floor elevation of the wet pools has been measured to be approximately elevation 1130', above mean sea level.

Table 13.14 - *Stage – Storage Relationship*

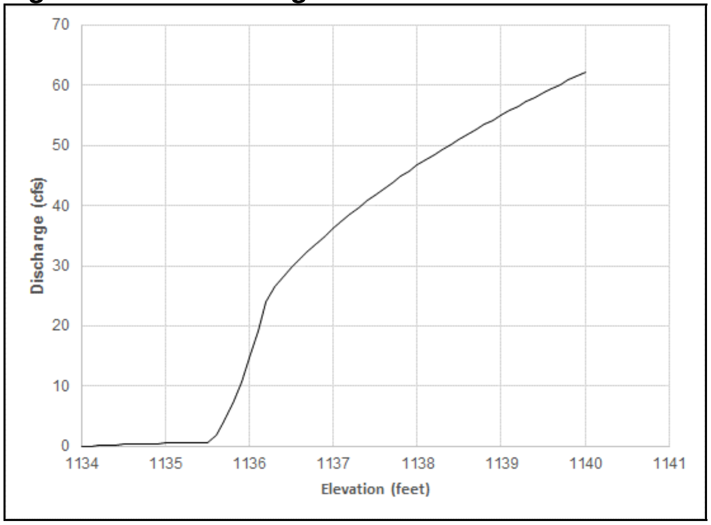
Elevation	Incremental Volume (ft^3)	Total Volume (ft^3)
1130	0	0
1131	1,405	1,405
1132	1,405	2,810
1132.5	702.5	3,513
1133	2,674	6,187
1133.5	2,674	8,861
1134*	6,124	14,984
1134.5	10,561	25,545
1135	11,000	36,545
1136	22,880	59,425
1138	47,520	106,945
1140	51,320	158,265

Step 7 - Design of 36 hour Water Quality Drawdown Structure

The proposed facility is designed to store 60% of the treatment volume above the permanent pool. The elevation corresponding to the treatment volume of 37,462 ft^3 is approximately 1135.04' (see **Table 13.14**). The volume above the permanent pool elevation (1,134.00') is required to have a drawdown of at least 36 hours. It is recommended that the designer use hydraulic design software that has the ability to model a multi-stage structure. It is typical that many iterations may be necessary to meet multiple criteria related to the design. Because these computations are not normally done by hand, detailed orifice and grate sizing computations are not shown in this example. If hand calculations are performed, the used is directed to the *VDOT Drainage Manual* for detailed guidance on orifice and grate sizing calculations.

For this particular installation, a combination 4" circular orifice at elevation 1134.0' and DI-7 Type 1 grate with top elevation 1135.50 is used as the multi-stage outlet structure. The discharge elevation curve associated with this design is shown in **Figure 13.2**. A VDOT SWM-1 Standard trash rack will be used on top of the control structure to prevent clogging. Note that for smaller installations, it is recommended that the drawdown and baseflow structure be a submerged inverted pipe to prevent clogging. However, due to the design volumes treated by this facility, the 4" circular orifice exceeds the size (3") that requires special precautions. The designer should determine if VDOT Hydraulics requires special precautions--in addition to a standard trash rack over--the low flow orifice. Drawdown calculations using the designed control structure are shown in **Table 13.15**. Abbreviated routing calculations for the 1-, 2-, 10-, and 100-year storms are shown in **Tables 13.16-13.19**, respectively. Note that routing calculations should assume that the starting pool elevation is at the permanent pool elevation of the facility (1134' in this case).

Figure 13.2 - Discharge-Elevation Curve for Outlet Structure Design



The conveyance pipe providing outfall from the riser structure is a 30" RCP pipe at 1.0% slope and invert (from riser) set at 1129.0'. This pipe size is adequate to convey the 100-year storm through the riser structure and to the receiving channel. A concrete cradle meeting the standards shown in the VDOT Drainage Manual will be installed on the 30" RCP pipe through the embankment to provide seepage control. As seen in **Table 13.19**, the peak 100-year storm elevation is 1137.70'. The top of berm of the facility, as designed, is 1140.00'. Because the freeboard is greater than 2' above the 100-year storm elevation, an emergency spillway is not required.

