Virginia Department of Transportation

Part II B BMP Design Manual of Practice

1401 East Broad Street, Richmond, Virginia 23219

PART II B BMP DESIGN MANUAL OF PRACTICE

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Chapter 1 Introduction

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 manual 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:

$$T_v = \left[\frac{(C_v)(1.0\text{ in.})\,(Rv_{composite})\,(A)}{12}\right] \quad \textbf{(1.1)}$$

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, $Rv_{composite}$ 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 $Rv_{composite}$ as:

$$Rv_{composite} = (Rv_I \times \%I) + (Rv_T \times \%T) + (Rv_F \times \%F) \quad \textbf{(1.2)}$$

where:

$Rv_{composite}$	= Composite weighted runoff coefficient
Rv_I	= Runoff coefficient for Impervious cover (**Table 1.1**)
Rv_T	= Runoff coefficient for Turf cover (**Table 1.1**)
Rv_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 $Rv_{composite}$ for the post-development condition.

Table 1.1 Land Cover Volumetric Runoff Coefficients (Rv)
Virginia Stormwater Management Handbook, Chapter 11, 2014

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 II B (9VAC25-870-62) of the Virginia Stormwater Management Regulations.

> “Part II B (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 II B technical criteria are applicable to non-grandfathered projects.”

This design manual applies to projects that must conform to Part II B technical criteria.

For projects that have been grandfathered, the requirements of Part II C (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 II C (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 II C technical criteria are applicable to grandfathered projects.

A separate design manual is provided for projects that must conform to Part II C technical criteria.

Chapter 2 Sheet Flow to Vegetated Filter Strip

2.1 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 Function
s 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 Contributing Drainage Area (CDA) and Contributing Flow Path

Vegetated filter strips should be restricted to treatment scenarios where the contributing drainage area (CDA) 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 Cover	Amended soils and dense turf cover or landscaped with herbaceous 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.*

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 and Sediment 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.

Chapter 3 Grass Channels

3.1 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, the performance of grass channels 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.

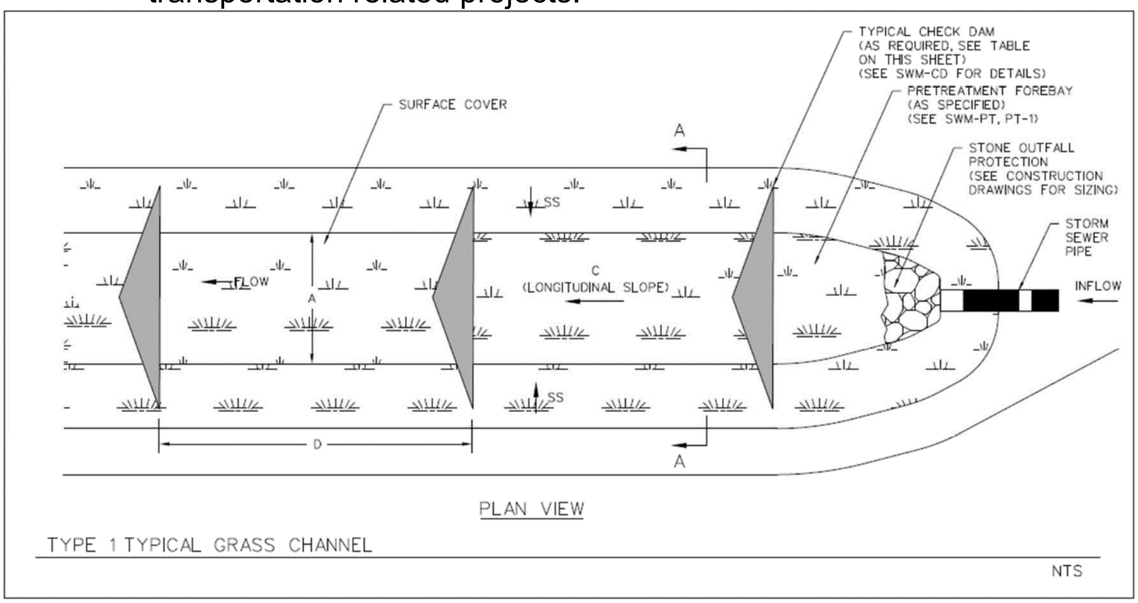

Figure 3.1 Schematic Grass Channels - Typical Plan
VDOT SWM-3 Grass Channels

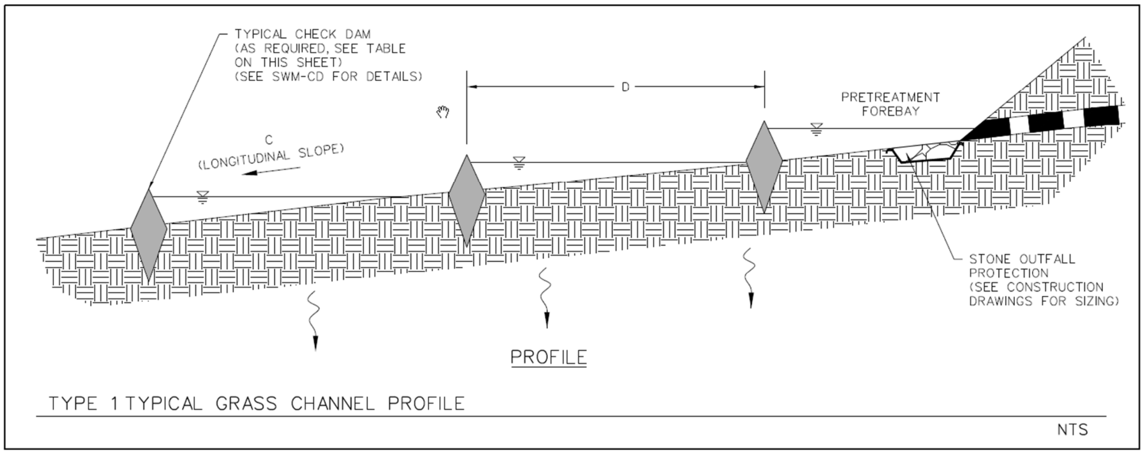

Figure 3.2 Schematic Grass Channels -Typical Profile
VDOT SWM-3 Grass Channels

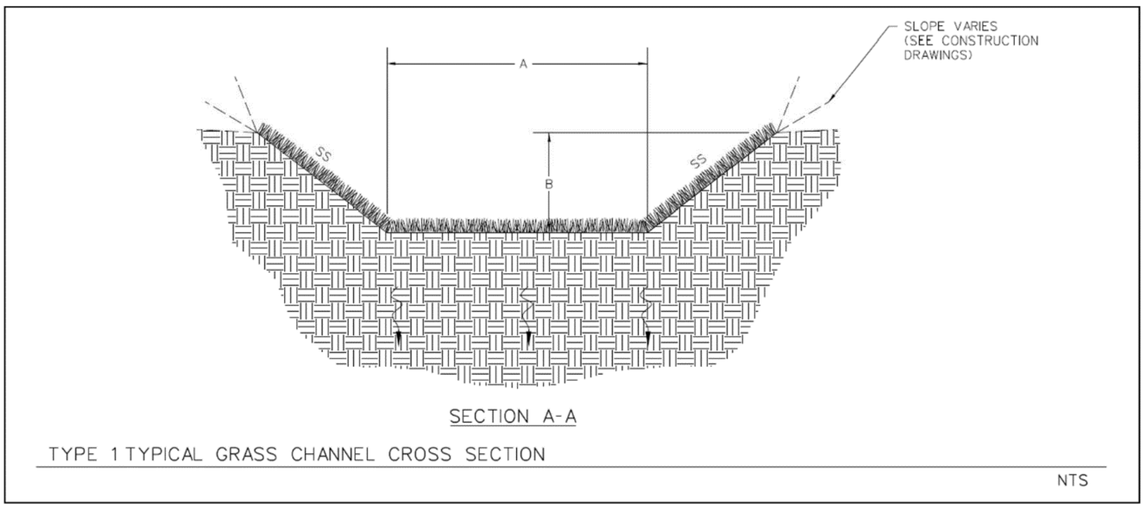

Figure 3.3 Schematic Grass Channels - Typical Section
VDOT SWM-3 Grass Channels

Table 3.1 Stormwater Functions Provided in Grass Channels[1]
Virginia Stormwater Design Specification No. 3, Grass Channels, 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)	20%	NA [3]	10%	20%
Total Phosphorus (TP) EMC Reduction[4] by BMP Treatment Process	15%		15%	
Total Phosphorus (TP) Mass Load Removal	32%		24% (no CA) to 32% (with CA)	
Channel & Flood Protection	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.			

[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:

$$V = \left[\left(\frac{1.49}{n}\right) R^{2/3} S^{1/2}\right] \quad (3.1)$$

where:

V = flow velocity (ft./sec)

n = Manning's roughness coefficient (see discussion below)
R =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))*

$$q_{pT_v} = VA = V(W \times D) \quad (3.2)$$

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**.

$$W = (n)(q_{pT_v})/(1.49D^{5/3}S^{1/2}) \quad (3.3)$$

$$V = q_{pT_v}/(W \times D) \quad (3.4)$$

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 downstreamtoe.
[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:

- ***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.

- ***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).

- ***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).

- ***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.

$$\mathrm{CN} = \frac{1000}{[10 + 5P + 10Q_a - 10(Q_a^2 + 1.25Q_aP)^{0.5}]} \qquad \textbf{(3.6)}$$

where:

CN = Adjusted curve number
P = Rainfall (inches), (1.0" in Virginia)
Q_a = Runoff volume (watershed inches), equal to Tv ÷ drainage area

$$Q_a = \frac{2{,}024\,\text{ft}^3}{1.1\ \text{ac}\ \left(\frac{43{,}560\ \text{ft}}{1\ \text{ac}}\right)}\left(\frac{12\text{in}}{1\ \text{ft}}\right) = 0.51\ \text{in}$$

$$\text{CN} = \frac{1000}{\left[10 + 5(1\ \text{in}) + 10(0.51\ \text{in}) - 10\left((0.51\ \text{in})^2 + 1.25(0.51\ \text{in})(1\ \text{in})\right)^{0.5}\right]}$$

$$\text{CN} = 94$$

$$q_{pTv} = (q_u)(A)(Q_a) \qquad \textbf{(3.7)}$$

where,

q_{pTv} = Treatment Volume peak discharge (cfs)
q_u = unit peak discharge (cfs/mi²/in)
A = drainage area (mi²)
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:

$$I_a = \frac{200}{\text{CN}} - 2 \qquad \textbf{(3.8)}$$

$$I_a = \frac{200}{94} - 2 = 0.13\ \text{inches}$$

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 *SCS TR-55 Handbook (1986)*. Reading the chart yields a value of 925 cfs/mi²/in.

$$q_{pTv} = \left(\frac{925\,\frac{\text{cfs}}{\text{mi}^2}}{\text{i}}\right)\left(\frac{1.1\ \text{ac}}{640\ \text{ac}/\text{mi}^2}\right)\left[\frac{2{,}024\,\text{ft}^3}{1.1\ \text{ac} \times \left(\frac{43{,}560\ \text{ft}}{1\ \text{ac}}\right)}\right]\left(\frac{12\text{in}}{1\ \text{ft}}\right)$$

$$q_{pTv} = 0.81\ \text{cfs}$$

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**.

$$W = (0.20)(0.81\ \text{cfs})/(1.49(0.33\ \text{ft})^{5/3}\ \ (0.01\frac{\text{ft}}{\text{ft}})^{1/2}$$
$$W = 6.9\ \text{ft}$$

Velocity can now be computed using **Equation 3.4** as:

$$V = \frac{0.81\ \text{cfs}}{(6.9\ \text{ft} \times 0.33\ \text{ft})} = 0.36\frac{\text{ft}}{\text{s}}$$

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:

$$L = 540V = (540\ \text{sec})\ (0.36\frac{\text{ft}}{\text{s}}) = 194\ \text{ft}$$

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.

Chapter 4 Soil Compost Amendments

4.1 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 are 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 4.2 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:

$$\frac{\mathrm{IC}}{\mathrm{SA}} = \frac{0.32\ \mathrm{acres}}{0.80\ \mathrm{acres}} = 0.40$$

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:

$$\frac{12\ \mathrm{in} - 10\ \mathrm{in}}{0.50 - 0.0} \times 0.4 + 10\ \mathrm{in} = 11.\ 6\ \mathrm{incMes}$$

Once the depth of the amendment has been computed, the estimated volume of compost in cubic yards is computed using **Equation 4.1** as:

$$\mathrm{C} = (1{,}000\ \mathrm{ft} \times 35\ \mathrm{ft} \times 11.6\ \mathrm{inc}h\mathrm{es} \times 0.0031\ \frac{\mathrm{CY}}{\mathrm{ft}^2 \cdot \mathrm{in}} = 1,\ 259\ \mathrm{CY}$$

Therefore, 1,259 cubic yards of compost will be required to be tilled into the amendment area to an average depth of 11.6”.

Chapter 5 Permeable Pavement

5.1 Overview of Practice

Permeable pavements 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.

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 pavements typically are used only for parking applications. Prior to use of permeable pavement in a road application, VDOT shall be consulted to confirm acceptance of use.