Elevations of design storms shown in **Tables 13.16-13.19** do not exceed the maximum depths over the permanent pool that is allowed by design standards for a Level 2 facility; therefore, the proposed basin configuration is adequate. Routed hydrographs (partially presented in **Tables 13.16-13.19**) may be used to complete downstream adequacy calculations for flood and erosion control (not shown in this example).

Table 13.15 - ***Extended Drawdown Calculations for 0.6T_v***

Elevation (ft)	Storage (acre-ft)	Outflow (cfs)	Time (hours)
1135.04	0.862	0.39	

1134.74	0.706	0.32	5.375
1134.21	0.446	0.07	16.140
1134.00	0.370	0.04	15.712
		Total	**37.227**

Table 13.16 - ***Portion of Modified Puls Routing Analysis of 1-Year Storm***

Event Time (hrs)	Hydrograph Inflow (cfs)	Storage Used (acre-ft)	Elevation MSL (feet)	Basin Outflow (cfs)
11.90	0.01	0.344	1134.00	0.00
12.00	0.06	0.344	1134.00	0.00
12.10	0.38	0.346	1134.00	0.00
12.20	1.27	0.353	1134.02	0.00
12.30	2.78	0.370	1134.05	0.01
12.40	4.14	0.398	1134.11	0.02
12.50	**4.49**	0.433	1134.18	0.06
12.60	4.15	0.468	1134.26	0.11
12.70	3.40	0.499	1134.32	0.15
12.80	2.83	0.523	1134.37	0.18
12.90	2.44	0.543	1134.41	0.21
13.00	2.05	0.560	1134.45	0.22
13.10	1.83	0.574	1134.47	0.23
~	~	~	~	~
20.50	0.35	0.760	1134.84	0.35
20.60	0.35	0.760	1134.84	0.35
20.70	0.35	0.760	1134.84	0.35
20.80	0.35	0.760	1134.84	0.35
20.90	0.35	0.760	1134.84	0.35
21.00	0.35	0.760	1134.84	0.35
~	~	~	~	~

Table 13.17 - ***Portion of Modified Puls Routing Analysis of 2-Year Storm***

Runoff Time (hrs)	Hydrograph Inflow (cfs)	Storage Used (acre-ft)	Elevation MSL (feet)	Basin Outflow (cfs)
12.20	3.58	0.371	1134.06	0.01
12.30	7.22	0.415	1134.15	0.04
12.40	10.04	0.486	1134.29	0.13
12.50	10.25	0.568	1134.46	0.23
12.60	8.92	0.645	1134.62	0.28
12.70	7.07	0.709	1134.74	0.32
~	~	~	~	~
16.80	0.80	1.114	1135.52	0.79
16.90	0.78	1.114	1135.52	0.79
17.00	0.77	1.114	1135.52	0.79
~	~	~	~	~

Table 13.18 - ***Portion of Modified Puls Routing Analysis of 10-Year Storm***

Runoff Time (hrs)	Hydrograph Inflow (cfs)	Storage Used (acre-ft)	Elevation MSL (feet)	Basin Outflow (cfs)
12.10	6.24	0.459	1134.24	0.09
12.20	13.27	0.538	1134.40	0.20
12.30	22.57	0.684	1134.69	0.30
12.40	28.06	0.890	1135.10	0.41
12.50	27.67	1.116	1135.53	0.82
12.60	22.93	1.285	1135.85	8.79
12.70	17.56	1.358	1135.99	14.10
12.80	13.67	1.367	1136.01	14.84
12.90	11.26	1.352	1135.98	13.69
13.00	8.86	1.330	1135.93	11.95
~	~	~	~	~

Table 13.19 - ***Portion of Modified Puls Routing Analysis of 100-Year Storm***

Runoff Time (hrs)	Hydrograph Inflow (cfs)	Storage Used (acre-ft)	Elevation MSL (feet)	Basin Outflow (cfs)
12.10	21.84	0.828	1134.98	0.38
12.20	40.66	1.083	1135.46	0.48
12.30	63.02	1.428	1136.12	19.79
12.40	72.77	1.771	1136.75	32.91
12.50	70.18	2.063	1137.28	39.30
12.60	56.20	2.246	1137.62	42.81
12.70	41.86	2.294	1137.70	43.68
12.80	32.02	2.242	1137.61	42.74
12.90	26.04	2.137	1137.42	40.75
13.00	20.06	2.002	1137.17	38.05
~	~	~	~	~

Step 8 - Water Balance Calculation

To ensure that the wetland permanent marsh does not become dry during extended periods of low or absent inflow, the designer must perform a water balance calculation. **Equation 11.1** (Section 11) calculates a recommended minimum pool depth to ensure that adequate pool volume will remain during drought conditions. The minimum deep pool depth as recommended is 22”. The deep pools in this analysis are proposed at 48”, which exceed the minimum depth for drought conditions.

A secondary analysis is performed for the anticipated low flow conditions. For Roanoke, Virginia, the month with the lowest average precipitation is February, at 2.87”. Using this average rainfall, **Equation 11.1** is evaluated as:

$$DP = 2.87\ in \times \frac{(23.6 \times 0.35)}{0.48} - 8\ in - 7.2\ in - 6\ in = 28\ inches$$

This analysis shows that the design pool depth of 48” is expected to be adequately maintained (drawing down to 28”) even during the month with the lowest average precipitation. If the equation is evaluated for the average July precipitation of 4.06” of rainfall, the estimated maintainable pool depth is 49”.

Step 9 - Buoyancy Calculation

A buoyancy calculation should be performed on every proposed riser structure. A minimum factor of safety of 1.25 should be provided between the weight of the structure and the uplifting buoyant force when the riser is submerged and the ground is saturated. When the summation of downward forces, including the riser's weight, are less than this buoyant force, *flotation will occur*.

The first step is to compute the buoyant force acting on the riser. The buoyant force is a function of the volume of water displaced by the riser. The calculation presented here also assumes that the basin ground is saturated, thus including the buoyant force of the volume of water displaced below grade by the riser footing. A VDOT SWM-1 is used in this design example.

Due to the use of the SWM-1 trash rack and the 30" outfall culvert, a 5' inner diameter (6' outer) manhole will be used. Displacement of water volume from the riser crest (DI-7 elevation) is calculated using the volume of the manhole [from base (typically invert minus 8")] to maximum storm depth. In this case, the total height is 1135.50' (DI-7) minus 1128.33' (base), or 7.17'.

Therefore, the volume of water displaced is computed as:

$$V_{dis} = \pi(3ft)^2(7.17ft) = 202.73\ ft^3$$

The unit weight of water is 62.4 lb/ft^3, with the buoyant force computed as:

$$F_{buoyant} = 202.73\ ft^3 \times 62.4\frac{lb}{ft^3} = 12{,}650\ lb$$

Applying the 1.24 factor of safety:

$$F_{design} = 12{,}650\ lb \times 1.25 = 15{,}813\ lb$$

The downward force is computed by calculating the summing the weights of the manhole, grates, and SWM-1 used for the structure.

Weight of manhole base:

$$F_{base} = (0.667\ ft)\pi(3ft)^2 \times 150\frac{lb}{ft^3} = 2{,}829\ lb$$

Weight of manhole riser:

$$F_{riser} = (5.84ft)[\pi(3ft)^2 - \pi(2.5ft)^2] \times 150\frac{lb}{ft^3} = 7{,}568\ lb$$

The weight of the SWM-1 trash rack is approximately 120 lbs, and the weight of the DI-7, Type 1 grate and top is approximately 2,000 lbs.