Table 5.1 Summary of Stormwater Functions Provided by Permeable Pavement
Virginia Stormwater Design Specification No. 7, Permeable Pavement (DCR/DEQ, 2013)

Stormwater Function	Level 1 Design	Level 2 Design
Annual Runoff Volume Reduction (RR)	45%	75%
Total Phosphorus (TP) EMC Reduction[1] by BMP Treatment Process	25%	25%
Total Phosphorus (TP) Mass Load Removal	59%	81%
Total Nitrogen (TN) EMC Reduction[1]	25%	25%
Total Nitrogen (TN) Mass Load Removal	59%	81%
Channel Protection	• 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.	
Flood Mitigation	Partial. May be able to design additional storage into the reservoir layer by adding perforated storage pipe or chambers.	
[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)

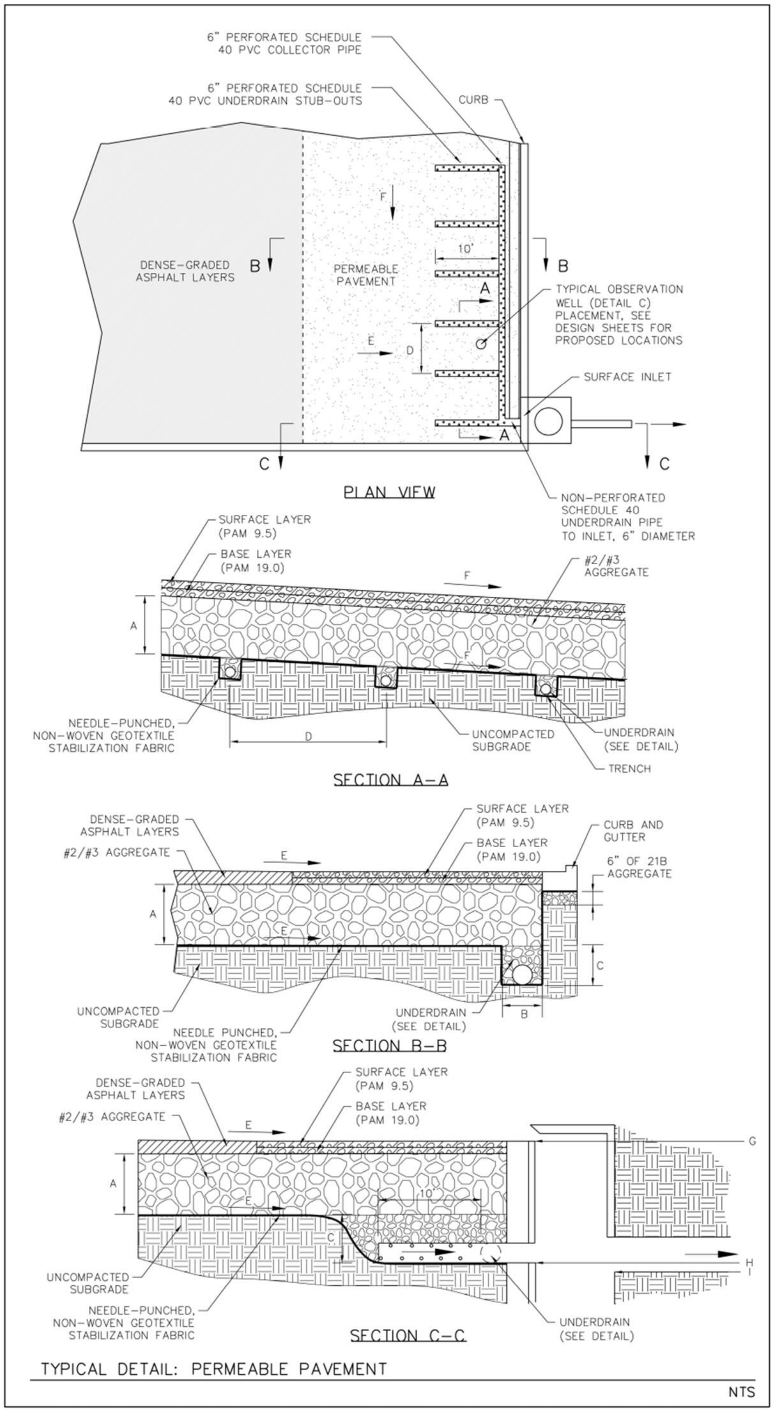

Figure 5.1 Typical Permeable Pavement Detail (Parking Lots)
(VDOT SWM-5, Filtering Practices, 2014)

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).

$$\mathrm{d_{ctone}} = \frac{(\mathrm{P} \times \mathrm{A} \times \mathrm{Rv_{compocite}}) + (\mathrm{P} \times \mathrm{A_P})}{5_\mathrm{r} \times \mathrm{A_P}} \qquad \textbf{(5.1)}$$

where:

$\mathrm{d_{ctone}}$ = Depth of stone reservoir layer (feet)
P = Rainfall depth (feet); Level 1 = 0.08'
A = Contributing drainage area (ft^2)
$\mathrm{Rv_{compocite}}$ = Composite runoff coefficient for contributing area
$\mathrm{A_P}$ = Area of permeable pavement (ft^2)
5_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, $\mathrm{A_P}$.

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, $\mathrm{Rv}_{\mathrm{compocite}}$ 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:

$$d_{ctone} = \frac{(0.08 \text{ feet} \times 8{,}712 \text{ ft}^2 \times 0.95) + (0.08 \text{ feet} \times 2{,}592 \text{ ft}^2)}{0.40 \times 2{,}592 \text{ ft}^2} = 0.84 \text{ feet}$$

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:

$$Q_i = C_w(L + 1.8W)d^{1.5}$$

where:

Q_i = 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:

$$Q_i = 2.3(2.5 + 1.8(2))0.34^{1.5}$$

$$Q_i = 2.3(2.5 + 1.8(2))0.34^{1.5} = 2.78 \text{ cfs}$$

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.

Chapter 6 Infiltration

6.1 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.

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 ft2), small-scale infiltration (2,500 to 20,000 ft2), and conventional infiltration (20,000 to 100,000 ft2). Specific criteria generally associated with each category are found in Table 6.2. A typical configuration and various cross-sections typically associated with infiltration facilities are found in Figures 6.1 to 6.4.

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)

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)

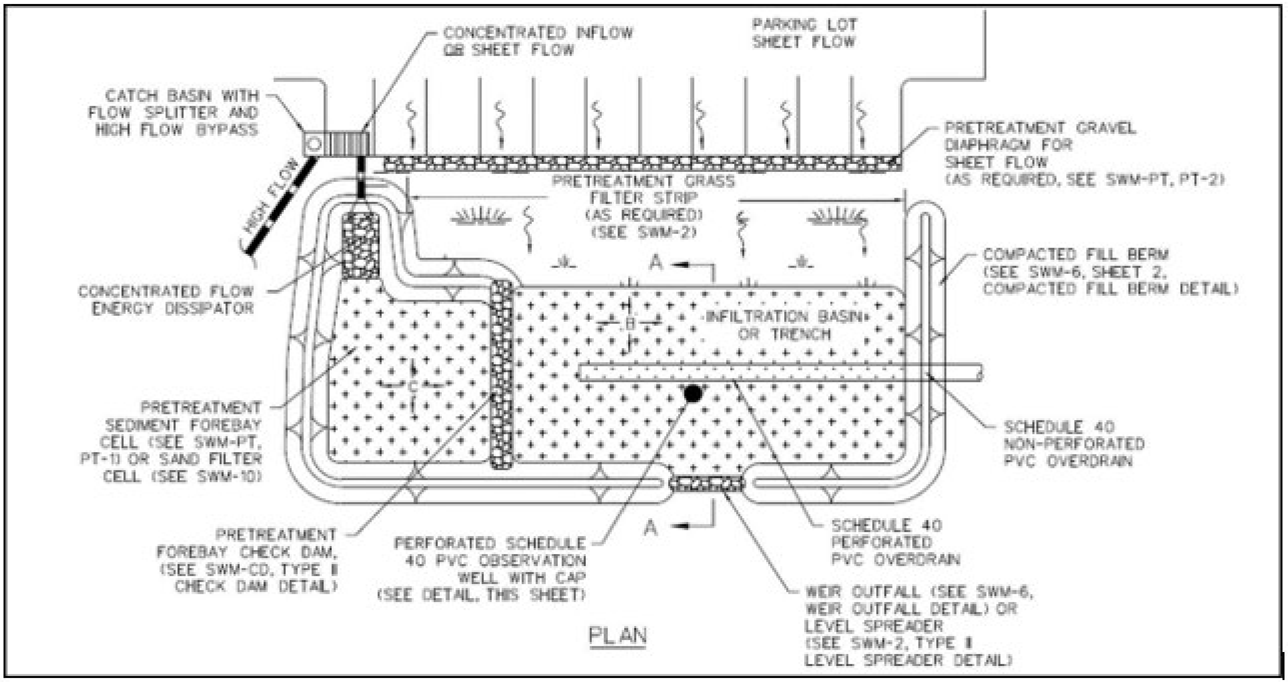

Figure 6.1 Typical Infiltration Basin
VDOT SWM-6 Infiltration Practices

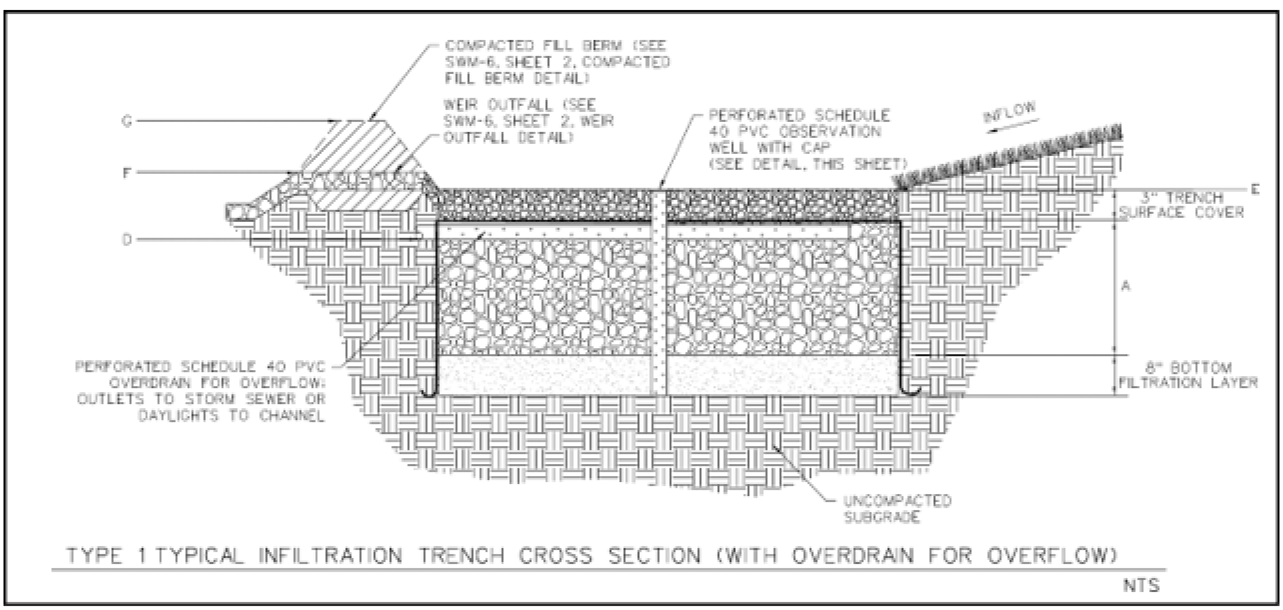

Figure 6.2 Typical Infiltration Trench Cross Section (with overdrain)
VDOT SWM-6 Infiltration Practices

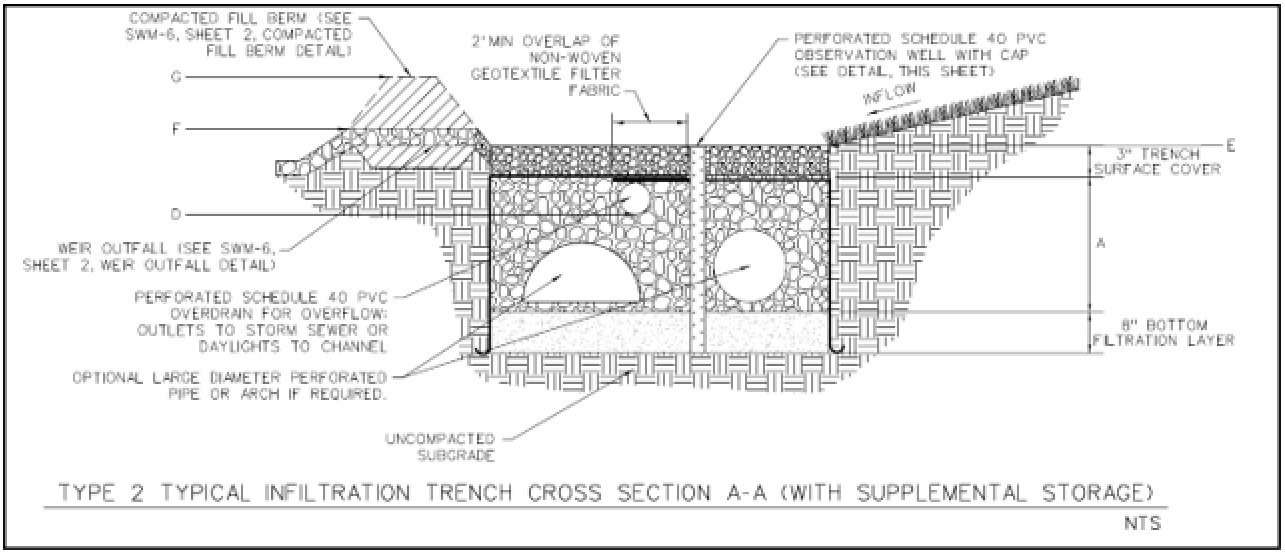

Figure 6.3 Infiltration Trench Cross Section (with supplemental storage)
VDOT SWM-6 Infiltration Practices

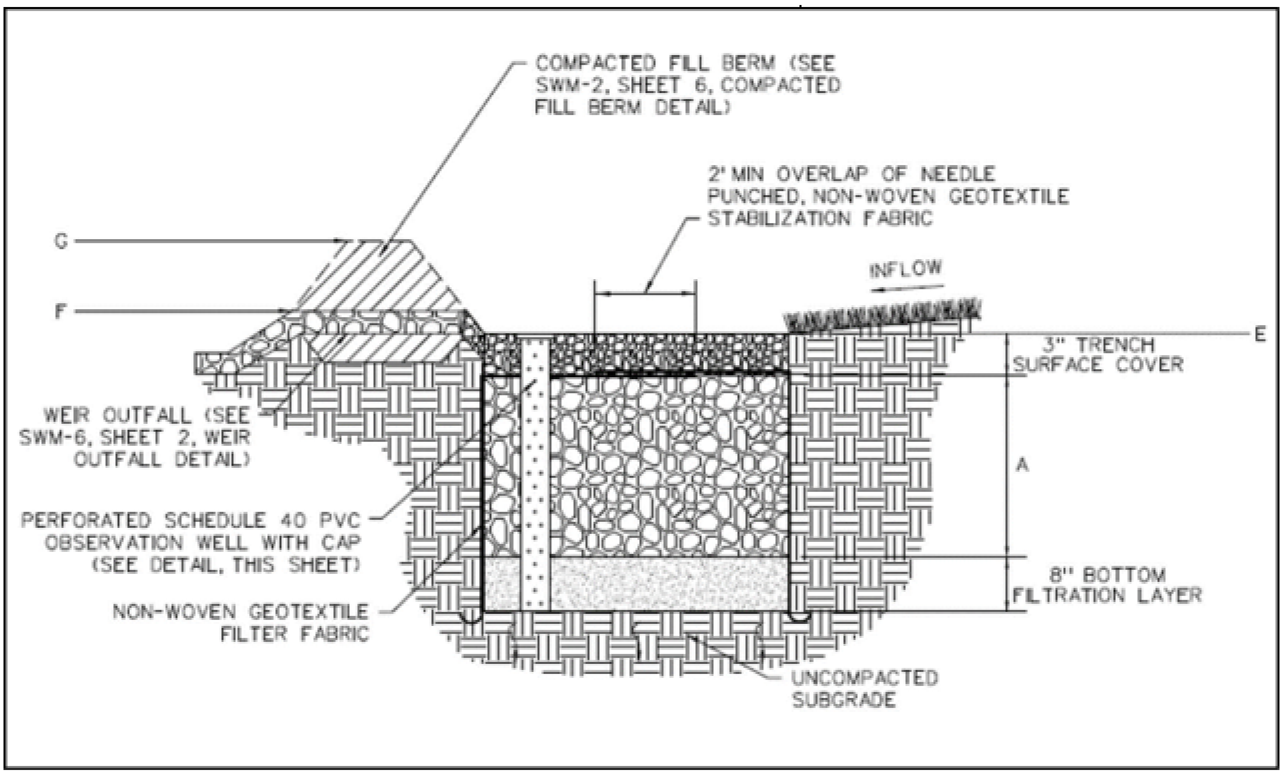

Figure 6.4 Infiltration Trench Cross Section (without subdrainage)
VDOT SWM-6 Infiltration Practices