Finally, the concrete weight lost due to the presence of the 4.5" orifice must be subtracted:

$$F_{orifice} = (0.5 ft)[\pi(0.1875\ ft)^2] \times 150 \frac{lb}{ft^3} = 8.3\ lb$$

The total force down is computed as:

$$2{,}829\ lb + 7{,}568\ lb + 120\ lbs + 2{,}000\ lbs - 8.3\ lbs = 12{,}509\ lbs$$

Because this weight is less than the buoyant force (with applied safety factor) of 15,813 lbs, additional weight must be added. The simplest method of providing this additional weight is to add additional concrete to the bottom of the manhole. If the manhole is ordered with additional depth (below the invert out), the invert may be placed on site with A3 concrete filling the base of the manhole up to the invert out elevation. This will provide the additional ballast necessary to counteract the buoyant force. The additional depth needed can be directly calculated using the difference in forces and the interior radius of the manhole (5') as:

$$D_{additional} = \frac{15{,}813\ lbs - 12{,}509\ lbs}{150 \frac{lb}{ft^3} \times \pi(2.5\ ft)^2} = 1.12\ feet$$

Therefore, when ordered, the interior manhole invert should be 1127.88 or less, and concrete will be placed in the bottom up to the pipe invert out of 1129.00.

Step 10 - Landscaping

As discussed previously, landscaping plans should be designed by a wetlands expert or a certified landscape architect with input from the design engineer regarding the aerial extent of various zones. The four inundation zones that must be evaluated for planting are:

- **Zone 1**: -6" to -12" below normal pool
- **Zone 2**: -6" to normal pool
- **Zone 3**: Normal pool to +12"
- **Zone 4**: +12" to +36"

Specific guidance on plant species suitable for each zone can be found in the Virginia Stormwater Design Specification No. 13, Constructed Wetland (DCR/DEQ, 2013). Invasive species such as cattails, Phragmites, and purple loosestrife should be avoided.

References

Center for Watershed Protection (CWP). 2007. *National Pollutant Removal Performance Database Version 3.0*. Center for Watershed Protection, Ellicott City, MD.

Natural Resource Conservation Service (NRCS). 1986. *Urban Hydrology for Small Watersheds, Technical Release 55*. Government Printing Office.

Schueler, T. 2008. *Technical Support for the Baywide Runoff Reduction Method*. Chesapeake Stormwater Network. Baltimore, MD. www.chesapeakestormwater.net

Virginia Department of Conservation and Recreation/Environmental Quality. 2013. *Virginia Stormwater Design Specification No. 2, Sheet Flow to a Vegetated Filter Strip or Conserved Open Space, Draft.* Richmond, VA.

Virginia Department of Conservation and Recreation/Environmental Quality. 2013. *Virginia Stormwater Management Handbook, Second Ed., Draft.* Richmond, VA.

Virginia Department of Environmental Quality. 1992. *Virginia Erosion and Sediment Control Handbook, 3rd Ed.* Richmond, VA.

Virginia Department of Transportation. 2014. *Special Provision for Sheet Flow to Vegetated Filter Strip.* Richmond, VA.

Virginia Department of Transportation. 2014. *Special Provision for Compost Amendments.* Richmond, VA.

Virginia Department of Conservation and Recreation/Environmental Quality. 2013. *Virginia Stormwater Design Specification No. 3, Grass Channels, Draft.* Richmond, VA.

Virginia Department of Conservation and Recreation/Environmental Quality. 2013. *Virginia Stormwater Design Specification No. 10, Dry Swales, Draft.* Richmond, VA.

Virginia Department of Conservation and Recreation/Environmental Quality. 2013. *Virginia Stormwater Design Specification No. 8, Infiltration, Draft.* Richmond, VA.

Virginia Department of Transportation. 2014. *VDOT Special Provision for Grass Channels.* Richmond, VA.

Virginia Department of Transportation. 2014. *VDOT Special Provision for Stormwater Miscellaneous.* Richmond, VA.

Composting Council (TCC). 1997. *Development of a Landscape Architect Specification for Compost Utilization*. Alexandria, VA. http://www.cwc.org/organics/org972rpt.pdf

Virginia Department of Conservation and Recreation/Environmental Quality. 2013. *Virginia DCR/DEQ Stormwater Design Specification No. 4, Soil Compost Amendment, Draft.* Richmond, VA.

Virginia Department of Transportation. 2014. *VDOT Special Provision for Permeable Pavement.* Richmond, VA.

Virginia Department of Environmental Quality. 2013. *Virginia DEQ Stormwater Design Specification No. 7, Permeable Pavement.* Richmond, VA.

Virginia Department of Transportation. 2014. *VDOT Special Provision for Infiltration Practices.* Richmond, VA.

Virginia Department of Environmental Quality. 2013. *Virginia DEQ Stormwater Design Specification No. 8, Infiltration Practices.* Richmond, VA.

Virginia Department of Conservation and Recreation/Environmental Quality. 2013. *Virginia Stormwater Design Specification No. 9, Bioretention, Draft.* Richmond, VA.

Virginia Department of Transportation. 2014. *VDOT Special Provision for Bioretention Facilities.* Richmond, VA.

Virginia Department of Transportation. 2014. *VDOT Special Provision for Dry Swales.* Richmond, VA.

Virginia Department of Conservation and Recreation/Environmental Quality. 2013. *Virginia Stormwater Design Specification No. 11, Wet Swales, Draft.* Richmond, VA.

Virginia Department of Conservation and Recreation/Environmental Quality. 2013. *Virginia Stormwater Design Specification No. 13, Constructed Wetlands, Draft.* Richmond, VA.

Virginia Department of Transportation. 2014. *VDOT Special Provision for Wet Swales.* Richmond, VA.

Virginia Department of Conservation and Recreation/Environmental Quality. 2013. *Virginia Stormwater Design Specification No. 12, Filtering Practices, Draft.* Richmond, VA.

Virginia Department of Transportation. 2014. *VDOT Special Provision for Filtering Practices.* Richmond, VA.

Virginia Department of Conservation and Recreation/Environmental Quality (DCR/DEQ). 1999. *Virginia Stormwater Management Handbook. Volumes 1 and 2*. Richmond, VA.

Virginia Department of Conservation and Recreation/Environmental Quality (DCR/DEQ). 2013. *Virginia Stormwater Management Handbook, Second Ed., Draft.* Richmond, VA.

Virginia Department of Transportation (VDOT). 2014. *VDOT Special Provision for Constructed Wetlands.* Richmond, VA.

Cappiella, K., T. Schueler and T. Wright. 2006. *Urban Watershed Forestry Manual: Part 2: Conserving and Planting Trees at Development Sites*. USDA Forest Service. Center for Watershed Protection. Ellicott City, MD

Virginia Department of Environmental Quality. 2013. *Virginia DEQ Stormwater Design Specification No. 13, Constructed Wetlands.* Richmond, VA.

Virginia Department of Environmental Quality. 2013. *Virginia DEQ Stormwater Design Specification No. 14, Wet Pond.* Richmond, VA.

Hirschman, D., L. Woodworth and S. Drescher. 2009. *Technical Report: Stormwater BMPs in Virginia's James River Basin: An Assessment of Field Conditions & Programs*. Center for Watershed Protection. Ellicott City, MD.

Virginia Department of Conservation and Recreation/Environmental Quality. 2013. *Virginia Stormwater Design Specification No. 15, Extended Detention Pond.* Richmond, VA.

Appendix 11B-1 Stormwater Management and Temporary Sediment Basin Summary

OUTFALL LOCATION STA.	Total Drainage Area (AC)	Road Area (AC)	Area of Impervious (AC)	Peak Discharge (CFS)	% Diff. (2-yr)	Peak Discharge (CFS)	% Diff. (10-yr)	Vel. (FPS)	Channel Depth (FT)	Normal Depth (FT)	Channel Adequacy	D.A Draining Across Dist. Area (AC)	SWM Req'd	TSB Req'd	Diverted Outfall 100-yr Adequacy	NOTES

Appendix 11B-2 Redevelopment/Surplus Credit Tracking Form

Sample sheet:

Redevelopment/Surplus Credit Tracking Form

page 1 of 3

Rev. 4/27/16

Project Information

Point of Contact Information

Name		Address		Phone Number		Email	
Title:							

Project Information

Project Name		ID #		UPC		District	
		GPIN		Lat/Long		County	
		River Basin		Trib Name		HUC	

Narrative / Project Summary:

Qualifying Criteria

Was the redevelopment project implemented after 7/1/09	Yes ☐	No ☐
Did the project result in net reductions in pollutants?	Yes ☐	No ☐
Is the credit for treatment of VDOT ROW exclusively?	Yes ☐	No ☐
Stormwater Quality Criteria Utilized	Part IIB (new) ☐	Part IIC (old) ☐
Is the project located in an urbanized area	Yes ☐	No ☐