6.2 Site Constraints and Siting of theFacility

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)
Sizing: T_v = [(Rv)(A)/12] – the volume reduced by an upstream BMP	Sizing: 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:

$$d_{mas} = \frac{\left(\frac{f}{2}\right)(t_d)}{12} \qquad (6.1)$$

where:

d_{mas} = 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**:

$$d_{mas} = \frac{\left(\frac{f}{2}\right)(t_d)}{\eta \times 12} \qquad (6.2)$$

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:

$$SA = \frac{(T_{vBMP})}{d + \left[\frac{\left(\left(\frac{1}{2}\right) f \times t_f\right)}{12}\right]} \qquad (\mathbf{6.3})$$

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)

$$SA = \frac{(T_{vBMP})}{(\eta \times d) + \left[\frac{\left(\left(\frac{1}{2}\right) f \times t_f\right)}{12}\right]} \qquad (\mathbf{6.4})$$

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 ft²	1	2*
2,500 ft² to 5,000 ft²	2	3
5,000 ft² to 7,500 ft²	2	4
7,500 ft² to 10,000 ft²	2	5
Greater than 10,000 ft²	Add 1 soil profile and 2 infiltration tests for each additional 5,000 ft² 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 ft²), 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:

$$\mathrm{d_{mas}} = \frac{(0.6\ \mathrm{in}/h\mathrm{r})(48\ \mathrm{hrs})}{0.4 \times 12} = 6\ \mathrm{feet}$$

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:

$$SA = \frac{(4{,}162\,\mathrm{ft}^3)}{(0.4 \times 4.5\,\mathrm{ft})} + \left[\frac{\left(\left(\frac{1}{2}\right)1.2\,\frac{\mathrm{in}}{hr} \times 2hr\right)}{12}\right] = 2{,}312\,\mathrm{ft}^2$$

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.

Chapter 7 Bioretention

7.1 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)

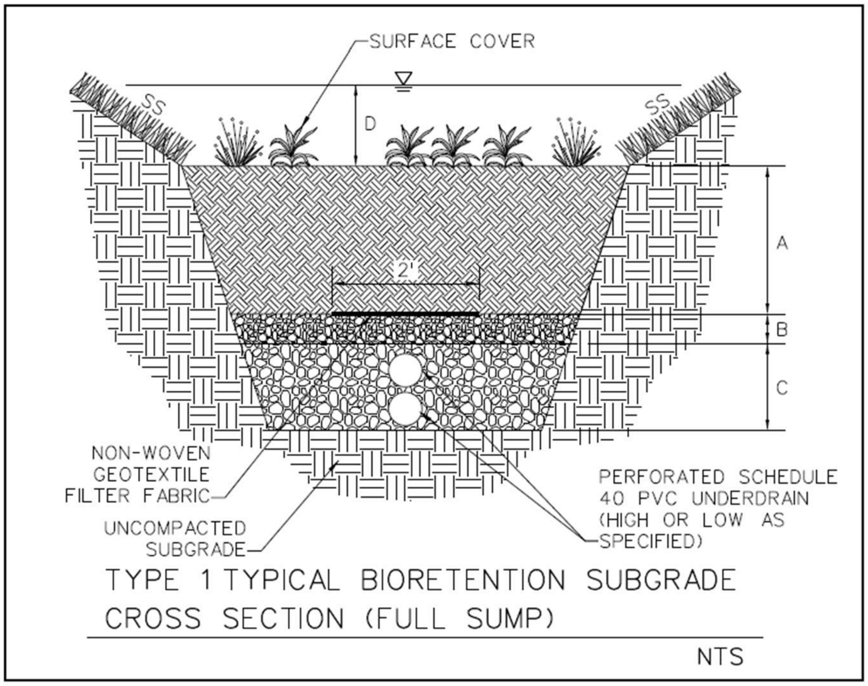

Figure 7.1 Bioretention Cross-section – Type 1
VDOT SWM-7 Bioretention

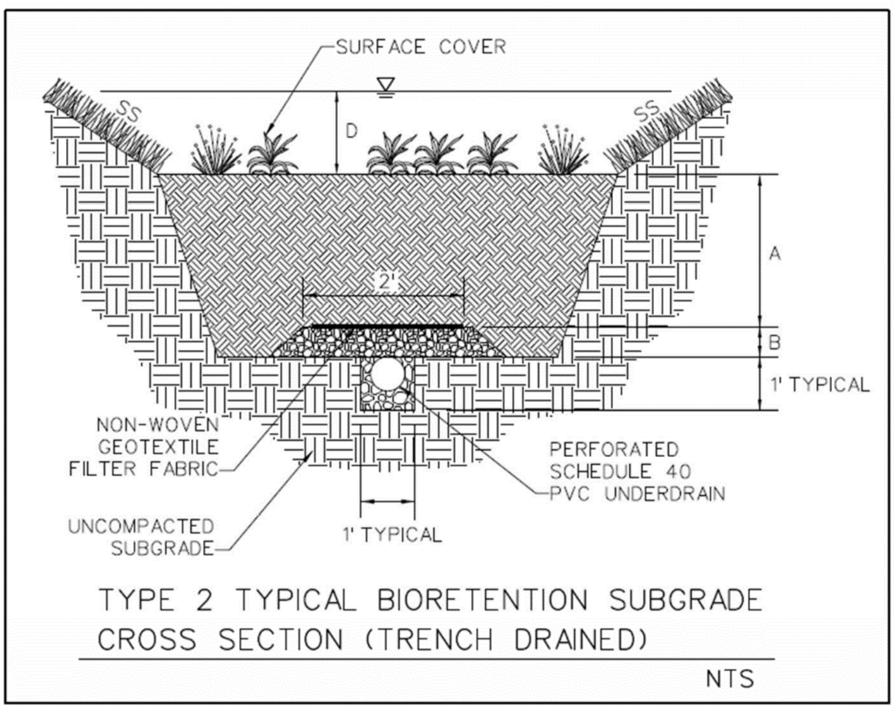

Figure 7.2 Bioretention Cross-section – Type 2
VDOT SWM-7 Bioretention

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 seasonal 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 (5) 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:

$$\mathrm{SD} = (0.25)\mathrm{M_d} + (0.40)\mathrm{G_d} + (1.0)\mathrm{SS_d} \qquad \textbf{(7.1)}$$

where:

SD = storage depth (ft);
$\mathrm{M_d}$ = proposed media depth (ft);
$\mathrm{G_d}$, = proposed gravel depth (ft) and
$\mathrm{SS_d}$ = 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:

$$\mathrm{SA} = \frac{[\mathrm{C_v} \times \mathrm{T_v} - \mathrm{V_u}]}{\mathrm{SD}} \qquad \textbf{(7.2)}$$

where:

SA = computed surface area (ft^2)

$\mathrm{C_v}$ = volume coefficient (1.0 for level 1 design and 1.25 for level 2 design);
$\mathrm{T_v}$, = computed treatment volume (acre-ft);
$\mathrm{V_u}$ = 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.

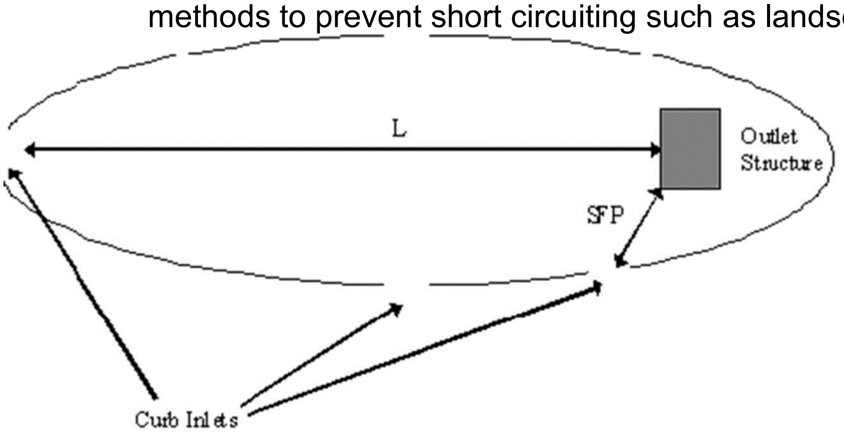

Figure 7.3 Basin Geometry Relating Shortest and Longest Flow Paths.
Virginia DEQ Stormwater Design Specification No. 9, 2013

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.

- ***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).

- ***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.

- ***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).

- ***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**.

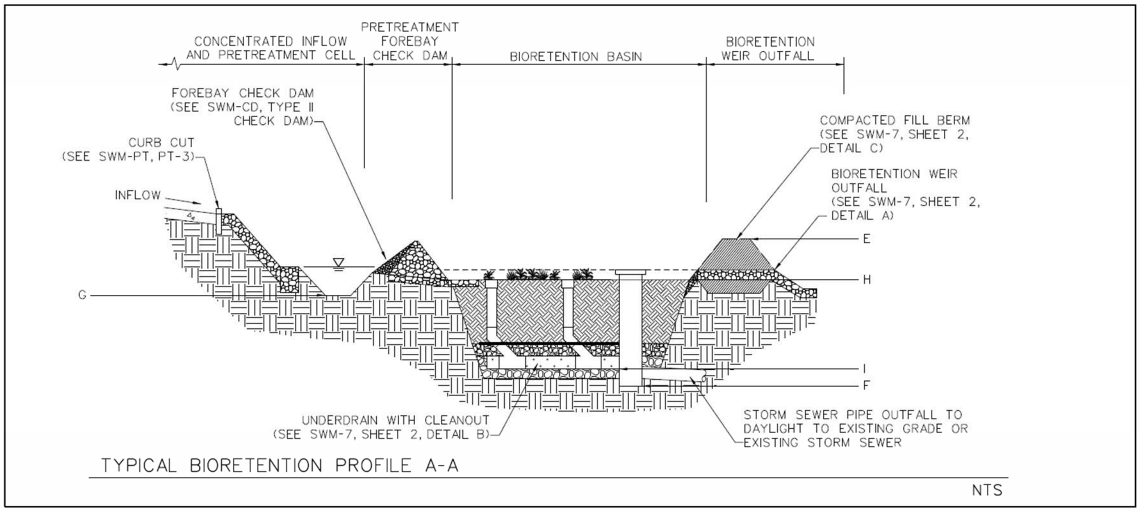

Figure 7.4 Typical Bypass Structure Configuration
VDOT BMP Standard SWM-7 Bioretention

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. 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:

$$T_v = 3{,}746\text{ ft}^3$$

The basin minimum surface area is determined through use of **Equation 7.2**.

$$SA = \frac{[1.0 \times 3{,}746\text{ft}^3 - 0]}{1.40\text{ ft}} = 2{,}676\text{ sq. ft}$$

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:

$$(0.15)(2{,}676\ \text{ft}^3)=401\ \text{ft}^3$$

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**.

Chapter 8 Dry Swales

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** to **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 8.1 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

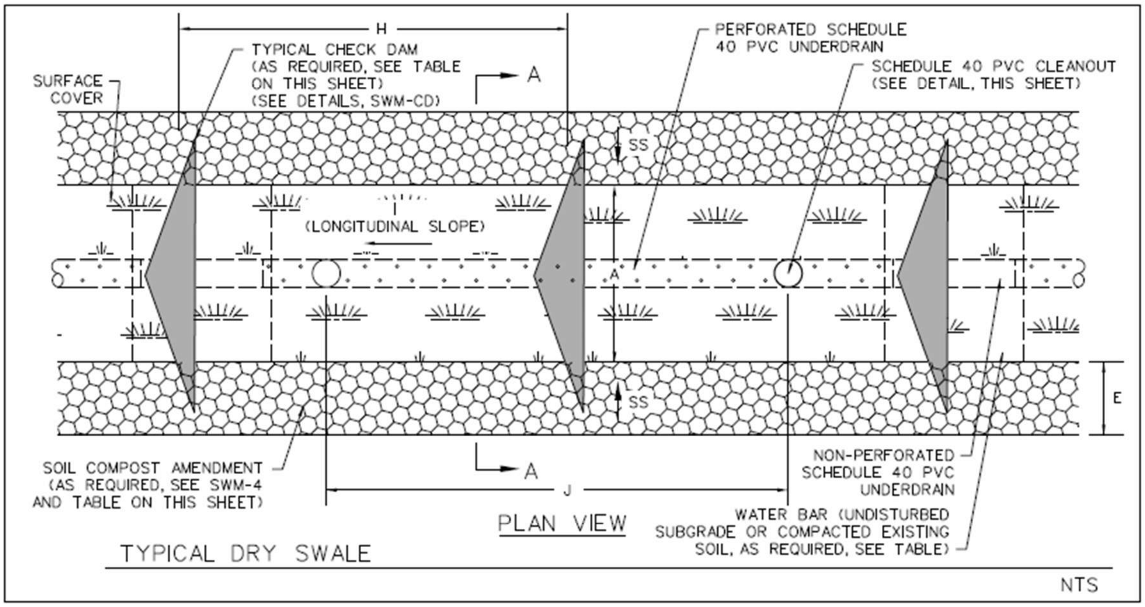

Figure 8.1 Typical Dry Swale [Plan View]
VDOT BMP Standard Detail SWM-8

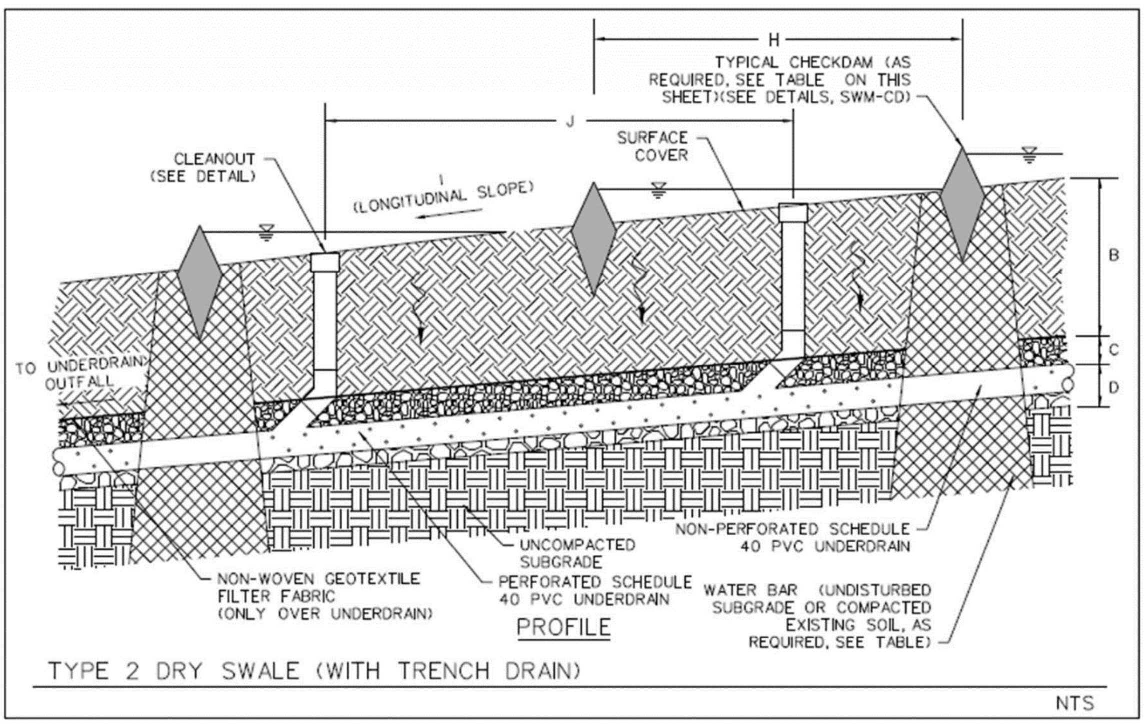

Figure 8.2 Typical Dry Swale – Level 1 [Profile View]
VDOT BMP Standard Detail SWM-8

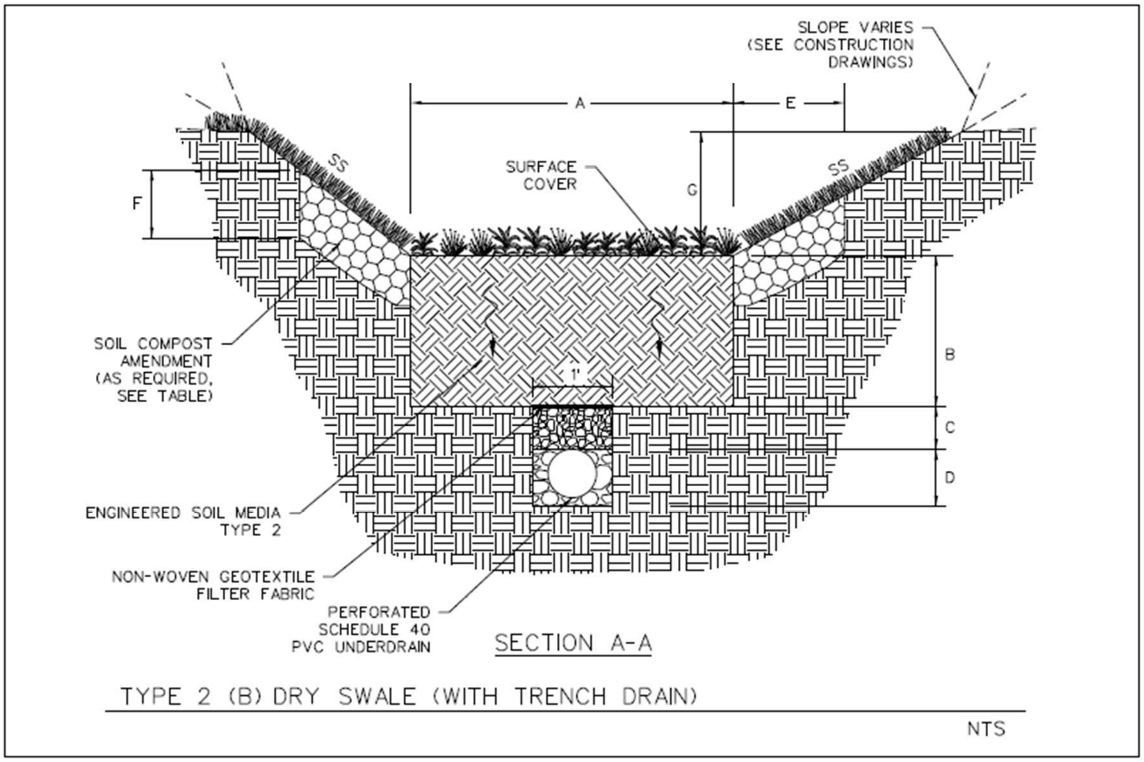

Figure 8.3 Typical Dry Swale – Level 1 [Cross-section View]
VDOT BMP Standard Detail SWM-8

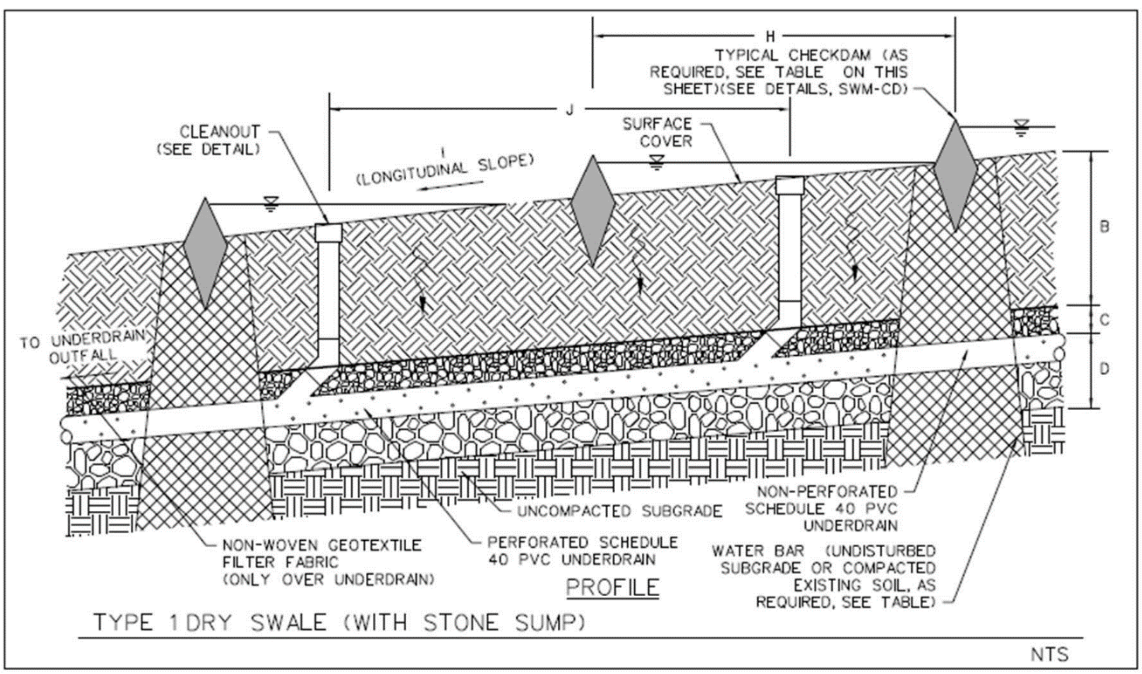

Figure 8.4 Typical Dry Swale – Level 2 [Profile View]
VDOT BMP Standard Detail SWM-8

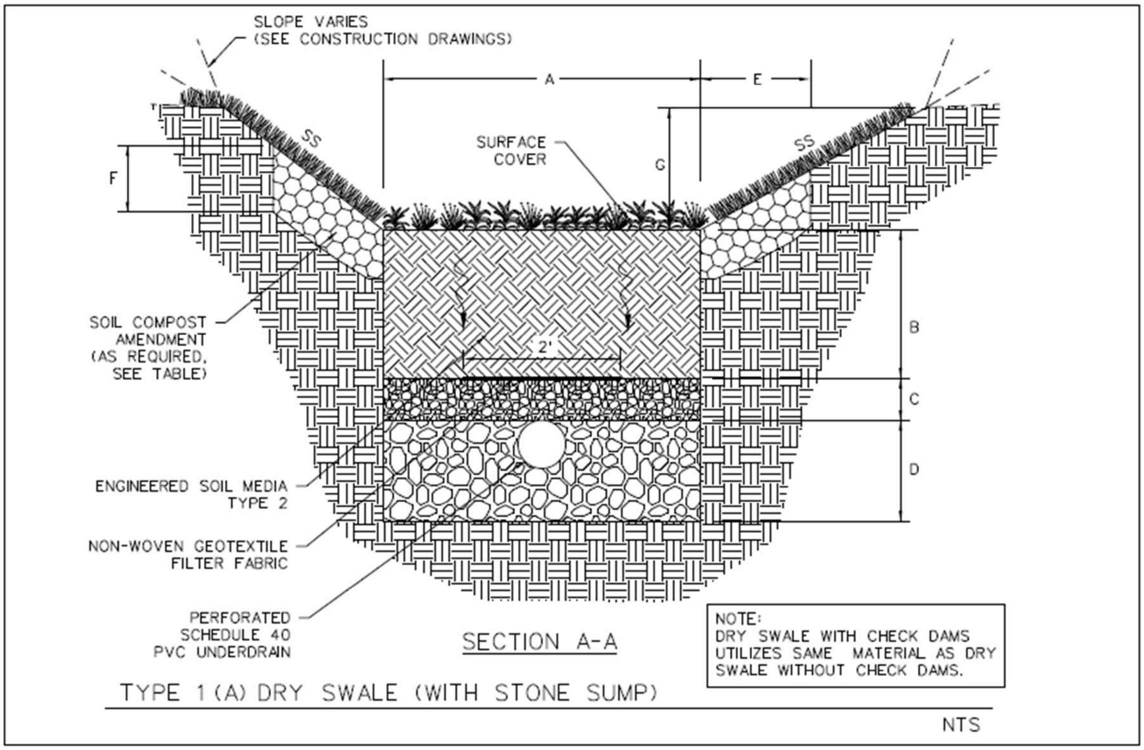

Figure 8.5 Typical Dry Swale – Level 2 [Cross-section View]
VDOT BMP Standard Detail SWM-8

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 – 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 8.2 Dry Swale Design Criteria

Virginia Stormwater Design Specification No. 10, Dry Swale, Draft (DCR/DEQ, 2013)

Level 1 Design (RR:40; TP:20; TN:25)	Level 2 Design (RR:60; TP:40; TN: 35)
Sizing (**Sec. 8.3.1**): Surface Area (ft^2) = (T_v– the volume reduced by an upstream BMP) / Storage depth [1]	Sizing (**Sec. 8.3.1**): Surface Area (ft^2) = {(1.1)(T_v) – the volume reduced by an upstream BMP } / Storage Depth [1]
Effective swale slope ≤ 2%[2]	Effective swale slope ≤ 1%[2]
Media Depth: minimum = 18"; Recommended maximum = 36"	Media Depth minimum = 24" Recommended maximum = 36"
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;	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
Underdrain (**Section 8.3.8**): Schedule 40 PVC with clean-outs	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**)
Media (**Section 8.3.10**): supplied by the vendor; tested for an acceptable hydraulic conductivity (or permeability) and phosphorus content [3]	
Inflow: sheet or concentrated flow with appropriate pre-treatment	
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.	
On-line design	Off-line design or multiple treatment cells
Turf cover	Turf cover, with trees and shrubs
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**)	
[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.	

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_{cc}), 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:

$$\mathrm{SA} = \frac{[T_v - V_{cc}]}{\mathrm{SD}} \qquad \textbf{(8.2)}$$

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.

- ***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).

- ***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.

- ***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).

- ***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:

$$L = \frac{Q_{10}}{3.33h^{\frac{3}{2}}} - 0.2h = \frac{4.92\text{ cfs}}{3.33(0.50\text{ft})^{3/2}} - 0.2(0.50\text{ ft}) = 4.1\text{ ft}$$

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:

$$3{,}594\text{ ft}^3 - (0.48\text{ft})(603\text{ft})(5.10\text{ft}) = 2{,}118\text{ ft}^3$$

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:

$$235\text{ ft}^3 = \frac{H^2}{S}\left(\frac{W}{2} + H\right) = \frac{(1.0\text{ ft})^2}{0.015}\left(\frac{W}{2} + 1.0\text{ ft}\right)$$

$$W = 5.1 \text{ ft}$$

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**).

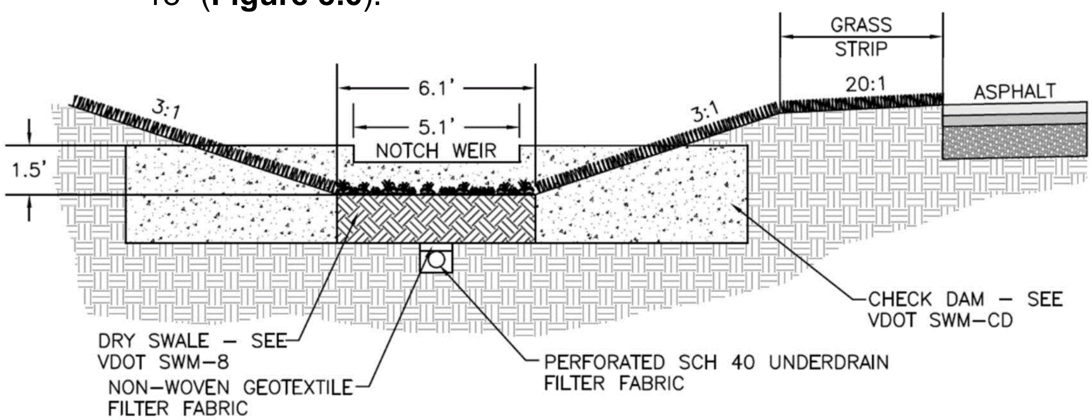

Figure 8.6 Cross-section through Downstream Check Dam

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.

Chapter 9 Wet Swales

9.1 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

Sources: CWP and CSN (2008), CWP, 2007

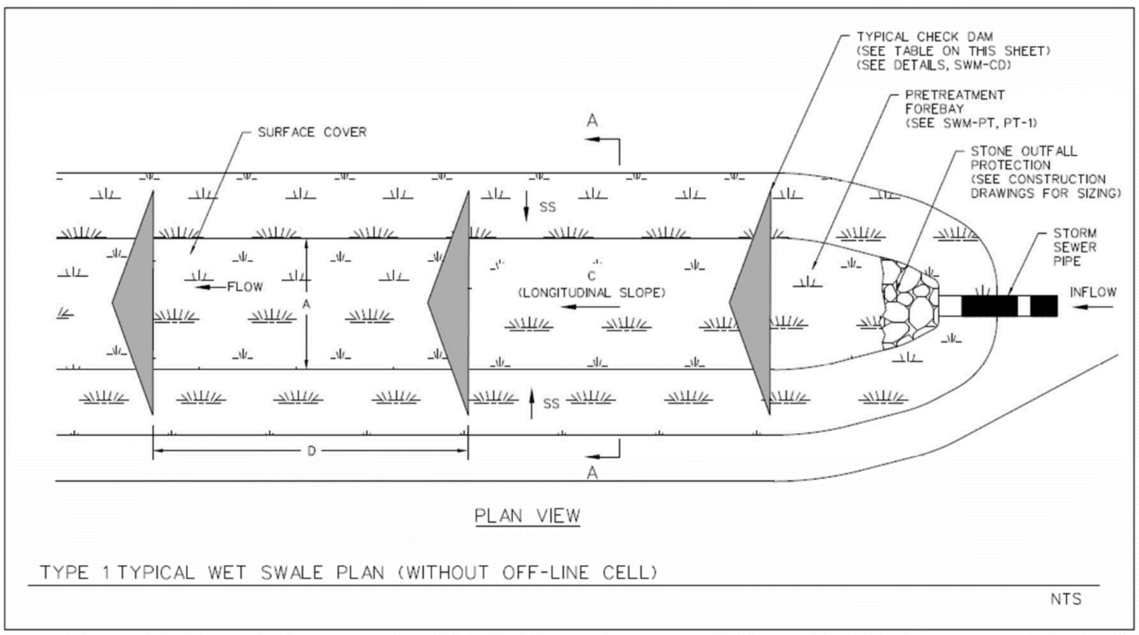

Figure 9.1 Wet Swale with Check Dams
VDOT BMP Standard Detail SWM-9 Wet Swale

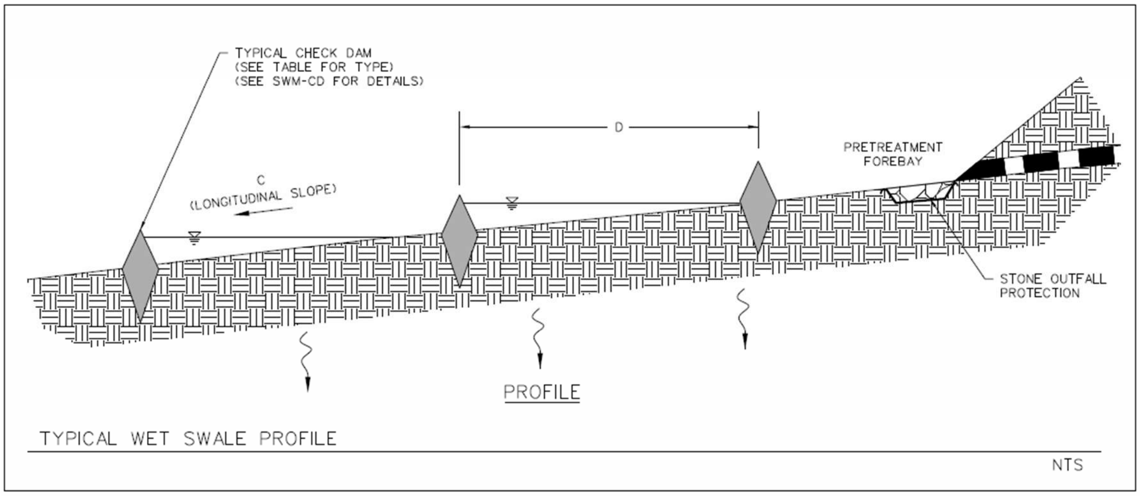

Figure 9.2 Typical Wet Swale Profile
VDOT BMP Standard Detail SWM-9 Wet Swale

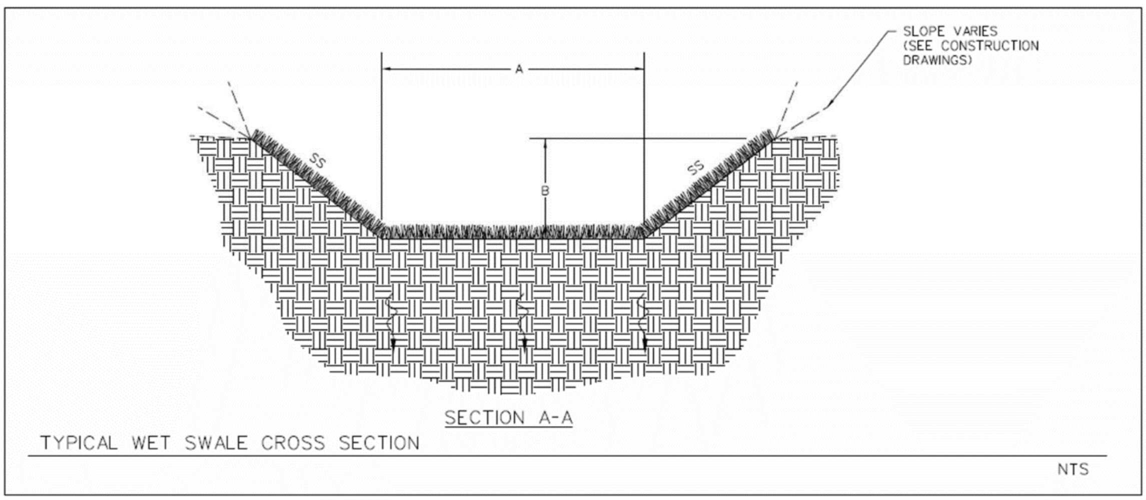

Figure 9.3 Typical Wet Swale Cross-section
VDOT BMP Standard Detail SWM-9 Wet Swale

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.

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.

$$\mathrm{CN} = \frac{1000}{[10+5P+10Q_a-10(Q_a^2+1.25Q_aP)^{0.5}]} \qquad \textbf{(9.1)}$$

where:

CN = Adjusted curve number
P = Rainfall (inches), (1.0" in Virginia)
Q_a = Runoff volume (watershed inches), equal to $\mathrm{Tv} \div \mathrm{drainage\ area}$

$$\mathrm{Q_a} = \frac{2{,}381\mathrm{ft}^3}{1.6\ \mathrm{ac}\ (^{43{,}560\ \mathrm{ft}}/_{1\ \mathrm{ac}})}(\frac{12\mathrm{in}}{1\ \mathrm{ft}}) = 0.41\ \mathrm{in}$$

$$\mathrm{CN} = \frac{1000}{[10+5(1\ \mathrm{in})+10(0.41\ \mathrm{in})-10((0.41\ \mathrm{in})^2+1.25(0.41\ \mathrm{in})(1\ \mathrm{in}))^{0.5}]}$$

$$\mathrm{CN} = 92$$

$$\mathrm{q_{pTv}} = (\mathrm{q_u})(\mathrm{A})(\mathrm{Q_a}) \qquad \textbf{(9.2)}$$

where:

$\mathrm{q_{pTv}}$ = Treatment Volume peak discharge (cfs)
$\mathrm{q_u}$ = unit peak discharge (cfs/mi^2/in)
A = drainage area (mi^2)
$\mathrm{Q_a}$ = runoff volume (watershed inches = $\mathrm{Tv/A}$)

All of the variables are known in the above equation with the exception of $\mathrm{q_u}$. To determine its value, first the initial abstraction must be computed using the equation:

$$\mathrm{I_a} = \frac{200}{\mathrm{CN}} - 2 \qquad \textbf{(9.3)}$$

$$\mathrm{I_a} = \frac{200}{92} - 2 = 0.17\ \mathrm{inches}$$

Compute $\mathrm{I_a/P}$ where P is the 1" rainfall (inches), which equates to 0.17.

Read the unit peak discharge, $\mathrm{q_u}$, from Exhibit 4-II of the *SCS TR-55 Handbook (NRCS, 1986)*. Reading the chart yields a value of 855 cfs/mi^2/in.

$$q_{pTv} = \left(\frac{855\ \frac{cfs}{mi^2}}{i}\right)\left(\frac{1.6\ ac}{640\ ac/mi^2}\right)\left[\frac{2{,}381\ ft^3}{1.6\ ac \times (43{,}560\ ft/1\ ac)}\right]\left(\frac{12in}{1\ ft}\right)$$

$$q_{pTv} = 0.88\, cfs$$

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.

$$W = (n)(q_{pT_v})/(1.49D^{5/3}S^{1/2})$$
$$W = (0.20)(0.88\ cfs)/(1.49(0.33\ ft)^{5/3}(0.008\ \frac{ft}{ft})^{1/2}$$
$$W = 8.4\ ft$$

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:

$$L = 540V = (540\ sec)\ (0.32\frac{ft}{s}) = 173\ ft$$

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.

Chapter 10 Stormwater Filtering Practices

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.

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.

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)

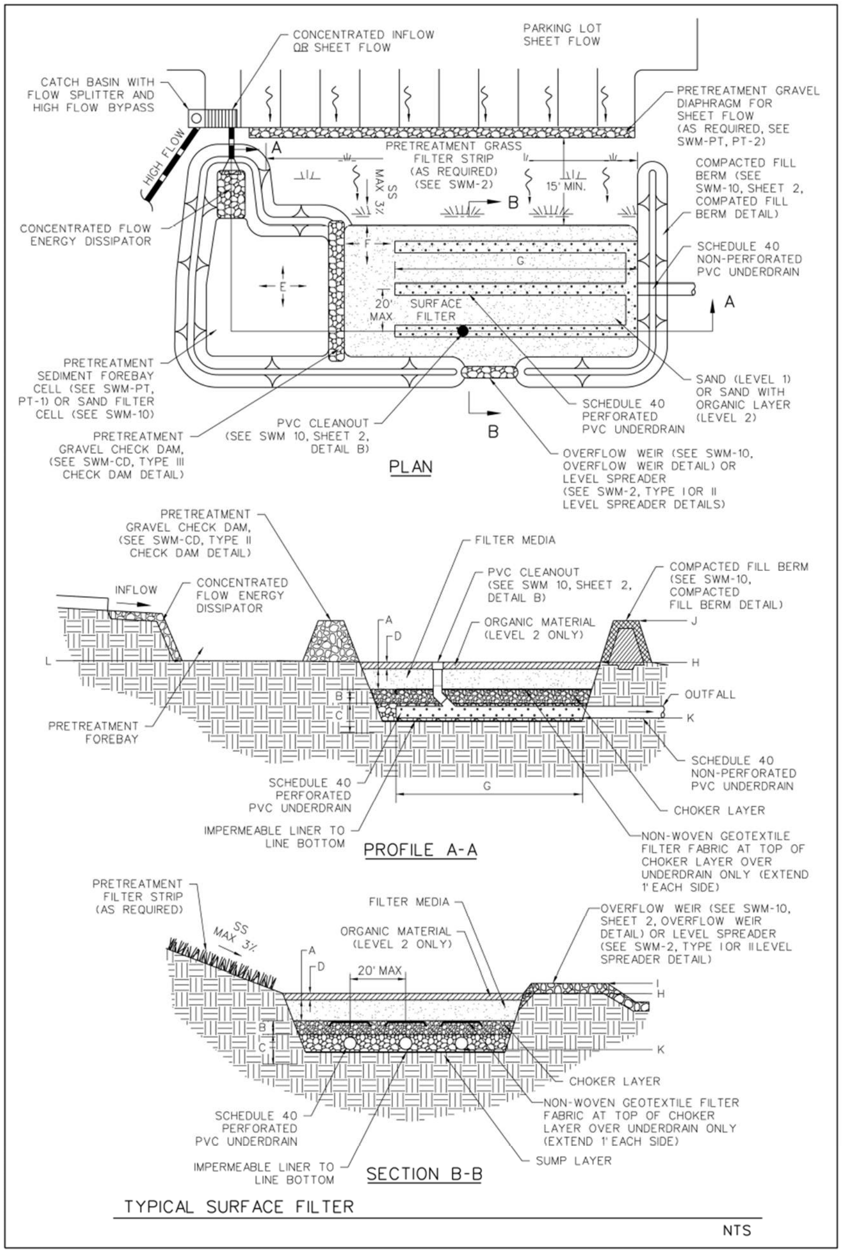

Figure 10.1 Typical Surface Filter
VDOT SWM-10 Filtering Practices

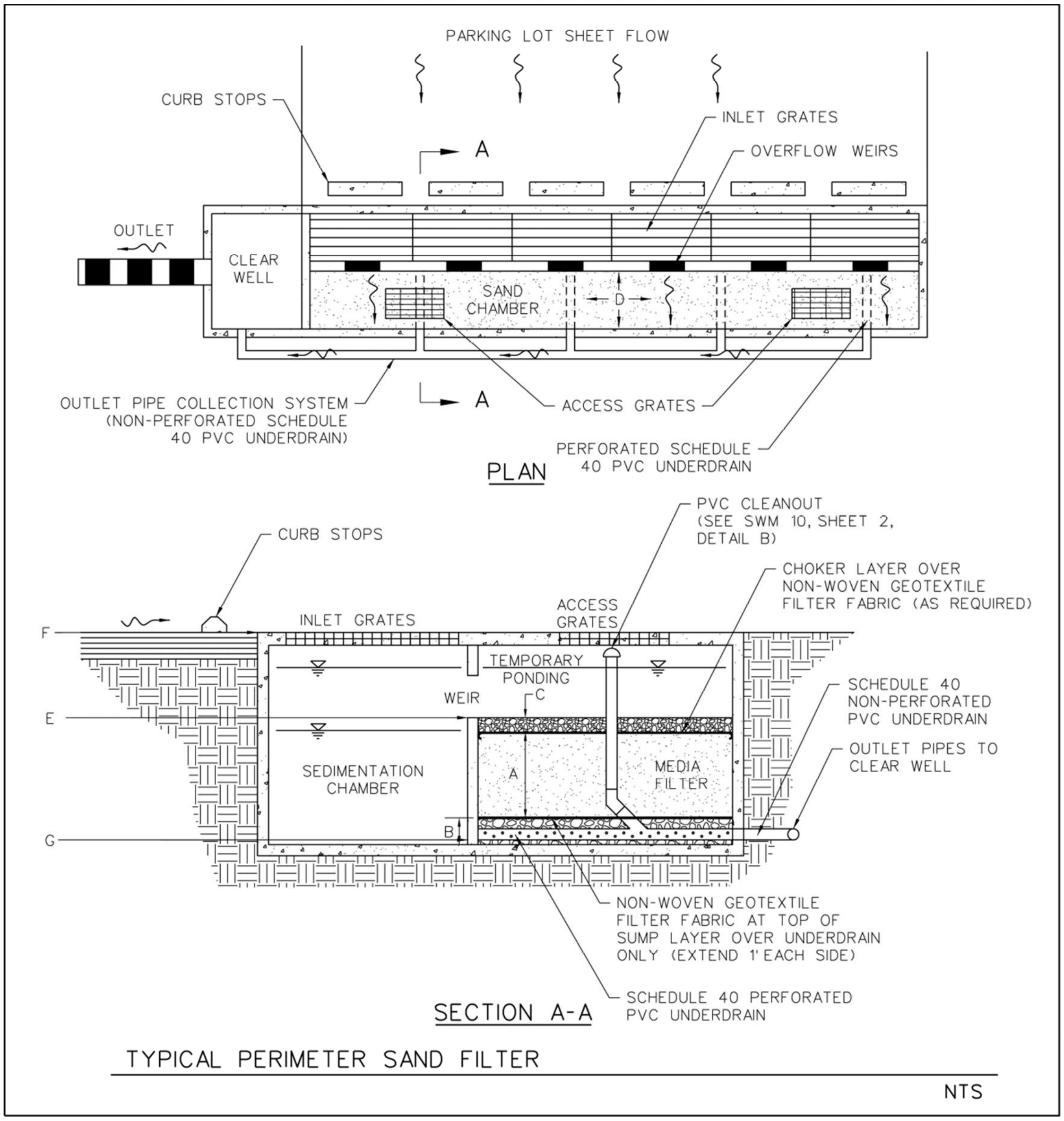

Figure 10.2 Typical Perimeter Sand Filter
VDOT SWM-10 Filtering Practices

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).

$$V_c = 0.75(T_v) \qquad \textbf{(10.2)}$$

where:

V_c = 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.

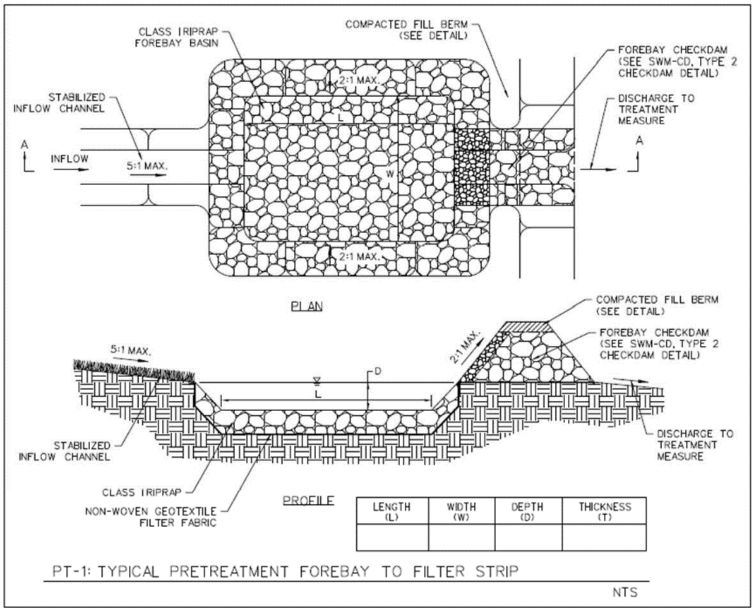

Figure 10.3 Typical Pretreatment Forebay
VDOT SWM-PT Pretreatment

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**].

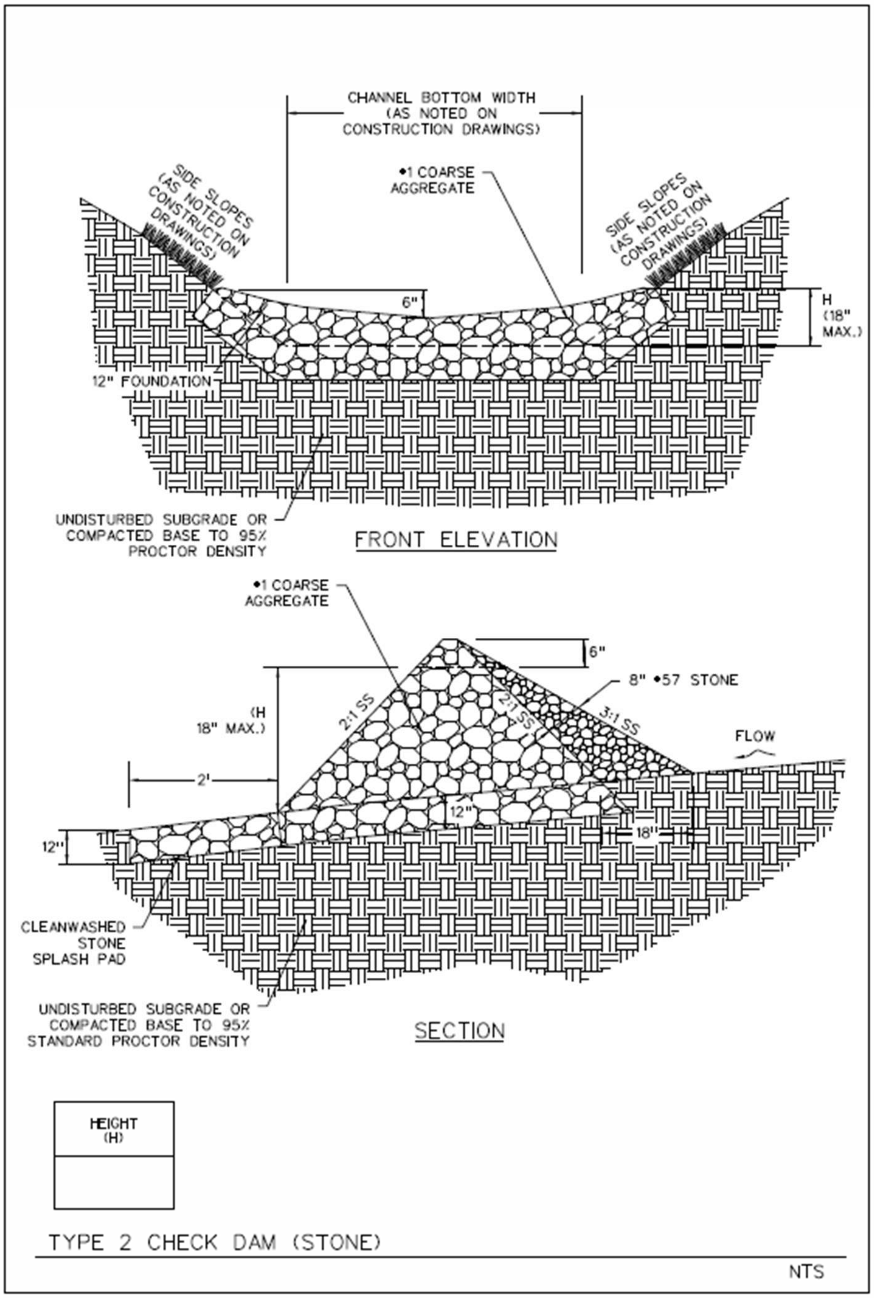

Figure 10.4 Type 2 Check Dam Separating Pretreatment Cell from Filter Cell
VDOT SWM-CD Check Dams

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 Specifications* 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.*

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 $\mathrm{R_{v_{compocite}}}$ for the post-development condition.

$$\mathrm{R_{v_{compocite}}} = (\mathrm{R_{v_I}} \times \%\mathrm{I}) + (\mathrm{R_{v\,T}} \times \%\mathrm{T}) + (\mathrm{R_{v_F}} \times \%\mathrm{F})$$

$$\mathrm{R_{v_{compocite}}} = (0.95 \times \frac{0.95\ \text{acres}}{1.05\ \text{acres}}) + (0.20 \times \frac{0.10\ \text{acres}}{1.05\ \text{acres}}) = 0.88$$

Once the $\mathrm{R_{v_{compocite}}}$ 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.

$$T_v = [\frac{(1.00)(1.0\text{ in.})\ (Rv_{compocite})\ (A)}{12}]$$

$$T_v = [\frac{(1.00)(1.0\text{ in.})(0.88)(1.05\text{ acres})}{12}] = 0.077\text{ acre-ft} = 3{,}354\text{ ft}^3$$

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:

$$T_v = 3{,}354\text{ ft}^3 - 1{,}638\text{ft}^3 = 1{,}716\text{ft}^3$$

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:

$$A_f = \frac{(1{,}716\ \text{ft}^3)(1.0\ \text{ft})}{(3.5\ \frac{\text{ft}}{\text{da}})\ (1.0\ \text{ft} + 1.0\ \text{ft})(1.67\ \text{days})} = 147\ \text{ft}^2$$

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).

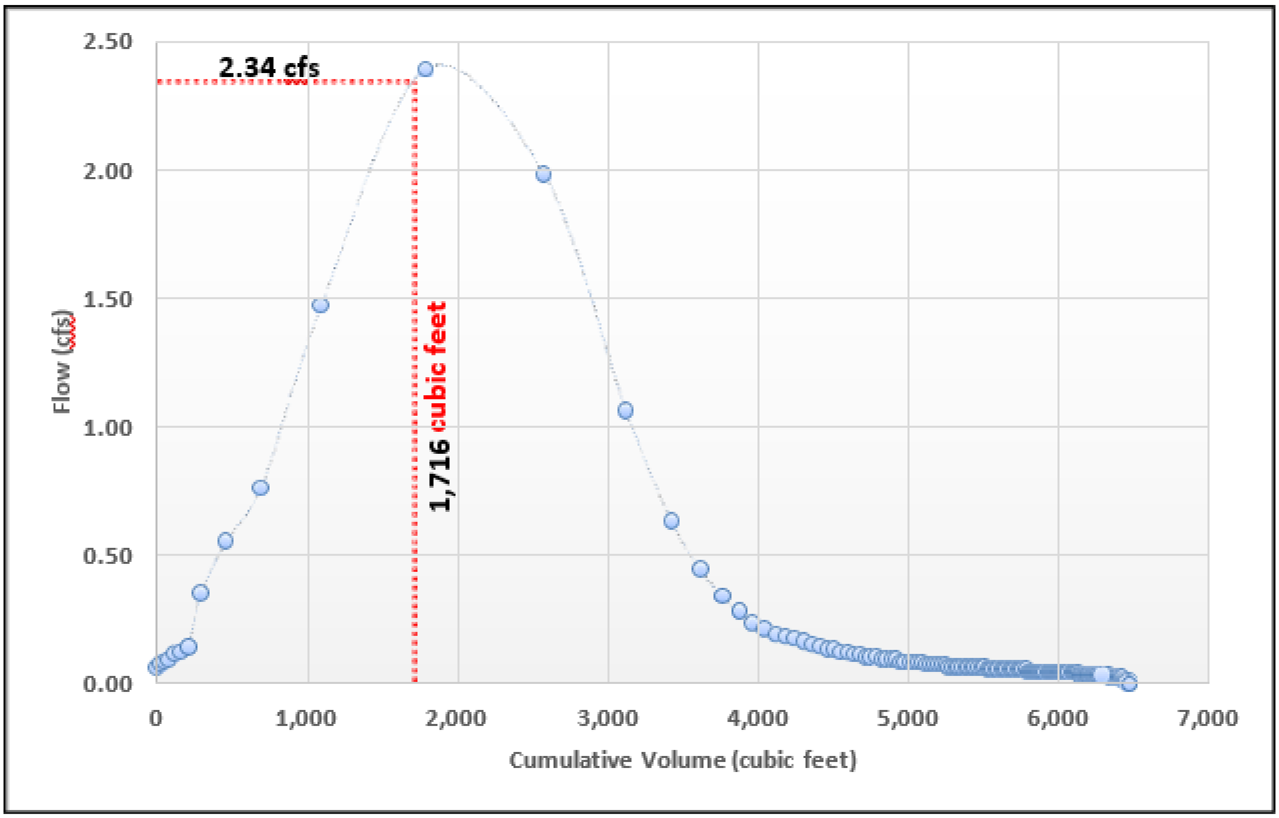

Figure 10.5 Discharge-Volume Curve for the 2-year 24-hour storm

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.

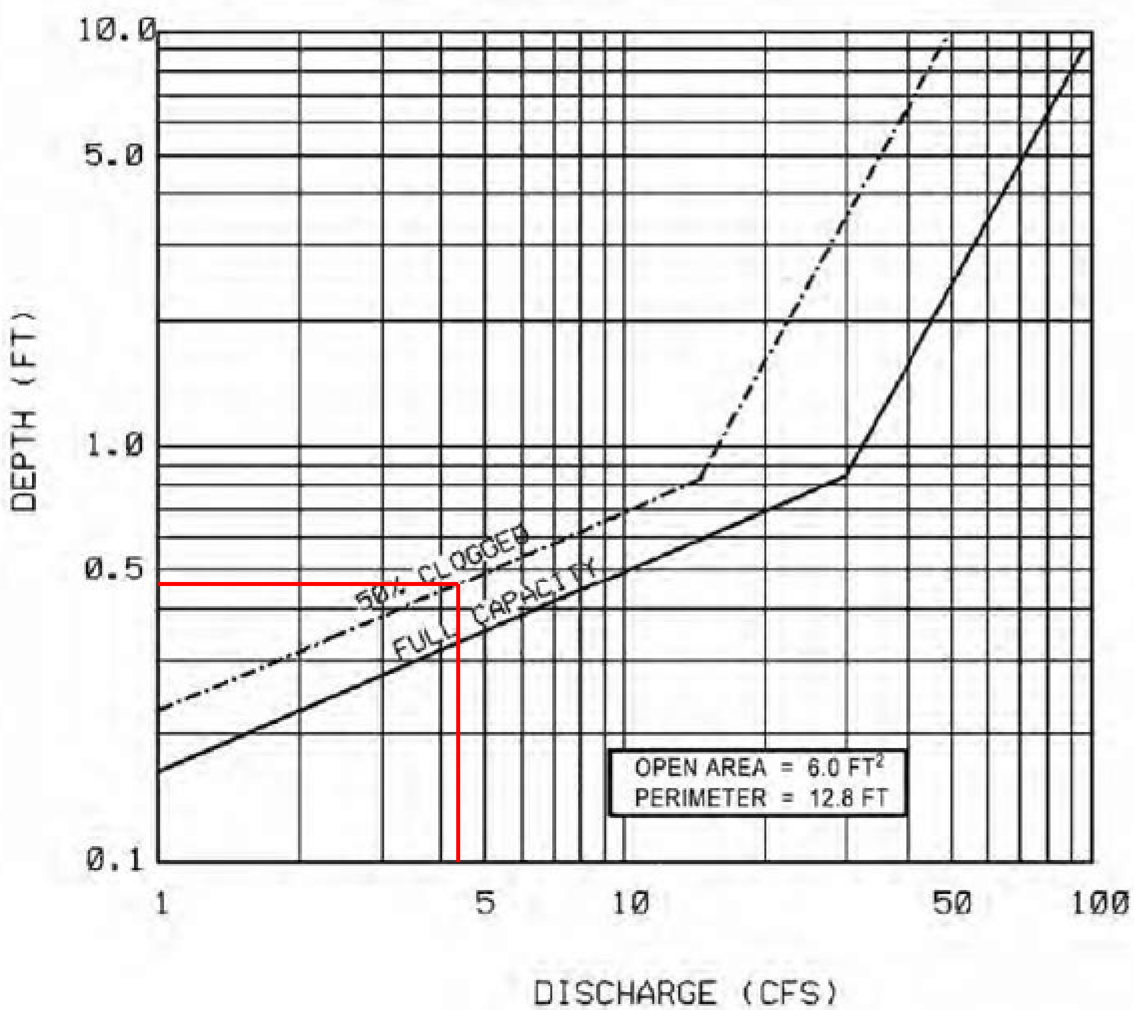

Figure 10.6 VDOT Performance Curve for DI-7 in Sump
VDOT Drainage Manual, Appendix 9C-14

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.

Chapter 11 Constructed Wetlands

11.1 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, 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.		

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.

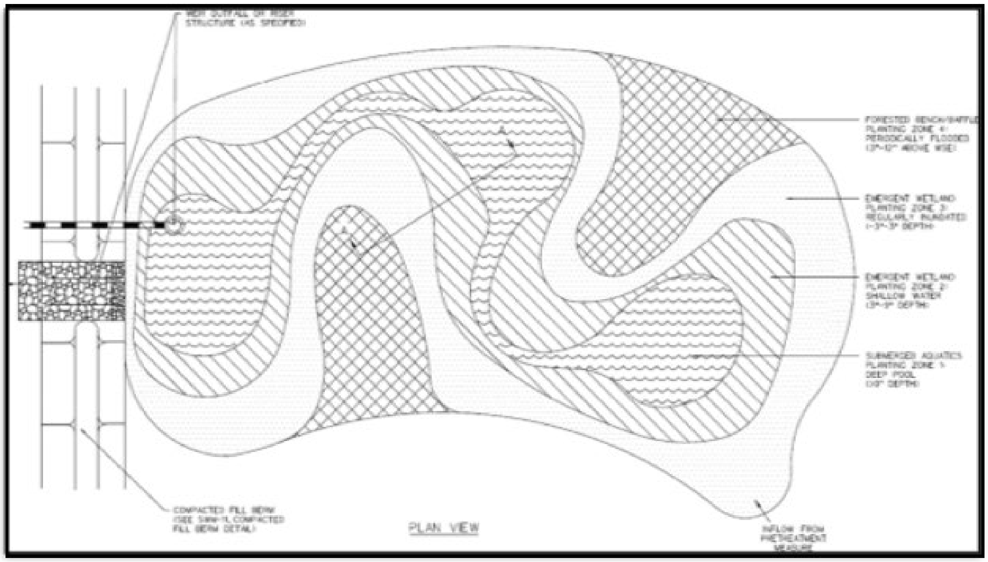

Figure 11.1 Typical Level I Constructed Wetland
VDOT SWM-11 Constructed Wetlands

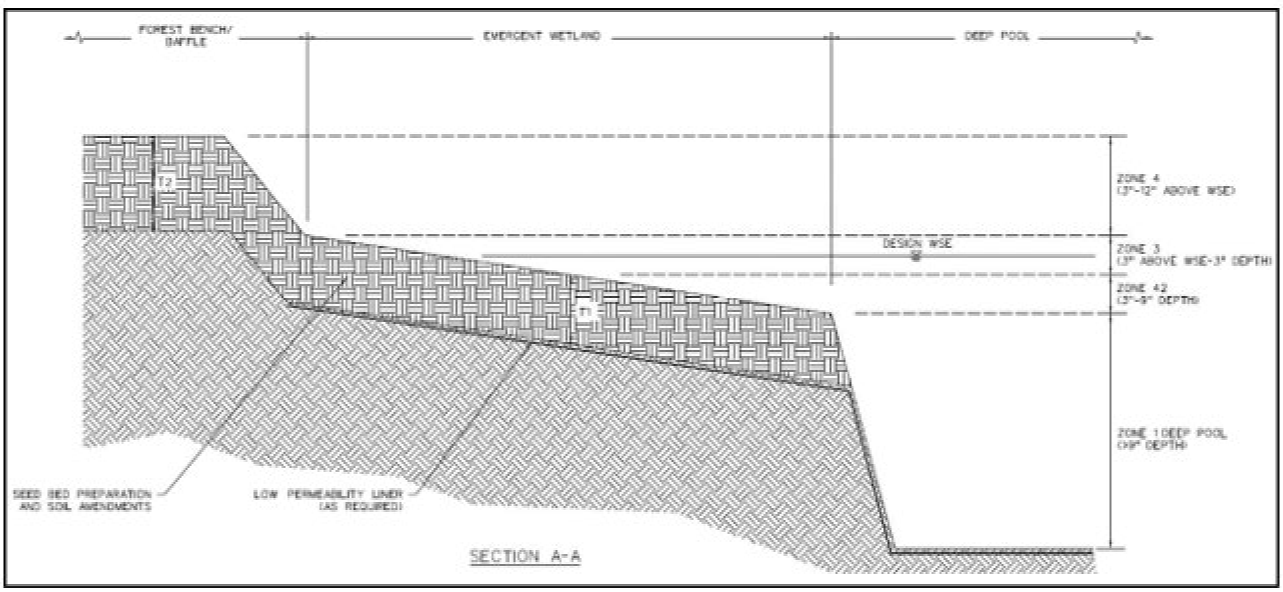

Figure 11.2 Varying Wetland Depth Zones (Profile)
VDOT SWM-11 Constructed Wetlands

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.

11.2.1 Adequate 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 is 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:

- Water volume stored below the normal pool (including deep pools).
- A 24-hour extended detention storage, including a maximum of $0.5T_v$ above the normal pool elevation.
- 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:

- Entire water volume stored below the normal pool of each cell (including deep pools).
- 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:

- 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:
  - Permanent pool volume, containing a minimum of 50% of the pond cell design volume.
  - Extended detention storage above the permanent pool, containing a maximum of 50% of the pond cell design volume.
- 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).

$$\mathrm{DP} = \mathrm{RF_m} \times \frac{(\mathrm{CDA} \times \mathrm{R_{v_{compocite}}})}{\mathrm{A_{wetSand}}} - \mathrm{ET} - \mathrm{INF} - \mathrm{RES} \qquad \textbf{(11.1)}$$

where:

DP = Depth of pool (inches)
$\mathrm{RF_m}$ = Monthly rainfall during drought (inches)
CDA = Contributing drainage area (acres)
$\mathrm{R_{v_{compocite}}}$ = Composite runoff volume coefficient for CDA
$\mathrm{A_{wetSand}}$ = 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:

- **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.

- **High Marsh:** Approximately 70% of the cell **surface area** should have elevations ranging between -6” to +6” relative to the normal pool elevation).

- **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:

- 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)).
- 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.
- 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.
- 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.
- 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.
- 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:
https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-D_Sediment-Forebays_03012011.pdf

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:

- Within the footprint of the proposed embankment(s).
- At the location of the proposed outlet structure.
- A minimum of two locations within the proposed treatment area.

Data from the borings will be used to:

- Determine the adequacy of excavated material for structural fill.
- Determine the need for and design requirements related to an embankment cut-off trench.
- Provide data regarding bearing capacity and buoyancy analysis for use in designing outlet works.
- Determine design depth to seasonal high groundwater and bedrock.
- 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:
https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-A_Earthen-Embankments_03012011.pdf

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:
https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-B_Principal-Spillways_03012011.pdf

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:

- 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.
- A maximum depth of 4’ over the normal pool elevation is recommended during 10-year and 100-year flooding events for on-line facilities.
- The designer should consider using flashboard risers to allow modification of normal pool elevation after construction.
- 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.
- A minimum 3” diameter orifice is recommended to prevent clogging; however, in certain situations, this may be reduced when using internal orifice plates within the pipe or other means to protect orifices < 3” diameter from clogging.
- 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: Vegetated Emergency Spillway** of the *Introduction to the New Virginia Stormwater Design Specifications*, as posted on the Virginia Stormwater BMP Clearinghouse web site at the following URL: https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-C_Vegetated-Emergency-Spillways_03012011.pdf

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: https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-E_Landscaping_03012011.pdf

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:

$$\text{Volume Forebay(s)} = 0.15 \times 22{,}992\ \text{ft}^3 = 3{,}449\text{ft}^3$$

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):

$$Q = \frac{[P - I_a]^2}{(P - I_a) + S} \qquad \textbf{(11.2)}$$

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:

$$S = \frac{1000}{CN} - 10 \qquad \textbf{(11.3)}$$

Substituting **Equation 11.3** into **Equation 11.2** yields:

$$Q = \frac{\left[P - 0.2\left(\frac{1000}{CN} - 10\right)\right]^2}{P + 0.8\left(\frac{1000}{C} - 10\right)} \qquad \textbf{(11.4)}$$

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.

$$Q_{Road} = \frac{\left[1\text{ in} - 0.2\left(\frac{1000\text{ in}}{85} - 10\text{ in}\right)\right]^2}{1\text{ in} + 0.8\left(\frac{1000\text{ in}}{8} - 10\text{ in}\right)} = 0.17\text{ inches}$$

$$Q_{Forect} = \frac{\left[1\text{ in} - 0.2\left(\frac{1000\text{ in}}{77} - 10\text{ in}\right)\right]^2}{1\text{ in} + 0.8\left(\frac{1000\text{ in}}{7} - 10\text{ in}\right)} = 0.05\text{ inches}$$

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:

$$\frac{SF_{Road}}{SF_{Forect}} = \frac{0.17\text{ in} \times \frac{7.51\text{ acres}}{15.3\text{ acres}}}{0.05\text{ in} \times \frac{7.79\text{ acres}}{15.3\text{ acres}}} = \frac{0.083}{0.025} \cong \frac{3.3}{1}$$

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:

$$V_{deep} = 0.25(T_V) = 0.25(22{,}992 \text{ ft}^3) = 5{,}748 \text{ ft}^3$$

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:

$$\frac{5{,}748 \text{ ft}^3}{4 \text{ ft}} = 1{,}437 \text{ ft}^2$$

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.

$$\frac{1{,}437 \text{ ft}^2}{0.08} = 17{,}963 \text{ ft}^2$$

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.

$$17{,}963 \text{ ft}^2 \times 0.70 = 12{,}574 \text{ ft}^2$$

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:

$$0.50(\mathrm{Tv}) - \mathrm{V_{deep}} = 0.50(22{,}992\ \mathrm{ft}^3) - 5{,}748\ \mathrm{ft}^3 = 5{,}748\ \mathrm{ft}^3$$

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:

$$0.5 \times 12{,}574\ \mathrm{ft}^2 \times 0.25\ \mathrm{ft} + 3{,}952\ \mathrm{ft}^2 \times 1\ \mathrm{ft} = 5{,}524\ \mathrm{ft}^3$$

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)	Elevation	Incremental Volume (ft^3)	Total Volume (ft^3)
48	0	0	51.5	2352.24	7187.4
48.5	696.96	696.96	52	4225.32	11412.72
49	696.96	1393.92	52.5	7361.64	18774.36
49.5	696.96	2090.88	53	9365.4	28139.76
50	696.96	2787.84	53.5	10367.28	38507.04
50.5	696.96	3484.8	54	11761.2	50268.24
51	1350.36	4835.16			

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:

$$\mathrm{DP} = 3.19\ \mathrm{in} \times \frac{(15.3\ \mathrm{ac} \times 0.41)}{0.41\ \mathrm{ac}} - 8\ \mathrm{in} - 7.2\ \mathrm{in} - 6\ \mathrm{in} = 28\ \mathrm{inc}hes$$

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.

Chapter 12 Wet Ponds

12.1 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)

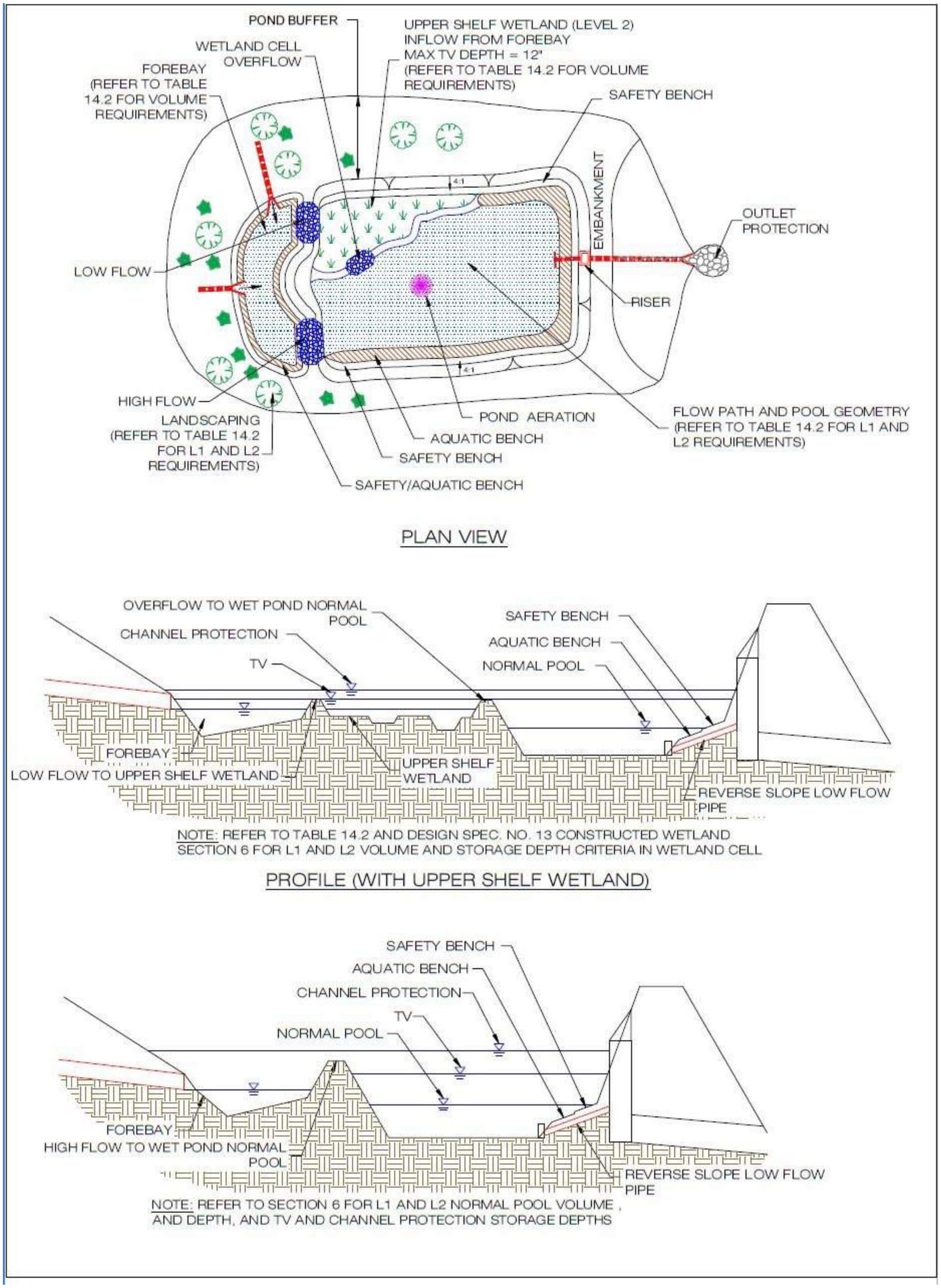

Figure 12.1 Schematic of Wet Pond Facility
Virginia Stormwater Design Specification No. 14, Wet Pond (DCR/DEQ, 2013)

12.2 Site Constraints and Siting ofthe 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 1×10^{-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 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:

$$Pond\ inches = ft^3/s * (2.592E6) * (12''/ft) / SA\ of\ Pond\ (ft^2) \qquad (\mathbf{12.2})$$

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:
https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-B_Principal-Spillways_03012011.pdf

12.3.10 Low Flow Orifice

Traditionally, orifice sizes would be a minimum of 3” in diameter to prevent clogging. However, newer basins designed for extended drawdown may require an orifice < 3” in diameter to meet the technical criteria. Risks to clogging of small orifices <3” in diameter 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; or
- 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:
https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-A_Earthen-Embankments_03012011.pdf

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.*

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: Vegetated 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:
https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-C_Vegetated-Emergency-Spillways_03012011.pdf

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.

- 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.

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 12.4**.

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: https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-E_Landscaping_03012011.pdf

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.

$$T_v = \left[\frac{(1.50)(1.0\ \text{in.})(0.61)(105.0\ \text{acres})}{12}\right] = 8.0\ \text{acre-ft} = 348{,}480\ \text{ft}^3$$

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:

$$\text{Volume Forebay} = 0.25\text{ in}\left(\frac{1\text{ ft}}{12\text{ in}}\right) \times 59.8\text{ ac}\left(\frac{43{,}560\text{ ft}}{1\text{ ac}}\right) = 54{,}268\text{ ft}^3$$

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**.

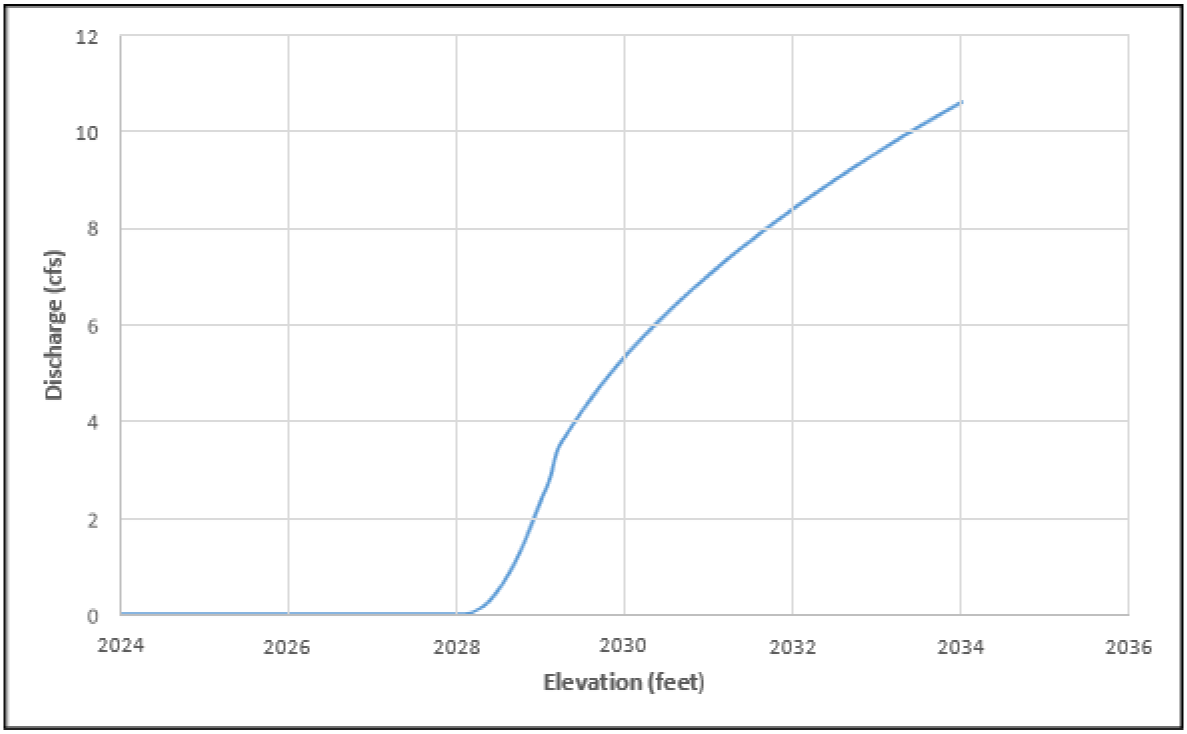

Figure 12.2 Discharge-Elevation Curve for Circular Orifice

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.

$$DP > 8'' + 7.2'' + 24'' - 0 \qquad (12.1)$$

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:

$$\frac{13{,}207\mathrm{ft}^2}{83{,}372\mathrm{ft}^2} \times 100 = 15.8\%$$

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 - Outlet Protection

Discharge locations should be evaluated using requirements set forth in the Virginia Erosion and Sediment Control Handbook, and specifically Minimm Standard 11 (MS-11), 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 were 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.

Chapter 13 Extended Detention

13.1 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)

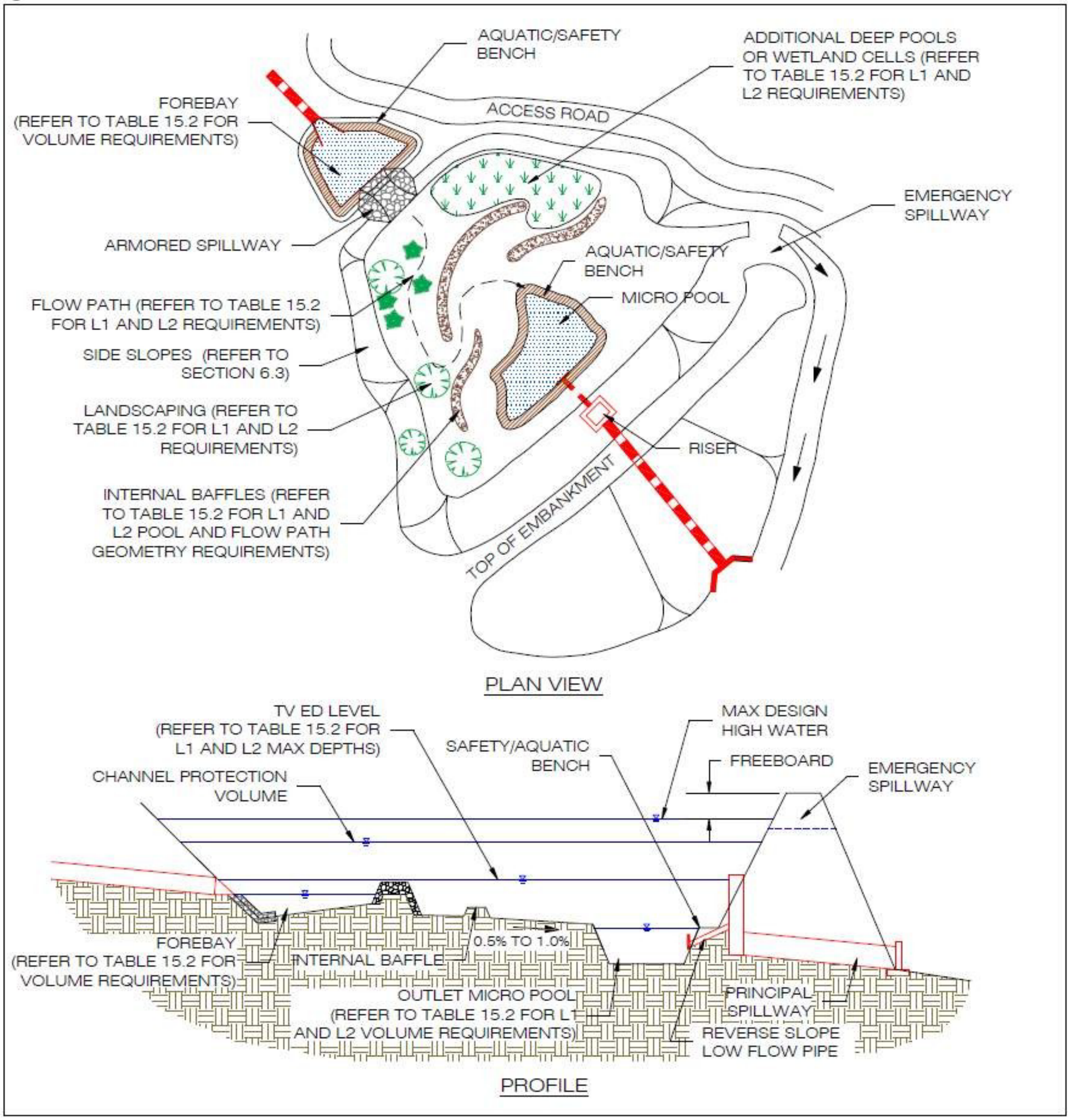

Figure 13.1 Schematic Extended Detention Basin Plan View
Virginia Stormwater Design Specification 15, Extended Detention Pond, (DCR/DEQ, 2013)

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:
https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-B_Principal-Spillways_03012011.pdf

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. 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 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:
https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-A_Earthen-Embankments_03012011.pdf

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

Traditionally, orifice sizes would be a minimum of 3” in diameter to prevent clogging. However, to meet the technical criteria of Part II B, orifices smaller than 3” may be necessary. 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 x 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 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 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:
https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-C_Vegetated-Emergency-Spillways_03012011.pdf

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:
https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-D_Sediment-Forebays_03012011.pdf

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:
https://www.swbmp.vwrrc.vt.edu/wp-content/uploads/2017/11/Introduction_App-E_Landscaping_03012011.pdf

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.

$$R_{v_I} = \frac{(4.60 \text{ acres})(0.95)}{4.60 \text{ acres}} = 0.95$$

$$R_{v_T} = \frac{(19.00 \text{ acres})(0.20)}{19.00 \text{ acres}} = 0.20$$

$$R_{v_{composite}} = (R_{v_I} \times \%I) + (R_{v_T} \times \%T)$$

$$R_{v_{composite}} = (0.95 \times \frac{4.60 \text{ acres}}{23.60 \text{ acres}}) + (0.20 \times \frac{19.00 \text{ acres}}{23.60 \text{ acres}}) = 0.35$$

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.

$$T_v = [\frac{(1.25)(1.0 \text{ in.})(0.35)(23.6 \text{ acres})}{12}] = 0.860 \text{ acre-ft} = 37{,}462 \text{ ft}^3$$

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:

$$\text{Volume(s)} = 0.15 \times 37{,}462 \text{ ft}^3 = 5{,}619 \text{ ft}^3$$

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:

$$\text{Total Volume} = (0.70 \times \text{Average Dept}h\ \text{Hig}h + 0.30 \times \text{Average Dept}h\ \text{Low}) * \text{Area}$$

Solving for Area:

$$9{,}366\ \text{ft}^3 = (0.70 \times 0.25\ \text{ft} + 0.30 \times 1\ \text{ft}) * \text{Area}$$

$$\text{Area} = 19{,}717\text{ft}^2$$

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:

$$\frac{5{,}619\ \text{ft}^3}{4\ \text{ft}} = 1{,}405\ \text{ft}^2$$

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).

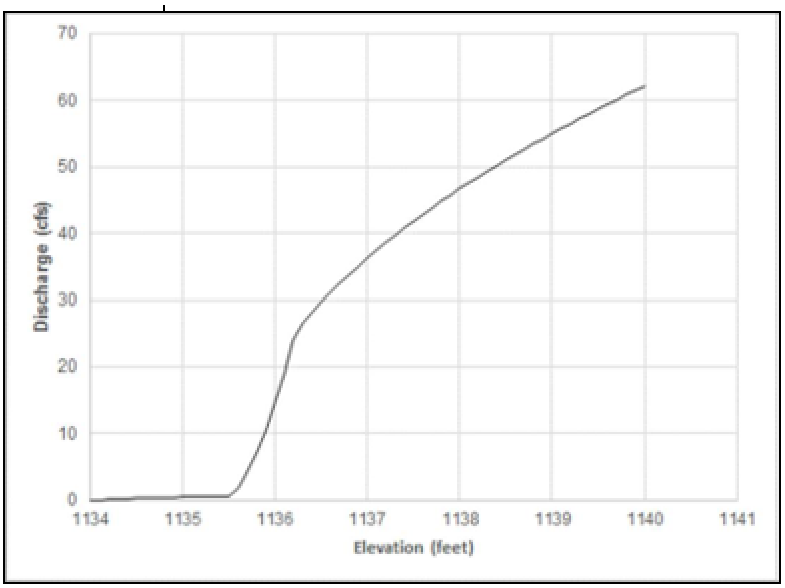

Figure 13.2 Discharge-Elevation Curve for Outlet Structure Design

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:

$$\mathrm{DP} = 2.87\ \mathrm{in} \times \frac{(23.6 \times 0.35)}{0.48} - 8\ \mathrm{in} - 7.2\ \mathrm{in} - 6\ \mathrm{in} = 28\ \mathrm{inc}hes$$

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:

$$\mathrm{V_{dic}} = \mathrm{n}(3\mathrm{ft})^2(7.17\mathrm{ft}) = 202.73\ \mathrm{ft}^3$$

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

$$\mathrm{F_{buoyant}} = 202.73\ \mathrm{ft}^3 \times 62.4\frac{\mathrm{lb}}{\mathrm{ft}^3} = 12{,}650\ \mathrm{lb}$$

Applying the 1.24 factor of safety:

$$\mathrm{F_{decign}} = 12{,}650\ \mathrm{lb} \times 1.25 = 15{,}813\ \mathrm{lb}$$

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:

$$F_{bace} = (0.667\ \text{ft})n(3\text{ft})^2 \times 150\frac{\text{lb}}{\text{ft}^3} = 2{,}829\ \text{lb}$$

Weight of manhole riser:

$$F_{ricer} = (5.84\text{ft})[n(3\text{ft})^2 - n(2.5\text{ft})^2] \times 150\frac{\text{lb}}{\text{ft}^3} = 7{,}568\ \text{lb}$$

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:

$$F_{orifice} = (0.5\text{ft})[n(0.1875\ \text{ft})^2] \times 150\frac{\text{lb}}{\text{ft}^3} = 8.3\ \text{lb}$$

The total force down is computed as:

$$2{,}829\ \text{lb} + 7{,}568\ \text{lb} + 120\ \text{lbs} + 2{,}000\ \text{lbs} - 8.3\ \text{lbs} = 12{,}509\ \text{lbs}$$

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:

$$D_{additionaS} = \frac{15{,}813\ \text{lbs} - 12{,}509\ \text{lbs}}{15\ \ \frac{\text{lb}}{\text{ft}^3} \times n(2.5\ \text{ft})^2} = 1.12\ \text{feet}$$

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.

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