Notes:

Crediting Information

(*include only the creditable portion associated with redevelopment or surplus that was not offsetting new development)

Summary of Nutrient Crediting

TP Credit (lb/yr)*:				
TN Credit (lb/yr)*:		DA Entirely in MS4 UA?	Yes ☐	No ☐
TSS Credit (lb/yr)*:				

* These values represent load reductions creditable for CB TMDL purposes

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)

			TP (CBPO)	TN (CBPO)	TSS (CBPO Established)
Type of Permanent BMP	Grass Swale Level 1				
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.

SITE SUMMARY **Creditable Reduction Computations**

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:	

Implementation Information

Page 3 of 3

Estimated Implementation Date(mo/yr):		Est. Time to Construct (mo):	
Actual Implementation Date:		Cost ($):	
Project Accepted and Completed* *includes planting and initial maintenance*	Yes ❑		No ❑
Inspection/Acceptance Form Location:			
Notes:			

Certified by:		Title:		Date:	
Address:		Email:		Phone:	

Maintenance/Inspection Information*

Maintenance ID:		Date Transferred to Maintenance:	

* All ongoing maintenance and verification is assumed to be associated with the VDOT Maintenance database and inspection/O&M program. Note the date xferred.

Documents/Tracking Information

Location of Plans on File:	
Location of Photos on File:	
Location of Permits on File:	

Appendix 11C-1 SWM Facility Tabulation Sheet

ORIGINAL SUBMISSION ____________ REVISED SUBMISSION ____________ (Check one)
(1) DISTRICT NO: ________ (2)RTE NO: ____________ (3)COUNTY/CITY ____________
(4)PROJECT NUMBER: ________________________ (5)AD DATE: ____________
(6)LOC./STA.: ________________________ (7)TYPE BASIN ____________
(8)STORAGE VOL.: ________________________ (in CU. FT. or AC. FT.)
(9)WATERSHED NAME: ________________________
(10)REM/MONITOR:

ORIGINAL SUBMISSION ____________ REVISED SUBMISSION ____________ (Check one)
(1) DISTRICT NO: ________ (2)RTE NO: ____________ (3)COUNTY/CITY ____________
(4)PROJECT NUMBER: ________________________ (5)AD DATE: ____________
(6)LOC./STA.: ________________________ (7)TYPE BASIN ____________
(8)STORAGE VOL.: ________________________ (in CU. FT. or AC. FT.)
(9)WATERSHED NAME: ________________________
(10)REM/MONITOR:

ORIGINAL SUBMISSION ____________ REVISED SUBMISSION ____________ (Check one)
(1) DISTRICT NO: ________ (2)RTE NO: ____________ (3)COUNTY/CITY ____________
(4)PROJECT NUMBER: ________________________ (5)AD DATE: ____________
(6)LOC./STA.: ________________________ (7)TYPE BASIN ____________
(8)STORAGE VOL.: ________________________ (in CU. FT. or AC. FT.)
(9)WATERSHED NAME: ________________________
(10)REM/MONITOR:

ORIGINAL SUBMISSION ____________ REVISED SUBMISSION ____________ (Check one)
(1) DISTRICT NO: ________ (2)RTE NO: ____________ (3)COUNTY/CITY ____________
(4)PROJECT NUMBER: ________________________ (5)AD DATE: ____________
(6)LOC./STA.: ________________________ (7)TYPE BASIN ____________
(8)STORAGE VOL.: ________________________ (in CU. FT. or AC. FT.)
(9)WATERSHED NAME: ________________________
(10)REM/MONITOR:

ORIGINAL SUBMISSION ____________ REVISED SUBMISSION ____________ (Check one)
(1) DISTRICT NO: ________ (2)RTE NO: ____________ (3)COUNTY/CITY ____________
(4)PROJECT NUMBER: ________________________ (5)AD DATE: ____________
(6)LOC./STA.: ________________________ (7)TYPE BASIN ____________
(8)STORAGE VOL.: ________________________ (in CU. FT. or AC. FT.)
(9)WATERSHED NAME: ________________________
(10)REM/MONITOR: