Virginia Department of Transportation

BMP Design Manual of Practice

1401 East Broad Street Richmond, Virginia 23219

BMP DESIGN MANUAL OF PRACTICE

Prepared for

Location and Design Division – Hydraulics/Utilities Program

and

Virginia Center for Transportation Innovation & Research

by

Virginia Tech

Effective April 2013

Virginia Department of Transportation

Approved By: ______________________
State Location and Design Engineer

Date: 4/4/13

COMMONWEALTH of VIRGINIA

DEPARTMENT OF TRANSPORTATION
1401 EAST BROAD STREET
RICHMOND, VIRGINIA 23219-2000

Gregory A. Whirley
Commissioner

April 4, 2013

MEMORANDUM

TO: All Virginia Department of Transportation Preliminary Engineering Design Staff

FROM: Mohammad Mirshahi, P.E. - State Location & Design Engineer

RE: 2013 BMP Design Manual of Practice

Effective April 4, 2013, the Virginia Department of Transportation (VDOT) adopted the 2013 *"BMP Design Manual of Practice"* (the "BMP Manual"), which was prepared by Virginia Tech under the direction of the Location & Design Division and the Virginia Center for Transportation Innovation and Research. The BMP Manual has been under development for some time, and has recently been updated with the latest applications of BMP design as recognized by public and private stakeholders. The BMP Manual's purpose is to provide guidance in the design of Best Management Practices capable of contributing to the goal of stormwater management as required under Part II C of the VSMP Regulations (4VAC50-60-93.1 et seq.) and defined by the Department's IIM-LD-195 ("Post Development Stormwater Management").

Users of the BMP Manual are encouraged to utilize it to aid in addressing water quality, and as a BMP selection tool (in terms of weighing physical site constraints versus BMP adaptability). The BMP Manual is not meant to be an all-encompassing guidance document that addresses water quality and quantity. The designer still has the obligation to address flooding and erosion control for each site/project, and is thus directed towards the Virginia Erosion and Sediment Control Handbook, the Virginia Stormwater Management Handbook, the Virginia Stormwater Management Program, the VDOT Drainage Manual, IIM-LD-11and IIM-LD-195.

If you have any questions or comments regarding the adoption of the 2013 *"BMP Design Manual of Practice"*, please contact the State Hydraulics and Utilities Engineer, Jeff Bragdon, P.E., at (804) 786-8025.

Sincerely,

Mohammad Mirshahi, P.E.
State Location & Design Engineer

BMP DESIGN MANUAL OF PRACTICE

TABLE OF CONTENTS

Chapter 1 – Introduction

TABLE OF CONTENTS

1.1 Introduction

This manual was prepared for the Virginia Department of Transportation by Virginia Tech under contract for the Virginia Center for Transportation Innovation and Research. It provides guidance in the design of Best Management Practices capable of contributing to the goal of stormwater management as defined in VDOT's Instructional and Informational Memorandum IIM-LD-195, under *"Post Development Stormwater Management"*.

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

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

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

1.2 Project Site

The *project site* as defined in the Stormwater Program Advisory SWPA 12-01 dated April 5, 2012, available at: http://www.virginiadot.org/business/resources/LocDes/SWPA_12-01.pdf as:

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

1.3 Water Quality Standards

Effective April 5, 2012, Stormwater Program Advisory SWPA 12-01 states that "Evaluation of water quality requirements shall be performed using the Performance Based Water Quality Criteria (see the Virginia Stormwater Management Handbook (1999) and VDOT IIM-LD-195. Although it is recognized that this is the standard for all new projects passed that date, there may be some projects underway prior to that date that may be designed under the direction found in VDOT IIM-LD-195.

Therefore, it is the designer's responsibility to determine and verify with the Department the methodology that is required on individual projects. Details on the Technology and Performance Based water quality calculation methodologies may be found in the Virginia Stormwater Management Handbook (1999) and VDOT IIM-LD-195.

The BMP selection table is shown in Table 1.1. While typically Table 1.1 would be used to select appropriate BMPs based on post-construction impervious cover using the "Technology Based" approach, it may also be used as a reference for projected BMP efficiencies when using a "Performance Based" approach.

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

Table 1.1 BMP Selection Table for VDOT Projects*

*Innovative or alternate BMPs not included in this table may be allowed at the discretion of DCR and with the concurrence of the VDOT State Hydraulics Engineer, as stated in IIM-LD-195.
(Refer to DCR website for current state of practice).

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

1.4 Water Quantity Standards

Although it is recognized that some BMPs used for water quality control implicitly have the ability to partially, or in some cases, fully meet the requirements for stormwater quantity control, this manual is not intended to cover Commonwealth of Virginia requirements for flooding or erosion control. The user is directed to the Virginia Erosion and Sediment Control Handbook (Third Edition, 1992) the Virginia Stormwater Management Handbook (First Edition, 1999) the Virginia Stormwater Management Program (VSMP) Permit Regulations (latest revision effective Nov. 21, 2012), the VDOT Drainage Manual (rev. July 2012) and any applicable VDOT Instructional and Information Memoranda (specifically IIM-LD-11; IIM-LD-195; IIM-LD-242; IIM-LD-246) for further discussion of specific state requirements and sample calculations related to stormwater quantity control.

Chapter 2 – Dry Extended Detention Basin

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

2.1 Overview of Practice

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

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

Figure 2.1 presents the schematic layout of a dry extended detention basin presented in the Virginia Stormwater Management Handbook (DCR, 1999, Et seq.). Of note is that the low flow rip rap lined channel has been removed from the drawing. Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management"*, Section 5.4.8.6, this channel is not recommended due to maintenance concerns.

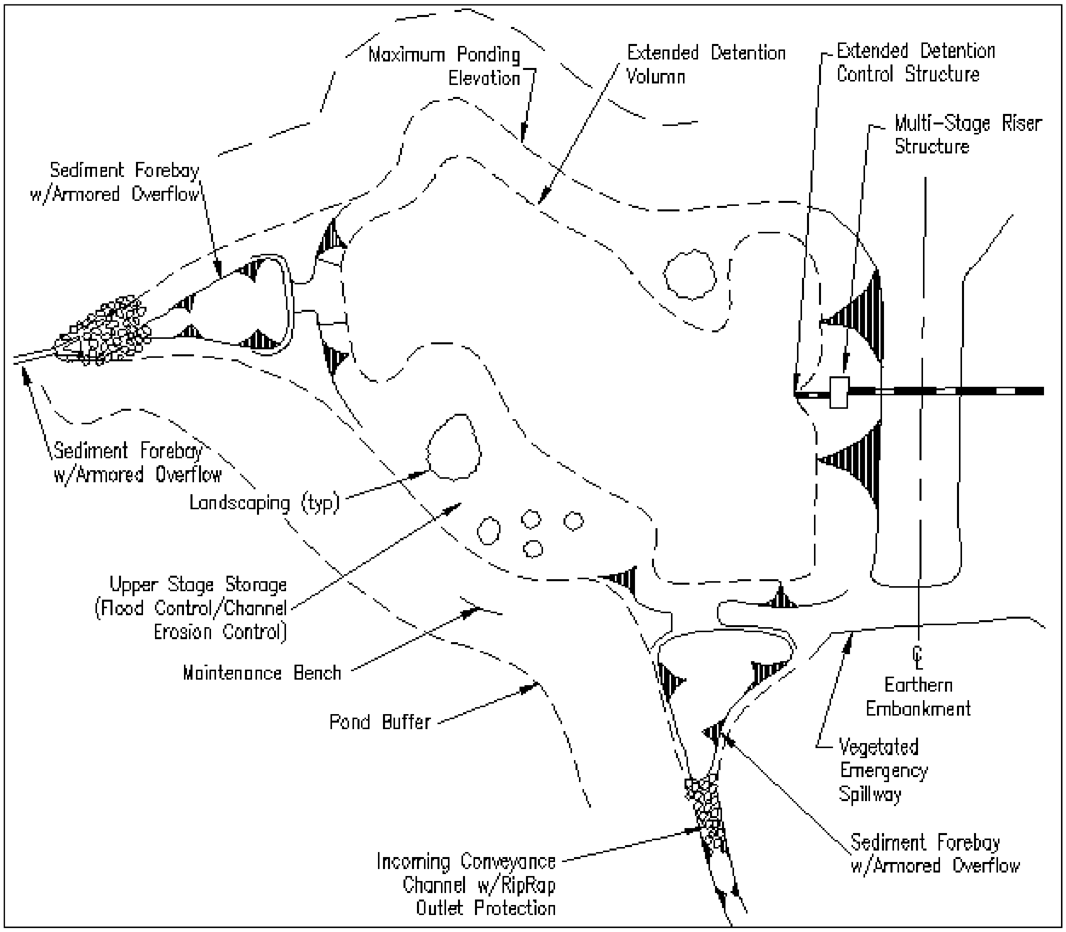


Figure 2.1. Schematic Dry Extended Detention Basin Plan View
(Virginia Stormwater Management Handbook, 1999, Et seq.)

2.2 Site Constraints and Siting of the Facility

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

2.2.1 Minimum Drainage Area

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

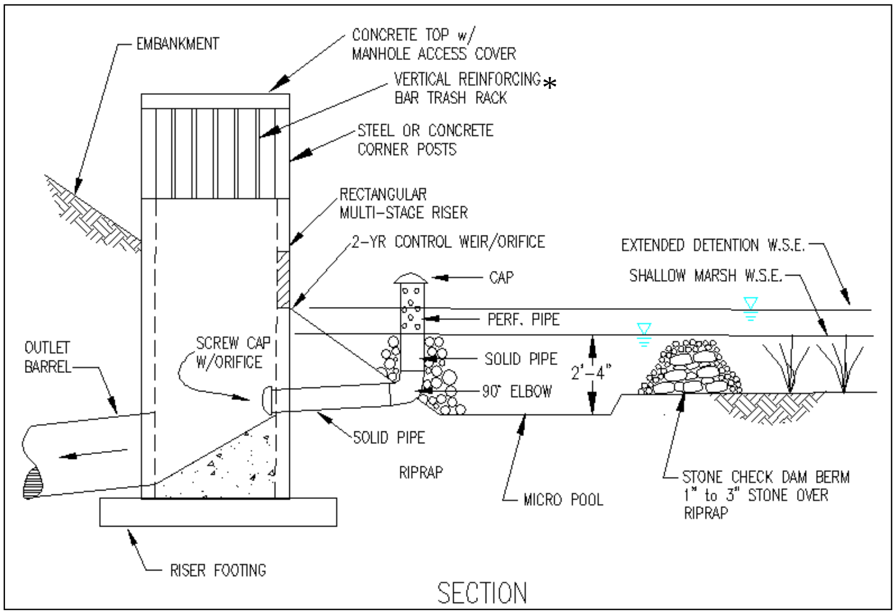


Figure 2.2. DCR Recommended Outlet Configuration 1 for the Control of Trash, Sediment and Debris (Virginia Stormwater Management Handbook, 1999, Et seq.)

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

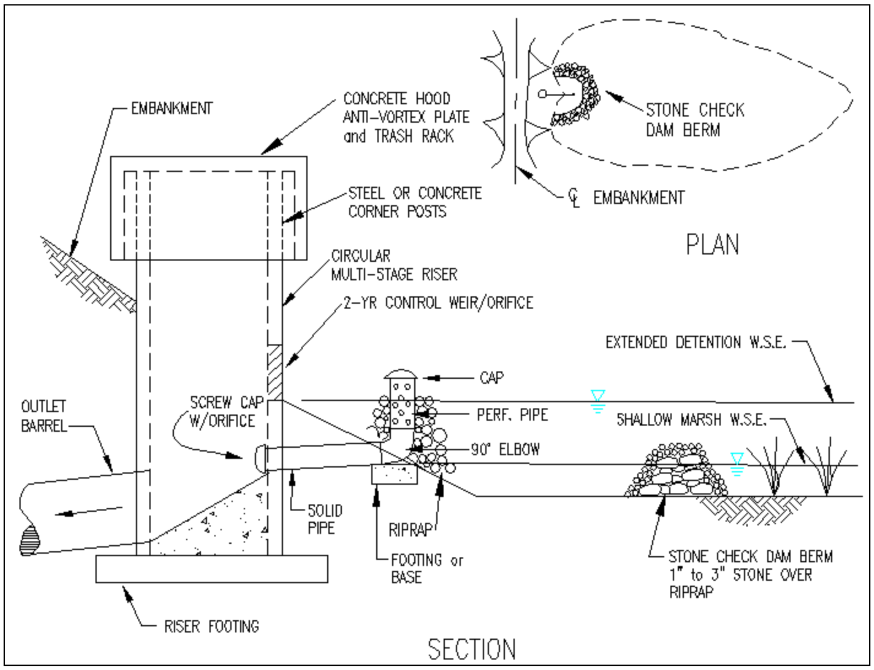


Figure 2.3. DCR Recommended Outlet Configuration 2 for the Control of Trash, Sediment and Debris (Virginia Stormwater Management Handbook, 1999, Et seq.)

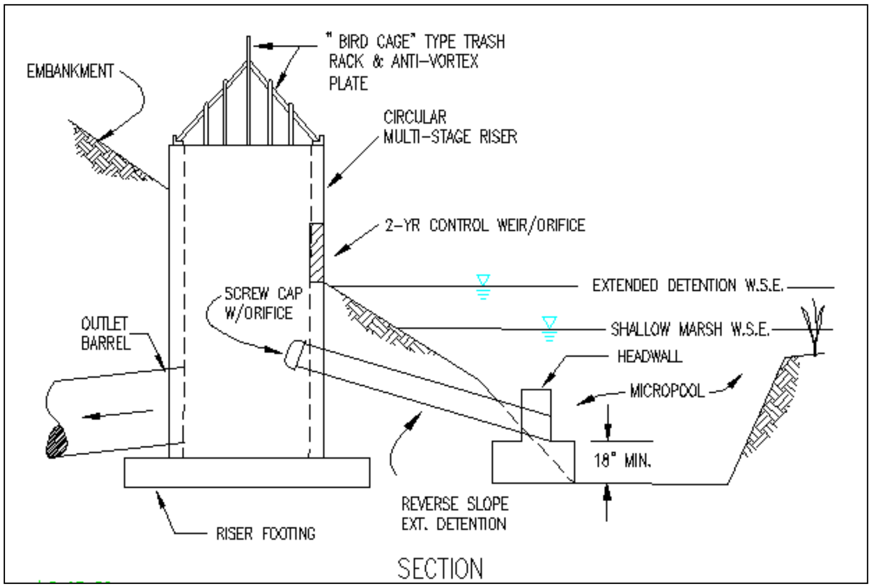


Figure 2.4. DCR Recommended Outlet Configuration 3 for the Control of Trash, Sediment and Debris (Virginia Stormwater Management Handbook, 1999, Et seq.)

2.2.2 Maximum Drainage Area

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

2.2.3 Separation Distances

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

2.2.4 Site Slopes

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

2.2.5 Site Soils

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

2.2.6 Rock

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

2.2.7 Existing Utilities

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

2.2.8 Karst

The presence of Karst topography places even greater importance on the subsurface investigation. Implementation of dry extended detention facilities in Karst regions may greatly impact the design and cost of the facility, and must be evaluated early in the planning phases of a project. Construction of stormwater management facilities within a sinkhole is prohibited. When the construction of such facilities is planned along the periphery of a sinkhole, the facility design must comply with the guidelines found in Instructional and Informational Memorandum IIM-LD-228, *"Sinkholes"* and DCR's Technical Bulletin #2 *"Hydrologic Modeling and Design in Karst" at:* http://www.dcr.virginia.gov/stormwater_management/documents/tecbltn2.PDF.

2.2.9 Wetlands

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

2.2.10 Upstream Sediment Considerations

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

2.2.11 Floodplains

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

2.2.12 Basin Location

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

Per Instructional and Informational Memorandum IIM-LD-195, under *"Post Development Stormwater Management,"* Section 6.9:

"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."

2.3 General Design Guidelines

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

2.3.1 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam. Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management"*, Section 12.1.1:

"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 feet or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."

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

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

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

2.3.2 Outfall Piping

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

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

2.3.3 Embankment

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

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

2.3.4 Embankment Height

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

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

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

2.3.5 Prevention of Short-Circuiting

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

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

It is preferable to construct the basin such that the length to width ratio is 3:1 or greater, with the widest point observed at the outlet end. If this is not possible, every effort should be made to design the basin with no less than a 2:1 length to width ratio. When this minimum ratio is not possible, consideration should be given to pervious baffles.

2.3.6 Ponded Depth

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

2.3.7 Principal Spillway Design

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

2.3.8 Emergency Spillway Stabilization

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

2.3.9 Fencing

Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management",* Section 13.1.1, fencing is typically not required or recommended on most VDOT detention facilities. However, exceptions do arise, and the fencing of a dry extended detention facility may be needed. Such situations include:

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

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

2.3.10 Sediment Forebays

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

2.3.11 Discharge Flows

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

2.4 Design Process

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

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

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

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

Table 2.1. Hydrologic Characteristics of Example Project Site

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

Table 2.2. Peak Rates of Runoff (cfs)

Step 1. Compute the Required Water Quality Volume

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

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

IA= Impervious Area (ft^2)

Dry extended detention basins should be designed to provide extended draw down for *two times the computed water quality volume (2xWQ$_V$).*

If the basin is to be implemented as a water quality basin, this computed volume of twice the WQ_V must be detained and released over a period of not less than 30 hours. The basin must completely drawdown within 72 hours.

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

Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management"*, Section 5.4.6, when the 1-year return frequency storm is detained for a minimum of 24 hours there is no need to provide additional or separate storage for the WQ_V provided it can be demonstrated that the WQ_V will be detained for approximately 24 hours.

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

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

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

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

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

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

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

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

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

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

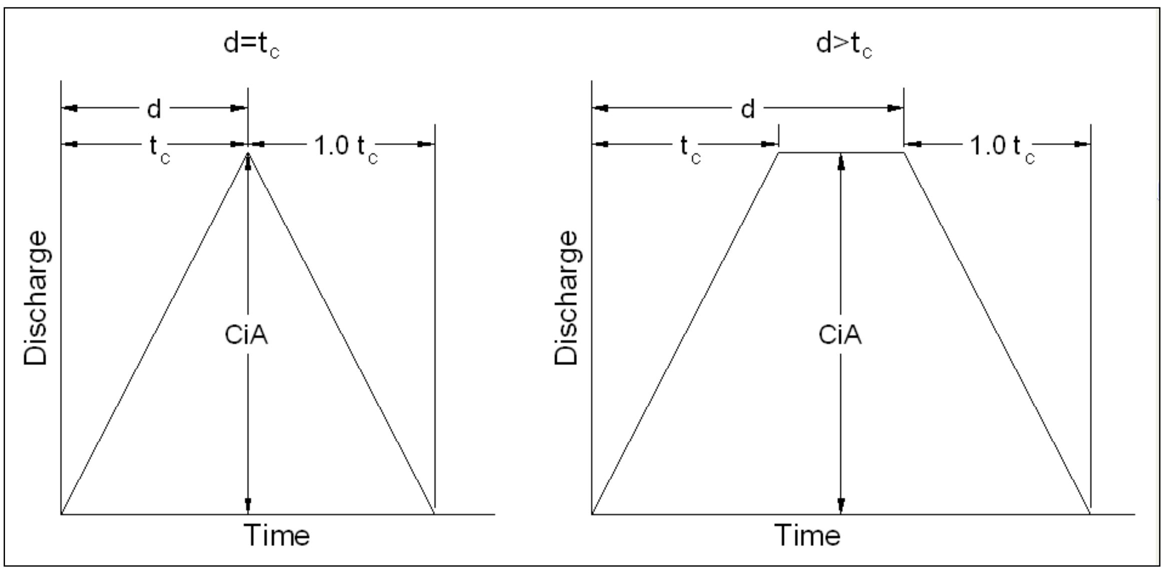


Figure 2.5. Modified Rational Runoff Hydrographs

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

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

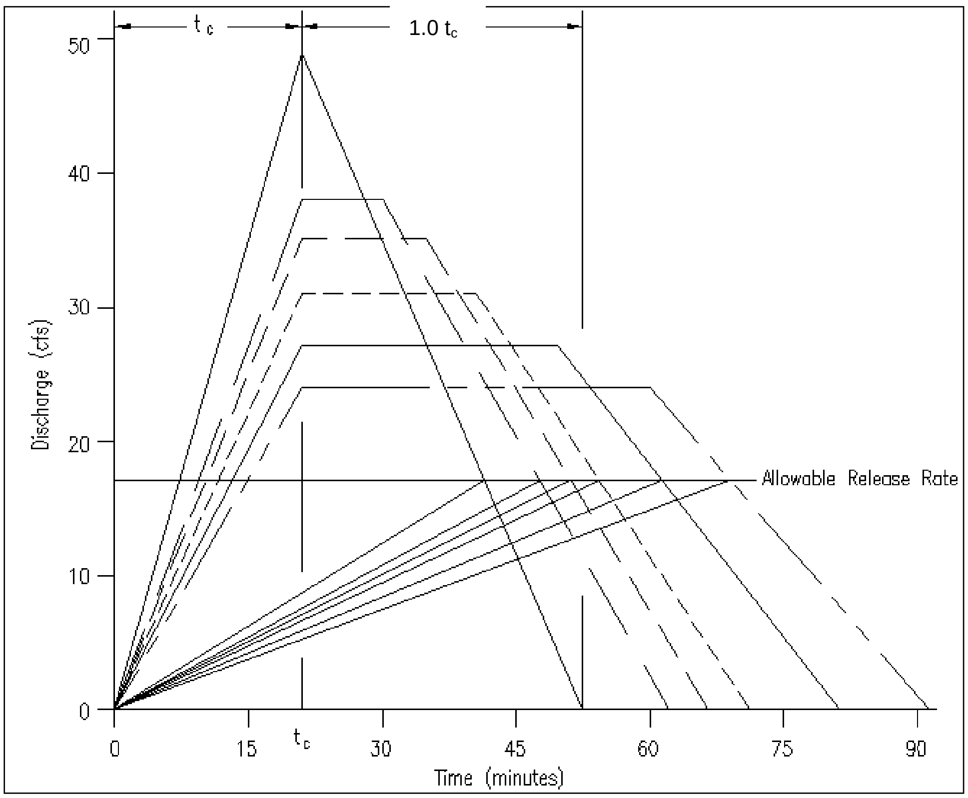


Figure 2.6. Graphical Determination of Critical Rainfall Intensity Duration
(Virginia Stormwater Management Handbook, 1999, Et seq.)

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

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

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

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

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

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

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

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

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

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

Table 2.3. Rainfall Regression Constants
Montgomery County

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

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

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

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

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

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

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

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

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

$$Q = CiAC_f$$

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

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

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

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

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

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

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

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

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

Step 3. Development of Runoff Hydrographs

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

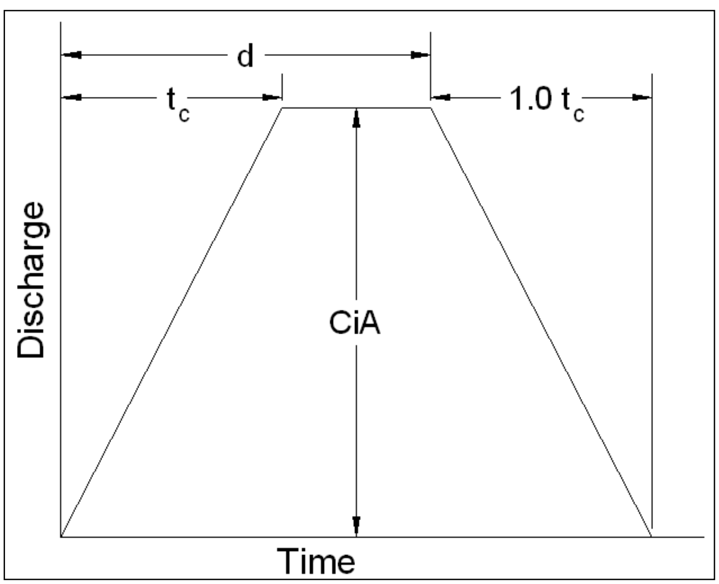


Figure 2.7. Modified Rational Hydrograph Shape

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

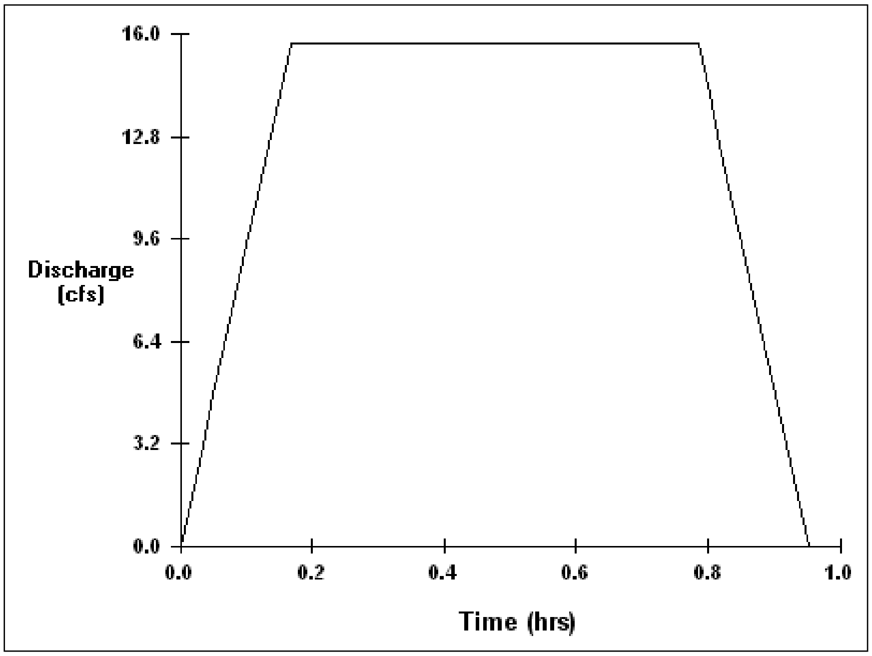


Figure 2.8. 2-Year Post-Development Modified Rational Hydrograph

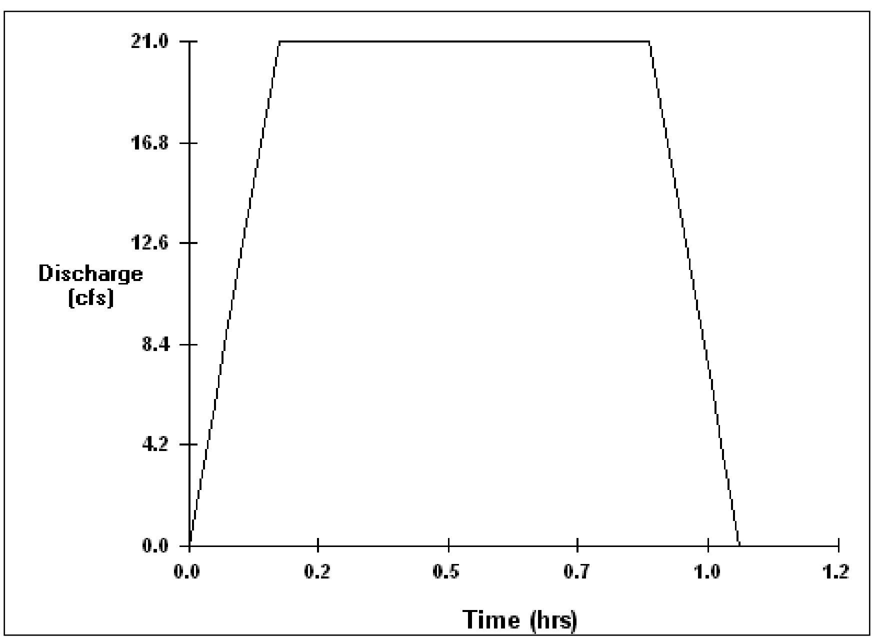


Figure 2.9. 10-Year Post-Development Modified Rational Hydrograph

Step 4. Development of Storage Versus Elevation Data

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

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

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

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

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

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

Table 2.4. Basin Storage Versus Elevation Data

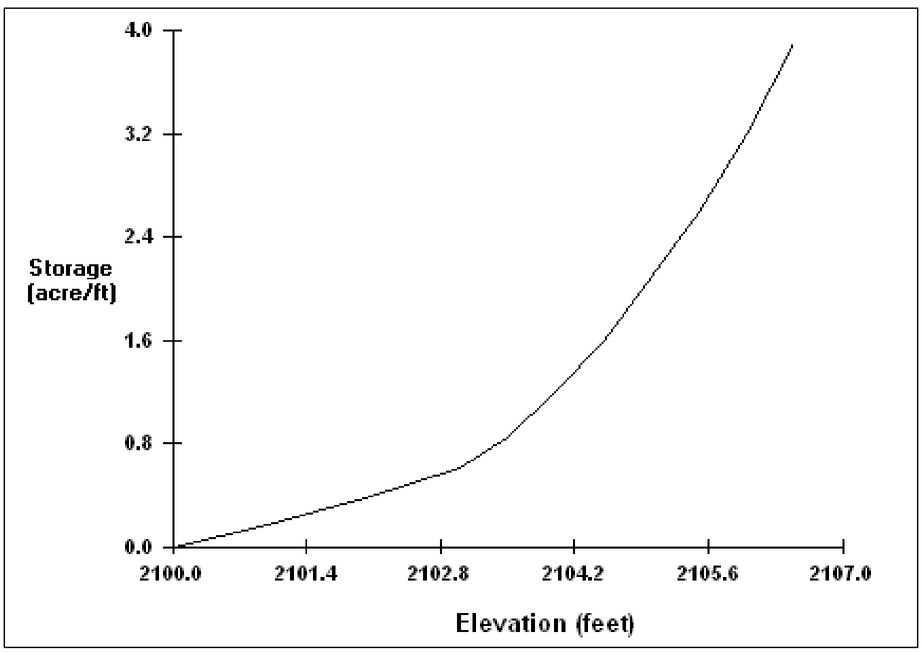


Figure 2.10. Basin Storage Versus Elevation Curve

Step 5. Design of the Water Quality Control Orifice

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

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

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

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

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

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

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

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

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

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

Rearranging the orifice equation, the orifice area is computed as

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

The diameter is then computed as:

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

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

Step 6. Design of the Principal Spillway

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

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

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

Step 6A. Size Basin Outfall Culvert

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

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

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

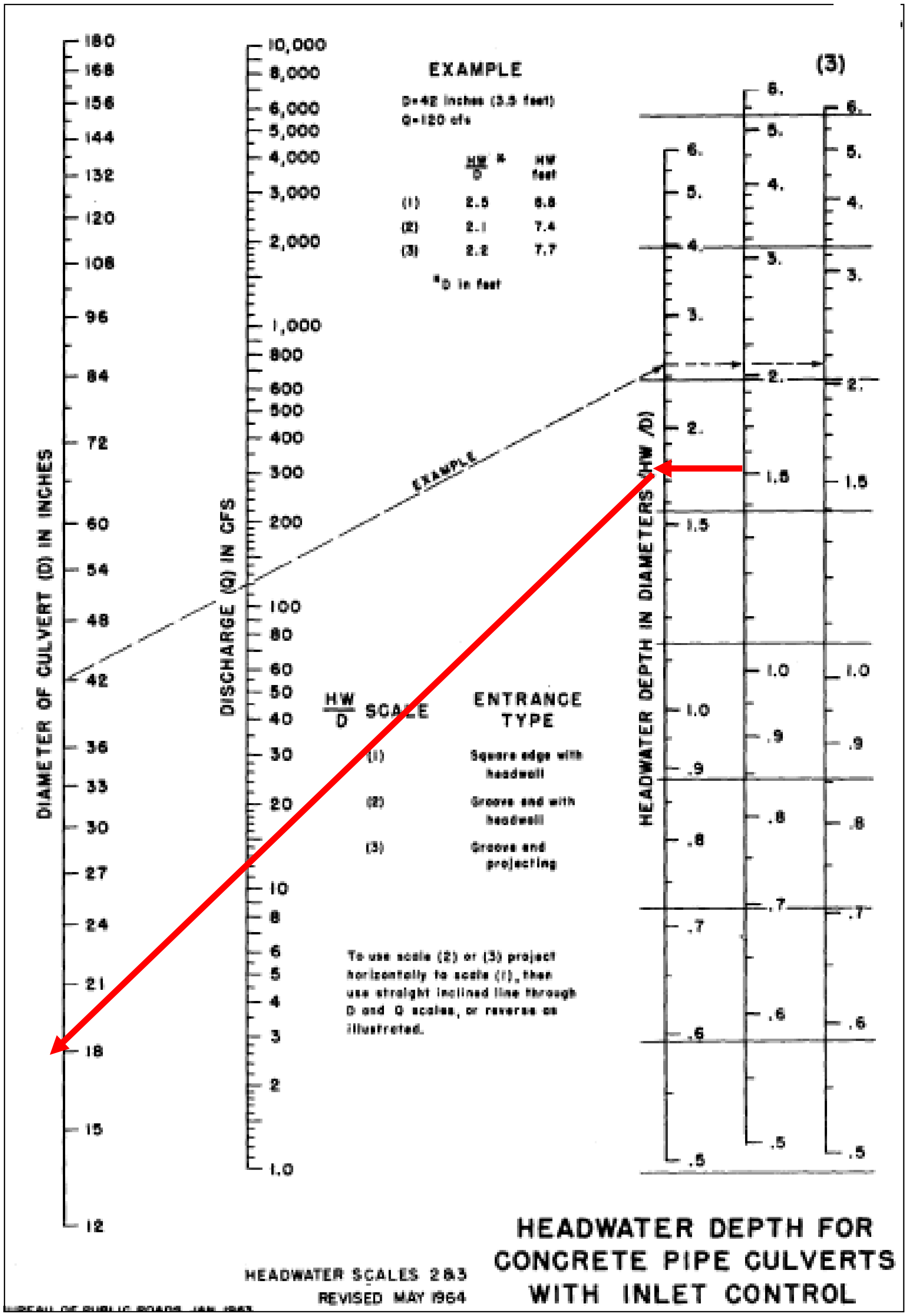


Figure 2.11 Culvert Design Chart (FHWA, 2001)

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

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

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

Step 6B. Design the 2-Year Control Outlet

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

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

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

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

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

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

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

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

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

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

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

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

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

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

The diameter is then computed as:

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

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

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

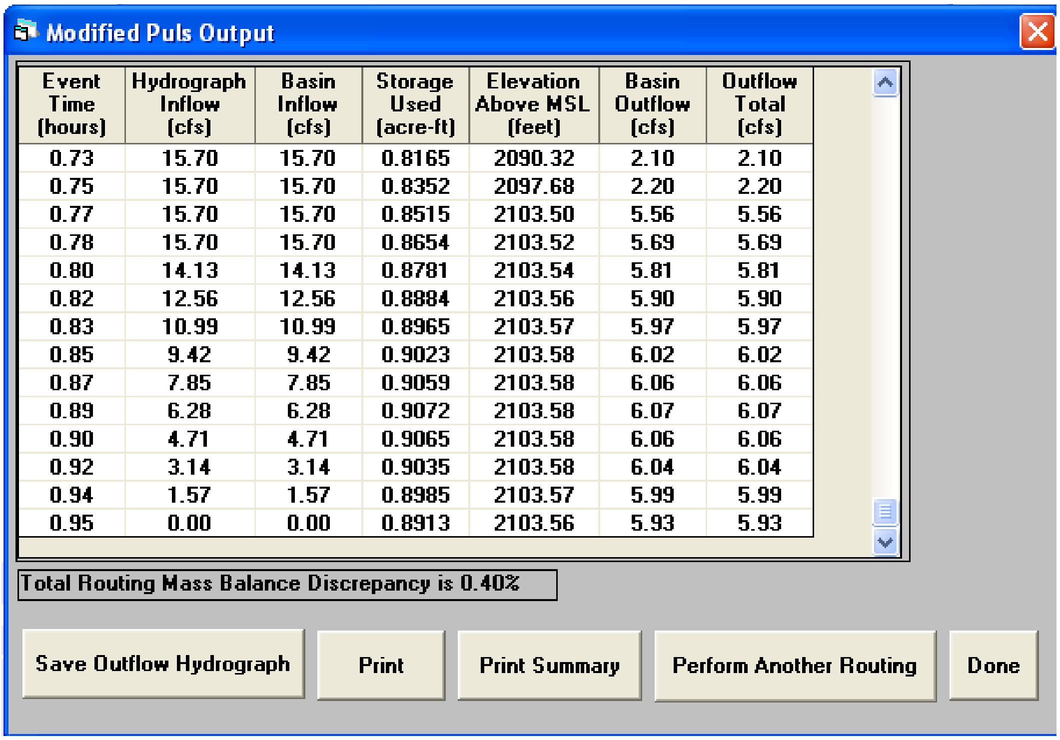
Stage 1: Circular Orifice
Invert = 2102.2
Discharge Coefficient = 0.6
Diameter = 18 in

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

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

Table 2.5. Preliminary Stage – Discharge Relationship

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



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

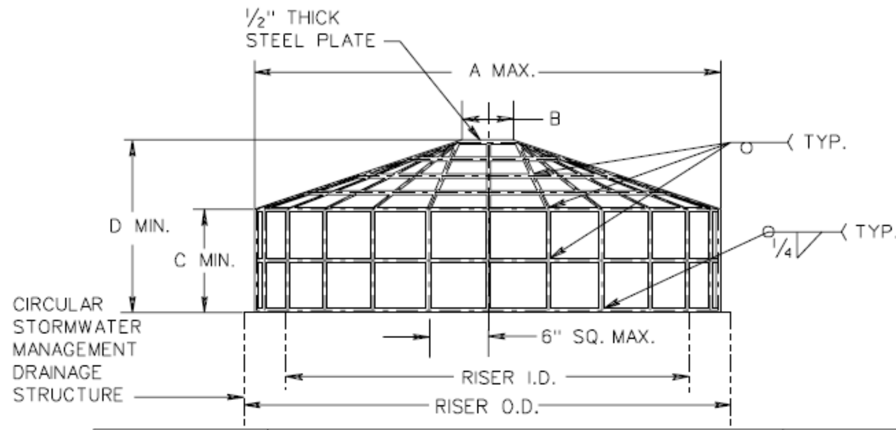
Figure 2.12. Preliminary Routing Results – 2-Year Inflow Hydrograph

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

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

Step 6C. Design the 10-Year Control Outlet

As with the 2-year control outlet, the designer has a multitude of options for the control of larger runoff producing events. These options range from circular riser tops equipped with a "bird cage" trash rack to various types of grated inlet tops. Regardless of the type of riser top selected, the effective weir length and total flow area of the configuration must be known in order to design and model the structure. This design example will employ a "bird cage" trash rack top consistent with the SWM-DR, 114.07 structure detailed in the Virginia Department of Transportation Road and Bridge Standards, (VDOT, 2008). A detail of this type of inlet top is shown in Figure 2.13.



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

Figure 2.13. VDOT SWM-DR Inlet Top (Metal)
VDOT Road and Bridge Standards (2008)

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

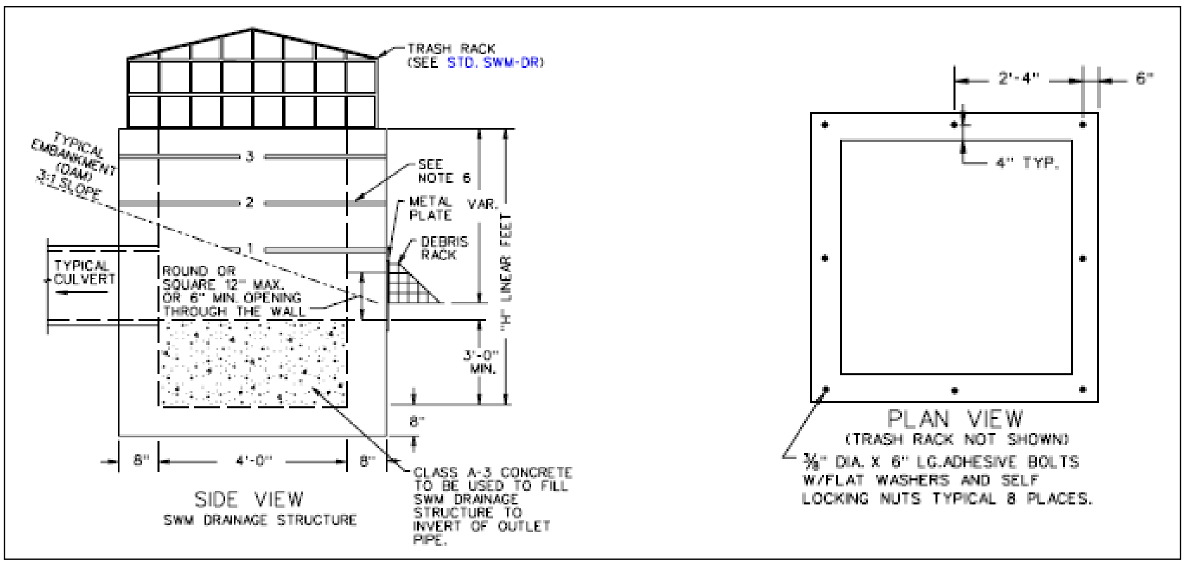


Figure 2.14. VDOT SWM-1 Riser
VDOT Road and Bridge Standards

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Table 2.6. Final Stage – Discharge Relationship

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

Modified Puls Output

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

Total Routing Mass Balance Discrepancy is 0.45%

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

Figure 2.15. Routing Results – 10-Year Inflow Hydrograph

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

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

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

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

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

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

$$Q = CPH^{1.5}$$

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

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

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

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

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

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

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

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

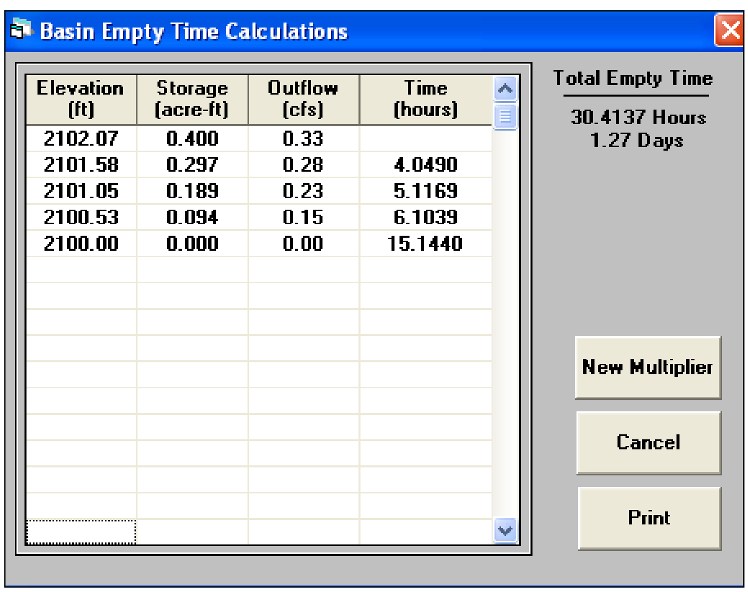
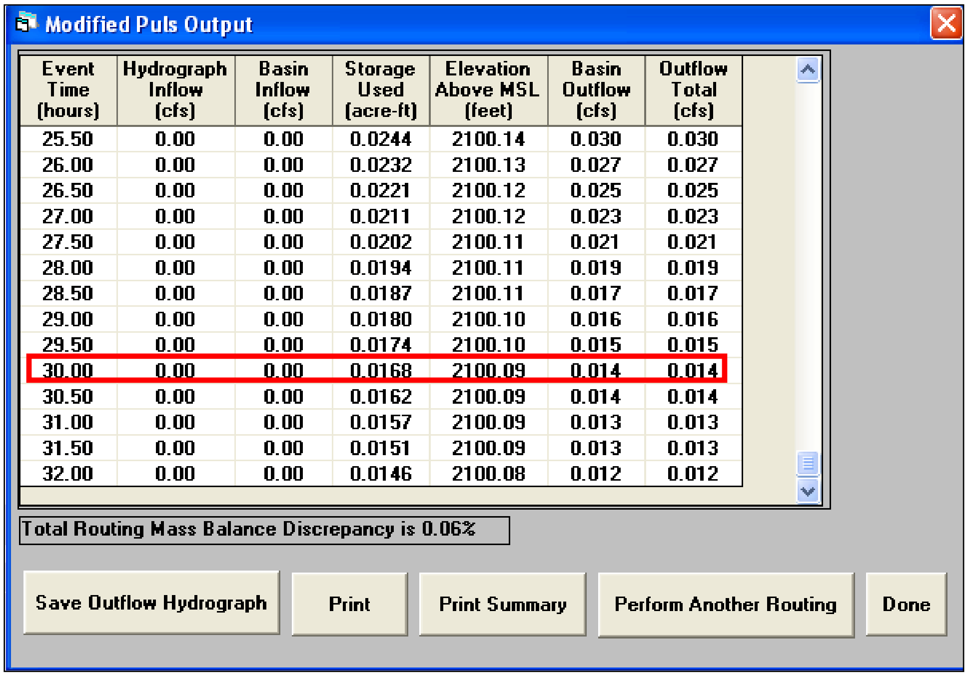


Figure 2.16. Water Quality Draw Down Calculator

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



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

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

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

Step 7. Design of the Emergency Spillway

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

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

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

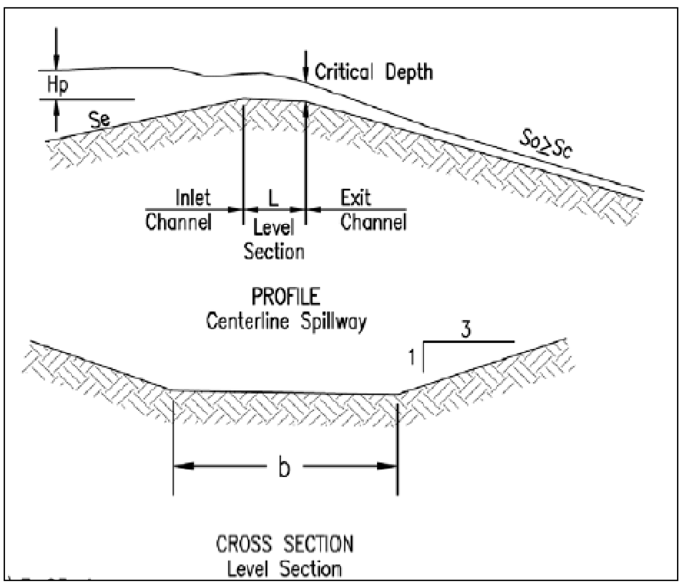


Figure 2.18. Profile and Cross Section of Typical Vegetated Emergency Spillway
(Virginia Stormwater Management Handbook, 1999, Et seq.)

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

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

Table 2.7. 100-Year Post-Development Runoff Parameters

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

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

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

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

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

Figure 2.19. Design Data for Earth Spillways
(Virginia Stormwater Management Handbook, 1999, Et seq.)

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

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

Table 2.8. Armored Emergency Spillway Parameters (1.4 ft. of Head Acting on Crest)

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

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

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

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

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

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

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

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

Table 2.9. Exit Channel Permissible Velocities
(Virginia Stormwater Management Handbook, 1999, Et seq.)

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

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

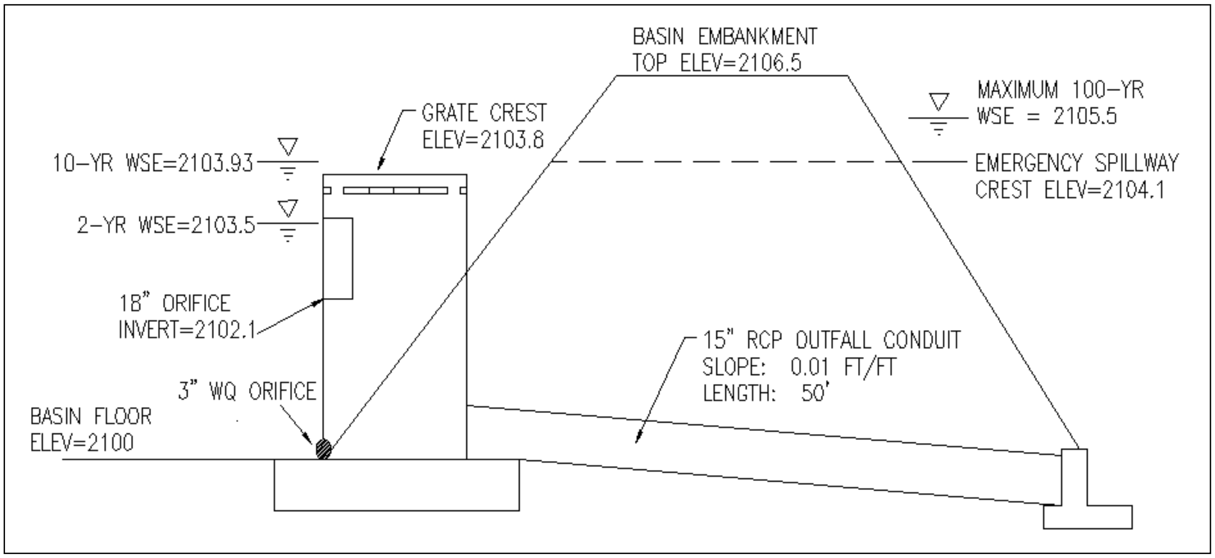


Figure 2.20. Schematic Illustration of Principal and Emergency Spillway Configuration and Resulting Water Surface Elevations

Step 8. Provision for Seepage Control

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

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

As an alternative to anti-seep collars, a variety of anti-seepage controls have been developed for *major* impoundments. By their nature, linear highway projects typically do not require large impoundment facilities for control of runoff. Therefore, per Instructional and Informational Memorandum IIM-LD-195, *"Management of Stormwater,"*, concrete cradles are recommended for seepage control on VDOT stormwater management basins. These cradles are to extend the *entire length of all outfall conduits penetrating earthen embankments*.

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

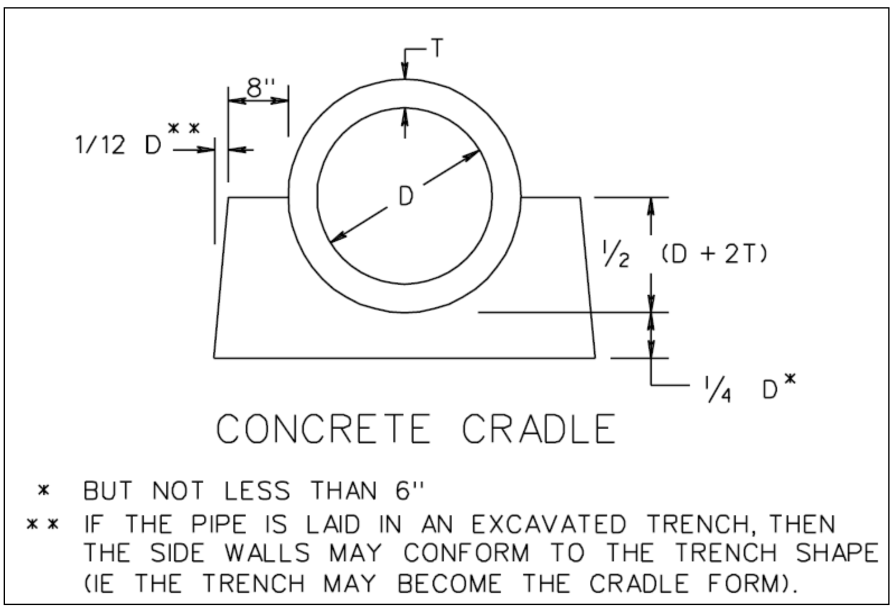


Figure 2.21. Typical Concrete Cradle for Minimization of Piping Along Outfall Conduits (VDOT Drainage Manual, 2002)

Step 9. Embankment Design

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

Early in the design stages of a project for which a detention basin is proposed, foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam. Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management,"* Section 12.1.1:

"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 feet or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."

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

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

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

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

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

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

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

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

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

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

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

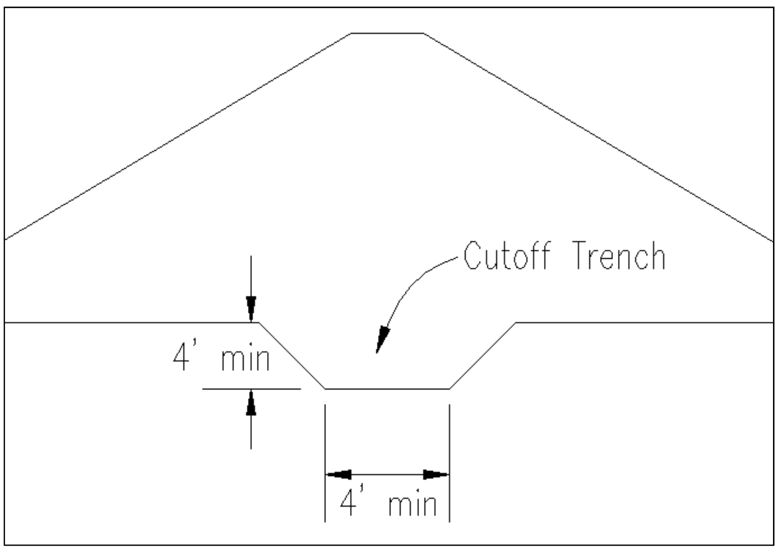


Figure 2.22 Typical Cutoff Trench Configuration

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

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

The designer is referenced to section 11.3.6 of the *VDOT Drainage Manual* for additional embankment details and specifications.

Step 10. Buoyancy Calculation

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

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

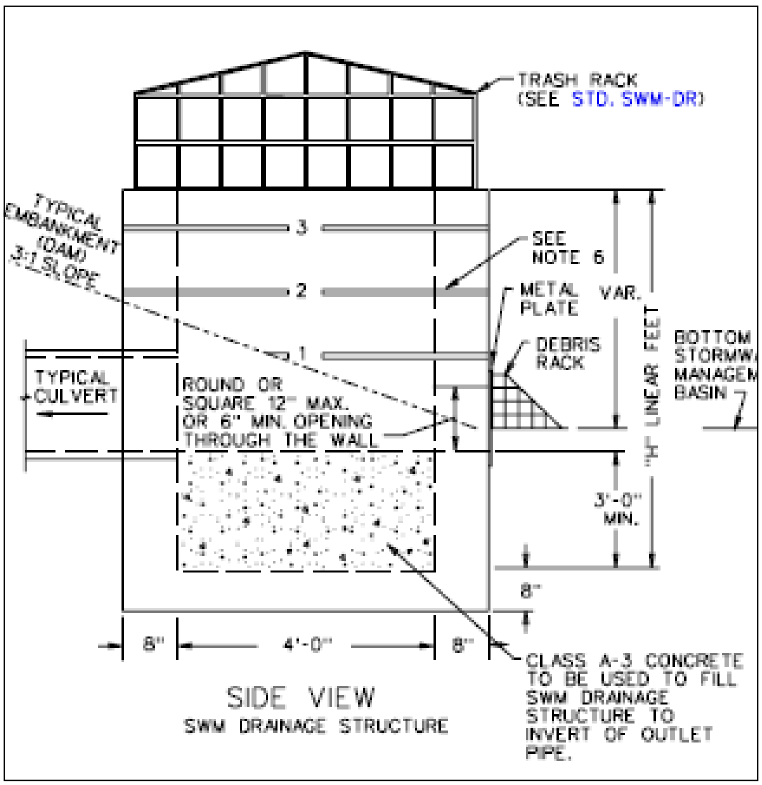


Figure 2.23. VDOT SWM-1 Side View
VDOT Road and Bridge Standards

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

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

Therefore, the volume of water displaced is computed as:

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

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

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

Applying the 1.25 factor of safety:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

The downward force of the riser weight is computed as:

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

Step 11. Design of Sediment Forebays

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

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

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

The storage volume in the sediment forebay is provided by separating the forebay from the rest of the basin. This separation is accomplished by means of an earthen berm, gabion baskets, concrete, or riprap. In a dry facility, the forebay outlet crest should be set at the elevation corresponding to the basin's water quality extended detention pool.

Depending on the type of material employed to construct the forebay embankment, the flows captured in the forebay may be detained over very long periods, with losses occurring only by means of infiltration and evaporation. Because the volume may be inundated at the onset of a runoff producing event, in a dry extended detention basin the forebay volume should not be considered part of the extended detention water quality volume.

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

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

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

Table 2.10. Summary of Pond Inflow Points

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

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

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

The sum of the forebay storage volumes:

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

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

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

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

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

The geometric layout of the forebay is dictated by site constraints and the designer's preference. The required forebay volume for entrance point 1 was found to be 817 ft^3.

Figures 2.24 and 2.25 illustrate the respective plan and cross-sectional view of a forebay providing this volume.

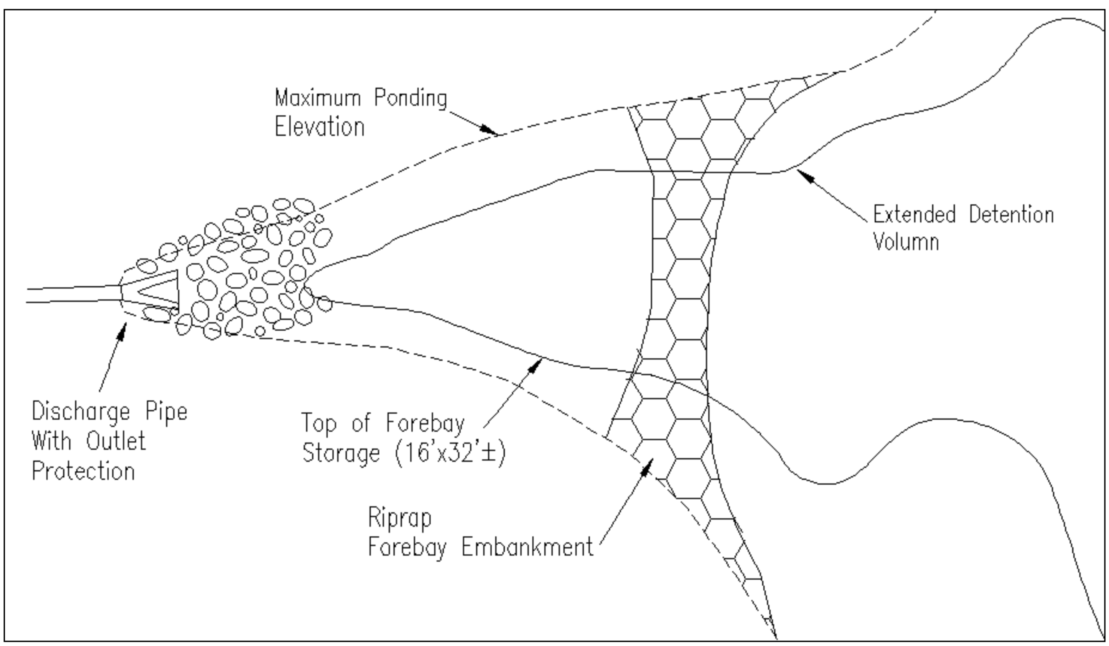


Figure 2.24. Plan View Sediment Forebay 1
(No Scale)

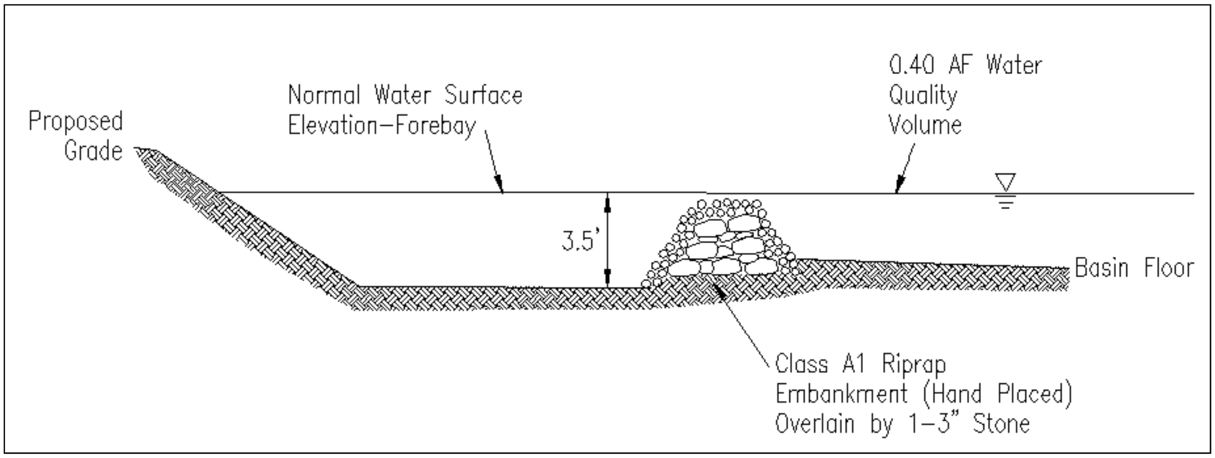


Figure 2.25. Cross-Section View Sediment Forebay 1
(No Scale)

Step 12. Landscaping

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

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

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

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

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

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

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

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

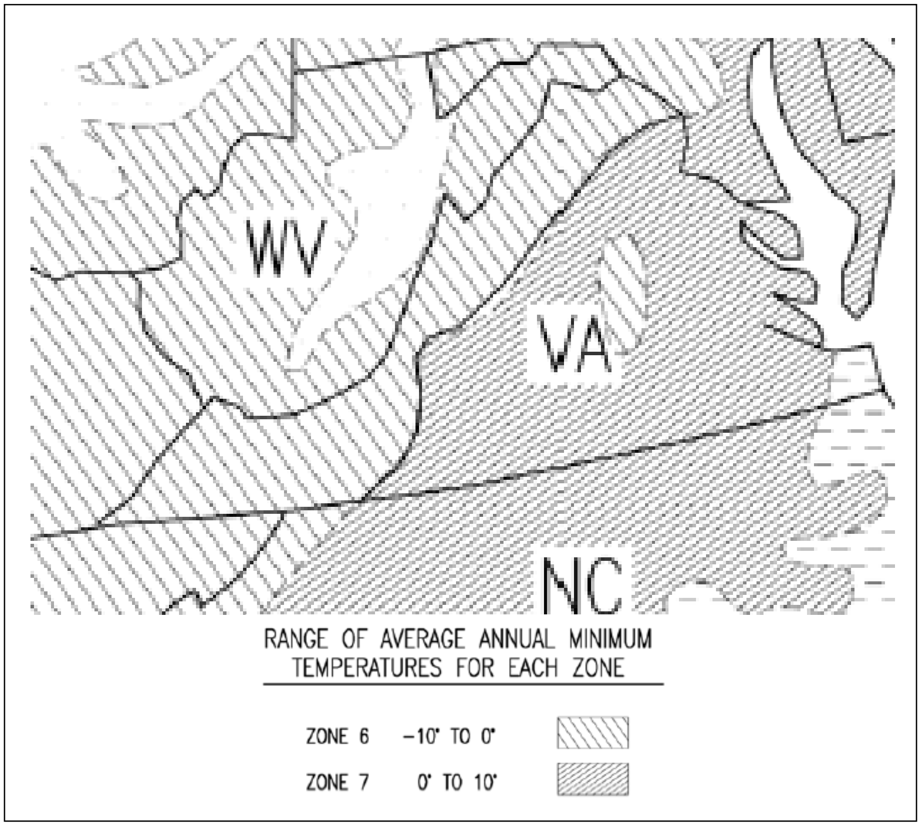


Figure 2.26. USDA Plant Hardiness Zones

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

Chapter 3 – Enhanced Dry Extended Detention Basin

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

3.1 Overview of Practice

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

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

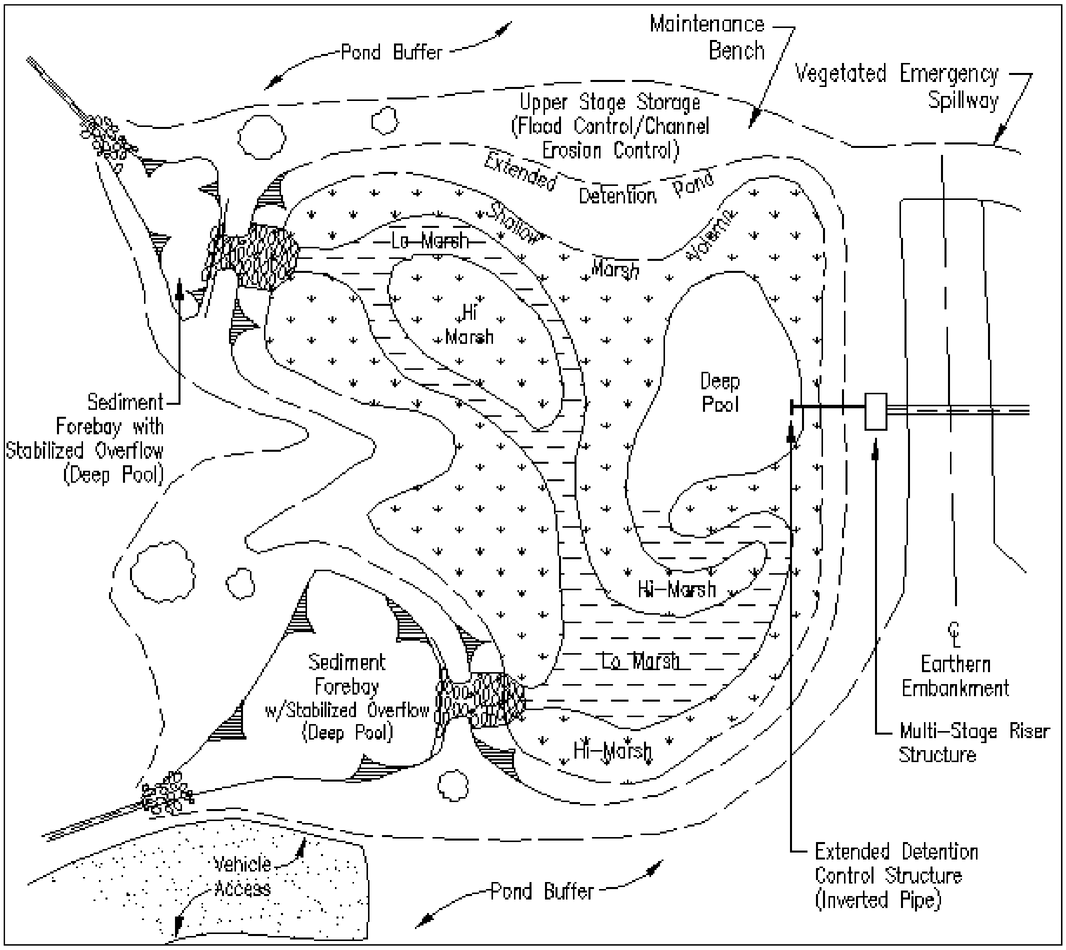


Figure 3.1. Schematic Dry Extended Detention Basin – Enhanced Plan View
(Virginia Stormwater Management Handbook, 1999, Et seq.)

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

3.2 Site Constraints and Siting of the Facility

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

3.2.1 Minimum Drainage Area

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

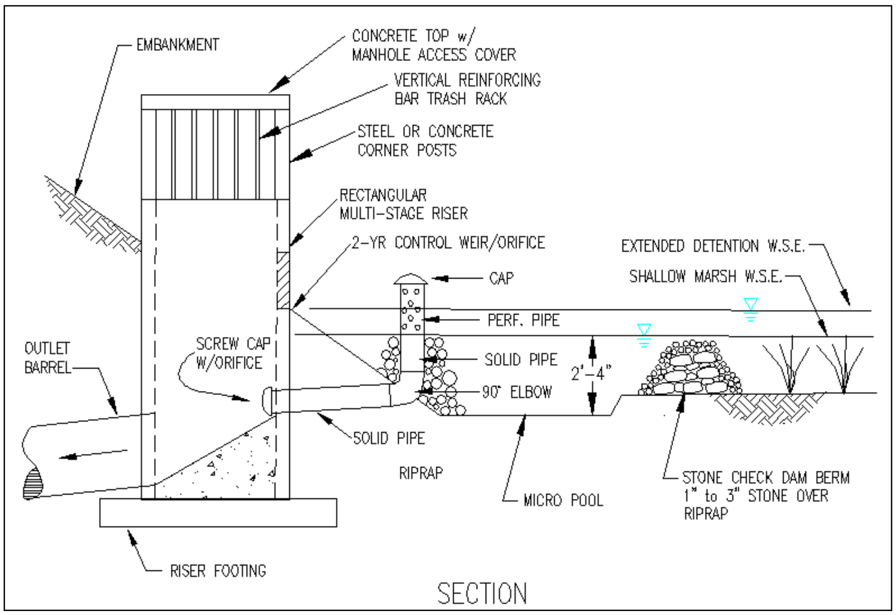


Figure 3.2. DCR Recommended Outlet Configuration 1 for the Control of Trash, Sediment and Debris (Virginia Stormwater Management Handbook, 1999, Et seq.)

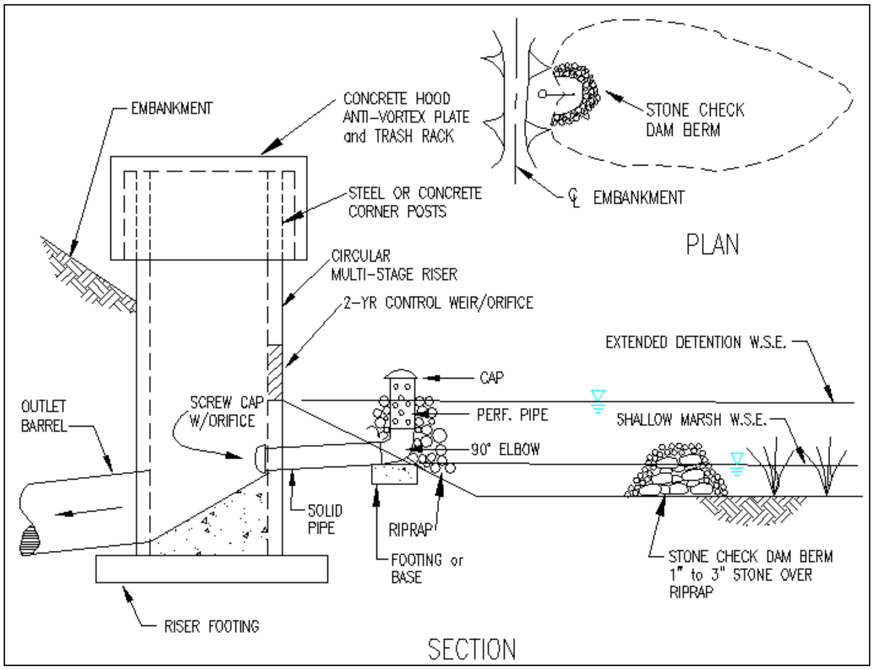


Figure 3.3. DCR Recommended Outlet Configuration 2 for the Control of Trash, Sediment and Debris (Virginia Stormwater Management Handbook, 1999, Et seq.)

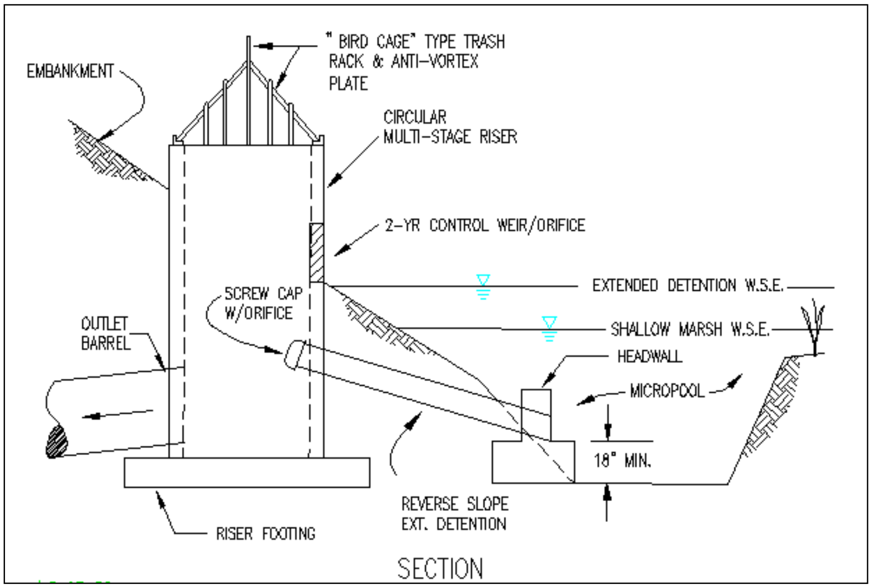


Figure 3.4. DCR Recommended Outlet Configuration 3 for the Control of Trash, Sediment and Debris (Virginia Stormwater Management Handbook, 1999, Et seq.)

3.2.2 Maximum Drainage Area

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

3.2.3 Separation Distances

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

3.2.4 Site Slopes

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

3.2.5 Site Soils

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

3.2.6 Rock

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

3.2.7 Existing Utilities

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

3.2.8 Karst

The presence of Karst topography places even greater importance on the subsurface investigation. Implementation of extended detention facilities in Karst regions may greatly impact the design and cost of the facility, and must be evaluated early in the planning phases of a project. Construction of stormwater management facilities within a sinkhole is prohibited. When the construction of such facilities is planned along the periphery of a sinkhole, the facility design must comply with the guidelines found in Instructional and Informational Memorandum IIM-LD-228 on *"Sinkholes"* and DCR's Technical Bulletin #2 *"Hydrologic Modeling and Design in Karst"* at:
http://dcr.cache.vi.virginia.gov/stormwater_management/documents/tecbltn2.PDF .

3.2.9 Existing Wetlands

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

3.2.10 Upstream Sediment Considerations

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

3.2.11 Floodplains

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

3.2.12 Basin Location

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

3.2.13 Hydrology

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

3.3 General Design Guidelines

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

3.3.1 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam. Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management,"* Section 12.1.1:

> *"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 feet or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."*

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

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

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

3.3.2 Outfall Piping

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

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

3.3.3 Embankment

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

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

The designer is referenced to section 11.3.6 of the VDOT Drainage Manual for additional embankment details and specifications.

3.3.4 Embankment Height

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

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

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

3.3.5 Prevention of Short-Circuiting

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

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

It is preferable to construct the basin such that the length to width ratio is 3:1 or greater, with the widest point observed at the outlet end. If this is not possible, every effort should be made to design the basin with no less than a 2:1 length to width ratio. When this minimum ratio is not possible, consideration should be given to pervious baffles.

3.3.6 Ponded Depth

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

3.3.7 Principal Spillway Design

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

3.3.8 Fencing

Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management,"*, Section 13.1.1, fencing is typically *not required or recommended* on most VDOT detention facilities. However, exceptions do arise, and the fencing of a dry extended detention facility may be needed. Such situations include:

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

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

3.3.9 Sediment Forebays

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

3.3.10 Discharge Flows

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

3.4 Design Process

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

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

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

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

Table 3.1. Hydrologic Characteristics of Example Project Site

Step 1. Compute the Required Water Quality Volume

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

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

IA= Impervious Area (square feet)

An enhanced dry detention basin must be sized to provide an extended detention volume of no less than *twice* the computed water quality volume. This volume should be distributed equally between the permanent marsh area and a separate extended detention volume.

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

Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management,"*, Section 5.4.6, when the 1-year return frequency storm is detained for a minimum of 24 hours there is no need to provide additional or separate storage for the WQ_V provided it can be demonstrated that the WQ_V will be detained for approximately 24 hours. It is noted that providing extended 24+ hour detention for the 1-year runoff volume may require the basin size to be 1.5 to 2 times the volume required to simply reduce the 2 and 10-year runoff events to pre-development levels.

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

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

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

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

Step 2. Sizing the Marsh Area Zones

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

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

- sediment forebays
- micro pools
- deep water channels

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

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

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

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

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

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

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

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

Table 3.2. Marsh Zone Surface Area Allocation

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

Step 2B. Compute the Minimum Marsh Surface Area

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

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

Step 2C. Size the Deep Pool Zone

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Table 3.3. Deep Pool Volume and Surface Area Allocation

Step 2D. Size the Low Marsh Area

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

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

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

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

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

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

Step 2E. Size the High Marsh Area

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

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

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

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

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

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

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

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

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

* Includes sediment forebays and micro-pool

Table 3.4. Marsh Surface Area and Volume Verification

Step 3. Construction of Storage Versus Elevation Data

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

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

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

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

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

Table 3.5. Stage – Storage Relationship

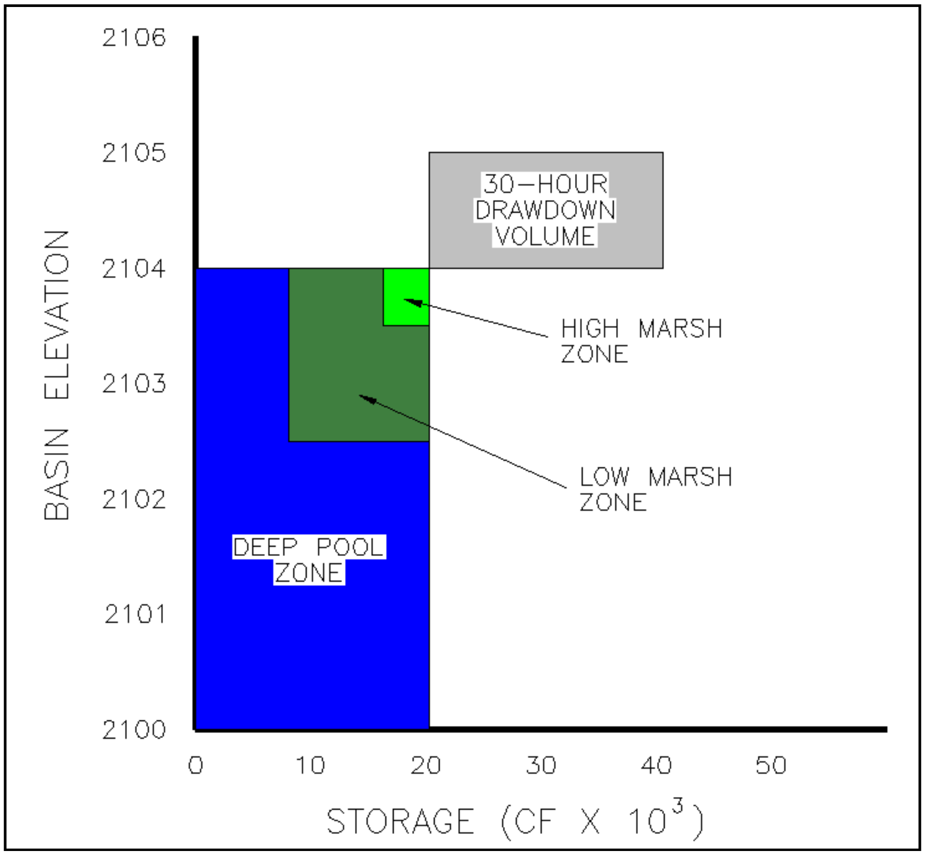


Figure 3.5. Graphical Elevation – Storage Relationship

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

Step 4. Water Balance Calculation

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

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

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

Table 3.6. Potential Evaporation Rates (Inches)
Virginia Stormwater Management Handbook, (DCR, 1999, ET SEQ.)

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Total contributing drainage area = 24.8 acres

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

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

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

Step 5. Landscaping

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

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

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

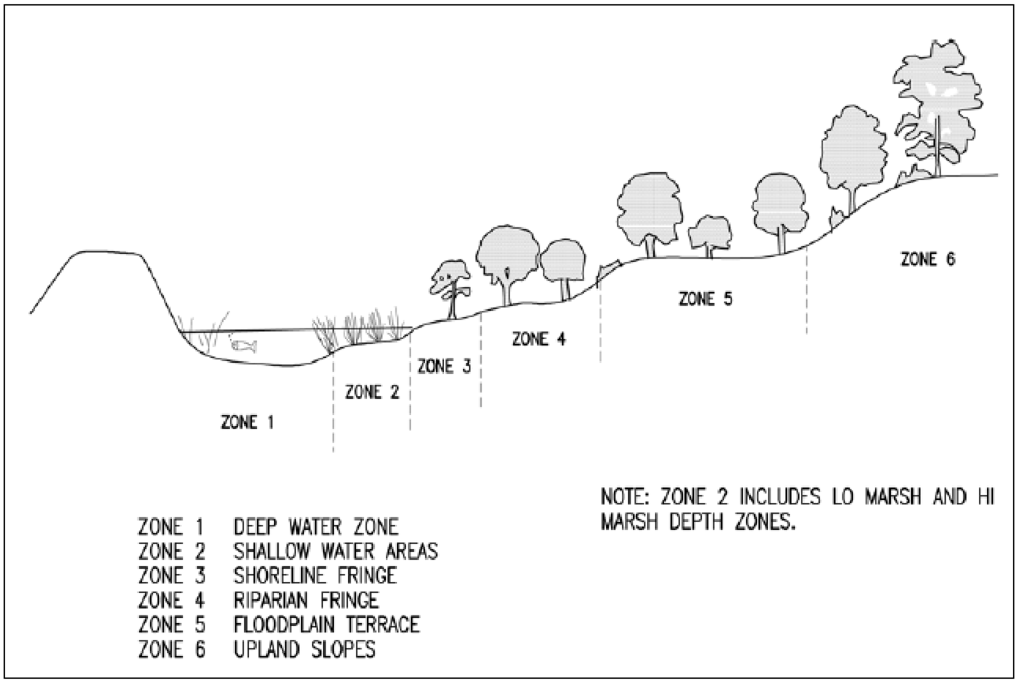


Figure 3.6. Planting Zones for Stormwater BMPs
Virginia Stormwater Management Handbook (DCR, 1999, ET SEQ.)

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

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

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

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

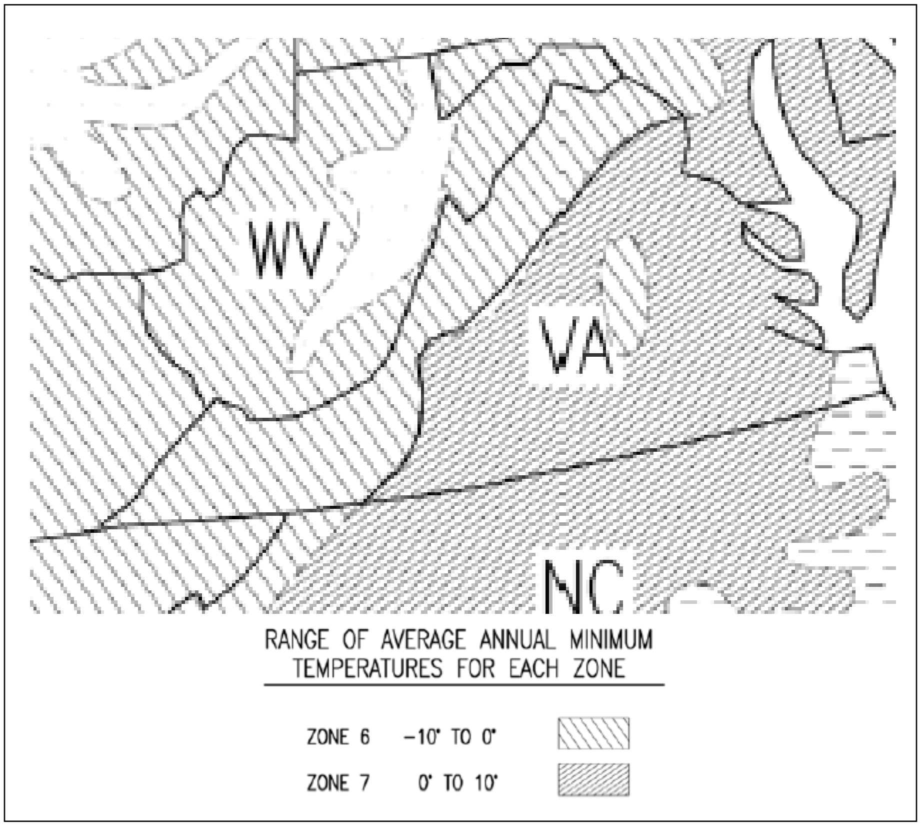


Figure 3.7. USDA Plant Hardiness Zones

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

Chapter 4 – Retention Basin

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

4.1 Overview of Practice

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

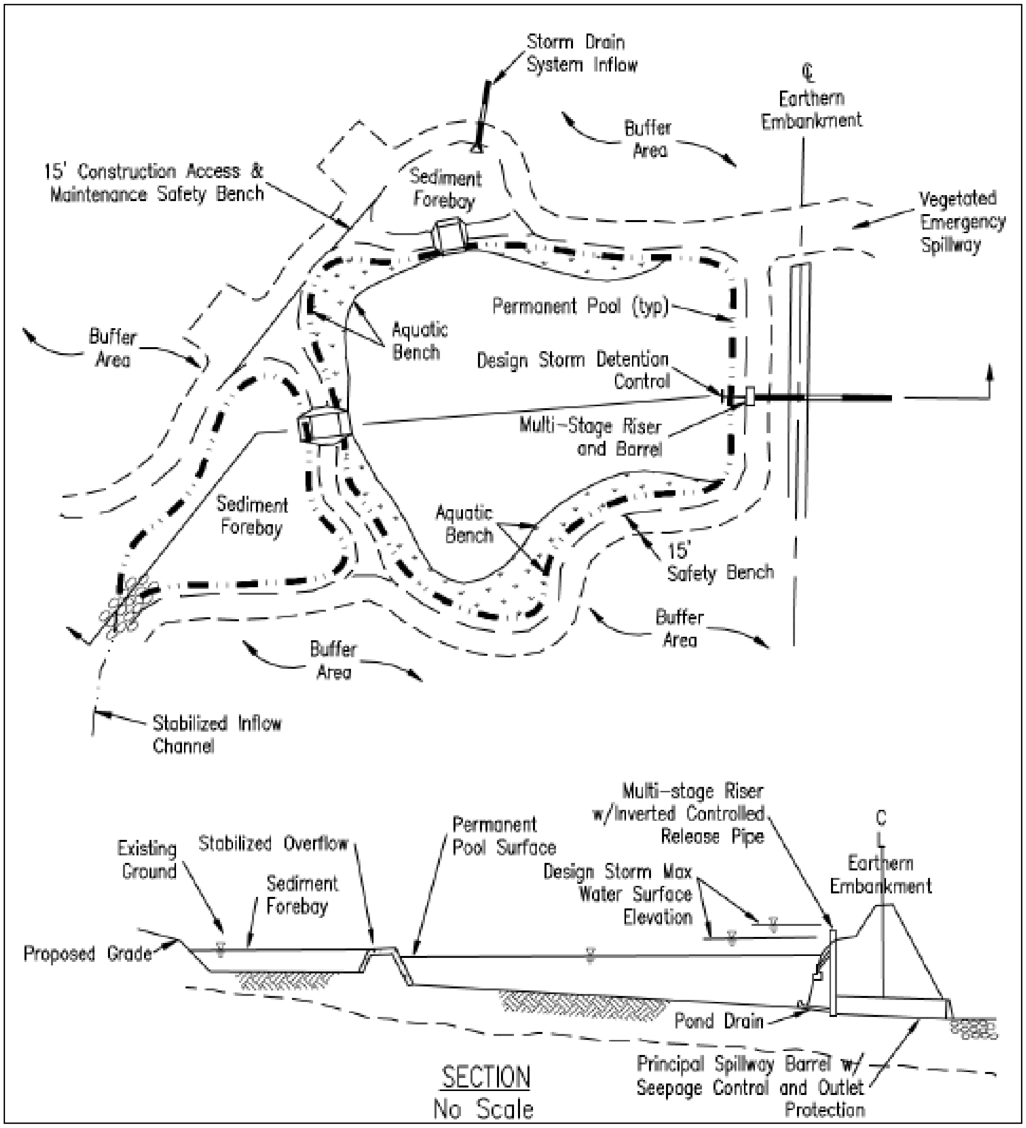


Figure 4.1. Schematic Retention Basin Plan and Sectional View
(Virginia Stormwater Management Handbook, 1999, Et seq.)

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

4.2 Site Constraints and Siting of the Facility

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

4.2.1 Minimum Drainage Area

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

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

4.2.2 Maximum Drainage Area

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

4.2.3 Separation Distances

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

4.2.4 Site Slopes

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

4.2.5 Site Soils

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

4.2.6 Rock

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

4.2.7 Existing Utilities

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

4.2.8 Karst

The presence of karst topography places even greater importance on the initial subsurface investigation. Implementation of retention basins in karst regions may greatly increase the design and construction cost of the facility, and must be evaluated early in the planning phases of a project. ***Construction of stormwater management facilities within a sinkhole is prohibited.*** When the construction of such a facility is planned along the periphery of a sinkhole, the facility design must comply with the guidelines found in Instructional and Informational Memorandum IIM-LD-228 on *"Sinkholes"* and DCR's Technical Bulletin #2 *"Hydrologic Modeling and Design in Karst* at " http://dcr.cache.vi.virginia.gov/stormwater_management/documents/tecbltn2.PDF .

4.2.9 Wetlands

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

4.2.10 Upstream Sediment Considerations

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

4.2.11 Downstream Considerations

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

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

4.2.12 Floodplains

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

4.2.13 Basin Location

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

4.2.14 Implementation as a Regional Stormwater Management Facility

The costs associated with constructing and maintaining a retention basin are often prohibitive; however, as the area contributing runoff to a retention basin increases, the total cost per acre decreases. Therefore, when a retention basin is chosen as the stormwater BMP it should, when possible, be implemented as part of a regional approach to stormwater management. The concept of regional stormwater management is endorsed by VDOT provided the following requirements are met per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management,"*, Section 7.0:

- Development and use of regional stormwater management facilities must be a joint undertaking by VDOT and the local governing body. The site must be part of a master stormwater management plan developed and/or approved by the local governing body and any agreements related to these facilities must be consummated between VDOT and the local governing body. VDOT may enter into an agreement with a private individual or corporation provided the local governing body has a SWM program that complies with the Virginia SWM regulations and the proper agreements for maintenance and liability of the regional facility have been executed between the local governing body and the private individual or corporation.

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

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

4.3 General Design Guidelines

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

4.3.1 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam. Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management,"*, Section 12.1.1:

> *"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 feet or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."*

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

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

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

4.3.2 Outfall Piping

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

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

4.3.3 Embankment

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

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

The designer is referenced to section 11.3.6 of the *VDOT Drainage Manual* for additional embankment details and specifications.

4.3.4 Embankment Height

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

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

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

4.3.5 Permanent Pool Volume

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

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

Table 4.1. Retention Basin Removal Efficiencies
(Virginia Stormwater Management Handbook, 1999, Et seq.)

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

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

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

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

4.3.6 Prevention of Short-Circuiting (Basin Geometry)

Short-circuiting occurs when flows entering the basin pass rapidly through the basin without displacing an equal volume of previously stored water. Short-circuiting of flow can greatly reduce the hydraulic residence time within the basin, thus negatively impacting the water quality benefit. While site conditions will ultimately dictate the geometric configuration of the basin, it is preferable to construct the basin such that the length-to-width ratio is 3:1 or greater, with the widest point observed at the outlet end. If this is not possible, every effort should be made to design the basin with no less than a 2:1 length-to-width ratio. When this minimum ratio is not possible, consideration should be given to baffles constructed of gabions, earthen berms, or other permeable materials.

In addition to increasing the basin length-to-width ratio, the likelihood of short-circuiting can be further reduced by designing meandering flow paths rather than straight line paths from stormwater entrance points to the basin principal spillway.

4.3.7 Ponded Depth

The depth of the basin permanent pool affects the planting species selected for the basin as well as the types of aquatic and wildlife species that will inhabit the basin and its surrounding areas. Additionally, the depth of the permanent pool has a significant impact on pollutant removal performance of the basin. Basins sized too shallow will not support a diverse population of aquatic species, while basins whose permanent pool is excessively deep will tend to stratify. This stratification can potentially create anaerobic conditions leading to the resuspension / resolubilization of captured pollutants. (DCR, 1999, Et seq.). The majority of the permanent pool volume should range in depth from 2 to 6 feet. Approximately 15 percent of the permanent pool volume should be comprised of regions less than 18 inches in depth. These regions are easily obtained with the inclusion of an *aquatic bench.* An aquatic bench provides not only improved pollutant removal efficiency in the basin, but also serves as an important safety feature (discussed later). Table 4.2 presents recommended surface area – pool depth relationships.

Pool Depth (ft)	**Surface Area (% of Total Surface Area)**
0 - 1.5	15%
1.5 - 2	15%
2 - 6	70%

Table 4.2. Surface Area – Permanent Pool Depth Relationships
(Virginia Stormwater Management Handbook, 1999, Et seq.)

4.3.8 Aquatic Bench

An aquatic bench is a 10 to 15 foot wide area that slopes from a depth of zero inches at the shoreline of the basin to a depth of approximately 18 inches in the basin permanent pool. The shallow depth of the aquatic bench supports a diverse mix of emergent and wetland plant species as well as providing ideal habitat to predatory insects that feed on mosquitoes and other nuisance insects. Table 4.1 shows a target phosphorus removal efficiency of 65 percent for a basin equipped with an aquatic bench, compared to 50 percent for a basin with an equal pool volume, but no bench. The ability of an aquatic bench to support a dense and diverse mix of vegetation will also make the shoreline of the basin less susceptible to the erosive action associated with fluctuating water levels. Figure 4.2 illustrates the general configuration of an aquatic bench.

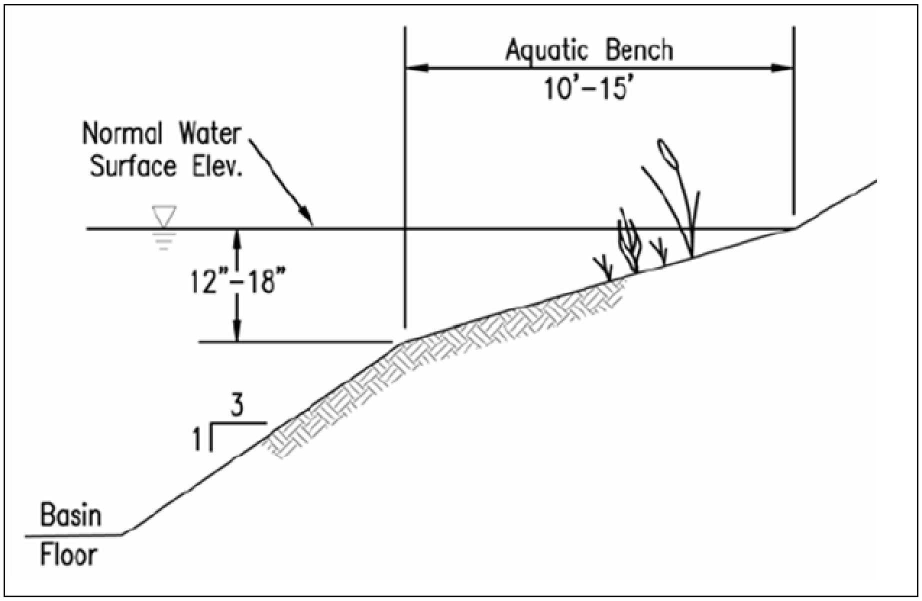


Figure 4.2. Schematic Aquatic Bench Section
(Virginia Stormwater Management Handbook, 1999, Et seq.)

The inclusion of an aquatic bench adds a significant safety feature to the basin, as it provides spatial disconnection from the basin's peripheral slope and its submerged slope. Whenever the total surface area of the basin permanent pool exceeds 20,000 ft^2 an aquatic bench should be considered an essential safety feature.

4.3.9 Principal Spillway Design

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

4.3.10 Fencing

Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management,"*, Section 13.1.1, fencing is typically *not required or recommended* on most VDOT detention facilities. However, exceptions do arise, and the fencing of a dry extended detention facility may be needed. Such situations include:

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

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

4.3.11 Signage

"No Trespassing" signs should be considered for inclusion on all stormwater impoundment facilities, whether fenced or unfenced. Additionally, retention basins should be identified as potentially exhibiting the following hazards:

- Deep water
- Waterborne disease
- Vortex conditions (if applicable)

Signs should be easily viewed from all streets, sidewalks, and paths adjacent to the basin.

4.3.12 Sediment Forebays

Each basin inflow point should be equipped with a sediment forebay. The forebay volume should range between 0.1 and 0.25" over the individual outfall's impervious area or 10 percent of the required WQ_V.

4.3.13 Discharge Flows

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

4.4 Design Process

Many of the design elements in a retention basin are identical to those of a dry extended detention basin. These elements include estimation of flood control storage volumes, design of a multi-stage riser, storage indication (reservoir) routing, emergency spillway design, riser buoyancy calculations, and the design of sediment forebays. For those design items, the reader is referred to *Chapter 2 – Dry Extended Detention Basin*.

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

The following example basin design is founded on the development scenario described in *Chapter 3 – Dry Extended Detention Basin Enhanced*. This example project entailed the construction of a small interchange and new section of two lane divided highway in Staunton. The total project site, including right-of-way and all permanent easements, consists of 24.8 acres. Pre and post-development hydrologic characteristics are summarized below in Table 4.3. Initial geotechnical investigations reveal a soil infiltration rate of 0.01 inches per hour with site soils classified as Hydrologic Soil Group C.

	Pre-Development	Post-Development
Project Area (acres)	24.80	24.80
Land Cover	Unimproved Grass Cover	11.28 acres impervious cover
Impervious Percentage	0	45

Table 4.3. Hydrologic Characteristics of Example Project Site

Step 1. Determine Permanent Pool Volume of the Basin as a Function of the Project Site Water Quality Volume

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

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

IA= Impervious Area (square feet)

For a retention basin serving a contributing drainage area comprised of 45 percent impervious cover, the permanent pool volume should be a minimum of four times the computed water quality volume (reference Table 4.1).

The demonstration project site is comprised of a total drainage area of 24.80 acres. The total impervious area within the project site is 11.28 acres. Therefore, the water quality volume is computed as follows:

$$WQV = \frac{11.28ac \times 43{,}560\frac{ft^2}{ac} \times \frac{1}{2}in}{12\frac{in}{ft}} = 20{,}473.2\,ft^3$$

The basin permanent pool volume is computed as:

$$4 \times 20{,}473.2\,ft^3 = 81{,}893\,ft^3$$

Step 2. Allocate the Computed Permanent Pool Volume into Regions of Varying Depth

The greatest pollutant removal efficiency of a retention basin is achieved when the surface area of the permanent pool is allocated to the regions of varying depth as shown in Table 4.2; however, initially, the total surface area of the basin permanent pool is unknown. The following steps illustrate the design process for sizing each of the three depth zones.

Approximately 15 percent of the total surface area of the permanent pool should be dedicated to depths ranging between zero and 18 inches. This depth zone may include or be comprised entirely of the aquatic bench, if one is proposed. Depths ranging between 18 and 24 inches should comprise an additional 15 percent of the total basin surface area. The remaining 70 percent of the basin surface area should be made up of deep water ranging in depth from 2 to 6 feet.

The total surface area of the basin is designated as *A*. Following this convention, the surface area of each depth zone can be expressed as follows:

$$A_1 = 0.15A$$
$$A_2 = 0.15A$$
$$A_3 = 0.70A$$

The average depth of zone A_1 ranges between zero and 18 inches. The 9 inch average depth can be employed as the zone's effective depth for purposes of volume calculations. Therefore, the total volume encompassed by the basin's shallowest pool zone is approximated as follows:

$$V_1 = 9in \times \frac{1ft}{12in} \times A_1 = (0.75\,ft)(0.15)(A)$$

Similarly, the effective depth of zone A_2 is computed as:

$$D_{e_2} = \frac{18in + 24in}{2} = 21in$$

The total volume encompassed by the basin's intermediate depth zone is approximated as follows:

$$V_2 = 21in \times \frac{1ft}{12in} \times A_2 = (1.75\,ft)(0.15)(A)$$

The deep water regions of the basin range in depth from 2 to 6 feet. Therefore the effective depth of zone A_3 is 4 feet and the volume is expressed as:

$$V_3 = 4\,ft \times A_3 = (4\,ft)(0.70)(A)$$

The sum of all incremental pool volumes must equal or exceed the previously established permanent pool volume of 4xWQV. Therefore, the basin surface area, *A*, is approximated as follows:

$$V = 81{,}893\,ft^3$$
$$V = (0.75\,ft)(0.15)(A) + (1.75\,ft)(0.15)(A) + (4\,ft)(0.70)(A)$$

Rearranging and solving for surface area, *A*:

$$3.175A = 81{,}893\,ft^3$$
$$A = 25{,}793\,ft^2$$

Table 4.4 summarizes the minimum surface area and approximate volume of each depth zone.

Zone / Depth	Surface Area (ft^2)	Approximate Volume (ft^3)
Shallow (0 - 18")	3,869	2,902
Intermediate (18 - 24")	3,869	6,771
Deep (2 - 6')	18,055	72,220*
Total	***25,793***	***81,893***

*Includes sediment forebay volume(s)

Table 4.4. Summary of Varying Depth Zones

It is noted that the permanent pool surface area of 25,793 ft^2 exceeds 20,000 ft^2. Therefore, the inclusion of an aquatic bench is required for purposes of safety.

Step 3. Estimate Total Land Area of the Retention Basin

The total proposed surface area of the basin permanent pool is 25,793 ft^2. This represents 2.4 percent of the total basin drainage area of 24.8 acres. Typically, the total surface area of a retention basin permanent pool will range between one and three percent of the total drainage area (FHWA, 1996).

At this point, to determine basin feasibility, the designer must consider the land area required for construction of the basin. Factors to examine include land acquisition costs, availability of right-of-way, and site topography. In addition to the area required for the basin permanent pool, area must be provided for flood control storage, freeboard, and the required 20-foot vegetated buffer strip that must occupy the basin periphery.

Applying the Modified Rational method (presented in detail in *Chapter 2 – Dry Extended Detention Basin*) we estimate the volume required to provide peak runoff rate reduction for the 10-year return frequency storm:

Peak pre-development runoff, q_{10} = 23.8 cfs

Peak post-development runoff, Q_{10} = 43.2 cfs

Critical duration storm, T_d = 23.5 minutes

Estimated detention volume, V_{10} = 33,978 ft^3

In this example, we will consider a basin of rectangular orientation, with a 2.5:1 length-to-width ratio. The demonstrated methodology is applicable to basins of other geometries. However, the results are only estimates of the total land area required for the basin.

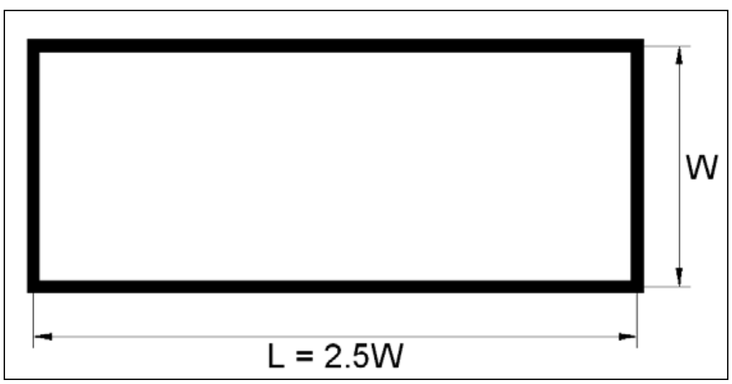


Figure 4.3. Schematic Basin Configuration

The dimensions of the basin permanent pool can then be approximated by solving the following expression:

$$W \times 2.5W = 25{,}793\,ft^2$$
$$W = 101.6\,ft$$
$$L = 254\,ft$$

The volume of flood control storage provided above the permanent pool can be approximated by the following equation:

$$V = \left(\frac{A_1 + A_2}{2}\right)d$$

V = volume of flood control storage (ft^3)
A_1 = surface area of permanent pool (25,793 ft^2)
A_2 = surface area above permanent pool dedicated to flood control storage
d = incremental depth between A_1 and A_2

Surface area, A_2, can be expressed as a function of depth, d:

$$A_2 = [101.6 + (2)(d)(Z)] \times [254 + (2)(d)(Z)]$$

Z = basin side slopes (ZH:1V)

In this example, we will consider that the basin side slopes are 3H:1V. The updated A_2 expression then becomes:

$$A_2 = [101.6 + (2)(d)(3)] \times [254 + (2)(d)(3)]$$

A total flood control volume of 33,978 ft^3 must be provided above the surface of the permanent pool. At this point, the designer can construct a plot of storage versus depth by employing the previously developed expression for volume, *V*. This plot is shown in Figure 4.4.

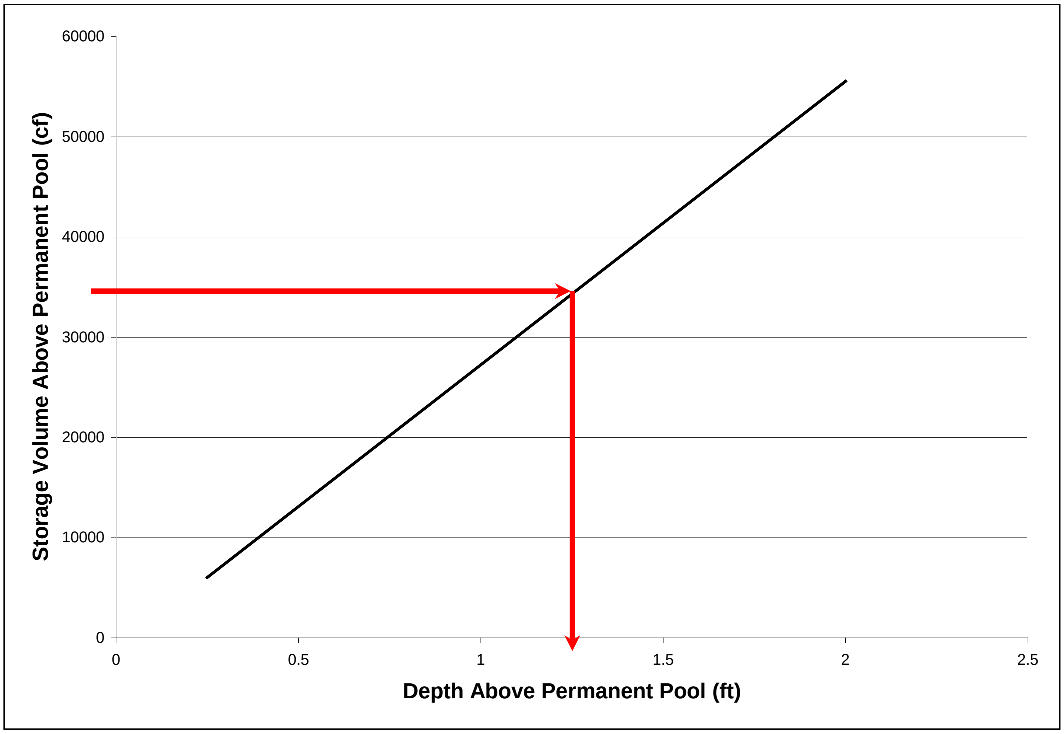


Figure 4.4. Plot of Storage Volume Versus Depth Above Permanent Pool

The plot indicates that the flood control storage is provided at an approximate depth of 1.25 feet above the permanent pool. This estimate can be verified as follows:

$$A_2 = [101.6 + (2)(1.25)(3)] \times [254 + (2)(1.25)(3)] = 28{,}530\ ft^2$$

The total storage volume provided above the permanent pool is then computed as:

$$V = \left(\frac{25{,}793 + 28{,}530}{2}\right)1.25 = 33{,}952\ ft^3$$

The volume is very close to the required storage volume of 33,978 ft^3, and is deemed adequate for the total basin land area estimate.

Maintaining the 2.5:1 length-to-width ratio, we now compute the surface area of the basin as:

$$W \times 2.5W = 28{,}530\ ft^2$$
$$W = 106.8\ ft$$
$$L = 267\ ft$$

Next, the required freeboard must be considered. The required freeboard depths under 100-year conditions are as follows (per DCR minimum standards):

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

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

We will assume that the basin is to be equipped with an emergency spillway and that approximately 0.5 feet of head is observed on the crest of the emergency spillway during conveyance of the 100-year event. At this point, these values are only estimates. The procedures detailed in *Chapter Two – Dry Extended Detention Basin* must be employed to determine the actual basin stage – storage relationship.

The freeboard depth (one foot) and the head on the emergency spillway (0.5 feet) increase the basin length and width as follows:

$$W = 106.8\,ft + (2)(3)(1.5\,ft) = 115.8\,ft$$
$$L = 267\,ft + (2)(3)(1.5\,ft) = 276\,ft$$

Finally, we must consider the required minimum 20-foot vegetated buffer located around the basin periphery. Adding this buffer width to the basin length and width results in the approximate basin surface dimensions shown in Table 4.5.

Length	156 ft	
Width	316 ft	
Area	49,296 ft^2	1.13 ac

Table 4.5. Basin Surface Dimensions

Step 4. Development of Stage – Storage Relationship

Having determined the required surface area and storage volume for the basin permanent pool, flood storage volume, and freeboard we move on to the next step of constructing a stage – storage relationship. Each site is unique, both in terms of constraints and required storage volume. Because of this, the development of a proposed basin grading plan may be an iterative process. The stage storage volume relationship for the example basin is shown in Figures 4.5 and 4.6. The basin floor is assumed to be at elevation 2000 MSL. Upon development of the basin stage – storage relationships, the next step(s) are to design and evaluate the basin for flood (peak rate) control. The reader is referred to *Chapter Two – Dry Extended Detention Basin*, Steps 6 – 8 for detailed methodology on these topics.

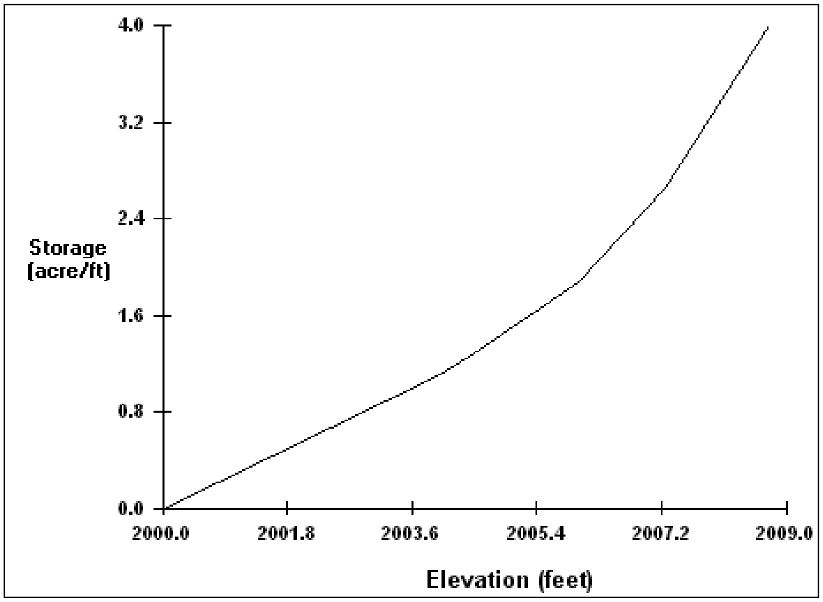


Figure 4.5. Retention Basin Stage – Storage Relationship

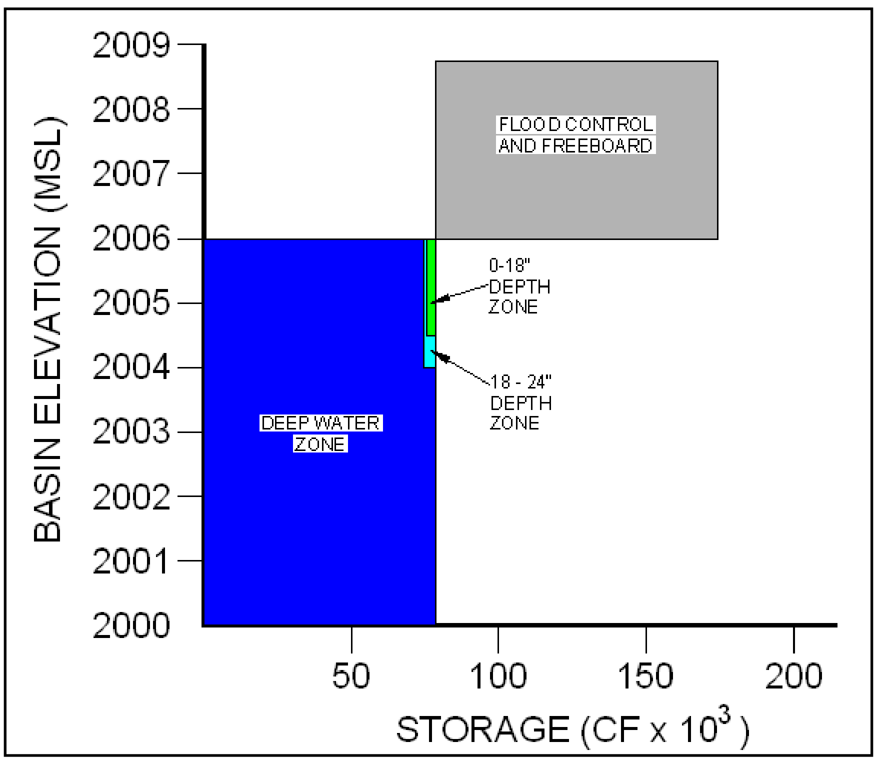


Figure 4.6. Graphical Depiction of Varying Depth Zones – Permanent Pool and Flood Control Storage

Step 5. Design of the Submerged Release Outlet

A retention basin must be equipped with a means by which baseflow can pass through the basin without accumulating and encroaching upon the volume of storage allocated to flood control. This conveyance is typically accomplished by a submerged, inverted pipe as shown in Figure 4.7.

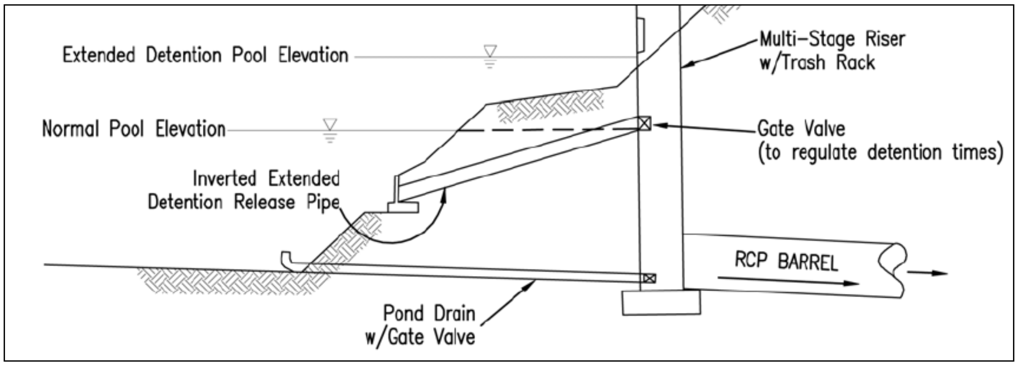


Figure 4.7. Schematic Retention Basin Outlet Configuration
Virginia Stormwater Management Handbook, 1999, Et seq.)

Generally, the highest quality of water in a retention basin is found at or near the surface of the permanent pool. In addition to the low levels of dissolved oxygen found near the basin floor, there are also potentially high levels of pollutants which have accumulated through gravitational settling. Though the pollutant levels near the pool surface tend to be lower than at points of greater depth in the water column, the water temperature tends to be higher. This elevated temperature arises from both solar heating and the influence of heated stormwater inflow. The release of heated runoff to downstream receiving channels may be detrimental to fish and other aquatic species inhabiting those channels. Consequently, a release depth of approximately 18 inches is recommended. (Virginia Stormwater Management Handbook, (DCR, 1999, Et seq.).

The first step in computing the required outlet size is to establish the maximum anticipated baseflow which must be conveyed through the basin once the permanent pool volume is present. This maximum baseflow arises during the month exhibiting the highest average precipitation. The Virginia State Climatology Office maintains an online database with monthly climate information from various stations across the state. This information can be obtained at: *http://climate.virginia.edu/online_data.htm#monthly*

Examining this data for the Staunton station, we see that the month exhibiting the highest average precipitation total is September, with *3.91 inches*.

This precipitation total must now be converted into a runoff rate. This is accomplished by first employing the NRCS runoff depth equation.

The post-development site is comprised of a total of 24.8 acres, 11.2 acres of which is impervious and 13.6 acres of which is unimproved grass cover. Appendix 6H-3 and 6H-4 of the VDOT Drainage Manual contain runoff curve numbers for various land covers and Hydrologic Soil Groups.

The site Hydrologic Soil Group is *C*. Because the site pervious cover is grass in fair condition, the runoff curve number taken from Appendix 6H-3 is *79*. The curve number for the site impervious fraction is *98*.

Next, the 2-year 24-hour precipitation depth must be obtained in order to estimate the average runoff efficiency. This information can be obtained from the National Weather Service at:

http://hdsc.nws.noaa.gov/hdsc/pfds/orb/va_pfds.html

Examining this data for the Staunton station reveals the 2-year 24-hour precipitation depth, *P*, to be *2.86 inches*.

Next, the NRCS runoff depth equations are employed to determine the 2-year 24-hour runoff depth for the post-developed site:

Pervious Fraction

$$S = \frac{1000}{CN} - 10 = \frac{1000}{79} - 10 = 2.66$$

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} = \frac{(2.86 - (0.2)(2.66))^2}{(2.86 + (0.8)(2.66))} = 1.09 inches$$

Impervious Fraction

$$S = \frac{1000}{CN} - 10 = \frac{1000}{98} - 10 = 0.20$$

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} = \frac{(2.86 - (0.2)(0.20))^2}{(2.86 + (0.8)(0.20))} = 2.63 inches$$

The total depth of runoff over the entire developed site is then computed as:

$$\frac{(1.09 inches)(13.6 acres) + (2.63 inches)(11.2 acres)}{24.8 acres} = 1.79 inches$$

The Efficiency of Runoff, *E*, is computed as the ratio of runoff depth to the total depth of precipitation for the 2-year event:

$$E = \frac{1.79 in}{2.86 in} = 0.63$$

Employing this efficiency ratio, we can estimate the average runoff volume for the month of September as:

$$3.91 inches \times 0.63 \times \frac{1 ft}{12 in} \times 24.8 ac \times \frac{43{,}560 ft^2}{ac} = 221{,}756 ft^3$$

The average baseflow rate is then computed as:

$$\frac{221{,}756\,ft^3}{30days} \times \frac{1day}{24hour} \times \frac{1hour}{3{,}600\sec} = 0.09cfs$$

The elevation at which the baseflow bypass outlet begins to discharge from the basin must be set equal to the basin elevation corresponding to the permanent pool volume. This ensures that the permanent pool volume is maintained in the basin at all times, while perennial baseflow is passed through the principal spillway and does not accumulate in the basin. Referencing Figures 4.5 and 4.6, we see that the permanent pool volume occurs at basin elevation 2006. The crest of the baseflow bypass outlet is therefore set at 2006 and sized as follows:

We will initially try a 3-inch diameter orifice, and restrict the maximum head to that occurring just as the outlet becomes submerged. Employing the orifice equation:

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

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

$$a = \pi r^2 = \pi \times \left(\frac{3in/2}{\frac{12in}{ft}} \right)^2 = 0.049\,ft^2$$

The head is measured from the centerline of the orifice. The head when the orifice has just become submerged by a small increment, 0.01 ft, is expressed as:

$$h = 1.5inches \times \frac{1\,ft}{12in} + 0.01\,ft = 0.135\,ft$$

Discharge is now computed as:

$$Q = (0.6)(0.049)\sqrt{(2)(32.2)(0.135)} = 0.09cfs$$

The selected 3-inch diameter orifice appears ideally suited for conveying the basin perennial baseflow.

Step 6. Embankment Design

When a stormwater impounding facility exceeds 15 feet in height or, as is the case with a retention basin, holds a permanent pool of water, the earthen embankment must be comprised of homogenous material with seepage controls or zoned embankments. The following steps provide guidance in designing a zoned embankment.

The steps presented in this example *do not* apply to embankments whose height exceed 25 feet and exhibit a maximum storage capacity of 50 acre feet or more. Such an embankment may be regulated under the Virginia Dam Safety Act, Article 2, Chapter 6, Title 10.1 (10.1-604 et seq.) of the Code of Virginia and Dam Safety Regulations established by the Virginia Soil and Water Conservation Board (VS&WCB). As previously stated, a retention basin embankment may be excluded from regulation if it meets any of the following criteria:

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

The design and construction of an earthen embankment is a complex process, and is inherently site-specific. Such a design must consider all unique site constraints, the characteristics of both native and imported construction materials, and the downstream hazard potential should the embankment fail. It is the engineer's responsibility to evaluate all of these considerations, including the potential for significant property damage and/or loss of life in the event of embankment failure. The guidance presented in this example does not constitute a standard or specification, and is not intended to replace the need for a thorough site investigation whenever a stormwater impounding facility is proposed.

The Virginia Stormwater Management Handbook, (DCR, 1999, Et seq.) defines a zoned embankment as containing a central impervious core, flanked by zones of more pervious material called shells. The pervious shells serve the function of enclosing, supporting, and protecting the impervious core. Often, the pervious shells are comprised of native site materials while the impervious core, comprised of material with very low permeability, is imported.

The first element in the design of an earthen embankment is that of a cutoff trench. The cutoff trench should be situated along the centerline of the embankment, or slightly upstream of the centerline. Along the width of the embankment, the trench should extend up the embankment abutments to a point coinciding with the 10-year water surface elevation.

When a zoned embankment is proposed, the cutoff trench material should be identical to that of the embankment core. The trench bottom width and depth should be no less than four feet, and the trench slopes should be no steeper than 1H:1V. (Virginia Stormwater Management Handbook (DCR, 1999, Et seq.) Figure 4.8 illustrates the *minimum* cutoff trench size configuration.

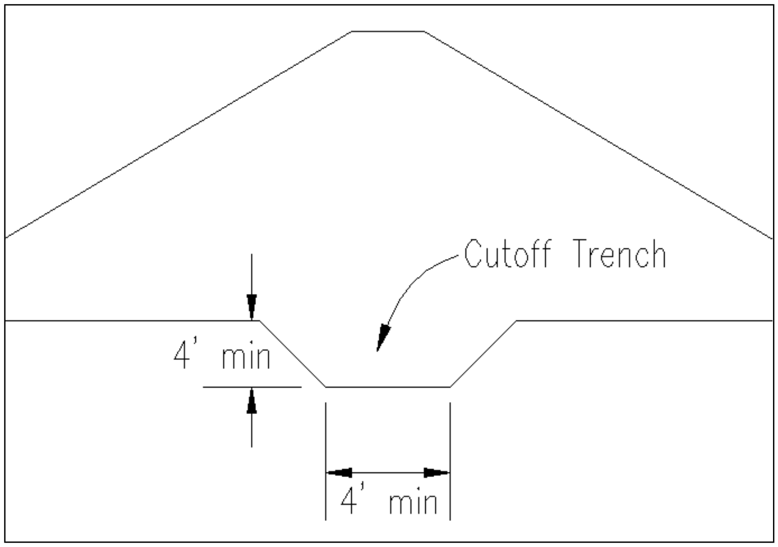


Figure 4.8. Typical Cutoff Trench Configuration

It must be noted that the dimensions shown in Figure 4.8 are absolute minimum values. Typically, as the ponded depth (and resulting hydraulic head) in a basin increase the bottom width of the trench should also increase. This increase in trench width may be reduced if the depth of the trench is also increased. The U.S. Bureau of Reclamation publication *Design of Small Dams* (revised 1977) gives the following relationship between head in the basin, trench width, and trench depth:

$$w = h - d$$

w = bottom width of cutoff trench
h = reservoir head above ground surface
d = depth of cutoff trench excavation below ground surface

The example basin permanent pool occurs at a basin depth of 6 feet (reference Figure 4.6). Fixing the cutoff trench depth as four feet and employing the trench width equation:

$$w = 6\,ft - 4\,ft = 2\,ft < \text{Minimum 4 ft}$$

Retention basins whose primary function is water quality improvement and flood control should typically exhibit permanent pool depths of less than 8 feet. Consequently, the minimum cutoff trench width and depth dimensions of four feet are generally adequate. However, when a proposed basin pool depth increases beyond the typical range, consideration should be given to increasing the dimensions of the embankment cutoff trench.

The next consideration is sizing the zones of the embankment. When a cutoff trench is provided, as required for a retention basin, sizing of the embankment zones should adhere to the guidelines illustrated in Figure 4.9.

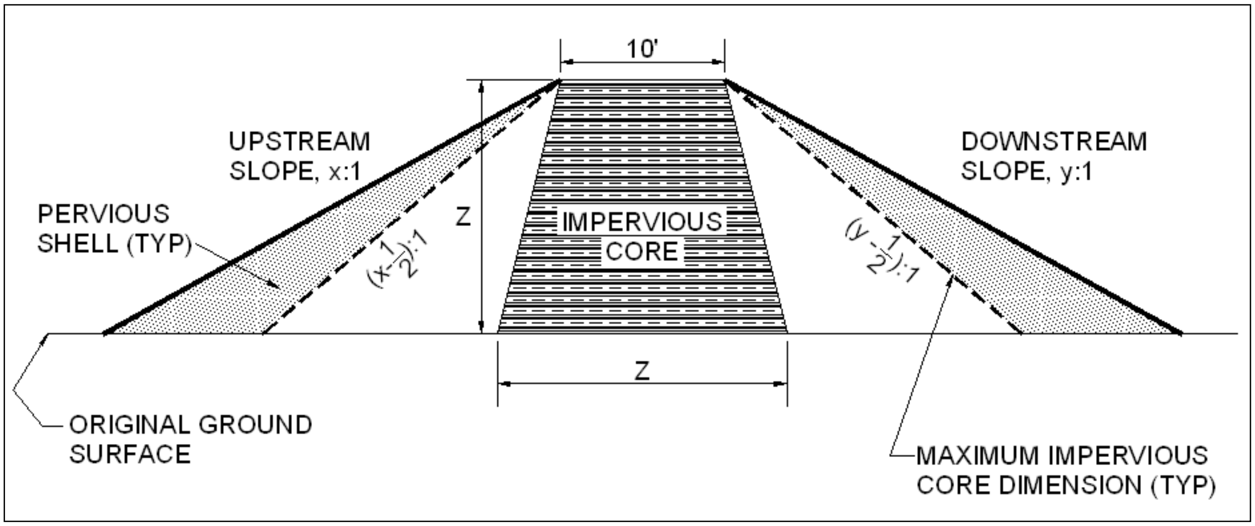


Figure 4.9. Minimum and Maximum Size of Embankment Core (U.S. Bureau of Reclamation, 1977)

As illustrated in Figure 4.9, the bottom width of the impervious core should, at a minimum, equal the total embankment height. This ensures that the core width at any basin elevation exceeds the height of embankment remaining above that elevation. Consequently, for all basin elevations, the hydraulic gradient through the core is less than unity and seepage potential is reduced. The maximum size of the impervious core is a function of the embankment's upstream and downstream external slopes. Should the impervious core be sized larger than these guidelines, the stabilization function of the pervious shell would be largely ineffective and, from a stabilization standpoint, the embankment would behave similar to a homogeneous type. (U.S. Bureau of Reclamation, 1977)

In the example problem, the proposed basin height is 9 feet (reference Figures 4.5 and 4.6), which is less than the embankment top width of 10 feet. Constructing the core bottom width equal to the embankment height would result in a negative slope for the sides of the impervious core. Such a configuration is impractical from a construction standpoint. The maximum side slope of the impervious core is a function of the embankment's external slopes, previously established as 3:1. Generally, the construction of the impervious core will require material to be imported to the site. It is both costly and unnecessary to size the core to its maximum dimensions (unless native site soils meet the classification for core material). In the example basin, we will consider impervious core side slopes of 1:1. This configuration is illustrated in Figure 4.10.

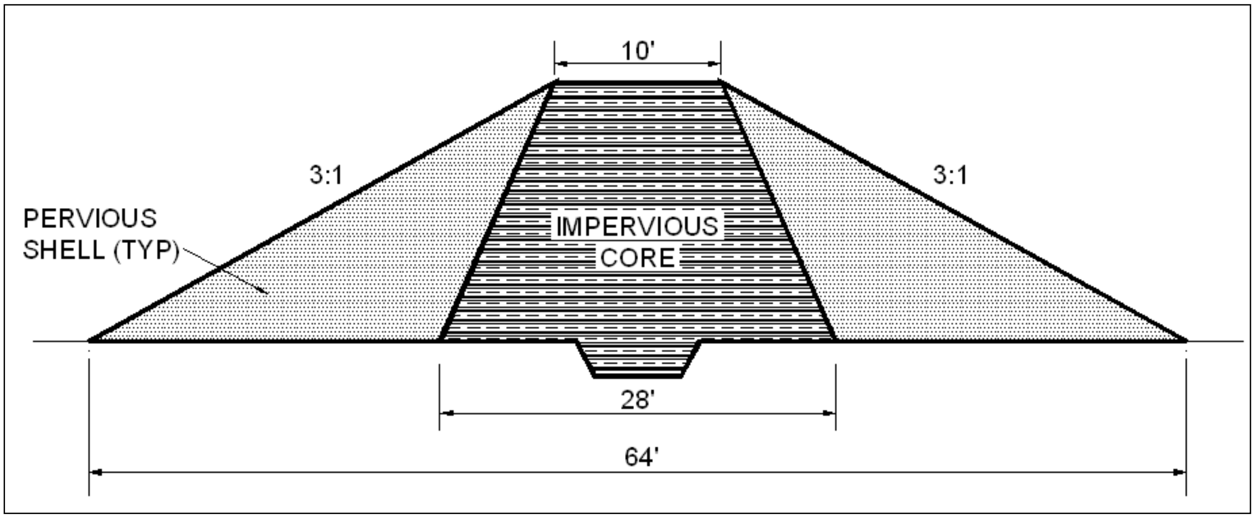


Figure 4.10. Example Basin Embankment Dimensions

Selection of core and pervious flanking material should conform to the Unified Soil Classifications shown in Table 4.6.

Zone	Core Material Classification
Impervious Core	GC, SC, CL*
Pervious Shell	Rockfill, GW, GP, SW, SP

Table 4.6. Suitable Embankment Material
(U.S. Bureau of Reclamation, 1977)

* Some materials approved by the U.S. Bureau of Reclamation have been omitted, and those shown are only those approved by the Virginia Department of Conservation and Recreation

When the classification of adjacent zone materials differs significantly, such as a clay impervious core adjoining a rockfill pervious shell, a transition zone is strongly recommended. The transition zone helps to prevent the fines of the core material from piping into the voids of the more pervious material. Additionally, on the embankment's upstream face, should voids or cracks appear in the core, the transition material can often effectively "plug" the voids, thus minimizing seepage. To facilitate ease of construction, the U.S. Bureau of Reclamation recommends that transition zones range between 8 and 12 feet in width; however, the effectiveness of a transition zone only a few feet wide can be significant. Transition zones are not required between impervious material and sand-gravel zones or between sand-gravel zones and rockfill.

The designer is referenced to section 11.3.6 of the VDOT Drainage Manual for additional embankment details and specifications.

Step 7. Water Balance Calculation

To ensure that the basin's permanent pool does not become dry during extended periods of low or absent inflow, the designer must perform a water balance calculation. Note that this water balance evaluation differs from the baseflow calculation made previously. Two approaches are described in the following section.

Step 7A. 45-Day Drought Condition

The first approach considers the extreme condition of a 45-day drought period with no precipitation and thus no significant surface runoff.

Table 4.7 presents potential evaporation rates for various locations in Virginia.

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

Table 4.7. Potential Evaporation Rates (Inches)
Virginia Stormwater Management Handbook, (DCR, 1999, Et seq.)

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

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

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

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

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

The total surface area of the permanent pool is 25,793 ft^2. Therefore, the total volume of water lost to evaporation is estimated as:

$$25{,}793\,ft^2 \times 0.64\,ft = 16{,}508\,ft^3$$

The volume of water lost to evaporation must be added to that lost to infiltration. As previously stated, the initial geotechnical tests revealed site soil infiltration rates to be 0.01 inches per hour. The infiltration is assumed to occur over the entire permanent pool, whose surface area is 25,793 ft^2. The volume of water lost to infiltration is estimated as:

$$25{,}793\,ft^2 \times 0.01\frac{in}{hr} \times \frac{1\,ft}{12in} \times 24\frac{hr}{day} \times 45days = 23{,}214\,ft^3$$

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

$$16{,}508\,ft^3 + 23{,}214\,ft^3 = 39{,}722\,ft^3$$

The total volume of the basin permanent pool is 1.88 ac – ft (81,893 ft^3). The estimated evaporation and infiltration loss over a 45-day drought period is slightly less than half of the total permanent pool volume. While the extended drought period does impact the basin pool significantly, a volume of more than twice the project site water quality volume does remain in the basin, and is thus considered adequate against drought.

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

Total contributing drainage area = 24.8 acres

Stored volume lost to evaporation and infiltration = 39,722 ft^3

$$\frac{39{,}722\,ft^3}{24.8ac \times \frac{43{,}560\,ft^2}{ac}} = .0368\ watershed\text{ - }feet = 0.44\ watershed\text{ - }inches$$

A precipitation event yielding a total runoff of 0.44 inches or more across the contributing watershed will replenish the depleted marsh volume.

Step 7B. Period of Greatest Evaporation (in Average Year)

The second water balance calculation examines impacts on the basin permanent pool during the one-month period of greatest evaporation. This calculation reflects an anticipated pool drawdown during the summer months of an average year. In contrast, the first calculation method reflects an extreme infrequent drought event.

From Table 4.7, the greatest monthly evaporation total for the project site is 5.52 inches in July. The Virginia State Climatology Office reports an average July rainfall for the Staunton station as 3.78 inches (reference Step 5 for link to data).

Applying the previously computed runoff efficiency ratio for the basin watershed, the average July inflow to the basin is computed as:

$$3.78inches \times 0.63 \times \frac{1ft}{12in} \times 24.8ac \times \frac{43{,}560\,ft^2}{ac} = 214{,}383\,ft^3$$

Evaporation losses are computed as the product of total monthly evaporation and the surface area of the permanent pool:

$$5.52inches \times \frac{1ft}{12in} \times 25{,}793\,ft^2 = 11{,}865\,ft^3$$

Infiltration losses over the entire month of July are estimated as:

$$25{,}793\,ft^2 \times 0.01\frac{in}{hr} \times \frac{1ft}{12in} \times 24\frac{hr}{day} \times 31days = 15{,}992\,ft^3$$

The water balance expression and total monthly loss/gains are computed as follows:

Monthly loss/gain = Inflow – Evaporation – Infiltration

$$= 214{,}383\,ft^3 - 11{,}865\,ft^3 - 15{,}992\,ft^3 = 186{,}526\,ft^3$$

The monthly climate data and site land cover characteristics indicate that the basin will not experience drawdown during the average period of highest evaporation.

Step 8. Landscaping

Generally, the non-inundated (dry storage) regions of a retention basin can be landscaped in the same manner as a dry basin (reference *Chapter Two – Dry Extended Detention Basin)*; however, careful attention must be given to the types of vegetation selected for the basin pool and aquatic bench areas. For these regions, the vegetative species must be selected based on their inundation tolerance and the anticipated frequency and depth of inundation.

The regions of varying depth within the basin are broadly categorized by zone as shown in Figure 4.11. Note the basin aquatic bench would be encompassed by Zone 2.

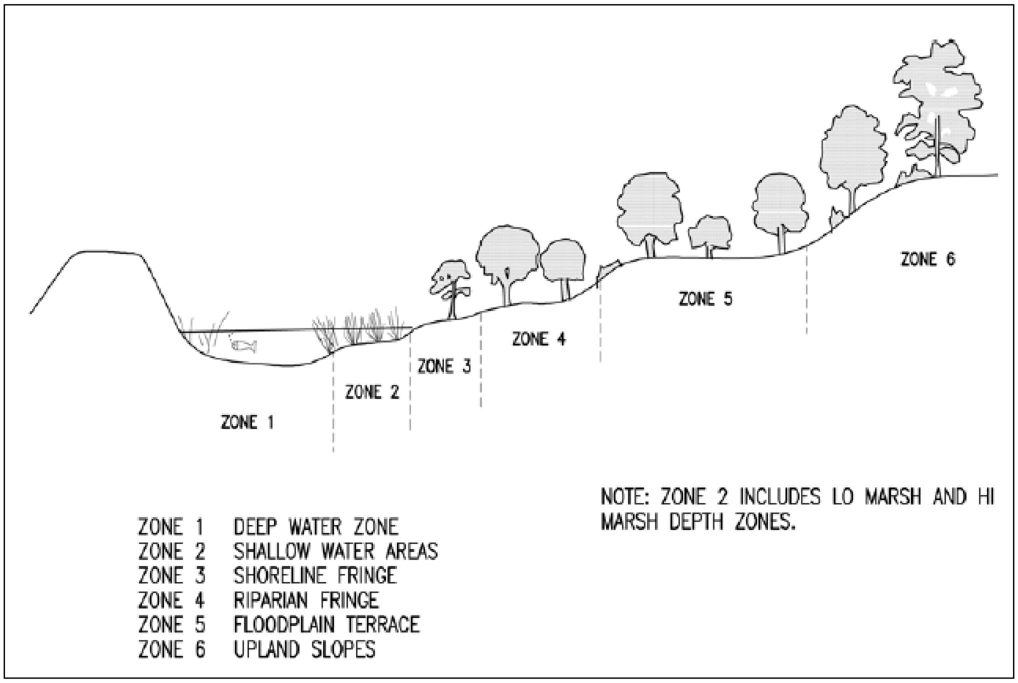


Figure 4.11. Planting Zones for Stormwater BMPs
Virginia Stormwater Management Handbook (DCR, 1999, Et seq.)

Suitable planting species for each of the zones identified in Figure 4.11 are recommended in Chapter 3-05 of the Virginia Stormwater Management Handbook, (DCR, 1999, Et seq.). Ultimately, the choice of planting species should be largely based on the project site's physiographic zone classification. Additionally, the selection of plant species should match the native plant species as closely as possible. Surveying a project site's native vegetation will reveal which plants have adapted to the prevailing hydrology, climate, soil, and other geographically-determined factors. Figure 3.05-4 of the Virginia Stormwater Management Handbook provides guidance in plant selection based on project location.

Generally, stormwater management basins should be permanently seeded within 7 days of attaining final grade. This seeding should comply with Minimum Standard 3.32, Permanent Seeding, of the Virginia Erosion and Sediment Control Handbook, (DCR, 1992, Et seq.). It must be noted that permanent seeding is *prohibited* in Zones one through four of Figure 4.11. The use of conventional permanent seeding in these zones will result in the grasses competing with the requisite wetland emergent species.

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

All chosen plant species should conform to the American Standard for Nursery Stock, current issue, and be suited for USDA Plant Hardiness Zones 6 or 7, see Figure 4.12.

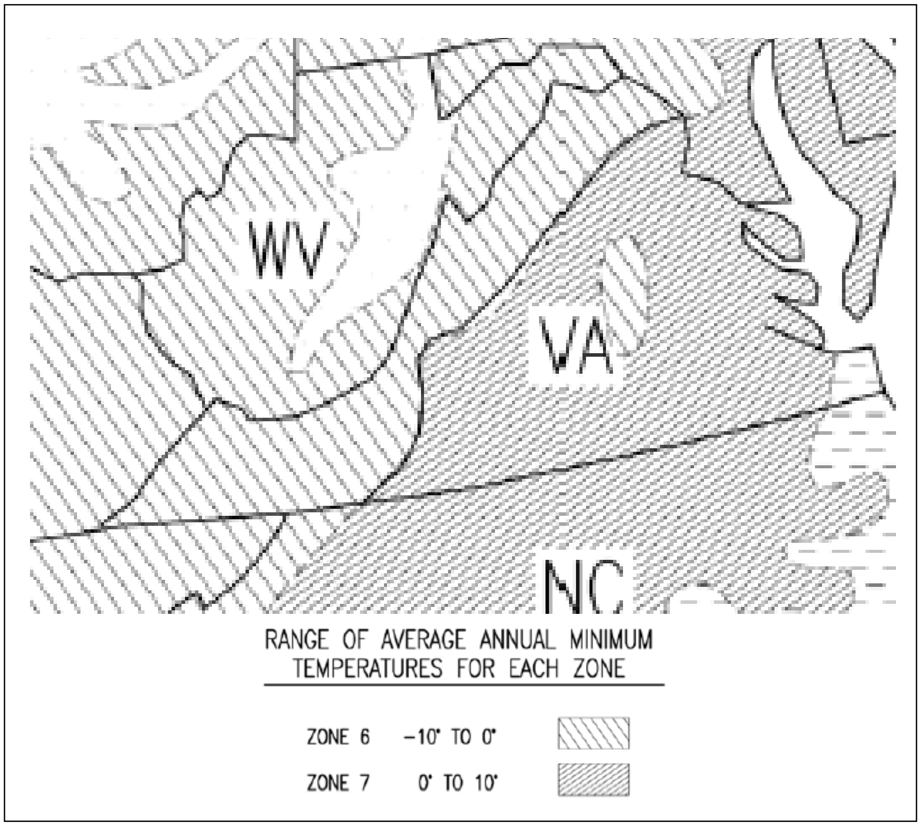


Figure 4.12. USDA Plant Hardiness Zones

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

Chapter 5 – Constructed Stormwater Wetlands

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

5.1 Overview of Practice

Constructed stormwater wetlands fall into a structural BMP category having the capacity to improve the quality of stormwater runoff in much the same manner as retention and enhanced extended detention basins. Like these impounding facilities, stormwater wetlands are seeded with a diverse mix of aquatic and emergent vegetation, which plays an integral role in the pollutant removal efficiency of the practice. Wetland BMPs improve the quality of runoff by physical, chemical, and biological means. The physical treatment of runoff occurs as a result of decreased flow velocities in the wetland, thus leading to evaporation, sedimentation, adsorption, and/or filtration. Chemical treatment arises in the form of chelation (bonding of heavy metal ions), precipitation, and chemical adsorption. The biological treatment processes occurring in wetlands include decomposition, plant uptake and removal of nutrients, and biological transformation and degradation. (FHWA, 1996)

Constructed stormwater wetlands should not be confused with naturally occurring wetlands. When proper pre-treatment measures are implemented, naturally occurring wetlands are *sometimes* capable of receiving runoff from development projects; however, constructed wetlands serve the primary function of receiving stormwater runoff, and generally exhibit less biodiversity than naturally occurring wetlands both in terms of plant and animal life (Yu, 2004). Similarly, constructed wetlands differ from *created wetlands*, which are intended to replace and mimic naturally occurring wetlands for mitigation purposes.

Constructed stormwater wetlands should, generally, *not be used for flood control or downstream channel control*. When a BMP is employed as a quantity control practice, there is an inherent expectation of rapidly fluctuating water levels in the practice following runoff producing events. Rapid fluctuations in water level subject emergent wetland and upland vegetation to enormous stress, and many wetland species cannot survive such conditions. In addition to producing large surges of stormwater runoff, land use conversion resulting in a loss of pervious cover will often result in a decrease of perennial baseflow from a watershed. The decrease or absence of such baseflow is problematic for the establishment of a diverse and healthy mix of wetland vegetation.

Figures 5.1, 5.2, and 5.3 present various schematic views of constructed stormwater wetlands.

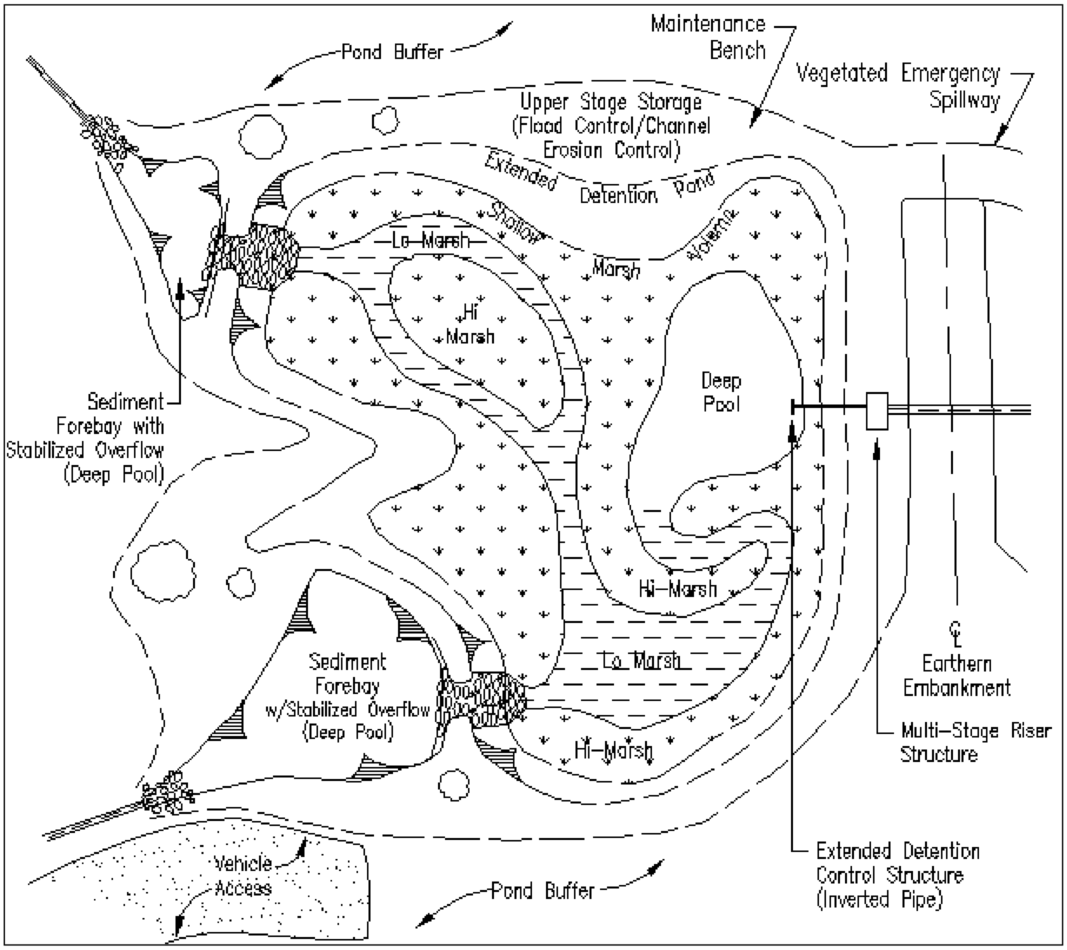


Figure 5.1. Constructed Stormwater Wetlands (Plan View)
(Virginia Stormwater Management Handbook, 1999, Et seq.)

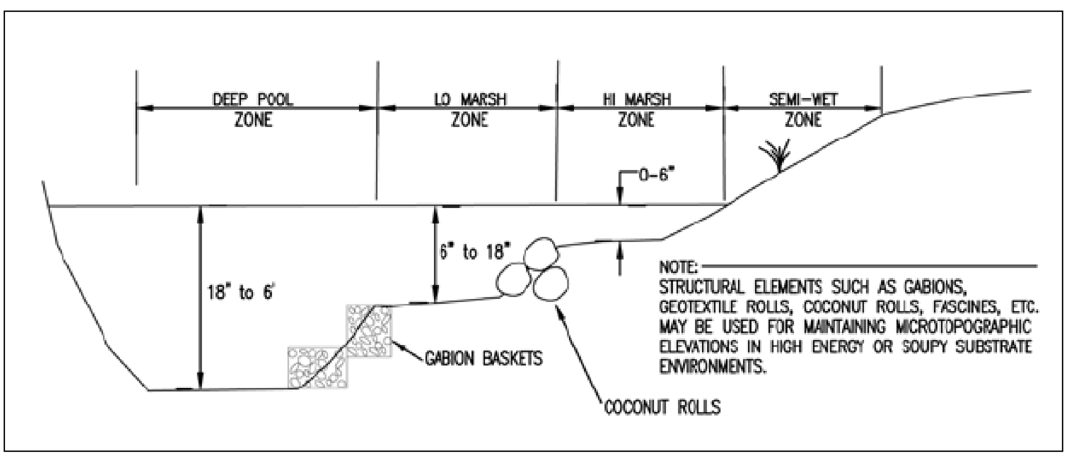


Figure 5.2. Varying Wetland Depth Zones (Profile)
(Virginia Stormwater Management Handbook, 1999, Et seq.)

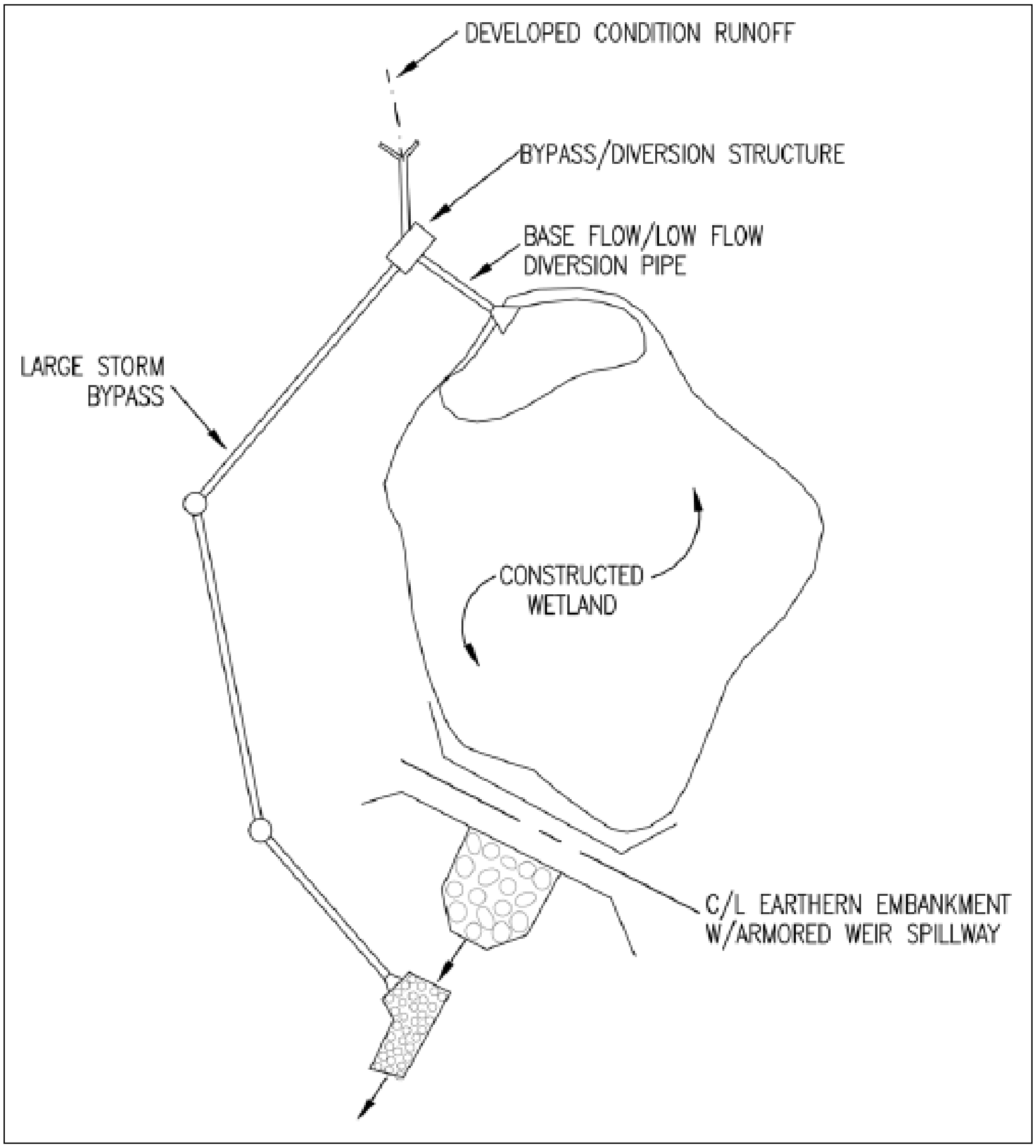


Figure 5.3. Offline Wetland Configuration
(Virginia Stormwater Management Handbook, 1999, Et seq.)

As evidenced in Figure 5.1, the wetland is comprised of three distinct zones – "low marsh," "high marsh," and "deep pool." These varying-depth zones introduce *microtopography* to the basin floor. Detailed surface area and depth requirements of the various marsh zones are discussed later in this section.

5.2 Site Constraints and Siting of the Facility

The engineer must consider a number of site constraints in addition to site impervious area when the implementation of constructed stormwater wetlands is proposed.

5.2.1 Minimum Drainage Area

Constructed stormwater wetlands should generally not be considered when contributing drainage area is less than 10 acres. Of critical concern is the presence of adequate baseflow to the facility. Many species of wetland vegetation cannot survive extreme drought conditions. Additionally, insufficient baseflow and the subsequent stagnation of wetland marsh areas can lead to the emergence of undesirable odors from the wetland. Regardless of drainage area, all proposed wetlands should be subjected to a low flow analysis to ensure that an adequate marsh volume is retained even during periods of dry weather when evaporation and/or infiltration are occurring at a high rate. The anticipated baseflow from a fixed drainage area can exhibit great variability, and insufficient baseflow may require consideration of alternate BMP measures. When infiltration losses from the wetland are excessive, a clay liner or geosynthetic membrane may be considered. Such a liner should meet the approval and specifications of the Materials Division.

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

5.2.2 Maximum Drainage Area

The maximum drainage area to a constructed stormwater wetland is not explicitly restricted. However, the designer must consider that, due to the needs of aquatic plant species, storage volume in the form of excessive pool depth (vertical storage) is typically not possible. Therefore, the land area required for constructed wetland may be two to three times the site area required of alternative BMPs. (MWCOG, 1992) The minimum surface area of the wetland marsh area is two percent of the contributing drainage area.

5.2.3 Separation Distances

Constructed stormwater wetlands should be located a minimum of 20 feet from any permanent structure or property line, and a minimum of 100 feet from any septic tank or drainfield.

5.2.4 Site Slopes

Stormwater wetlands should, generally, not be constructed within 50 feet of any slope steeper than 10 percent. When this is unavoidable, or when the facility is located at the toe of a slope greater than 10 percent, a geotechnical report should be performed to address the potential impact of the facility in the vicinity of such a slope.

5.2.5 Site Soils

The implementation of constructed stormwater wetlands can be successfully accomplished in the presence of a variety of soil types. However, when such a facility is proposed, a subsurface analysis and permeability test is required. The required subsurface analysis should investigate soil characteristics to a depth of no less than three feet below the proposed bottom of the wetland. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils. To ensure the long-term success of a constructed wetland, it is essential that water inflows (baseflow, surface runoff, and groundwater) be greater than losses to evaporation and infiltration. This requires the designer to calculate a monthly water budget. Due to excessive infiltration losses, soils exhibiting high infiltration rates are not suited for the construction of stormwater wetlands. Often, soils of moderate permeability (on the order of $1x10^{-6}$ cm/sec) are capable of supporting the shallow marsh areas of a stormwater wetland. However, the hydraulic head (pressure) generated from deeper regions, such as the wetland micro-pool, may increase the effective infiltration rate rendering similar soils unsuitable for wetland construction. Mechanical compaction of existing subsoils, a clay liner, geosynthetic membrane, or other material (as approved by the Materials Division) may be employed to combat excessively high infiltration rates. The wetland embankment material must meet the specifications detailed later in this section and/or be approved by the Materials Division.

5.2.6 Rock

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

5.2.7 Existing Utilities

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

5.2.8 Karst

The presence of Karst topography places even greater importance on the subsurface investigation. Construction of stormwater wetlands in Karst regions may greatly impact the design and cost of the facility, and must be evaluated early in the planning phases of a project. *Construction of stormwater management facilities within a sinkhole is prohibited.* When the construction of such facilities is planned along the periphery of a sinkhole, the facility design must comply with the guidelines found in Instructional and Informational Memorandum IIM-LD-228 on *“Sinkholes”* and DCR Technical Bulletin #2 *“Hydrologic Modeling and Design in Karst.”*

5.2.9 Existing Wetlands

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

5.2.10 Upstream Sediment Considerations

Close examination should be given to the flow velocity at all points discharging concentrated runoff to the wetland. When entering flows exhibit erosive velocities, they have the potential to greatly increase maintenance requirements by depositing large amounts of sediment within the wetland. Regardless of entering flow velocities, a highly disturbed contributing drainage area can hinder the wetland pollutant removal performance through the deposition of excessive sediment. Constructed wetlands are extremely vulnerable to sediment loading, as excessive sediment loading has the potential to greatly alter the microtopography of the marsh floor. The negative impacts associated with excessive sediment loading reinforce the need for sediment forebays as discussed in Section 5.3.

5.2.11 Location

When properly designed, landscaped, and maintained, constructed wetlands may be suitable for high visibility locations. However, when a constructed wetland is proposed in a high visibility location, ongoing maintenance of the facility is critical to its acceptance by neighboring landowners. Additionally, early in the project planning stages, careful attention should be given to the general characteristics of neighboring land uses. The landscape of a constructed wetland exhibits natural and sometimes rapid growth and vegetative colonization. This may be undesirable in the vicinity of an otherwise manicured landscape. The designer must also be aware of the significant land area requirements of a constructed stormwater wetland.

5.2.12 Hydrology

To achieve the pollutant removal efficiencies expressed in Table 1.1, the marsh area of a constructed wetland must support aquatic and emergent plant species. While a quantified volumetric flow rate is not explicitly required, the wetland's contributing watershed should supply enough runoff to ensure that the marsh pools of varying depth are maintained as intended.

5.3 General Design Guidelines

The following presents a collection of issues to be considered when designing a constructed stormwater wetland.

5.3.1 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam. Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management"*, Section 12.1.1:

> *"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 feet or the amount recommended by the VDOT Materials Division. The backfill and embankment material must meet the soil classification requirements identified herein or the design of the dam may incorporate a trench lined with a membrane (such as bentonite penetrated fabric or an HDPE or LDPE liner). Such designs shall be reviewed and approved by the VDOT Materials Division before use."*

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

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

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

5.3.2 Embankment Geometry

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

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

The designer is referenced to section 11.3.6 of the VDOT Drainage Manual for additional embankment details and specifications.

5.3.3 Embankment Height

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

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

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

5.3.4 Principal Spillway Design

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

The primary outlet of a constructed stormwater wetland should be a weir if at all possible. Weirs can be configured to convey large volumetric flow rates with relatively low head. Minimization of ponding depth in a wetland helps to avoid unnecessarily stressing the sensitive vegetative species.

5.3.5 Outfall Piping

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

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

5.3.6 Prevention of Short-Circuiting (Wetland Geometry)

Short-circuiting occurs when entering flows pass rapidly through the wetland without achieving effective hydraulic residence times. Short-circuiting of flow negatively impacts the observed water quality benefit of the wetland. While site conditions will ultimately dictate the geometric configuration of a constructed wetland, it is preferable to construct the facility such that the dry length-to-width ratio is 2:1 or greater, and the *wet* length-to-width ratio is at least 1:1.

The dry length-to-width ratio is computed by dividing the dry weather flow path length (from entrance point to primary outlet) by the wetland's average width. The wet length-to-width ratio is calculated by dividing the straight line distance (from entrance point to primary outlet) by the wetlands average width. The dry weather length-to-width ratio is easily increased through the creative use of microtopography, such as situating high marsh berms perpendicular to straight line flow paths. This reduces the likelihood of short-circuiting by creating meandering flow paths rather than straight line paths from stormwater entrance points to the principal spillway.

5.3.7 Volume

The pollutant removal efficiency of a constructed stormwater wetland (expressed in Table 1.1) is based on a permanent pool/marsh volume of twice the computed water quality volume ($2xWQ_V$) from the contributing drainage area.

5.3.8 Surface Area

The surface area of the wetland permanent marsh should, at a minimum, be two percent of the area contributing runoff to the wetland. A permanent pool surface area of three percent (or greater) of the wetland's contributing drainage area is optimal.

5.3.9 Ponded Depth

The depth of the wetland marsh affects the planting species selected for the wetland as well as the types of aquatic and wildlife species that will inhabit the wetland and its surrounding areas. Additionally, the depth allocation of the permanent pool has a significant impact on the pollutant removal performance of the wetland. Table 5.1 presents the recommended surface area and volume allocation for the various permanent pool depth zones. The characteristics of each zone are discussed later in the context of a design example.

Depth Zone	Surface Area (% of Total Surface Area)	Treatment Volume (% of Total Treatment Volume)
Deep Water (1.5 – 6 feet deep)	10	20
Low Marsh (0.5 – 1.5 feet deep)	40	*
High Marsh (0 – 0.5 feet deep)	50	*

Table 5.1. Recommended Allocation of Surface Area and Treatment Volume for Various Depth Zones (Virginia Stormwater Management Handbook, 1999, Et seq.)

* The combined marsh areas should sum to approximately 80 percent of the total treatment volume. If the surface area criteria conflict with volume allocations, the surface area allocations are considered more critical to an effective design. (DCR, 1999, Et seq.)

5.3.10 Maximum Flood Control Ponded Depth

The use of constructed stormwater wetlands for flood control is strongly discouraged. Offline configurations, such as that shown in Figure 5.3, can provide effective water quality improvement while not subjecting the wetland to the extreme water fluctuations typically associated with flood control facilities. When a proposed wetland will be subjected to storm inflows beyond the water quality volume, it is critical to restrict the vertical ponding depth to as shallow as practically possible. Outlet structures must be sized to pass the 10-year return frequency storm with a maximum ponded depth of 2 feet above the wetland marsh pool. (DCR, 1999, Et seq.)

5.3.11 Fencing

Per Instructional and Informational Memorandum IIM-LD-195 under *"Post Development Stormwater Management,"*, Section 13.1.1, fencing is typically not required or recommended on most VDOT detention facilities. However, exceptions do arise, and the fencing of a dry extended detention facility may be needed. Such situations include:

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

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

5.3.12 Sediment Forebays

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

5.3.13 Discharge Flows

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

5.4 Design Process

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

The following design example is founded on the development scenario described in *Chapter Two – Dry Extended Detention Basin*. The project entails the construction of a section of two lane divided highway situated in Montgomery County. The total project site, including right-of-way and all permanent easements, consists of 17.4 acres. Pre and post-development hydrologic characteristics are summarized below in Tables 5.2 and 5.3. Peak rates of runoff for both pre and post-development conditions were computed by the Rational Method and the regional NOAA NW-14 factors recommended in the VDOT Drainage Manual. Initial geotechnical investigations reveal a soil infiltration rate of 0.02 inches per hour.

	Pre-Development	**Post-Development**
Project Area (acres)	17.4	17.4
Land Cover	Unimproved Grass Cover	4.8 acres impervious cover
Rational Runoff Coefficient	0.30	0.50*
Time of Concentration (min)	45	10

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

Table 5.2. Hydrologic Characteristics of Example Project Site

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

Table 5.3. Peak Rates of Runoff (cfs)

Step 1. Compute the Required Water Quality Volume

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

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

IA= Impervious Area (square feet)

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

$$WQV = \frac{4.8ac \times 43{,}560\frac{ft^2}{ac} \times \frac{1}{2}in}{12\frac{in}{ft}} = 8{,}712\,ft^3$$

The permanent marsh area of the wetlands will be sized to provide twice this volume (17,424 ft^3).

Step 2. Sizing the Marsh Area Zones

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

The *Deep Pool Zone* ranges in depth from 1.5 to 6 feet, and may be comprised of the following three categories:

- o sediment forebays
- o micro pools
- o deep water channels

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

Micro-pools provide open water areas which promote plant and wildlife diversity. When the wetland is equipped with a riser structure, a micro-pool should be provided near the riser. When a baseflow conveyance pipe is provided, it should be constructed on a negative slope that extends to an approximate depth of 18 inches below the normal surface of the micro-pool.

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

The sum of all forebay, micro-pool, and deep channel volumes should be 10 percent of the marsh surface area and provide approximately 20 percent of the water quality volume (reference Table 5.1).

Low Marsh Zones are those regions of the marsh ranging in depth between 6 and 18 inches. The sum of all low marsh zones should equal 40 percent of the total marsh surface area.

High Marsh Zones are those regions of the marsh ranging in depth from 0 to 6 inches. The high marsh zone is capable of supporting the most diverse mix of vegetation. The sum of all high marsh zones should comprise 50 percent of the total marsh surface area.

Semi-Wet Zones are those regions of the marsh that are situated above the permanent marsh pool. During non runoff-producing periods, the semi-wet zone is generally dry. This zone becomes inundated during runoff-producing events.

When designing the marsh area of a constructed stormwater wetlands, both surface area and volume guidelines must be considered. The following steps illustrate this process for the example project site. As indicated earlier, the example site is a section of two lane divided highway in Montgomery County.

Step 2B. Compute the Minimum Marsh Surface Area

The summation of all "wet" marsh zone surface areas must not be less than two percent of the wetland's total contributing drainage area. The minimum marsh surface area is therefore computed as:

$$17.4ac \times \frac{43{,}560\,ft^2}{ac} \times 0.02 = 15{,}159\,ft^2$$

This minimum area must be distributed across the three "wet" marsh zones as shown in Table 5.1. The total volume provided by this distribution should yield the computed treatment volume of 17,424 ft^3. If the surface area criteria conflict with storage volume requirements, the surface area allocations are considered more critical to an effective wetland design. (DCR, 1999, Et seq.) Consequently, it is considered essential to attain the surface area distributions shown in Table 5.1. The following steps illustrate a procedure for meeting the surface area allocation targets while also achieving the desired water quality volume.

Step 2C. Size the Zones of Varying Depth

50 percent of the total surface area of the marsh should be dedicated to the *high marsh zone* (depths ranging between zero and 6 inches). The *low marsh zone* (depths ranging between 6 and 18 inches) should comprise an additional 40 percent of the total marsh surface area. The remaining 10 percent of the marsh surface area should be made up of the *deep water zone* (ranging in depth from 1.5 to 6 feet).

The total surface area of the marsh is designated as *A*. Following this convention, the surface area of each depth zone can be expressed as follows:

$$A_1 = 0.50A$$
$$A_2 = 0.40A$$
$$A_3 = 0.10A$$

Because of its shallow depth, the side slopes of the high marsh zone can be considered negligible, and the *effective* depth of the zone is assumed to be the maximum depth of 0.5 feet.

This effective depth can be employed for purposes of volume calculations. Therefore, the total volume encompassed by the marsh's shallowest pool zone is approximated as follows:

$$V_1 = 0.5\,ft \times A_1 = (0.5\,ft)(0.50)(A)$$

The effective depth of the low marsh zone is computed as its average depth:

$$D_e = \frac{6in + 18in}{2} = 12in = 1\,ft$$

With the total volume encompassed by the low marsh zone approximated as follows:

$$V_2 = 1\,ft \times A_2 = (1\,ft)(0.40)(A)$$

For this example, the deep water zone of the marsh (sediment forebays and micro pool) will be designed at an average depth of 4 feet. Therefore, the effective depth is 2 feet and the volume is expressed as:

$$V_3 = 2\,ft \times A_3 = (2\,ft)(0.10)(A)$$

The sum of all incremental marsh volumes should equal or exceed 0.40 acre-feet. Therefore, the basin surface area, *A*, is approximated as follows:

$$V = 17{,}424\,ft^3$$
$$V = (0.5\,ft)(0.50)(A) + (1\,ft)(0.40)(A) + (2\,ft)(0.1)(A)$$

Rearranging and solving for surface area, *A*:

$$0.85A = 17{,}424\,ft^3$$
$$A = 20{,}499\,ft^2$$

This value exceeds the minimum allowable surface area of 15,159 ft^2 and is therefore acceptable. The computed surface area is 2.7 percent of the wetland contributing drainage area of 17.4 acres.

Tables 5.4 and 5.5 summarize the surface area and approximate volume of each marsh depth zone.

Zone / Depth	Surface Area (ft^2)	Percentage of Total Surface Area (%)
High Marsh (0 - 6")	10,250	50
Low Marsh (6 - 18")	8,199	40
Deep (0 - 4')	2,050	10
Total	***20,499***	***100***

Table 5.4. Surface Area Summary of Varying Depth Zones

Zone / Depth	Approximate Volume (ft^3)	Percentage of Total Treatment Volume (%)
High Marsh (0 - 6")	5,125	30
Low Marsh (6 - 18")	8,199	47
Deep (0 - 4')	4,100*	23
Total	***17,424***	***100***

Table 5.5. Volume Summary of Varying Depth Zones

*Includes sediment forebay and micro pool volumes

It is noted that the treatment volume provided in the deep water zone is 23 percent of the total treatment volume. This slightly exceeds the target of 20 percent. However, as previously stated, attainment of surface area allocation targets is of greater importance than volume distribution.

The computed deep pool surface area must be distributed among two sediment forebays and the outlet micro-pool. Obtained from *Chapter Two – Extended Dry Detention Basin*, Table 5.6 presents the respective storage volume of each sediment forebay.

Basin Location	Volume (ft^3)
Forebay 1	817
Forebay 2	908

Table 5.6. Deep Pool Volume Allocation

The total forebay volume is 1,725 ft^3. The remaining deep pool volume (2,375 ft^3) is allocated to the micro-pool located at the wetland outlet.

Step 3. Construct Elevation – Storage Relationship

Having determined the required surface area and storage volume for each of the three “wet” marsh zones, the next step is to construct a stage – storage relationship. This step is required in order to perform final flood routing for selected storms, thereby testing the final grading plan and outlet structure design for adequacy. The reader is referred to Step 6 of *Chapter Two – Dry Extended Detention Basin* for detailed flood routing procedure. Each site is unique, both in terms of constraints and required storage volume. Because of this, the development of a proposed grading plan may be an iterative process. The reader is referred to *Chapter Four – Retention Basin* for detailed embankment design procedures.

Table 5.7 presents the stage – storage relationship for the computed marsh area. The wetland floor elevation is assumed to be 2000 ft MSL.

Elevation	Incremental Volume (ft^3)	Total Volume (ft^3)
2100	0	0
2100.5	512.5	512.5
2101	512.5	1025
2101.5	512.5	1537.5
2102	512.5	2050
2102.5	512.5	2,562.5
2103	3245.5	5,808
2103.5	3245.5	9,053.5
2104	8,370.5	17,424

Table 5.7. Stage – Storage Relationship

Step 4. Evaluate Impact of the 10-Year Runoff Producing Event

The use of constructed stormwater wetlands for flood control is strongly discouraged. Offline configurations, such as that shown in Figure 5.3, can provide effective water quality improvement while not subjecting the wetland to the extreme water fluctuations typically associated with a flood control facility. When a proposed wetland will be subjected to storm inflows beyond the water quality volume, it is critical to restrict the vertical ponding depth to as shallow as practically possible. Outlet structures must be sized to pass up to the 10-year return frequency storm with a maximum ponded depth of 2 feet above the surface of the wetland marsh. (DCR, 1999, Et seq.) The following steps illustrate a procedure for ensuring that the 10-year return frequency storm is routed through the example wetland facility without inducing a ponded depth of more than two feet above the marsh surface. The reader is referred to *Chapter Two – Dry Extended Detention Basin* for detailed routing and principal spillway design steps.

This design example will employ a riser consistent with the SWM-1 structure detailed in the Virginia Department of Transportation's Road and Bridge Standards. A detail of this type of inlet top is shown in Figure 5.4.

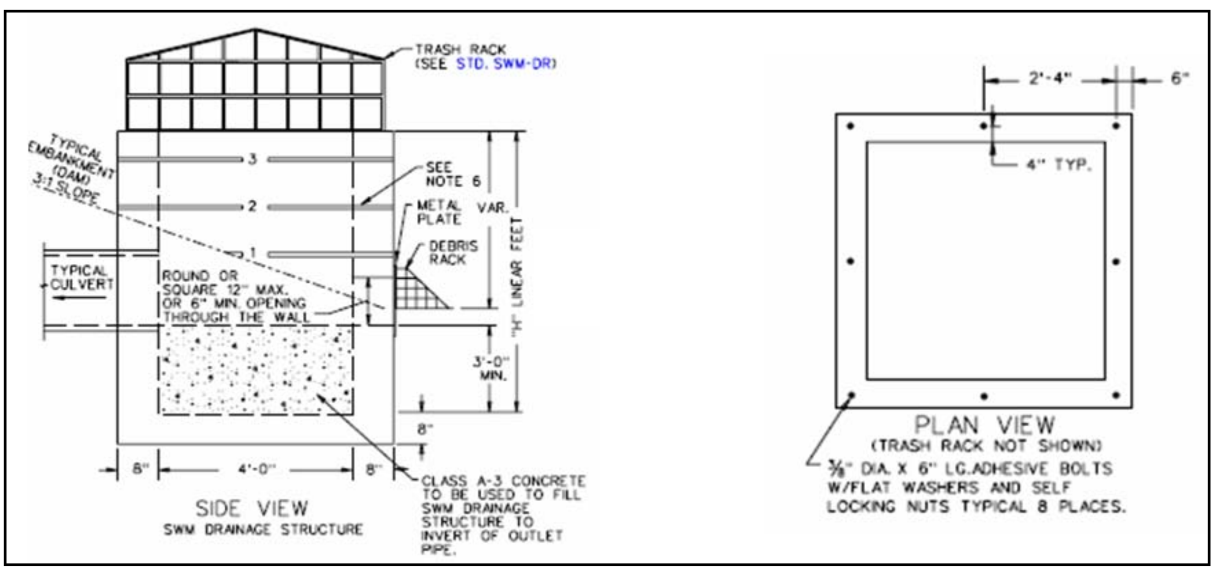


Figure 5.4. VDOT SWM-1 Plan and Section
VDOT Road and Bridge Standards

Obtained from *Chapter Two – Extended Dry Detention Basin* the effective weir length and flow area of the SWM-1 grate top is:

Effective flow perimeter (weir length): 16 ft

Effective flow area: 16 ft^2

The crest of the grate will be set at an elevation just above the surface of the wetland permanent pool – 2004.1. This will minimize the depth of ponding observed during runoff producing events.

The next step is to estimate the volume of storage provided above the permanent marsh in the wetland *semi-dry zone*.

In this example, we will consider a wetland of rectangular orientation, with a 2.5:1 length-to-width ratio. The demonstrated methodology can be adapted to wetlands exhibiting different geometry.

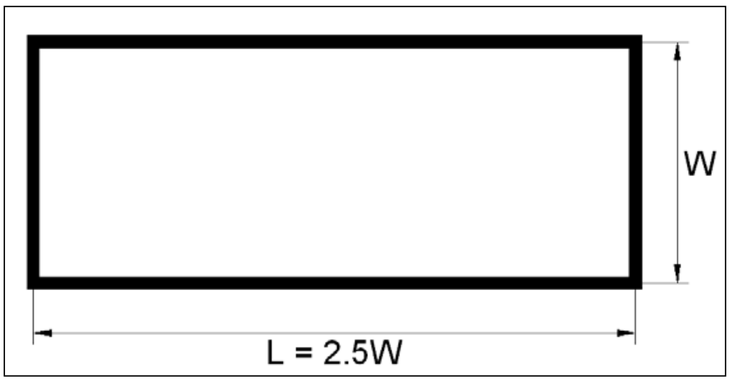


Figure 5.5. Schematic Wetland Orientation

The dimensions of the basin permanent pool can be approximated by solving the following expression:

$$W \times 2.5W = 20{,}499\, ft^3$$
$$W = 90.6\, ft$$
$$L = 226.5\, ft$$

Considering side slopes of 4H:1V, at a depth of two feet above the permanent pool the wetland area is computed as:

$$W = 90.6 + (2)(4)(2) = 106.6\, ft$$
$$L = 226.5 + 16 + (2)(4)(2) = 242.5\, ft$$
$$A = (106.6\, ft)(242.5\, ft) = 25{,}851\, ft^2$$

The storage volume provided between the surface of the permanent marsh and a depth of 2 feet above the marsh is computed by the trapezoidal rule as follows:

$$V = \left[\frac{20{,}499\, ft^2 + 25{,}851\, ft^2}{2} \right] \times 2\, ft = 46{,}350\, ft^3$$

Using the procedures described at length in *Chapter Two – Dry Extended Detention Basin*, we can develop elevation – discharge and elevation – storage relationships. The permanent marsh pool is assumed to be present in the basin at the onset of the 10-year runoff producing event. Therefore, only storage above the marsh surface elevation is considered. The discharge – elevation relationship is for a VDOT SWM-1 riser structure as shown in Figure 5.4. This relationship is shown in Table 5.8 and Figure 5.6.

Wetland Water Elevation (ft)	Basin Outflow (cfs)
2104.00	0.00
2104.50	12.55
2105.00	42.35
2105.50	82.16
2106.00	106.19

Table 5.8. Stage – Discharge Relationship

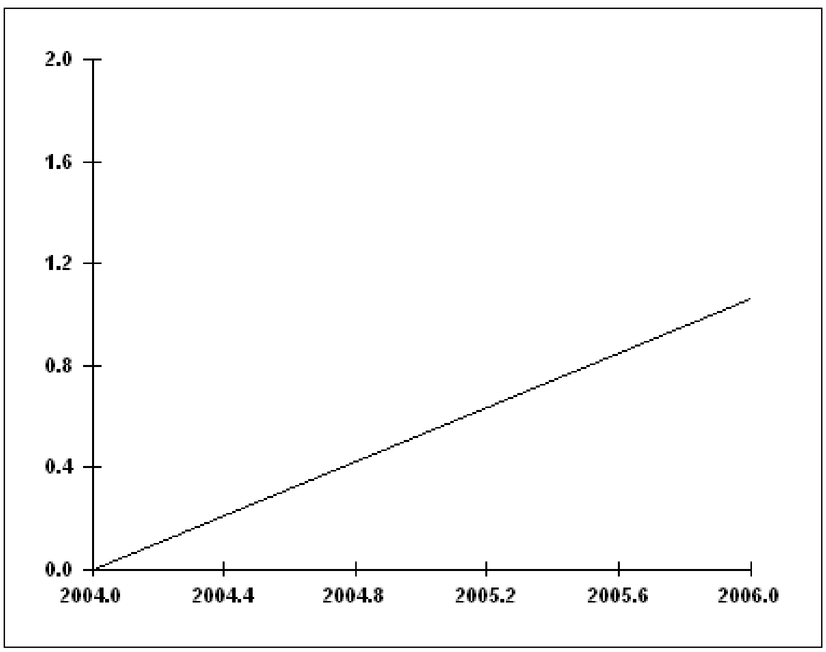
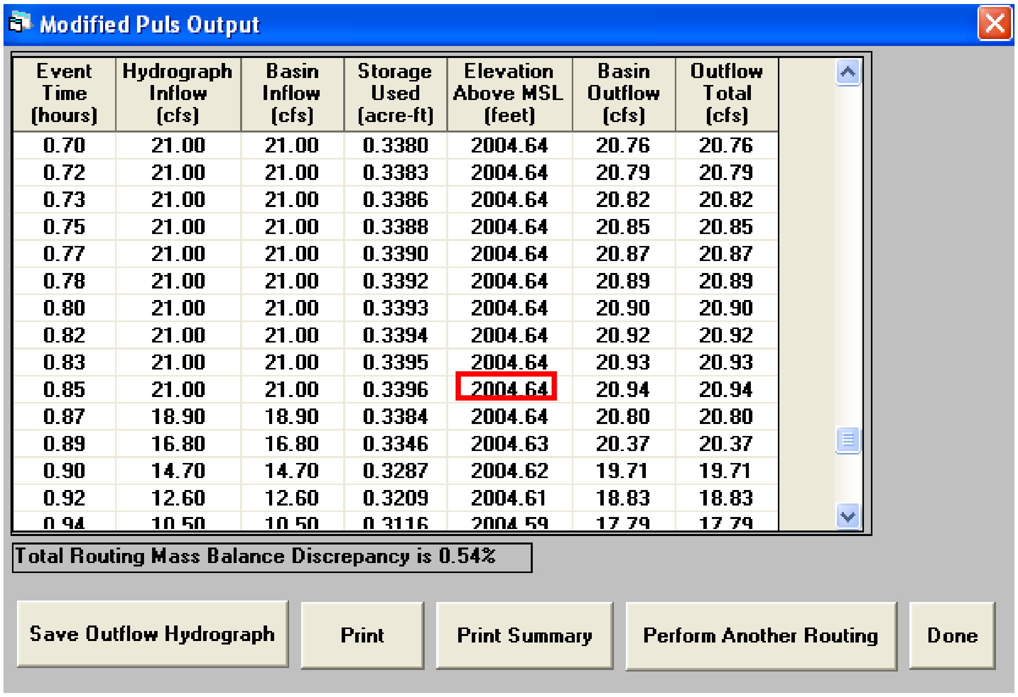


Figure 5.6. Stage – Storage Relationship

Next, we utilize the 10-year return frequency Modified Rational hydrograph from *Chapter Two – Dry Extended Detention Basin* and route it through the wetland. While this Modified Rational hydrograph does not exhibit the maximum volumetric runoff *rate* from the project site, it does reflect the storm event which generates the greatest volume of required storage. It is this event which yields the greatest ponding depth in the wetland, and therefore it must be evaluated. The results of this routing are shown in Figure 5.7.



Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
0.70	21.00	21.00	0.3380	2004.64	20.76	20.76
0.72	21.00	21.00	0.3383	2004.64	20.79	20.79
0.73	21.00	21.00	0.3386	2004.64	20.82	20.82
0.75	21.00	21.00	0.3388	2004.64	20.85	20.85
0.77	21.00	21.00	0.3390	2004.64	20.87	20.87
0.78	21.00	21.00	0.3392	2004.64	20.89	20.89
0.80	21.00	21.00	0.3393	2004.64	20.90	20.90
0.82	21.00	21.00	0.3394	2004.64	20.92	20.92
0.83	21.00	21.00	0.3395	2004.64	20.93	20.93
0.85	21.00	21.00	0.3396	2004.64	20.94	20.94
0.87	18.90	18.90	0.3384	2004.64	20.80	20.80
0.89	16.80	16.80	0.3346	2004.63	20.37	20.37
0.90	14.70	14.70	0.3287	2004.62	19.71	19.71
0.92	12.60	12.60	0.3209	2004.61	18.83	18.83
0.94	10.50	10.50	0.3116	2004.59	17.79	17.79

Figure 5.7. Routing of 10-Year Modified Rational Hydrograph Through Wetland

Figure 5.7 shows the maximum water surface in the wetland as 2004.64. Therefore, the 10-year runoff producing event is conveyed through the wetland with a maximum depth of 0.64 feet above the surface of the wetland marsh. This value is less than the 2.0 feet allowable, and therefore is acceptable.

Step 5. Design of the Submerged Release Outlet

Generally, a constructed wetland facility must be equipped with a means by which baseflow can pass through the wetland without continually accumulating. This conveyance is typically accomplished by a submerged, inverted pipe (see detail in *Chapter Four – Retention Basin*. The submerged outlet pipe should extend into the outlet micro-pool to a depth of approximately 18 inches in order to reduce the likelihood of clogging by debris and floating plant matter.

The first step in computing the required outlet size is to establish the maximum anticipated baseflow which must be conveyed through the wetland once the permanent marsh/pool volume is present. This maximum baseflow arises during the month exhibiting the highest average precipitation. The Virginia State Climatology Office maintains an online database with monthly climate information from various stations across the state. This information can be obtained at:

http://climate.virginia.edu/online_data.htm#monthly

Examining this data for the Montgomery County (Blacksburg) station reveals the month exhibiting the highest average precipitation total as May, with *4.00 inches*.

This precipitation total must now be converted into a runoff rate. This is accomplished by employing the NRCS/SCS runoff depth equation.

The post-development site is comprised of a total of 17.4 acres, 4.75 acres of which is impervious and 12.65 acres of which is unimproved grass cover. Appendix 6H-3 and 6H-4 of the VDOT Drainage Manual contain runoff curve numbers for various land covers and Hydrologic Soil Groups.

The site's Hydrologic Soil Group is *B*. Estimating the site's pervious cover as grass in fair condition, the runoff curve number taken from Appendix 6H-3 is *69*. The curve number for the site's impervious fraction is *98*.

Next, the 2-year 24-hour precipitation depth must be obtained. This information can be obtained from the National Weather Service at:

http://hdsc.nws.noaa.gov/hdsc/pfds/orb/va_pfds.html

Examining this data for the Blacksburg station reveals the 2-year 24-hour precipitation depth, *P*, to be *2.76 inches*.

Next, the SCS runoff depth equations are employed to determine the 2-year 24-hour runoff depth for the post-developed site:

Pervious Fraction

$$S = \frac{1000}{CN} - 10 = \frac{1000}{69} - 10 = 4.49$$

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} = \frac{(2.76 - (0.2)(4.49))^2}{(2.76 + (0.8)(4.49))} = 0.55 inches$$

Impervious Fraction

$$S = \frac{1000}{CN} - 10 = \frac{1000}{98} - 10 = 0.20$$

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} = \frac{(2.76 - (0.2)(0.20))^2}{(2.76 + (0.8)(0.20))} = 2.53 inches$$

The total depth of runoff over the entire developed site is then computed as:

$$\frac{(0.55 inches)(12.65 acres) + (2.53 inches)(4.75 acres)}{17.4 acres} = 1.09 inches$$

The Efficiency of Runoff, *E*, is computed as the ratio of runoff depth to the total depth of precipitation for the 2-year event:

$$E = \frac{1.09 in}{2.76 in} = 0.39$$

Employing this efficiency ratio, the estimated average runoff volume for the month of May is computed as:

$$4.00inches \times 0.39 \times \frac{1\,ft}{12in} \times 17.4ac \times \frac{43{,}560\,ft^2}{ac} = 98{,}533\,ft^3$$

The baseflow rate is then computed as:

$$\frac{98{,}533\,ft^3}{31days} \times \frac{1day}{24hour} \times \frac{1hour}{3{,}600\sec} = 0.04cfs$$

The elevation at which the baseflow bypass outlet begins to discharge from the wetland must be set equal to the elevation corresponding to the surface of the wetland marsh. This ensures that the permanent pool volume is maintained in the wetland at all times, while perennial baseflow is passed through the principal spillway and does not accumulate. Referencing Figure 5.4, we see that the permanent pool volume occurs at elevation 2004. The crest of the baseflow bypass outlet is therefore set at 2004 and sized as follows:

We will initially try a 3-inch diameter orifice, and restrict the maximum head to that occurring just as the outlet becomes submerged. Employing the orifice equation:

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

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

$$a = \pi r^2 = \pi \times \left(\frac{3in/2}{\frac{12in}{ft}} \right)^2 = 0.049\,ft^2$$

The head is measured from the centerline of the orifice. The head when the orifice has just become submerged by a small increment, 0.01 ft, is expressed as:

$$h = 1.5inches \times \frac{1\,ft}{12in} + 0.01\,ft = 0.135\,ft$$

Discharge is now computed as:

$$Q = (0.6)(0.049)\sqrt{(2)(32.2)(0.135)} = 0.09cfs$$

The selected 3-inch diameter orifice will easily convey the perennial baseflow (0.04 cfs) entering the wetland. A smaller diameter orifice would meet the required hydraulic function. However, a smaller orifice would be susceptible to clogging by debris and floating/suspended plant matter and is therefore not recommended.

Step 6. Water Balance Calculation

To ensure that the wetland permanent marsh does not become dry during extended periods of low or absent inflow, the designer must perform a water balance calculation. Two approaches are described in the following section.

Step 6A. 45-Day Drought Condition

The first approach considers the extreme condition of a 45-day drought period with no precipitation and thus no significant surface runoff.

Table 5.9 presents potential evaporation rates for various locations in Virginia.

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

Table 5.9. Potential Evaporation Rates (Inches)
Virginia Stormwater Management Handbook, (DCR, 1999, Et seq.)

The greatest potential evaporation for the station nearest the project site (Roanoke) occurs during the months of July and August, 5.85 inches and 5.30 inches respectively. Therefore, the total evaporation over a 45-day period is estimated as follows:

Average evaporation per month = $\dfrac{5.85in + 5.30in}{2} = 5.58in$

Average evaporation per day = $\dfrac{5.58\dfrac{in}{month}}{31\dfrac{day}{month}} = 0.18\dfrac{in}{day}$

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

$$45\,\text{days}\ X\ 0.18\frac{in}{day} = 8.1in = 0.68\,ft$$

The total surface area of the marsh is 20,499 ft^2. Therefore, the total volume of water potentially lost to evaporation is estimated as:

$$20{,}499\,ft^2 \times 0.68\,ft = 13{,}939\,ft^3$$

The volume of water lost to evaporation must be added to that lost to infiltration. As previously stated, the initial geotechnical tests revealed site soil infiltration rates to be 0.02 inches per hour. The infiltration is assumed to occur over the entire marsh, whose surface area is 15,160 ft^2. The volume of water lost to infiltration is estimated as:

$$20{,}499\,ft^2 \times 0.02\frac{in}{hr} \times \frac{1\,ft}{12in} \times 24\frac{hr}{day} \times 45days = 36{,}898\,ft^3$$

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

$$13{,}939\,ft^3 + 36{,}898\,ft^3 = 50{,}837\,ft^3$$

This value exceeds the total marsh volume of 17,424 ft^3, implying that a 45-day drought period will leave the marsh area in a completely dry state. Over time, it is quite likely that the infiltration rate of the basin soil will decrease considerably due to clogging of the soil pores. However, the aquatic and wetland plant species will likely not survive an extended period of drought that occurs prior to this clogging. Therefore, at this point in the design, it would be recommended to install a clay or synthetic basin liner as approved by the Materials Division. A typical infiltration rate for synthetic liner may be on the order of 3x10^{-7} in/sec. The calculation is repeated for this rate of infiltration.

$$20{,}499\,ft^2 \times 3x10^{-7}\,\frac{in}{\sec} \times \frac{1\,ft}{12in} \times 3{,}600\frac{\sec}{hr} \times 24\frac{hr}{day} \times 45days = 1{,}993\,ft^3$$

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

$$13{,}939\,ft^3 + 1{,}993\,ft^3 = 15{,}932\,ft^3$$

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

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

Total contributing drainage area = 17.4 acres

Stored volume lost to evaporation and infiltration = 15,932 ft^3

$$\frac{15{,}932\,ft^3}{17.4ac \times \frac{43{,}560\,ft^2}{ac}} = 0.02\,Watershed\text{ - }Feet = 0.24\,Watershed\text{ - }Inches$$

A precipitation event yielding a total runoff of 0.24 inches or more across the contributing watershed will replenish the depleted marsh volume.

Step 6B. Period of Greatest Evaporation (in Average Year)

The second water balance calculation examines impacts on the marsh during the one-month period of greatest evaporation during an average year. This calculation reflects an anticipated marsh drawdown during the summer months. In contrast, the first calculation method reflects an extreme infrequent drought event.

From Table 5.9, the greatest monthly evaporation total for the station nearest the project site is 5.85 inches in July. The Virginia State Climatology Office reports an average July rainfall for the Blacksburg station as 3.99 inches (reference Step 5 for link to data).

Applying the previously computed runoff efficiency ratio for the basin watershed, the average July inflow to the basin is computed as:

$$3.99inches \times 0.39 \times \frac{1\,ft}{12in} \times 17.4ac \times \frac{43{,}560\,ft^2}{ac} = 98{,}286\,ft^3$$

Evaporation losses are computed as the product of total monthly evaporation and the surface area of the permanent pool:

$$5.85inches \times \frac{1\,ft}{12in} \times 20{,}499\,ft^2 = 9{,}993\,ft^3$$

Infiltration losses (with synthetic liner) over the entire month of July are estimated as:

$$20{,}499\,ft^2 \times 3x10^{-7}\,\frac{in}{\sec} \times \frac{1\,ft}{12in} \times 3{,}600\,\frac{\sec}{hr} \times 24\,\frac{hr}{day} \times 31days = 1{,}373\,ft^3$$

The water balance expression and total monthly loss/gains are computed as follows:

Monthly loss/gain = Inflow – Evaporation – Infiltration

$$= 98{,}286\, ft^3 - 9{,}993\, ft^3 - 1{,}373\, ft^3 = 86{,}920\, ft^3$$

The monthly climate data and site land cover characteristics indicate that the wetland marsh will not experience drawdown during the average period of highest evaporation.

Step 7. Landscaping

Generally, the non-marsh regions of constructed stormwater wetlands (i.e. the *semi wet* zone) can be landscaped in much the same manner as a typical stormwater impounding facility. However, careful attention must be given to the types of vegetation selected for the wetland marsh areas. For these regions, the vegetative species must be selected based on their inundation tolerance and the anticipated frequency and depth of inundation.

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

The regions of varying depth within the wetland are broadly categorized by zone as shown in Figure 5.8.

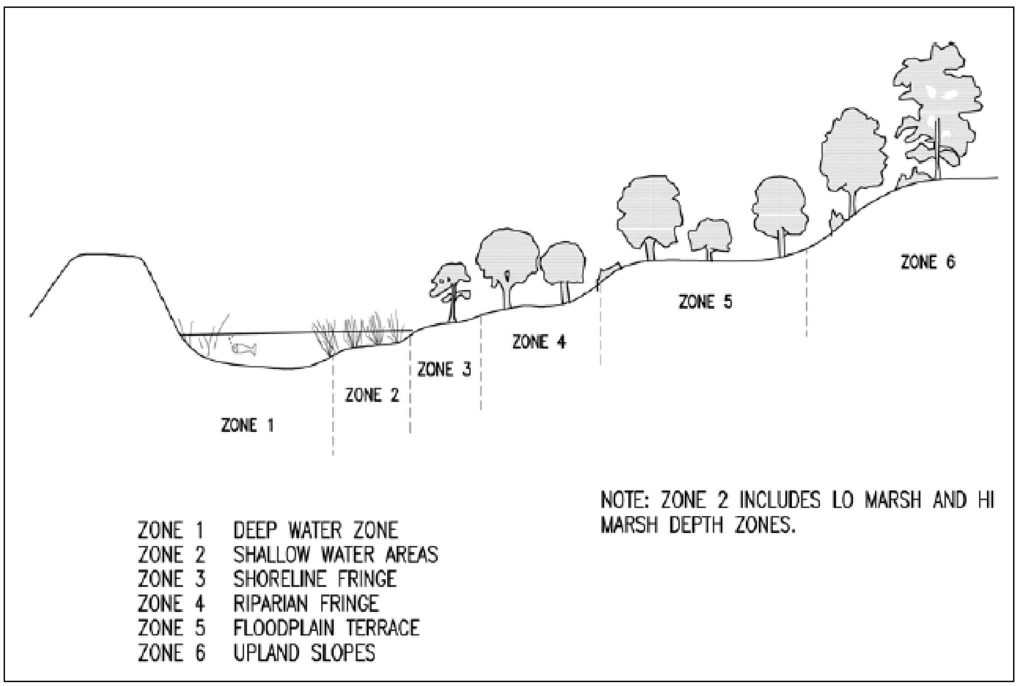


Figure 5.8. Planting Zones for Stormwater BMPs
Virginia Stormwater Management Handbook (DCR, 1999, Et seq.)

Suitable planting species for each of the zones identified in Figure 5.9 are recommended in Chapter 3-05 of the Virginia Stormwater Management Handbook, (DCR, 1999, Et seq.). Ultimately, the choice of planting species should be largely based on the project site's physiographic zone classification. Additionally, the selection of plant species should match the native plant species as closely as possible. Surveying a project site's native vegetation will reveal which plants have adapted to the prevailing hydrology, climate, soil, and other geographically-determined factors. Figure 3.05-4 of the Virginia Stormwater Management Handbook provides guidance in plant selection based on project location.

Generally, stormwater management facilities should be permanently seeded within 7 days of attaining final grade. This seeding should comply with Minimum Standard 3.32, Permanent Seeding, of the Virginia Erosion and Sediment Control Handbook, (DCR, 1992, Et seq.). It must be noted, however, that permanent seeding is *prohibited* in Zones one through four of Figure 5.9. The use of conventional permanent seeding in these zones will result in the grasses competing with the requisite wetland emergent species.

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

All chosen plant species should conform to the American Standard for Nursery Stock, current issue, and be suited for USDA Plant Hardiness Zones 6 or 7, see Figure 5.9.

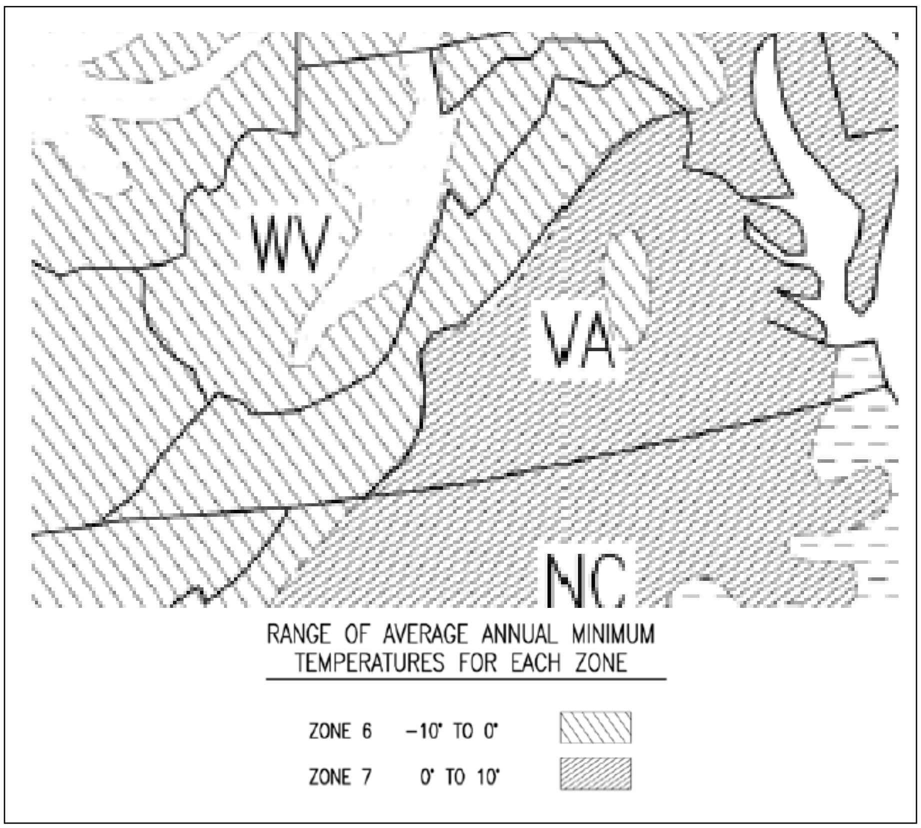


Figure 5.9. USDA Plant Hardiness Zones

If the wetland is equipped with an impounding embankment, under no circumstances should trees or shrubs be planted on the basin embankment. The large root structure may compromise the structural integrity of the embankment.

Chapter 6 – Vegetated Water Quality Swale

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

6.1 Overview of Practice

Vegetated swales are broadly described as surface depressions which collect and convey stormwater runoff from roadways, driveways, rooftops, and other impervious surfaces. However, when applied as a *Best Management Practice*, an engineered grassed swale functions beyond simple collection and conveyance, seeking to also improve the quality of stormwater runoff through sedimentation and filtration. The inherent linear orientation of a vegetated swale makes it an attractive option for treatment and conveyance of highway runoff.

Vegetated swales function by minimizing flow velocity and inducing ponding behind strategically placed check dams. While infiltration of some runoff associated with ponding can attenuate peak runoff rates, this attenuation can be considered minimal at best. Vegetated swales are water quality improvement practices, and cannot be considered effective flood control strategies.

The Virginia Stormwater Management Handbook, (DCR, 1999) identifies two categories of vegetated conveyance BMPs – “Grassed Swales” and “Water Quality Swales” (Minimum Standard 3.13). Grassed swales, also termed “dry swales,” function by slowing the velocity of runoff and inducing ponding behind strategically placed check dams. The swale’s controlled velocity permits filtration of runoff pollutants by the dense vegetation lining the channel. Ponding increases the hydraulic residence time within the swale, thus providing an increased opportunity for the gravitational settling of pollutants. Water quality swales, or wet swales, can be conceptualized as a linear wetland. Their underlying soils, in contrast to dry swales, are comprised of a very specific mixture in order to permit controlled infiltration as well as the growth of wetland vegetation. The rigid underlying soil characteristics of a wet swale will typically require native site soils to either be amended or excavated completely and replaced with imported material. While wet water quality swales are considered capable of achieving phosphorus removal beyond that of dry swales, they are best suited for contributing drainage areas whose impervious cover ranges from 16 – 37%. When a project site’s impervious cover enters that range, there will be a need for flood control in the form of mitigation of post-developed runoff rates to those of pre-developed levels. The inability of a wet water quality swale to also provide peak attenuation will generally render it cost prohibitive, with BMPs capable of providing both water quality improvement and peak mitigation preferred. Therefore, as evidenced in Table 1.1, the VDOT BMP selection table only considers the grassed, or dry, variation of a water quality swale.

6.2 Site Constraints and Siting of the Facility

In addition to the contributing drainage area's impervious cover, a number of site constraints must be considered when the implementation of a grassed swale is proposed. These constraints are discussed as follows.

6.2.1 Minimum Drainage Area

The minimum drainage area contributing to a vegetated swale is not restricted. Vegetated swales are particularly well suited to small drainage areas.

6.2.2 Maximum Drainage Area

The water quality improvement function of a vegetated swale is predicated on its ability to maintain minimal flow velocities within the channel. Therefore, within the confines of feasible cross-sectional areas, such channels cannot simultaneously be designed to convey large flow rates and/or volumes. The channel cross-section geometry, roughness, longitudinal slope, and design discharge will ultimately dictate flow velocity within the channel. The design discharge is a function of the contributing drainage area, and therefore the area must be limited such that desired velocities are maintained. In addition to meeting velocity restrictions (discussed later), the swale must be designed to convey the 10-year flow with a minimum of six inches of freeboard.

6.2.3 Site Slopes

Sites on which a vegetated swale is proposed should exhibit relatively flat topography. The maximum permissible slope of a grassed swale is *six percent*. Alternative BMPs should be considered when site topography is such that this maximum slope is exceeded. Grassed swales function best when their slope is a flat as practically possible.

6.2.4 Site Soils

The implementation of a grassed swale can be successfully accomplished in the presence of a variety of soil types exhibiting at least moderate permeability. However, when such a practice is proposed, *a* permeability test is strongly recommended. This data should be provided to the Materials Division early in the project planning stages to determine if a grassed swale is feasible on native site soils. Because ponding is induced within the swale, site soils should permit the emptying of the swale through infiltration. The inability of native site soils to completely drain a swale within a period of less than 72 hours can introduce undesirable marshy conditions and mosquito habitat. The minimum soil infiltration rate considered for construction of a grassed swale is 0.27 inches per hour. Soils underlying a vegetated grass should be USDA *ML, SM, or SC*. Sites exhibiting sandy soils should conform to *ASTM C-33*, VDOT fine aggregate grading *A* or *B*, or as otherwise approved by the Materials Division.

6.2.5 Depth to Water Table

Grassed swales inevitably infiltrate detained runoff into the subsurface. The infiltrated runoff may potentially carry a significant pollutant load. Therefore, grassed swales should not be used on sites exhibiting a seasonally-high water table of less than two feet below the proposed swale bottom.

6.2.6 Existing Utilities

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

6.2.7 Wetlands

When the construction of a grassed swale is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify the wetland boundaries, their protected status, and the feasibility of BMP implementation in their vicinity. The presence of existing wetlands may reveal native soils capable of accommodating a wet water quality swale at the site.

6.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing a vegetated swale for improvement of water quality.

6.3.1 Swale Geometry

Because the fundamental goal of a grassed swale is to improve the *quality* of runoff, it is essential to avoid any concentration of the flow within the channel. In addition to presenting problems of constructability, parabolic and triangular channels will concentrate low flows, and thus are undesirable. Similarly, rectangular channels should be avoided because of the inherent instability of their side slopes. Therefore, to satisfy both the issues of constructability and that of desired flow regime, only *trapezoidal cross section* channels are considered. Channel side slopes should be no steeper than 3H:1V.

6.3.2 Bottom Width

Channel bottom widths of less than two feet are essentially non-constructible, and should not be considered. Conversely, bottom widths greater than six feet will tend to concentrate small flow events thereby reducing the pollutant removal ability of the swale. With a range of two to six feet established as acceptable, the precise channel bottom width becomes largely a function desired flow depth. This topic is discussed later in this section in the context of an example swale design.

6.3.3 Channel Depth

The swale should be designed such that the water quality volume flows at a depth approximately equal to the grass height. For most applications this will be four inches. The overall depth should permit conveyance of the 10-year runoff event while providing a minimum of six inches of freeboard. Additionally, channel depth should be such that the check dam height does not exceed one half of the total channel depth.

6.3.4 Longitudinal Slope

The generally accepted minimum constructible slope is 0.75%. The slope of a grassed swale should be as flat as practically possible for the given site topography. The site-specific allowable longitudinal slope will ultimately be governed by the desired flow depth and velocity. In general, however, this maximum slope should not exceed six percent.

6.3.5 Flow Velocity

The flow velocity should be as low as practically possible in order to achieve maximum pollutant removal. Additionally, the swale must be designed such that larger runoff events do not result in re-suspension of previously deposited sediments. The following design velocities should be met:

Design Flow	Permissible Velocity (fps)
2-year	4
10-year	7

Table 6.1. Permissible Flow Velocities

Source: Virginia Stormwater Management Handbook, (DCR, 1999)

6.3.6 Shear Stress

In addition to considering the velocity in the channel, the shear stress exhibited by the flow must also be examined. Table 5.2 presents permissible shear stresses for five different classes of vegetative linings. These classes are further described later in the context of a design example.

		Permissible Shear Stress, τ_p	
Lining Category	**Lining Type**	**lb/ft^2**	**kg/m^2**
Vegetative	Class A	3.70	18.06
	Class B	2.10	10.25
	Class C	1.00	4.88
	Class D	0.60	2.93
	Class E	0.35	1.71

Table 6.2. Permissible Shear Stresses

Source: FHWA/Chen and Cotton (1988)

6.3.7 Swale Length

The length of a grassed swale is not restricted, but rather must be sized together with the channel cross-sectional area and check dam height to provide the desired water quality storage volume.

6.3.8 Discharge Flows:

When a grassed swale empties into an existing swale or other surface conveyance system, the receiving channel must be evaluated for adequacy as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR, 1992). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

6.4 Design Process

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

The following swale design will provide the technology-based water quality requirements arising from the construction of approximately 1,800 linear feet of secondary subdivision roadway in the City of Hampton. Topography is such that runoff from the road is collected in VDOT CG-6 curb and gutter and conveyed to curb inlets in a sump near the mid station of the road. The runoff is then discharged into the proposed swale. The total project site, including right-of-way and all permanent easements, consists of 5.27 acres. Pre and post-development hydrologic characteristics are summarized below in Tables 5.3 and 5.4. The project site exhibits topography typical of the coastal region of Virginia, with slopes less than two percent. Site constraints limit the swale length to 275 feet.

	Pre-Development	Post-Development
Project Area (acres)	5.27	5.27
Land Cover	Unimproved Grass Cover	1.03 acres impervious cover
Impervious Percentage	0	19.5

Table 6.3. Hydrologic Characteristics of Example Project Site

		York County - 10 Year				
Acreage	**Rational C**	**A Constant**	**B Constant**	**t_c (min)**	**i_{10} (iph)**	**Q_{10} (cfs)**
1.03	0.9	186.78	21.22	8	6.39	5.9

York County - 2 Year				
A Constant	**B Constant**	**t_c (min)**	**i_2 (iph)**	**Q_2 (cfs)**
122.93	16.72	8	4.97	4.6

Table 6.4. Peak Roadway Runoff

Step 1. Compute the Required Water Quality Volume

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

$$WQ_V = \frac{IA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

IA= Impervious Area (ft^2)

The project site in this example is comprised of a total drainage area of 5.27 acres. The total impervious area within the site is 1.03 acres (19.5 percent of the total site area). Therefore, the water quality volume for this site is computed as follows:

$$WQ_V = \frac{1.03ac \times \frac{1}{2} in \times \frac{43{,}560\ ft^3}{ac}}{12 \frac{in}{ft}} = 1{,}870\ ft^3$$

A vegetated swale must be sized to provide ponding for the computed water quality volume. This ponding occurs behind check dams (height and longitudinal spacing discussed later).

Step 2. Determine the Cross-Sectional Dimensions of the Channel

Ponding in the swale will occur behind check dams 18" in height. Because the cross-sectional size and configuration of the channel remain constant throughout its length, the total volume of water detained throughout the swale can be estimated by the average end area method. This volume calculation simply averages the wet cross-sectional area at the upstream and downstream ends of the channel and computes the stored volume as the product of this average area and the channel length. This approach assumes that the available ponding depth at the downstream end of the channel is equal in depth to the check dam height. The depth of water at the most remote upstream point in the channel is assumed to be zero. For a trapezoidal channel with 3:1 side slopes and 18" (1.5') check dams, the downstream wet cross-sectional area is computed as:

$$A = (w_b)(1.5) + (2)\left(\frac{1}{2}\right)(1.5)(3)(1.5)$$

With: w_b= channel base width (ft)

Because the ponded upstream depth is zero, the effective cross sectional area of the swale is one half this value, expressed as:

$$A_{avg} = \frac{(w_b)(1.5) + (2)\left(\frac{1}{2}\right)(1.5)(3)(1.5)}{2}$$

The design is continued for a total channel length of 275 ft, longitudinal slope of 2%, and side slopes of 3:1. The required average cross-sectional area of the channel is computed by dividing the required water quality volume by the channel length.

$$A_{avg} = \frac{1{,}870\ ft^3}{275\ ft} = 6.80\ ft^2$$

Rearranging the earlier channel cross-sectional area expression in terms of base width, w_b:

$$w_b = \frac{2A_{avg} - (1.5)(3)(1.5)}{1.5}$$

The required channel base width is then computed as:

$$w_b = \frac{(2)(6.80) - (1.5)(3)(1.5)}{1.5} = 4.56\ ft$$

To address any underestimation in storage volume arising from the average end computation, the base width of the channel is increased to *five feet*.

Step 3. Determine the Depth of the Channel

The ten-year flood peak, Q_{10}, is selected as the design discharge for establishing the conveyance properties of the channel, while providing a minimum six inches of freeboard. The presence of check dams in the swale introduces difficulty in modeling flow through the channel. Two approaches are presented in this example for determining the required channel depth. The first approach conceptualizes the swale as linear detention facility, with storage-indication routing employed to establish the maximum water surface elevation under 10-year runoff producing conditions. This approach yields accurate results, yet is computationally intensive. The second approach simply ignores the presence of check dams and computes the normal depth in the channel under 10-year flow conditions. This computed normal depth is added to the check dam height and the required six inch freeboard. While computationally simpler, the second approach tends to oversize the channel because it does not consider that a significant portion of the 10-year runoff volume is detained behind the check dams and, thus not contributing to computed *flow* depth.

Step 3A. Channel Depth – Method 1

Because water is ponded in the swale behind 18” check dams, the swale behaves much like a detention facility, with flow through the swale occurring as weir flow over the check dams. Thus a reasonable approach to determining the required swale depth is to perform storage indication routing. This approach yields the maximum water surface elevation under 10-year inflow conditions. Adding 6” of freeboard to this depth provides the minimum swale depth.

The first step is to establish a stage – storage relationship for the swale. Storage volumes are computed based on channel geometry, with all variables as defined:

$$V = \left[\frac{(w_b)(d) + (2)\left(\frac{1}{2}\right)(d)(Z)(d)}{2} \right] \times L$$

V = ponded volume (ft^3)
w_b = channel base width (ft)
d = ponded depth (ft)
Z = channel side slope (ZH:1V)
L = channel length (ft)

Employing the previously established channel parameters, the ponded volume can be computed solely as a function of ponded depth:

$$V = \left[\frac{(5)(d) + (2)\left(\frac{1}{2}\right)(d)(3)(d)}{2} \right] \times 275$$

This calculation is employed for various incremental depths. The results are shown in Table 5.5 below, assuming a downstream bottom channel elevation of 300 ft mean sea level (MSL). Note that the approximate water quality volume is provided at a depth of 1.5 feet, equaling the check dam height.

Elevation	Volume (ft^3)
300	0
300.5	447
301	1,100
301.5	1,959
302	3,025
302.5	4,297
303	5,775
303.5	7,459
304	9,350
304.5	11,447
305	13,750
305.5	16,259
306	18,975

Table 6.5. Swale Stage – Storage Relationship

Next, the stage – discharge relationship is constructed. The channel check dams function as broad-crested weirs. At a depth of 18", the weir length is calculated as follows, with parameters as previously defined:

$$L = w_b + (2)(d)(z)$$
$$= 5\,ft + (2)(1.5\,ft)(3) = 14\,ft$$

Discharge over a broad-crested weird is a function of the head acting on the weir crest. The weir equation is as follows, and used to establish the stage – discharge relationship shown in Table 5.6. Note there is no flow occurring below the check dam crest elevation.

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

Q = Weir discharge (cfs)
C_W = Weir coefficient (3.0)
L = Weir length (14 ft)
h = hydraulic head acting on weir crest (ft)

Elevation	Discharge (cfs)
301.5	0
302	15
302.5	42
303	77
303.5	119
304	166
304.5	218
305	275
305.5	336
306	401

Table 6.6. Swale Stage – Discharge Relationship

Next, using the stage – storage data, stage – discharge data, and the 10-year return frequency post-development runoff hydrograph, storage-indication routing is performed to determine the actual water surface elevation observed in the swale during this event. Figure 5.1, below, illustrates the 10-year post-development runoff hydrograph developed using the NOAA NW-14 regional rainfall I-D-F parameters recommended in the VDOT Drainage Manual.

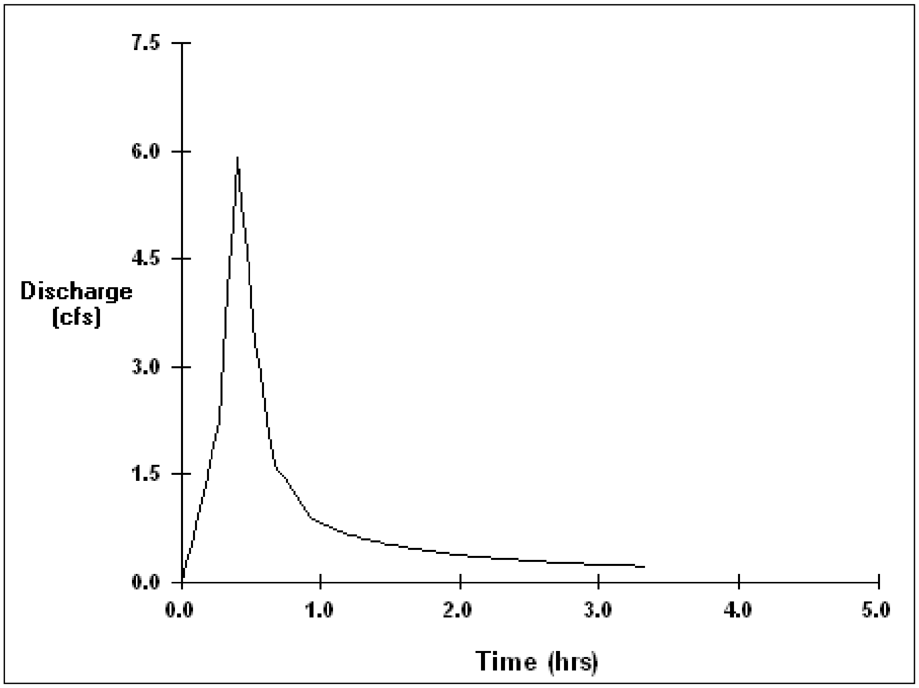


Figure 6.1. 10-Year Post-Development Flow Entering Swale

Figure 5.2 on the following page illustrates the results of the storage-indication routing operation.

Modified Puls Output

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
0.23	1.96	1.96	0.0184	300.78	0.000	0.000
0.27	2.26	2.26	0.0242	300.97	0.000	0.000
0.30	3.20	3.20	0.0317	301.17	0.000	0.000
0.33	4.13	4.13	0.0418	301.42	0.000	0.000
0.37	5.06	5.06	0.0515	301.63	2.17	2.17
0.40	5.99	5.99	0.0567	301.74	5.11	5.11
0.43	5.34	5.34	0.0575	301.76	5.64	5.64
0.47	4.69	4.69	0.0566	301.74	5.05	5.05
0.50	4.03	4.03	0.0556	301.72	4.40	4.40
0.53	3.38	3.38	0.0546	301.70	3.74	3.74
0.57	2.95	2.95	0.0536	301.68	3.27	3.27
0.60	2.52	2.52	0.0528	301.66	2.83	2.83
0.63	2.09	2.09	0.0519	301.64	2.40	2.40
0.67	1.67	1.67	0.0511	301.63	1.97	1.97
0.70	1.58	1.58	0.0505	301.61	1.68	1.68

Figure 6.2. Routing of 10-Year Flow Through Swale

The routing reveals a maximum flow depth of 1.76 feet, equal to 0.26 feet (3.12 inches) over the check dams. Therefore, the minimum swale depth is computed as the sum of the computed water depth and the required freeboard:

$$1.76\,ft + 0.5\,ft = 2.26\,ft = 27.12in$$

Step 3B. Channel Depth – Method 2

An alternative approach for determining the necessary swale depth is to compute the normal flow depth observed during the 10-year runoff producing event, under the assumption that there is water stored behind each check dam at the onset of the 10-year runoff event. This depth is then added to the check dam height and the required freeboard depth to determine the minimum swale depth. This is a conservative approach, as it does not consider that a significant portion of the 10-year runoff volume is detained behind the check dams and, thus not contributing to computed flow depth.

The computed 10-year post-development runoff exhibits a peak discharge of 5.9 cfs. The first step is to compute the flow depth (normal depth) of the 5.9 cfs discharge in the proposed channel. This task is accomplished by employing both the continuity and Manning's equations.

In order to apply Manning's Equation, the roughness coefficient of the channel must first be established. This coefficient can be estimated initially and then adjusted as needed to satisfy flow velocity and hydraulic radius requirements. It is an iterative process since these hydraulic parameters depend, in turn, on the Manning's n value.

The first step in computing the Manning roughness coefficient is to estimate the retardance class of the vegetation lining the channel. The channel retardance factor is based on the type of vegetative lining, and can be found in Table 5.7.

For this example, the proposed swale will be seeded with Kentucky bluegrass and maintained at a height of approximately six inches. This vegetative cover falls in retardance class C.

The next step is to select an initial value of Manning's *n* and then estimate the product of the flow velocity and hydraulic radius (VR_h) in the channel, using the following SCS graph.

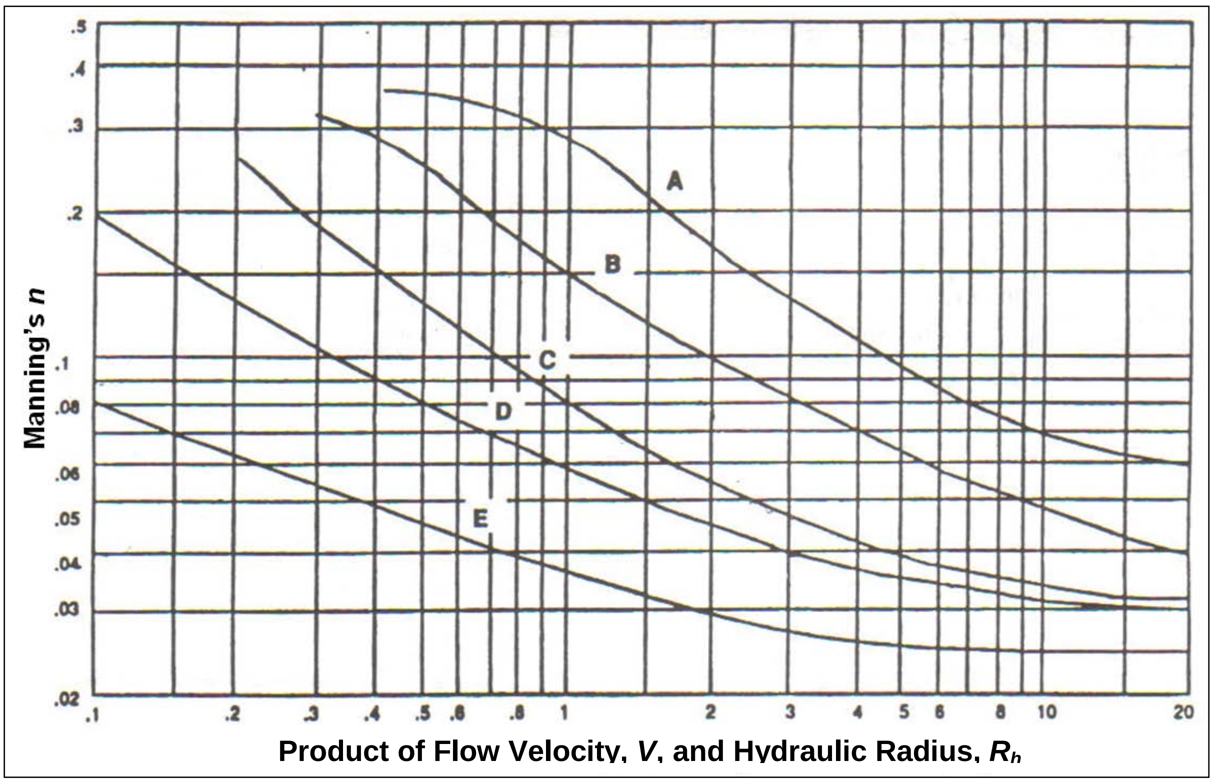


Figure 6.3. Relationship of Manning's n to VR_h

Sources: U.S. Department of Transportation. Federal Highway Administration. <u>Evaluation and Management of Highway Runoff Water Quality</u>. Washington, D.C., 1996. Presents part of SCS Tech. Paper 61, 1954.

USDA, Soil Conservation Service, Technical Paper 61, Handbook of Channel Design for Soil and Water Conservation, 1954.

Retardance Class	Cover	Condition
A	Weeping Lovegrass	Excellent stand, tall (average 30in [76cm])
	Yellow bluestem *Ischaemum*	Excellent stand, tall (average 36in [91cm])
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12in [30cm])
	Native grass mixture	Good stand, unmowed
	(little bluestem, bluestem, blue gamma,	
	and other long and short midwest grasses)	
	Weeping Lovegrass	Good stand, (average 24in [61cm])
	Lespedeza sericea	Good stand, not woody, tall (average 19in [48cm])
	Alfalfa	Good stand, uncut (average 11in [28cm])
	Weeping Lovegrass	Good stand, unmowed (average 13in [28cm])
	Kudzu	Dense growth, uncut
	Blue gamma	Good stand, uncut (average 11in [28cm])
C	Crabgrass	Fair stand, uncut (10-48in [25-120cm])
	Bermuda grass	Good stand, mowed (average 6in [15cm])
	Common lespedeza	Good stand, uncut (average 11in [28cm])
	Grass-legume mixture -- summer	Good stand, uncut (6-8in [15-20cm])
	(orchard grass, redtop, Italian ryegrass,	
	and common lespedeza)	
	Centipedegrass	Very dense cover (average 6in [15cm])
	Kentucky bluegrass	Good stand, headed (6-12in [15-30cm])
D	Bermuda grass	Good stand, cut 2.5in height (6cm)
	Common lespedeza	Excellent stand, uncut (average 4.5in [11cm])
	Buffalo grass	Good stand, uncut (3-6in [8-15cm])
	Grass-legume mixture -- fall	Good stand, uncut (4-5in [10-13cm])
	(orchard grass, redtop, Italian ryegrass,	
	and common lespedeza)	
	Lespedeza sericea	After cutting to 2in in height (5cm)
		Very good stand before cutting
E	Bermuda grass	Good stand, cut to 1.5in in height (4cm)
	Bermuda grass	Burned stubble

Table 6.7. Classes of Retardance by Vegetation Type and Height
Source: Adapted from Mays (2005), and FHWA (1996).

Employing an initial trial Manning's roughness coefficient of 0.10, Figure 5.3 yields an estimated value of VR_h as 0.73 ft^2/s. Next, the actual value of VR_h corresponding to a roughness coefficient of 0.10 is computed. The actual VR_h value is determined using the Manning's equation as follows:

$$VR_h = \frac{1.49}{n} R_h^{\,1.67} S^{\,0.5}$$

The following flow parameters are considered for this example:

Channel base width	5ft
Channel side slopes	3H:1V
Channel longitudinal slope	2.00%
Manning's Roughness Coefficient	0.10
Design Discharge	5.9 cfs

Employing VTPSUHM to solve the Manning's equation for these parameters yields the following results:

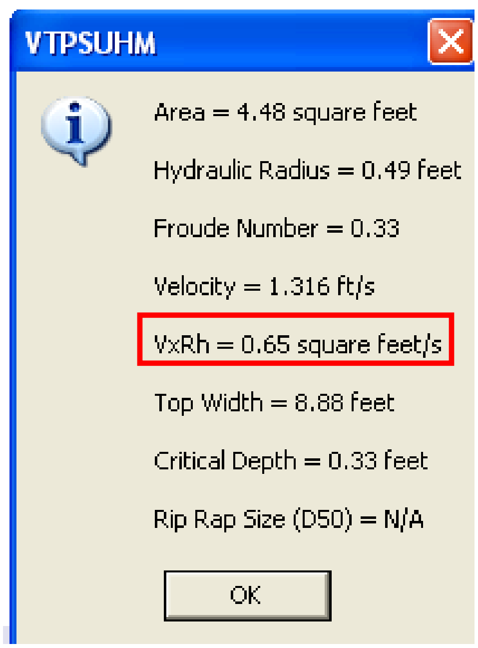


Figure 6.4. Results of Initial Manning's Roughness of 0.10

The product of the flow velocity and hydraulic radius is found to be 0.65 ft^2/s. This value is now used to determine a new Manning's roughness value from Figure 5.3. Entering Figure 5.3 with a VR_h value of 0.65 ft^2/s and a vegetative retardance class of C yields a roughness coefficient of 0.12.

Employing the new roughness coefficient with all previously defined flow and channel size parameters yields the following results:

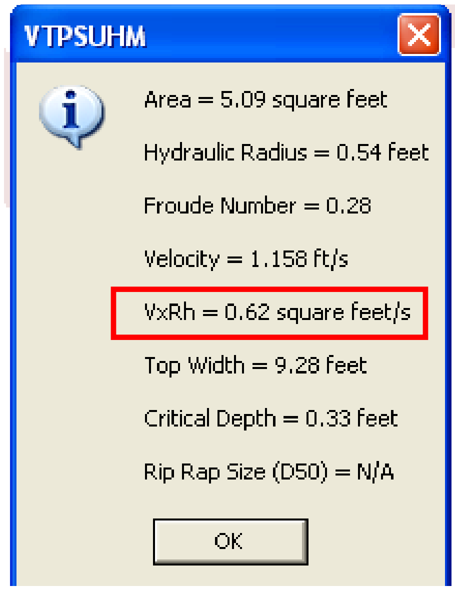


Figure 6.5. Results of Second Manning's Roughness of 0.12 (Q_{10})

The new product of the flow velocity and hydraulic radius is found to be 0.62 ft^2/s. This value is less than five percent different than the estimated value of 0.65 ft^2/s, and thus is acceptable. Had the results yielded a discrepancy of greater than five percent, subsequent iterations would have been carried out until convergence was observed.

With an acceptable Manning's roughness coefficient established, the next step is to compute the required channel depth. Employing the aforementioned flow parameters, we now compute the 10-year flow depth (normal depth) in the channel by Manning's equation. The VTPSUHM results of this calculation are shown as follows.

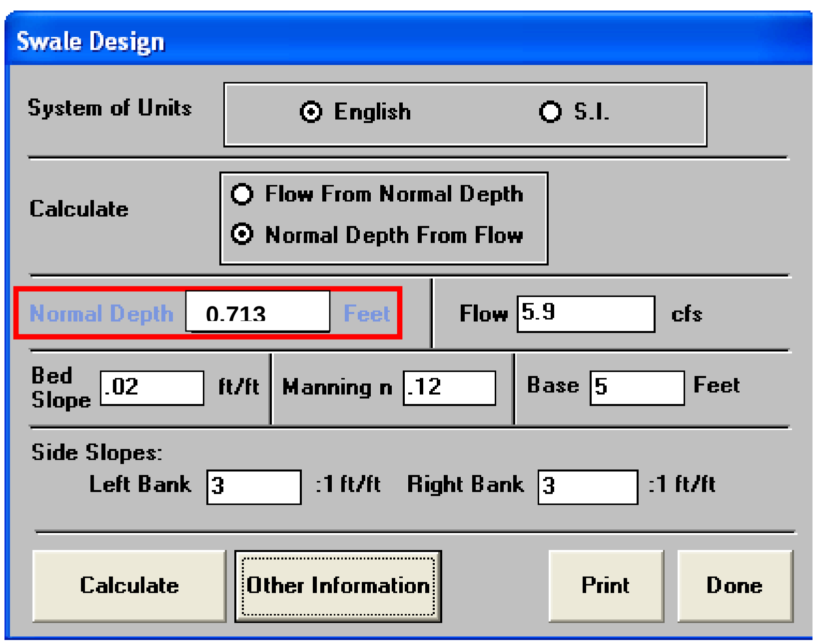


Figure 6.6. Results of Normal Depth Calculation (Q_{10})

The output exhibits a 0.713 ft flow depth (normal depth) for the 10-year return frequency discharge.

Examining the VTPSUHM output (Figure 5.5) on the previous page reveals that the flow velocity of 1.16 fps is less than the maximum allowable velocity of 7 fps for the 10-year return frequency flow.

The minimum depth of the channel can now be computed by summing the segmental depths, based on the conservative assumption that there is an 18-inch ponded depth in the swale prior to the arrival of the 10-year storm hydrograph. The Q_{10} normal depth will then be added to the ponded depth under this assumption.

$$d_{min} = d_{Ponded} + d_{10\text{ yr. storm}} + d_{Freeboard}$$

$$d_{min} = 1.5\mathit{ft} + 0.71\mathit{ft} + 0.5\mathit{ft} = 2.71\mathit{ft} = 32.5\mathit{in}$$

This approach yields a required channel depth predictably greater than that found by storage indication routing.

The next step is to evaluate the 2-year flow conditions for compliance with the maximum permissible flow velocity of 4 fps. Employing VTPSUHM to perform the Manning's equation calculation:

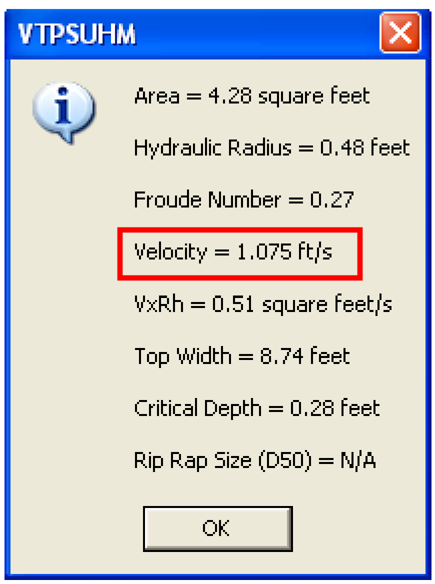


Figure 6.7. Flow Parameters (Q_2)

The output reveals that the flow velocity of 1.08 fps is less than the allowable velocity of 4 fps for the 2-year return frequency discharge. Additionally, it should be noted that the Froude number of 0.27 indicates a sub critical flow regime. Designs for which the Froude number approaches unity should be avoided.

Step 3C. Channel Depth – Method 3

A third alternative for computing the required channel depth was developed by Dr. Osman Akan, Associate Dean of Engineering and Professor of Civil Engineering at Old Dominion University. First reported in 2001 by Akan and Hager in the ASCE *Journal of Hydraulic Engineering*, this method employs charts developed from a dimensionless form of the Manning equation. Application of these charts permits a direct solution of channel depth and width. The results obtained by this method are, generally, comparable to the previously described Method 2 normal depth calculation. However, for side slopes milder than 2:1, the Akan direct solution approach may overdesign the swale size by approximately 5%. Readers interested in applying the Akan direct solution method are referred to:

Akan, A. O. (2006). Open Channel Hydraulics. Elsevier/Butterworth-Heinemann, Burlington, MA, ISBN-13:978-0-7506-6857-6 and ISBN-10: 0-7506-6857-1

Table 5.8 summarizes the computed channel depth for the three design approaches.

Design Method	Computed Swale Depth (ft)
1 - Hydrograph Routing	2.26
2 - Normal Depth Calculation	2.71
3 - Akan-Hager Direct Solution Method	2.72*

Table 6.8. Summary of Computed Channel Depth

*Computed value provided by Akan (personal communication).

It should be noted at this point that, (adhering to previously established design guidelines) the channel check dam height should not exceed one half of the total channel depth. The check dams employed in this design were assumed to be 18 inches in height. Therefore, the minimum channel depth that should be considered is three feet. Per the calculations presented in Step 3, a channel depth of three feet yields a conservative design which provides more than the minimum six inches of a freeboard under 10-year inflow conditions. The check dam height could be reduced, but doing so would necessarily require an increased channel cross-sectional area to provide storage for the computed water quality volume. Increased channel area results in a need for greater right-of-way acquisition, and this is generally undesirable. A channel depth of three feet is therefore adopted.

Step 4. Ensure Allowable Levels of Shear Stress

The final step in verifying the adequacy of the proposed design is a check to ensure that the shear stress exhibited by the flow does not exceed the allowable values previously presented (Table 5.2).

The average shear stress associated with the flow is given by the following equation:

$$\tau_{Design} = \gamma R S_0$$

γ = specific weight of water ($62.4^{lb}/_{cf}$)
R = design hydraulic radius for the 10-year event (ft)
S_0 = channel longitudinal slope (ft/ft)

We note parenthetically that due to non-unifrom velocity distribution in the cross section, the maximum shear stress developed on the bed and sides of most trapezoidal channels of practical interest will be approximately 1.0 and 0.75 times the average shear, respectively. (Chow,1959).

The output from the 10-year flow reveals a hydraulic radius of 0.54 ft. Employing the previously presented equation, shear stress on the channel is found as follows:

$$\tau_{Design} = (62.4\frac{lb}{ft^3})(0.54\,ft)(.020\frac{ft}{ft}) = 0.67\frac{lb}{ft^2}$$

For a vegetative lining with a Class C retardance factor, the permissible shear stress is 1 lb/sf. Thus, the proposed design is acceptable.

Step 5. Investigation of Alternative Swale Designs

Best Hydraulic Section

In the design of non-erodible *st*ormwater conveyance channels, the concept of the best hydraulic section is often employed. The best hydraulic section is the channel configuration for which wetted perimeter is minimized for a fixed cross-sectional area and desired discharge. In other words, the hydraulic radius is maximized. The best hydraulic section exhibits side slopes of 0.58:1. These excessively steep side slopes lend themselves well to concrete or other manmade systems, but are usually impractical for vegetated swales.

For the swale of interest in this design (base width of 5 ft and side slopes of 3:1), computing the swale depth by the best hydraulic section methodology yields a value of 15.4 feet. While potentially useful as a starting design point, best hydraulic section methodology will usually require significant modification to section properties to accommodate local site conditions. Design of an erodible channel, such as the vegetated water quality swale, should be carried out according to allowable shear stress principles, as shown in the above example.

Vegetated Swale Without Check Dams

Another design possibility is to construct the swale with no check dams. The primary purpose of the check dams is to level the grade, decrease erosion, and increase the contact time for the flow as it passes through the vegetative cover. Without check dams the length of equivalent swale must increase. For many sites, this alternative will not be feasible because of the excessive length required to achieve an acceptable hydraulic residence time for the flow entering the channel. This length calculation is shown as follows:

$$L = V\,T_r\,(60s/min)$$

L = Required swale length (ft)
V = Flow velocity for the 10-year return event (ft/s)
T_r = Hydraulic residence time in minutes (9minutes minimum, FHWA, 1996)

Previous calculations show a flow velocity of 1.2 ft/s for the 10-year return event. For the example presented here, the required swale length is calculated as:

$$L = (1.2\ ft/s)(9min)(60\ s/min) = 648\ ft$$

When vegetated swales employ check dams, ponding results in easy attainment of the 9 minute hydraulic residence time. Consequently, swale length can be reduced greatly, as illustrated in the initial design where the length was 275 feet. BMP swales without check dams are intended to serve only as a single treatment step in a series of multiple BMPs. In the absence of check dams, infiltration of runoff in the swale is negligible.

Step 6. Check Dam Design

Check dam materials and construction techniques shall conform to those described in Minimum Standard 3.13 of the Virginia Stormwater Management Manual (DCR, 1999). All check dams shall be equipped with toe protection as described in Minimum Standard 3.13. When the check dam material is riprap or gabion baskets, the check dams shall be underlain by a filter fabric approved by the Materials Division.

Check dams shall be placed longitudinally in the channel such that the dam height and the channel slope combine to provide the desired water quality volume. After establishing the swale dimensions as previously outlined, the total number of check dams required is computed as follows:

$$L_d = \frac{H}{S}$$

L_d = longitudinal distance behind each check dam (ft)
H = depth of ponding behind check dam (ft)
S = channel longitudinal slope (ft/ft)

$$L_d = \frac{(18'')(\frac{1\,ft}{12''})}{0.02} = 75\,ft$$

The total number of check dams is then computed by dividing the overall swale length by L_d:

$$\#\,Dams = \frac{275\,ft}{75\,ft} = 3.67$$ Use four check dams

In addition to providing a minimum of six inches of freeboard during 10-year flow conditions, the check dams should be equipped with a notch to ensure that the 2-year flow does not contact the check dam abutments. At the check dam height of 18 inches, the channel width is 14 feet. Providing 6 inches of abutment freeboard on each end, the 2-year flow notch can be evaluated as a broad-crested weir of length 13 feet. The required depth of the notch can then be determined by the weir equation as follows.

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

Rearranging the equation to solve for head:

$$h = \left[\frac{Q}{C_W L}\right]^{\frac{2}{3}}$$

The peak 2-year discharge is 4.6 cfs, and the flow depth, h, is computed as:

$$h = \left[\frac{4.6}{(3.0)(13.0)}\right]^{\frac{2}{3}} = 0.24\,ft = 2.9in$$

Therefore, a notch 2.9 inches or greater in depth will ensure that the 2-year flow is conveyed through the channel without contacting the check dam abutments.

Step 7. Selection of Vegetation

The chosen vegetative channel lining must be water-tolerant, erosion–resistant and be suited to site-specific climate, soils, and topography. Selection of vegetation should conform to Standard and Specification 3.32 of the Virginia Erosion and Sediment Control Handbook (DCR, 1992) The use of fertilization should be minimized as it contradicts the water quality improvement function of the swale.

The example channel is shown in profile and cross-section in Figures 5.8 and 5.9 respectively.

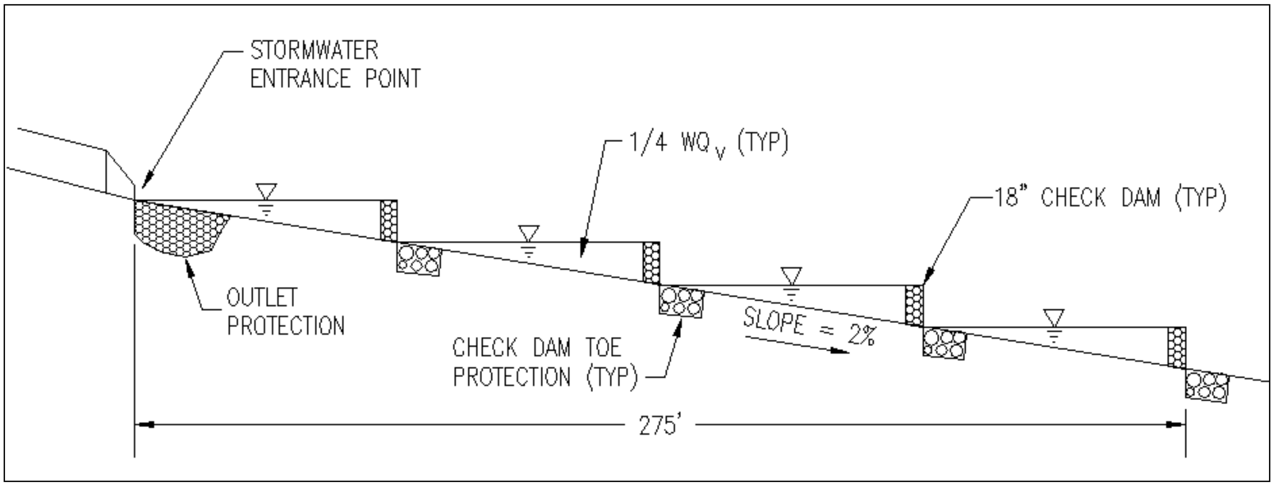


Figure 6.8. Profile of Example Swale
Not to Scale

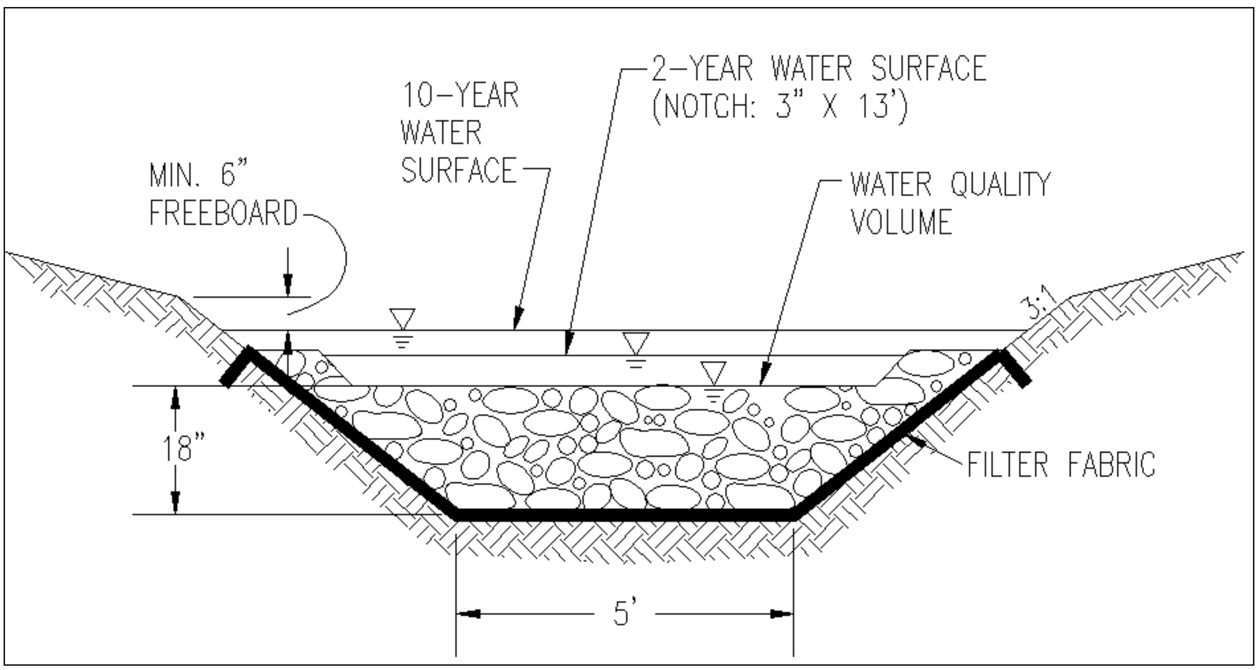


Figure 6.9. Cross-Sectional View of Example Swale
Not to Scale

Chapter 7 – Vegetated Filter Strip

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

7.1 Overview of Practice

A vegetated filter strip is a densely vegetated strip of land, similar to a grassed swale, but engineered to accept runoff from upstream development only as *overland sheet flow* (Yu, 2004). The type of vegetation selected may range from native species, to grass meadow, to forest. In addition to serving as a primary water quality improvement practice, vegetated filters strips function extremely well as pre-treatment measures for other BMPs whose function may be compromised if sediment loading is excessive.

Vegetated filter strips are water quality improvement practices, and cannot be considered effective flood control strategies.

7.2 Site Constraints and Siting of the Facility

A number of site constraints must be considered in addition to the contributing drainage area's impervious cover when the implementation of a vegetated filter strip is proposed. These constraints are discussed as follows.

7.2.1 Minimum Drainage Area

The minimum drainage area contributing to a vegetated filter strip is not restricted. Vegetated filter strips are particularly well suited to small drainage areas.

7.2.2 Maximum Drainage Area

The water quality improvement function of a vegetated filter strip is predicated on its ability to maintain sheet flow across the strip. When flow on the strip becomes concentrated, forming channels, the hydraulic residence time on the strip is reduced to ineffective levels. As contributing drainage area increases, so does the difficulty in ensuring that the volume of runoff generated from the area can remain as sheet flow across the strip. The contributing area to a filter strip should never exceed five acres. Regardless of the strip's contributing drainage area, flow entering onto the strip must never be concentrated. If sheet flow cannot be maintained upstream of the filter strip, a level spreader should be employed to convert concentrated flows back to sheet flow prior to their entrance onto the strip.

7.2.3 Site Slopes

Sites upon which a vegetated filter strip is proposed should exhibit relatively flat topography. Alternative BMPs should be considered when site topography is such that slopes exceed five percent.

7.2.4 Site Soils

The implementation of a vegetated filter strip is restricted to those soils having an infiltration rate of at least 0.52 inches per hour. A permeability test is required for this BMP. This data should be provided to the Materials Division early in the project planning stages to determine if a vegetated filter strip is feasible on native site soils. In addition to infiltration rate restrictions, the soil must be capable of sustaining a dense stand of vegetation with minimal fertilization.

7.2.5 Depth to Water Table

The presence of a shallow water table in the vicinity of a proposed filter strip may hinder the infiltration function of the strip. The lowest elevation of the filter strip should be a minimum of two feet above the local seasonally high water table.

7.2.6 Existing Utilities

Filter strips often can be constructed over existing easements, provided permission to construct the strip over these easements is obtained from the utility owner *prior* to design of the strip.

7.2.7 Wetlands

When the construction of a vegetated filter strip is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation in their vicinity.

7.3 General Design Guidelines

The following presents a collection of broad design issues to be considered when designing a vegetated swale for improvement of water quality.

7.3.1 Length

Ultimately, the required length of a filter strip (in the direction of flow) is a function of the target hydraulic residence time for flows entering onto the strip. A 9 minute hydraulic residence time is recommended with five minutes being the absolute minimum for water quality improvement (FHWA, 1996). Generally, for strips exhibiting a longitudinal slope of less than two percent, the *minimum* strip length that should be considered is 25 feet. For any one percent increase in slope, the filter length should increase by at least four feet. These values, however, are only estimates and computational procedures (discussed later in this chapter) must be used to ensure target hydraulic residence times are met. Optimal filter strip lengths will range from 80 to 100 feet. Flow over pervious surfaces tends to become concentrated when the flow path exceeds 150 feet (CWP, 1996). Therefore, strips of excessive length are discouraged.

7.3.2 Width

Ideally, the width of the filter strip (perpendicular to the flow direction) should, if at all possible, be equal to the width of the area contributing runoff to the strip. When this is not possible, a level spreader may be used to distribute flow evenly onto the strip. The *minimum* width of the filter strip should be the greater of the two values:

0.2 x Filter Length

or

8 feet

7.3.3 Slope

The filter strip slope should be as flat as practically possible while still providing positive drainage across the strip. Excessive ponding of runoff is undesirable as this will lead to saturation of the strip's underlying soil, resulting in difficulty maintaining a dense stand of vegetation on the strip. The slope of a vegetated filter strip is not restricted to any specific maximum value. However, as the strips slope is increased the flow velocity on the strip increases. The increase in velocity will necessarily require lengthening of the strip to attain an effective hydraulic residence time. As filter strip length increases so does the likelihood of the flow becoming concentrated. Filter strips function best on slopes of five percent or less (Yu, 2004). Table 7.1 presents maximum recommended filter strip slopes as a function of Hydrologic Soil Group and vegetative cover.

Filter Strip Soil Type	Hydrologic Soil Group	Maximum Filter Strip Slope (Percent)	
		Turf Grass, Native Grasses, and Meadows	Planted and Indigenous Woods
Sand	A	7	5
Sandy Loam	B	8	7
Loam, Silt Loam	B	8	8
Sandy Clay Loam	C	8	8
Clay Loam, Silty Clay, Clay	D	8	8

Table 7.1. Recommended Maximum Filter Strip Slopes

Source: Pennsylvania Department of Environmental Protection. Stormwater Best Management Practices Manual. 2006.

7.3.4 Pervious Berm

When soil infiltration rates, site groundwater depths, and/or slopes do not adhere to the guidelines previously described, the filter strip may be equipped with a berm at its downstream end. Such a berm will effectively force ponding on the surface of the strip, thus increasing the hydraulic residence time of the entering flows. The berm should be constructed of moderately permeable soils as approved by the Materials Division. Generally acceptable soils are ASTM *ML, SM,* or *SC* or soils meeting USDA sandy loam or loamy sand texture with a minimum of 10 – 25% clay. The berm must be equipped with an armored overflow section to permit safe passage of large flows which would otherwise overtop the berm. The maximum depth of ponding behind the berm should not exceed one foot. The use of a berm should only be considered as a last resort, as the forced ponding of runoff on the strip will hinder the establishment of a dense stand of vegetation.

7.3.5 Discharge Flows

When a grassed swale empties into an existing swale or other surface conveyance system, the receiving channel must be evaluated for adequacy as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR, 1992). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

7.4 Design Process

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

The following filter strip design will provide the technology-based water quality requirements arising from a linear development scenario similar to that described in *Chapter Six – Vegetated Swale*. The new scenario entails the construction of approximately 1,300 linear feet of secondary subdivision roadway in the City of Hampton. Topography is such that runoff from the road is collected in roadside ditches and conveyed to a low point near the mid station of the road. The concentrated runoff is discharged into a level spreader from which it then enters onto the proposed filter strip as overland sheet flow. The total project site, including right-of-way and all permanent easements, consists of 4.6 acres. Pre and post-development land cover characteristics and peak rates of runoff are summarized below in Tables 7.2 and 7.3. The project site exhibits topography typical of the coastal region of Virginia, with slopes generally less than two percent. Site soils are categorized as a sandy loam (Hydrologic Soil Group B).

	Pre-Development	**Post-Development**
Project Area (acres)	4.6	4.6
Land Cover	Unimproved Grass Cover	0.75 acres impervious cover
Impervious Percentage	0	16.3

Table 7.2. Land Cover Characteristics of Example Project Site

		York County - 10 Year				
Acreage	**Rational C**	**A Constant**	**B Constant**	**t_c (min)**	**i_{10} (iph)**	**Q_{10} (cfs)**
0.75	0.9	186.78	21.22	8	6.39	4.3

Table 7.3. Peak 10-Year Runoff from Example Project Site

Step 1. Compute the Required Water Quality Volume

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

$$WQ_V = \frac{IA \times \frac{1}{2} in}{12 \frac{in}{ft}}$$

IA = Impervious Area (ft^2)

The project site is comprised of a total drainage area of 4.6 acres. With impervious area within the project site of 0.75 acres, the water quality volume is computed as:

$$WQ_V = \frac{0.75ac \times \frac{1}{2} in \times \frac{43{,}560\,ft^2}{ac}}{12\frac{in}{ft}} = 1{,}361\,ft^3$$

The vegetated filter strip should be sized to provide a minimum hydraulic residence time of five minutes for the computed water quality volume.

Step 2. Estimate the Required Strip Length

The next step is to estimate the strip's required length. Making an initial estimate of the required length will assist in evaluating the feasibility of the practice for the given site conditions. The following nomographs, Figures 7.1 – 7.5 (obtained from the Pennsylvania Department of Environmental Protection Stormwater Best Management Practices Manual, 2006), provide a means by which to estimate the required filter strip length as a function of the underlying Hydrologic Soil Group (HSG), strip slope, and type of vegetative cover. As stated previously, the proposed strip's underlying soil is a sandy loam of HSG *B*. At this point in the design, the vegetative cover is assumed to be native grasses. Figure 7.2 reflects the site-specific conditions.

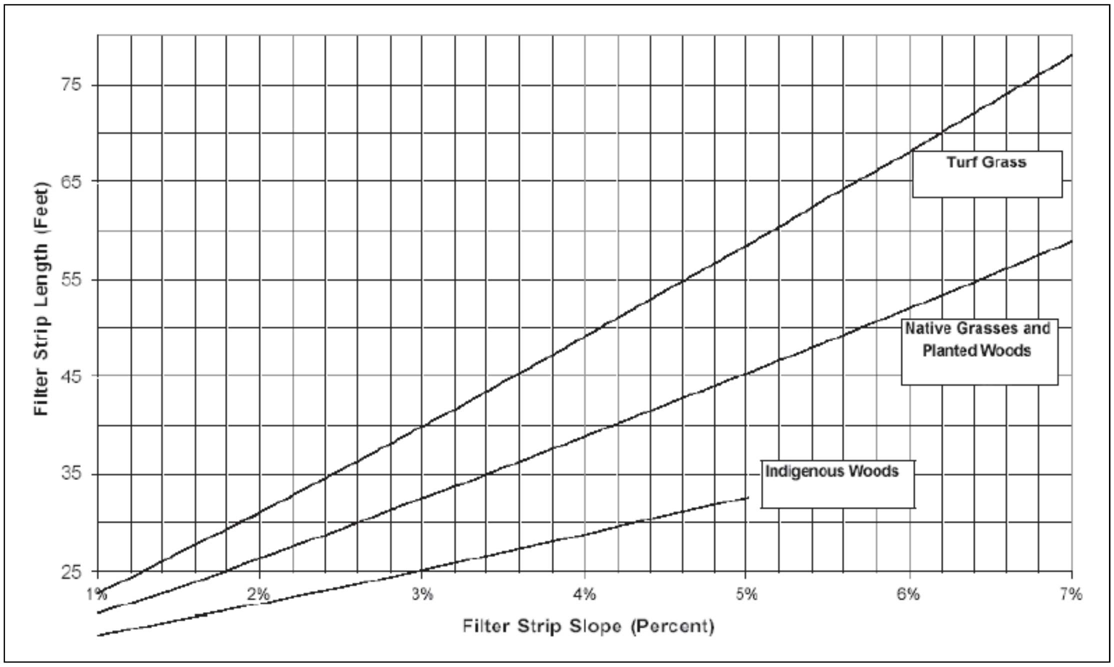


Figure 7.1. Filter Strip Length – Sand, HSG A (PADEP, 2006)

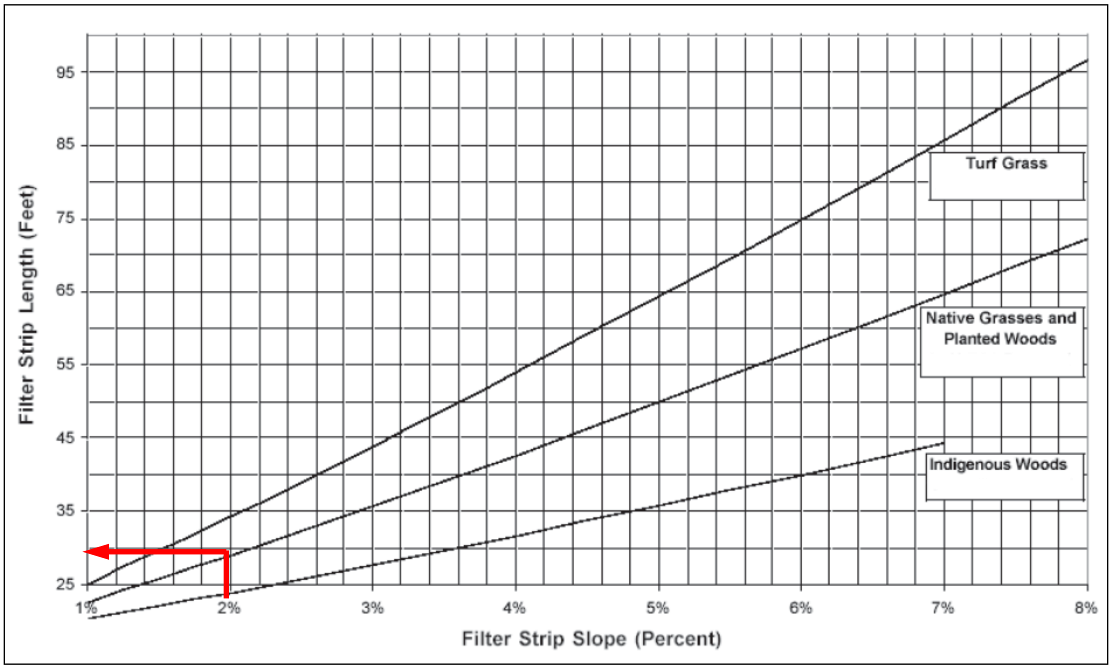


Figure 7.2. Filter Strip Length – Sandy Loam, HSG B (PADEP, 2006)

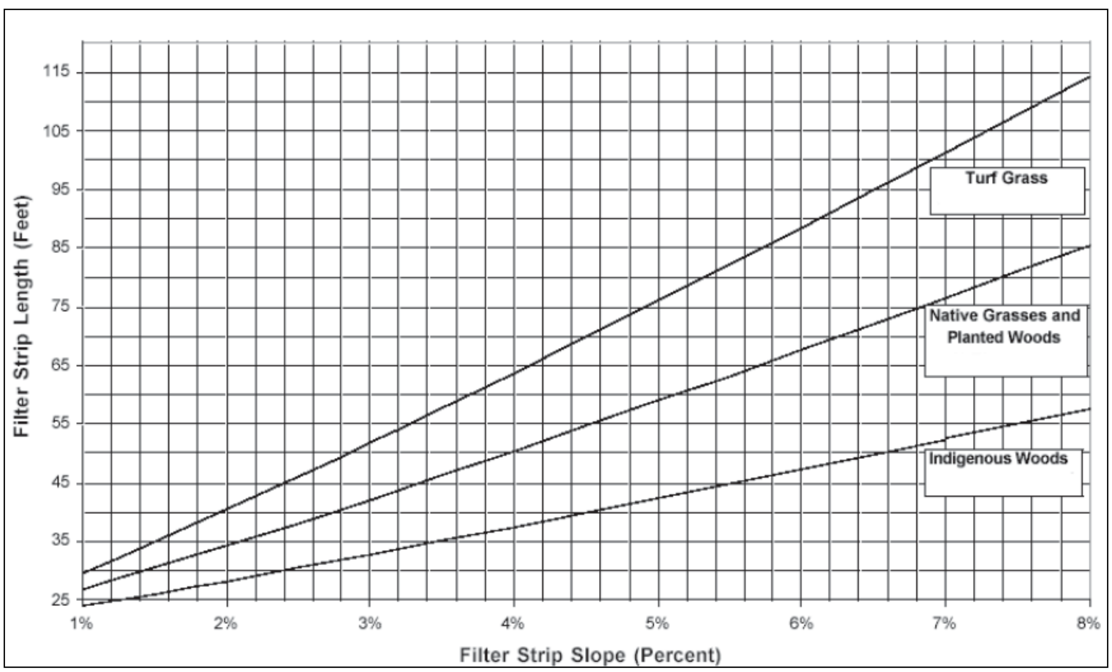


Figure 7.3. Filter Strip Length – Loam / Silt Loam, HSG B (PADEP, 2006)

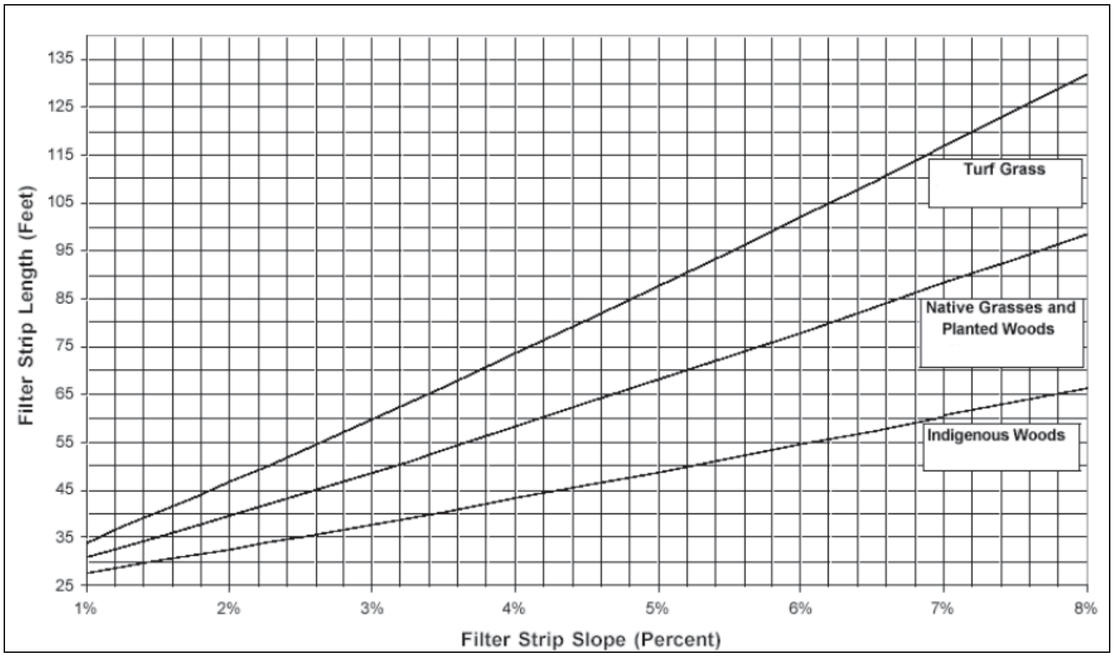


Figure 7.4. Filter Strip Length – Sandy Clay Loam, HSG C (PADEP, 2006)

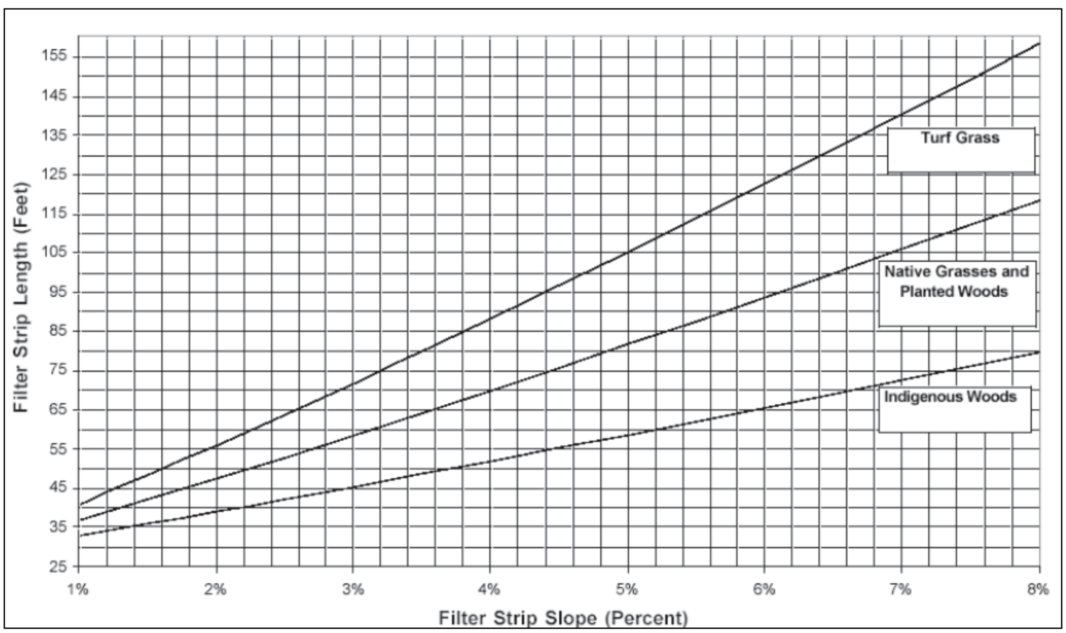


Figure 7.5. Filter Strip Length – Clay Loam / Silty Clay / Clay, HSG D (PADEP, 2006)

Figure 7.2 provides an estimated filter strip length of 29 feet. It should be noted that this is a short strip, whose estimated length is largely a function of the relatively high permeability rates exhibited by sandy loams categorized as HSG *B*. While the filter strip may be able to infiltrate a large portion of its received runoff under ideal conditions, conservative design practice will size the strip to provide effective hydraulic residence times even when antecedent moisture conditions are such that the underlying soils are in a near-saturated condition. This sizing procedure is discussed in the next steps. The estimated strip length of 29 feet is the absolute minimum length that should be considered for this example.

Step 3. Estimate the Peak Rate of Runoff Corresponding to the Water Quality Volume

A detailed filter strip design requires that the design discharge onto the strip be known. The length of the strip can then be sized to accommodate this discharge while providing the desired hydraulic residence time. The site's water quality volume was computed previously as 1,361 ft^3. The peak volumetric rate of discharge which generates this runoff volume can be estimated by examining the basic Rational Method hydrograph shape shown in Figure 7.6.

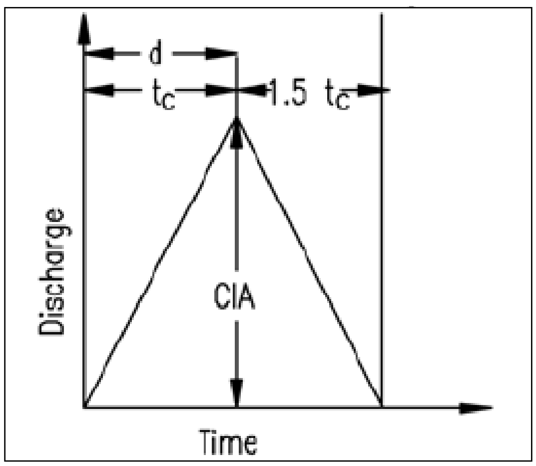


Figure 7.6. Basic Rational Hydrograph Shape
Virginia Stormwater Management Handbook, (DCR, 1999)

The time of concentration is known to be 8 minutes. Therefore, the "base" of the triangular shaped hydrograph is 20 minutes (1,200 seconds). The total area under the hydrograph is the water quality volume (1,361 ft^3). Therefore, employing the area relationship of a triangle, the lone unknown, Q, is computed as follows:

$$A = \left(\frac{1}{2}\right) \times b \times h$$

$$h = \frac{2A}{b}$$

$$Q = \frac{(2)(1,361\,ft^3)}{1,200s} = 2.3cfs$$

The water quality volume from the 0.75 acre impervious development generates an estimated peak discharge of 2.3 cfs. This value is now used to size the strip.

Step 4. Compute the Strip Length (Flow Direction)

Runoff will enter onto the strip from a level spreader. The size of the level spreader is a function of the 10-year flow from the contributing drainage area. The required level spreader dimensions are shown in Table 7.4.

Q10 (cfs)	Depth (ft)	Width of Lower Side Slope of Spreader (ft)	Length (ft)
0-10	0.5	6	10
20-10	0.6	6	20

Table 7.4. Minimum Level Spreader Dimensions
Virginia Erosion and Sediment Control Handbook (DCR, 1992)

The 10-year peak rate of runoff from the roadway is 4.3 cfs. Therefore, the minimum level spreader "lip" length that will discharge runoff onto the strip is 10 feet.

In order to assure that the minimum five minute hydraulic residence time is achieved, the length of the strip (in the direction of flow) must be sized as a function of the anticipated flow velocity on the strip.

Flow velocity is computed by the Manning's equation. A Manning roughness coefficient of 0.20 is typically used in grass filter strip flow calculations. If the filter strip is mowed infrequently, a roughness coefficient of 0.24 may be used. (FHWA, 1996, pg 325; also, Horner, 1993). This Manning roughness coefficient is derived from employing the anticipated flow velocity and flow depth on the filter strip. Manning's n values for various categories of vegetative ground covers are presented in Table 7.5.

Surface	Recommended Value	Range of Values
Range (natural)	0.13	0.01-0.32
Range (clipped)	0.08	0.02-0.24
Grass (bluegrass sod)	0.45	0.39-0.63
Short Grass Prairie	0.15	0.10-0.20
Dense Grass	0.24	0.17-0.30
Bermuda Grass	0.41	0.30-0.48

Table 7.5. Recommended Manning's n Values for Overland Flow

Source: Mays, Larry W. Water Resources Engineering. John Wiley & Sons, Inc. New York, NY, 2001.

By the principal of continuity, flow on the strip can be expressed as:

$$Q = V \times W \times h$$

Q = volumetric flow rate (cfs)
V = average flow velocity on the strip (fps)
W = strip width (ft)
H = flow depth on the strip (ft)

For shallow overland flow, the anticipated flow depth is assumed equal to the hydraulic radius. Expressing flow in terms of the Manning's equation, the previous expression becomes:

$$Q = \frac{1.49}{n} \times h^{\frac{2}{3}} \times S^{\frac{1}{2}} \times (W \times h)$$

n = Manning roughness coefficient
S = filter strip slope (ft/ft)
Other terms as previously defined

This equation can then be rearranged to isolate the desired unknown, *h.*

$$\frac{Q}{W} = \frac{1.49}{n} \times h^{\frac{5}{3}} \times S^{\frac{1}{2}}$$

At this stage in the design, the filter strip width is unknown. Therefore, an assumption must be made and its adequacy later verified. We will assume a filter strip width of 25 feet. Then, solving for h:

$$\frac{2.3}{25} = \frac{1.49}{0.20} \times h^{\frac{5}{3}} \times 0.02^{\frac{1}{2}}$$

h = 0.23 ft

Employing the previously established parameters, flow velocity on the strip is computed as follows:

$$V = \frac{1.49}{n} R_h^{\frac{2}{3}} S^{\frac{1}{2}}$$

V = velocity (fps)
N = Manning roughness coefficient
R_h = hydraulic radius (ft, equal to flow depth for shallow overland flow)
S = filter strip slope (ft/ft)

$$V = \frac{1.49}{0.20}(0.23)^{\frac{2}{3}}(0.02)^{\frac{1}{2}} = 0.395\frac{ft}{s}$$

Next, the filter strip length can be computed as a function of this flow velocity and the target hydraulic residence time. First, the minimum residence time of five minutes is considered:

$$L = t \times V$$

L = filter strip length (ft)
t = target hydraulic residence time (sec)
V = flow velocity (fps)

$$L = 5\,\text{min} \times \frac{60\,\text{sec}}{\text{min}} \times 0.395\frac{ft}{\text{sec}} = 119\,ft$$

It is again noted that this approach does not consider that a portion of the water quality volume will infiltrate into the strip's subsoil. Additionally, the accumulation of flow depth and subsequent decrease in velocity is not considered. Therefore, the computed length of 119 feet reflects a conservative design which can reasonably be assumed to provide a hydraulic residence time in excess of the minimum value of five minutes.

Step 5. Verify Adequacy of the Assumed Strip Width (Perpendicular to Flow Direction)

The *minimum* width of the filter strip should be the greater of the two values:

0.2 x Filter Length

or

8 feet

Therefore, the minimum strip width is computed as follows:

$$0.2 \times 119\,ft = 23.8\,ft$$

The assumed strip width of 25 feet is therefore adequate.

Ideally, the filter strip width will equal the width of the contributing drainage area. When a level spreader is used, as in this example, the lip of the spreader must extend to within *a minimum* of 10 feet of the filter strip on each end (Virginia Stormwater Management Handbook*, (DCR, 1999)*. The proposed level spreader lip is 10 feet in length. Therefore the spreader extends to within 7.5 feet of the filter edges (see calculation below):

$$\frac{25\,ft - 10\,ft}{2} = 7.5\,ft$$

If this value was found to exceed 10 feet the level spreader length would need to be increased.

Step 6. Selection of Vegetation

Filter strips must be constructed of dense, soil-binding deep rooted water-resistant plants. If a grass filter strip is to be employed, a dense turf is necessary to achieve desirable pollutant removal percentages while avoiding erosion. If turf grass is used, the height shall be maintained between two and four inches. The specific species of vegetation should be appropriate for the climatic conditions and expected maintenance.

Filter strips should be planted with a minimum of two of the following vegetation types, per the Virginia Stormwater Management Handbook (DCR, 1999):

- deep-rooted grasses, ground covers, or vines
- deciduous and evergreen shrubs
- under-and over-story trees

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

All chosen plant species should conform to the American Standard for Nursery Stock, current issue, and be suited for USDA Plant Hardiness Zones 6 or 7, see Figure 7.7 on the following page.

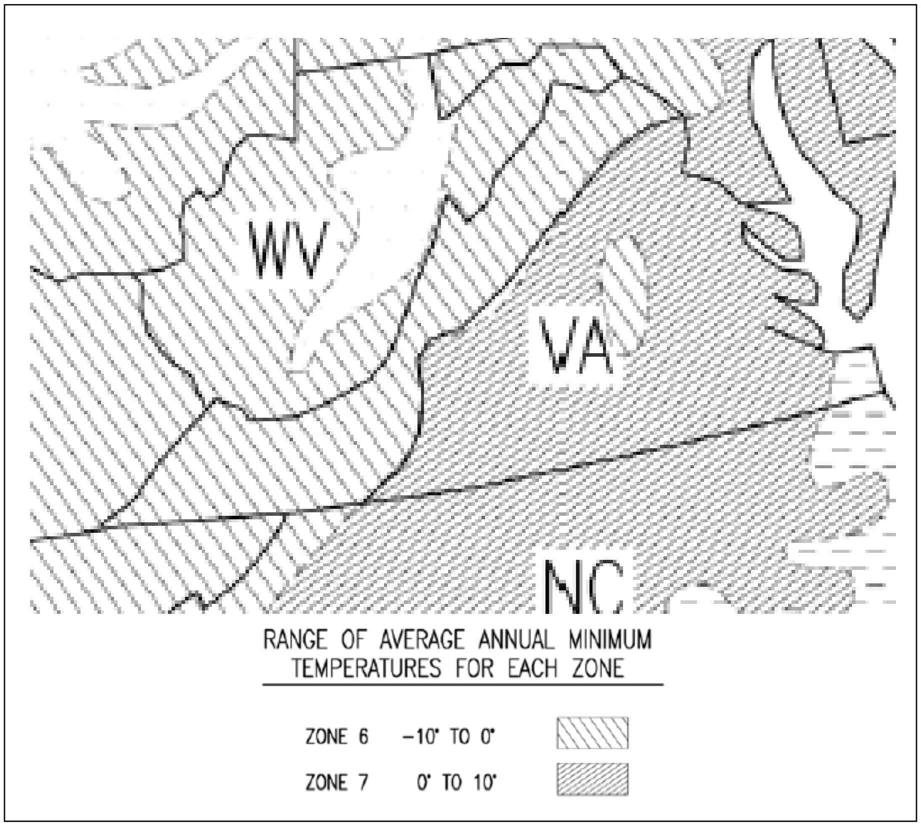


Figure 7.7. USDA Plant Hardiness Zones

The presences of trees, shrubs, and other woody vegetation can further increase the water quality performance of vegetated filter strips. In addition to intercepting a portion of stormwater before it even reaches the ground, trees and shrubs increase the infiltration and retention present in the filter strip. However, when trees are incorporated into the filter strip design, one must be aware that the overall density of vegetation is decreased. Consequently, while filter strips with trees and other woody vegetation can demonstrate higher pollutant removal efficiencies than their strictly grass counterparts, they require that the filter strip be longer in length to account for the reduced vegetation density. Additionally, tree and shrub trunks have the potential to support the development of gullies and channels in the strip. To offset this phenomenon, filter strips equipped with trees and shrubs should be designed with flatter slopes than those employing only grass.

Chapter 8 – Infiltration Trench

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

8.1 Overview of Practice

Infiltration trenches are shallow trenches equipped with an underground reservoir comprised of coarse stone aggregate. The void space created by the aggregate provides storage for surface runoff that has been diverted into the trench. This runoff then infiltrates into the surrounding soil, through the bottom and sides of the trench.

Infiltration trenches act primarily as water quality BMPs; however, when equipped with underground piping, the temporary storage volume of the trench may be increased to a volume that provides peak runoff rate reduction for the one and two year return frequency storms. Peak rate control of the 10-year and greater storm events is typically beyond the capacity of an infiltration practice.

8.2 Site Constraints and Siting of the Facility

The designer must consider a number of site constraints in addition to the contributing drainage area's impervious cover when an infiltration trench is proposed. These constraints are discussed as follows.

8.2.1 Minimum Drainage Area

The minimum drainage area contributing to an infiltration trench is not restricted. Infiltration trenches are particularly well suited to small drainage areas.

8.2.2 Maximum Drainage Area

The maximum drainage area to a single infiltration trench should be restricted to no more than five acres. Multiple trenches may be employed to receive runoff from larger drainage areas; however, when considering required trench maintenance, the implementation of multiple infiltration trenches is often undesirable.

8.2.3 Site Slopes

Infiltration trenches are suitable for installation on sites exhibiting slopes generally less than 20 percent. Infiltration trenches should be located a minimum of 50 feet away from any slope steeper than 15 percent. When site slopes exceed 20 percent, alternative BMP measures should be considered.

8.2.4 Site Soils

The soil infiltration rate is a critical design element of an infiltration trench. When such a facility is proposed, *a subsurface analysis and permeability test is required*. The required subsurface analysis should investigate soil characteristics to a depth of no less than three feet below the proposed bottom of the stone trench. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils.

The soil's infiltration rate should be measured when the soil is in a saturated condition. Soil infiltration rates which are deemed acceptable for infiltration trenches range between *0.52 and 8.27 inches per hour* (DCR, 1999, Et Seq.). Infiltration rates falling within this range are typically exhibited by soils categorized as loam, sandy loam, and loamy sand.

Soils exhibiting a clay content of greater than 30 percent are unacceptable for infiltration facilities. Similarly, soils exhibiting extremely high infiltration rates, such as sand, should also be avoided. Table 8.1 presents typical infiltration rates observed for a variety of soil types. This table is provided as a reference only, and does not replace the need for a detailed site soil survey.

Texture Class	Effective Water Capacity (C_w) (inch per inch)	Minimum Infiltration Rate (*f*) (inch per hour)	Hydrologic Soil Grouping
Sand	0.35	8.27	A
Loamy Sand	0.31	2.41	A
Sandy Loam	0.25	1.02	B
Loam	0.19	0.52	B
Silt Loam	0.17	0.27	C
Sandy Clay Loam	0.14	0.17	C
Clay Loam	0.14	0.09	D
Silty Clay Loam	0.11	0.06	D
Sandy Clay	0.09	0.05	D
Silty Clay	0.09	0.04	D
Clay	0.08	0.02	D

Table 8.1. Hydrologic Soil Properties Classified by Soil Texture
Source: (Virginia Stormwater Management Handbook, 1999)

8.2.5 Depth to Water Table

Infiltration trenches should not be installed on sites with a high groundwater table. Inadequate separation between the trench bottom and the surface of the water table may result in contamination of the water table. This potential contamination arises from the inability of the soil surrounding the trench to filter pollutants prior to their entrance into the water table. Additionally, a high water table can flood an infiltration trench and render it inoperable during periods of high precipitation and/or runoff. A separation distance of no less than two feet is required between the bottom of an infiltration trench and the surface of the *seasonally* high water table. Unique site conditions may arise which require an even greater separation distance. The separation distance provided should allow the trench to empty completely within a maximum of 48 hours following a runoff producing event.

8.2.6 Separation Distances

Infiltration trenches should be located at least 20 feet down-slope and at least 100 feet up-slope from building foundations. Infiltration trenches should not be located within 100 feet of any water supply well. Local health officials should be consulted when the implementation of an infiltration trench is proposed within the vicinity of a septic drainfield.

8.2.7 Bedrock

A minimum of two feet of separation is required between the bottom of an infiltration trench and bedrock, with four feet or greater recommended.

8.2.8 Placement on Fill Material

Infiltration trenches should not be constructed on or nearby fill sections due to the possibility of creating an unstable subgrade. Fill areas are vulnerable to slope failure along the interface of the in-situ and fill material. The likelihood of this type of failure is increased when the fill material is frequently saturated, as anticipated when an infiltration BMP is proposed.

8.2.9 Karst

The concentration of runoff into an infiltration trench may result in the formation of flow channels. Such channels may lead to collapse in karst areas, and therefore the implementation of infiltration trenches in known karst areas should be avoided.

8.2.10 Existing Utilities

Infiltration trenches can often be constructed over existing easements, provided permission to construct the strip over these easements is obtained from the utility owner *prior* to design of the strip.

8.2.11 Wetlands

When the construction of an infiltration trench is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation in their vicinity.

8.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing an infiltration trench for improvement of water quality.

8.3.1 Design Infiltration Rate

To provide a factor of safety, and to account for the decline in performance as the facility ages, the soil infiltration rate upon which a trench design is founded should be one-half the infiltration rate obtained from the geotechnical analysis.

8.3.2 Maximum Storage Time

Infiltration trenches should be designed to empty within 48 hours following a runoff producing event.

8.3.3 Trench Sizing

Generally, the trench's total depth ranges from 2 to 10 feet. The surface area of the trench is that area which, when multiplied by the trench depth and the aggregate porosity, provides the computed treatment volume. Trench widths greater than 8 feet require large excavation equipment rather than smaller trenching equipment. When treatment volumes require a width greater than 8 feet, an infiltration basin or other BMP should be considered.

8.3.4 Runoff Pretreatment

Infiltration trenches *must* be preceded by a pretreatment facility. Roadways and parking lots often produce runoff with high levels of sediment, grease, and oil. These pollutants can potentially clog the pore space in the trench, thus rendering its infiltration and pollutant removal performance ineffective. Suitable pretreatment practices include vegetated buffer strips, sediment forebays, and proprietary water quality inlets.

All infiltration trenches that receive surface runoff as sheet flow should be equipped with a vegetated buffer strip at least 20-feet wide (see *Chapter Seven – Vegetated Filter Strip)*.

8.3.5 Aggregate Material

The infiltration trench material should be comprised of clean aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches. Aggregate meeting this specification should be VDOT No. 1 Open-graded Coarse Aggregate or its equivalent as recommended by the Materials Division.

An 8-inch deep sand layer must be installed at the bottom of the trench. This material should be VDOT Fine Aggregate, Grading A or B, or equivalent as approved by the Materials Division.

8.3.6 Observation Well

An observation well is recommended at an interval of every 50 feet along the entire trench length. Observation wells provide a means by which dewatering times can be observed to ensure that the trench is emptying within the maximum allowable time of 48 hours. Generally, the observation well is constructed of 4 or 6 inch perforated PVC pipe, configured as shown in Figure 8.1

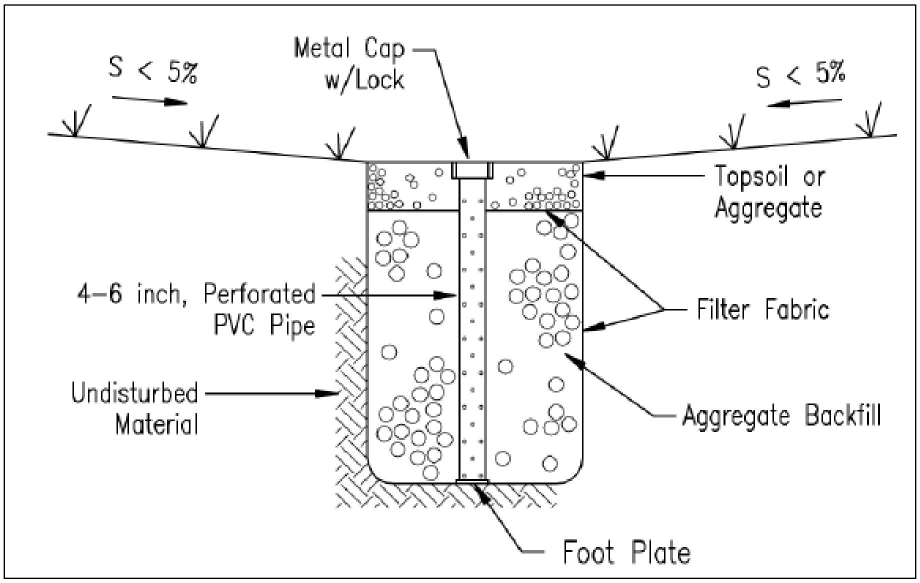


Figure 8.1. Infiltration Trench Observation Well Configuration
(Virginia Stormwater Management Handbook, 1999, Et seq.)

8.3.7 Filter Fabric

The trench aggregate material should be surrounded with filter fabric as shown in Figure 8.2. The filter fabric should be a material approved by the Materials Division. Filter fabric should *not* be placed on the trench bottom. When the trench is constructed as a "surface trench" with no soil overlay, a separate piece of filter fabric should be used as the top layer. This enables replacement of the upper filter fabric upon its eventual clogging.

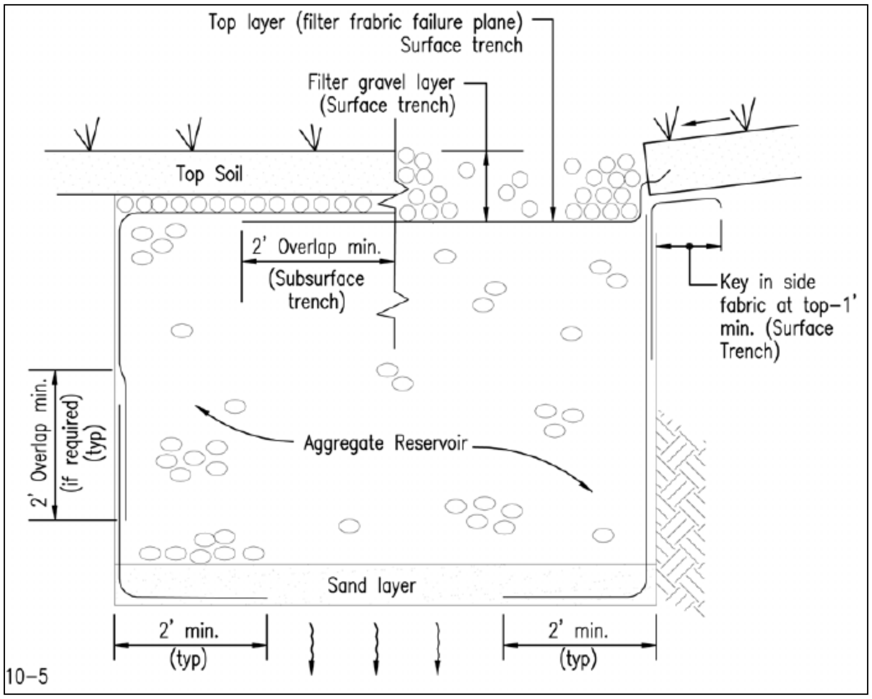


Figure 8.2. Infiltration Trench Filter Fabric Installation
(Virginia Stormwater Management Handbook, 1999, Et seq.)

8.4 Design Process

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

The infiltration trench design will meet the technology-based water quality requirements arising from the construction of approximately 2,000 linear feet of roadway in Halifax County. Topography is such that runoff from the road is collected in VDOT CG-6 curb and gutter and conveyed to curb inlets along the road. The runoff is then discharged into sediment forebays from which it then enters onto the surface of the proposed trench, which is located in the median of the divided roadway. The total project site, including right-of-way and all permanent easements, consists of 6.2 acres. Pre and post-development hydrologic characteristics are summarized below in Table 8.2. Approximately 300 linear feet is available for construction of the trench. Geotechnical investigations reveal the site's saturated soil infiltration rate to be 2.3 inches per hour. The project site does not exhibit a high or seasonally high groundwater table.

	Pre-Development	Post-Development
Project Area (acres)	6.2	6.2
Land Cover	Unimproved Grass Cover	3.4 acres impervious cover
Impervious Percentage	0	54.8

Table 8.2. Hydrologic Characteristics of Example Project Site

Step 1. Compute the Required Water Quality Volume

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

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

IA= Impervious Area (ft^2)

The project site in this example has a total drainage area of 6.2 acres. The total impervious area within the site is 3.4 acres. Therefore, the water quality volume is computed as follows:

$$WQV = \frac{3.4ac \times \frac{1}{2}in \times \frac{43{,}560\,ft^2}{ac}}{12\frac{in}{ft}} = 6{,}171\,ft^3$$

The impervious cover within the project site is less than 67 percent of the total project site. Therefore, in accordance with Table 1.1, the infiltration trench will be sized to treat the computed water quality volume of 6,171 ft^3.

Step 2. Compute the Design Infiltration Rate

Per DCR guidelines, the design infiltration rate, f_d, is computed as one-half the infiltration rate obtained from the required geotechnical analysis. For the given site conditions, the infiltration rate is computed as:

$$f_d = 0.5f = (0.5)\left(2.3\frac{in}{hr}\right) = 1.15\frac{in}{hr}$$

Step 3. Compute the Maximum Allowable Trench Depth

The trench must be designed such that it is completely empty within a maximum of 48 hours following a runoff producing event. To ensure compliance with this requirement, we will compute the maximum allowable trench depth by the following equation:

$$d_{\max} = \frac{f_d \times T_{\max}}{V_r}$$

d_{max} = maximum allowable trench depth (ft)
f_d = design infiltration rate (in/hr)
T_{max} = maximum allowable drain time (48 hours)
V_r = void ratio of the stone trench (0.40 for VDOT No. 1 Coarse-graded Aggregate)

The maximum allowable trench depth is therefore computed as:

$$d_{\max} = \frac{\left(1.15\frac{in}{hr}\right)\left(\frac{1\,ft}{12in}\right)(48hrs)}{0.40} = 11.5\,ft$$

Step 4. Compute the Minimum Allowable Trench Bottom Area

Employing the principles of Darcy's Law, and assuming one-dimensional flow through the bottom of the trench, we can compute the minimum allowable surface area of the trench by the following equation:

$$SA_{\min} = \frac{WQV}{(f_d)(T_{\max})}$$

SA_{min} = minimum trench bottom surface area (ft^2)
WQV = treatment volume (ft^3)
f_d = design infiltration rate (in/hr)
T_{max} = maximum allowable drain time (48 hours)

The minimum allowable trench surface area is computed as follows:

$$SA_{\min} = \frac{6{,}171\, ft^3}{\left(1.15\dfrac{in}{hr}\right)\left(\dfrac{1\, ft}{12 in}\right)(48 hr)} = 1{,}342\, ft^2$$

Step 5. Size the Trench Based on Site-Specific Parameters

The example trench is to be located in the median of a divided highway. Per the problem statement, approximately 300 linear feet are available for construction of the trench. This entire length will be utilized in an effort to minimize the trench depth.

The maximum desirable trench width is 8 feet. Employing this maximum width with the available 300 foot length results in a trench bottom surface area computed as follows:

$$SA = (300\, ft)(8\, ft) = 2{,}400\, ft^2$$

This value is greater than the minimum value (computed previously as 1,342 ft^2), and is therefore considered acceptable.

Next, the trench depth must be computed. The volume of storage provided in the void space of the trench aggregate must provide the computed treatment volume. Therefore, the minimum trench depth is computed by the following equation, with variables as previously defined.

$$d = \frac{WQ_V}{(V_r)(SA)}$$

The trench depth is then computed as:

$$d = \frac{6{,}171\,ft^3}{(0.4)(2{,}400\,ft^2)} = 6.43\,ft$$

The computed trench depth is less than the maximum value (computed previously as 11.5 ft), and is therefore considered acceptable.

A summary of the trench parameters are provided in Table 8.3.

Length	300 ft
Width	8 ft
Depth	6.5 ft
Storage Volume	6,240 ft^3

Table 8.3. Summary of Trench Dimensions

Step 6. Alternative Trench Sizing Procedure

The addition of a large perforated pipe(s) within the trench can greatly increase the trench storage capacity. This increased storage capacity can be used to reduce the overall dimensions of the trench, or, keeping the trench size fixed, provide a greater overall infiltration volume. The following steps illustrate the procedure for decreasing the trench depth by providing perforated corrugated metal pipes within the trench. The demonstrated methodology can also be adapted to resize the trench length and/or depth.

In this example, we will consider placement of two 36-inch perforated corrugated metal pipes within the trench. Assuming the pipes extend the full length of the trench, we can compute the total volume provided by the pipes as follows:

$$V_{Pipe} = L \times \pi \times r^2$$

$$V_{Pipe} = \left[300 \times \pi \times 1.5^2\right] \times 2Pipes = 4{,}242\,ft^3$$

The volume provided by the stone aggregate to be replaced by the pipes is computed as:

$$V_{Stone} = 4{,}242\,ft^3 \times 0.4 = 1{,}696.8\,ft^3$$

Therefore, the net "gain" in storage volume by replacing the aggregate with the pipes is computed as:

$$V_{Net} = 4{,}242\,ft^3 - 1{,}696.8\,ft^3 = 2{,}545.2\,ft^3$$

The reduction in trench depth can then be computed as a function of the net gain in storage volume and the trench's length and width:

$$D_{\text{Re}\,duction} = \frac{2{,}545.2\,ft^3}{300\,ft \times 8\,ft \times 0.4} = 2.65\,ft$$

The new trench depth is computed as:

$$D = 6.43\,ft - 2.65\,ft = 3.8\,ft$$

The overall volume provided by the re-sized trench is then computed as:

$$V_{Trench} = 300\,ft \times 8\,ft \times 3.8\,ft \times 0.4 = 3{,}648\,ft^3$$

This volume is then added to the net gain in volume provided by the two 36-inch diameter pipes:

$$V_{Total} = 3{,}648\,ft^3 + 2{,}545\,ft^3 = 6{,}193\,ft^3$$

A schematic illustration of the re-sized trench is shown in Figure 8.3.

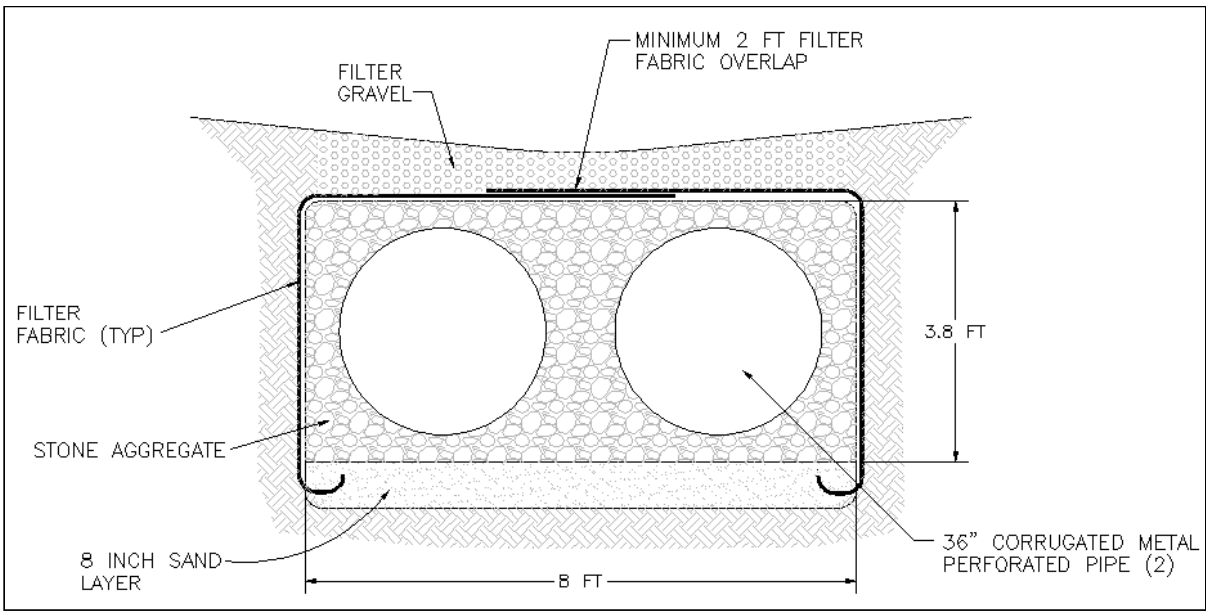


Figure 8.3. Infiltration Trench Equipped with Perforated Pipes

Step 7. Provide Provision for Overflow

Infiltration trenches serve primarily as water quality BMPs. Typically, it is impractical to size the trench to accommodate a volume of runoff beyond that which must be captured for water quality purposes. Therefore, provisions must be provided for runoff conveyance when the capacity of the trench is exceeded. Because of the small drainage area served by an infiltration trench, an emergency spillway is typically not required; however, a non-erosive channel or storm sewer system must be located at the downstream end of the trench. The channel or sewer should carry excess flows to an adequate receiving channel as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR, 1992, Et seq.). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

When a storm sewer or other conduit is used to convey excess runoff, the invert must be located at an elevation that is not below the surface of the infiltration trench's aggregate storage volume. *Only the volume of storage provided below the invert of the bypass pipe can be considered infiltration (treatment) volume.* A typical bypass configuration is shown below in Figure 8.4.

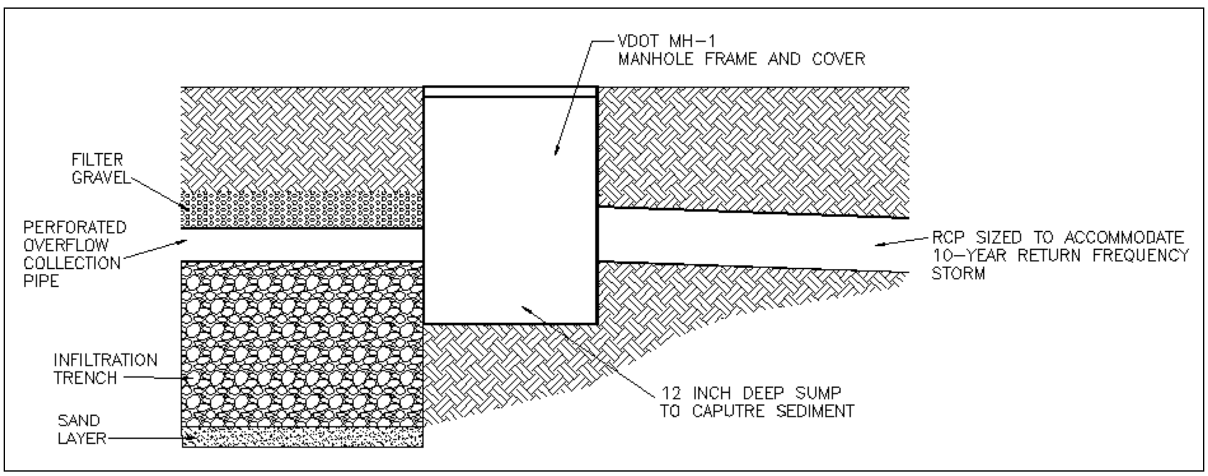


Figure 8.4. Infiltration Trench Section Equipped with RCP Overflow Pipe

Step 8. Landscaping

Trenches that are not designed to function as a surface trench (as shown in Figure 8.2) must exhibit a dense vegetative cover before any stormwater runoff is directed to the facility. Careful attention must be given to the types of vegetation selected for the trench surface. The vegetative species must be selected based on their inundation tolerance and the anticipated frequency and depth of inundation. The designer is referred to the Virginia Erosion and Sediment Control Handbook (DCR, 1992, Et seq.) for recommendations of specific vegetative species based on the facility's geographic location. Generally, low-growing stoloniferous grasses are good candidates for infiltration facilities as they permit long intervals between mowing, thus minimizing the frequency of traffic on the surface of the facility.

Maintenance of the facility's vegetative cover is essential to the long-term performance of the facility. A dense vegetative stand enhances infiltration, minimizes surface erosion, and deters invasive and detrimental vegetative species. Any bare spots on the surface of the facility should be re-seeded immediately.

The use of fertilizers should be minimized and avoided completely if practically possible. Excessive use of fertilizers on highly permeable soil may lead to groundwater contamination. Reference the Virginia Erosion and Sediment Control Handbook (DCR, 1992, Et seq.) for recommendations on appropriate fertilizer types and minimum effective application rates.

Chapter 9 – Infiltration Basin

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

9.1 Overview of Practice

Infiltration basins are impounding facilities which temporarily store surface runoff and infiltrate a designated portion of it into the soil strata.

Unlike infiltration trenches, infiltration basins may also serve as peak mitigation facilities. This is accomplished by providing “dry” storage above the designated infiltration volume. This dry, flood control volume is then released through a multi-stage riser and barrel system. Conceptually, an infiltration basin can be viewed as an extended dry detention basin whose water quality volume is infiltrated into the soil strata rather than released through a small orifice over a 30 hour period.

As shown in Table 1.1, the water quality volume of an infiltration trench can vary, and the anticipated pollutant removal performance of the trench varies as a function of this volume.

9.2 Site Constraints and Siting of the Facility

The designer must consider a number of site constraints in addition to the contributing drainage area's impervious cover when an infiltration basin is proposed. These constraints are discussed as follows.

9.2.1 Minimum Drainage Area

The minimum drainage area contributing to an infiltration trench is not restricted. However, when contributing drainage areas are particularly small, infiltration trenches will often provide a more cost-effective option.

9.2.2 Maximum Drainage Area

The drainage area contributing runoff to an infiltration basin should be restricted to no more than 50 acres.

9.2.3 Site Slopes

Infiltration basins are suitable for installation on sites exhibiting slopes generally less than 20 percent. Infiltration basins should be located a minimum of 50 feet away from any slope steeper than 15 percent. When site slopes exceed 20 percent, alternative BMP measures should be considered. The floor slope of an infiltration basin should be as flat as practically possible in order to maximize the area upon which effective infiltration can occur.

9.2.4 Site Soils

When an infiltration basin is proposed the soil infiltration rate is of critical design importance. *A subsurface analysis and permeability test is required*. The required subsurface analysis should investigate soil characteristics to a depth of no less than three feet below the proposed bottom of the basin. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils.

The soil's design infiltration rate should be measured when the soil is in a saturated condition. Soil infiltration rates which are deemed acceptable for infiltration trenches range between *0.52 and 8.27 inches per hour* (DCR, 1999, Et Seq.). Infiltration rates falling within this range are typically exhibited by soils categorized as loam, sandy loam, and loamy sand.

Soils exhibiting a clay content of greater than 30 percent are unacceptable for infiltration facilities. Similarly, soils exhibiting extremely high infiltration rates, such as sand, should also be avoided. Table 9.1 presents typical infiltration rates observed for a variety of soil types. This table is provided as a reference only, and does not replace the need for a detailed site soil survey.

Texture Class	Effective Water Capacity (C_w) (inch per inch)	Minimum Infiltration Rate (*f*) (inch per hour)	Hydrologic Soil Grouping
Sand	0.35	8.27	A
Loamy Sand	0.31	2.41	A
Sandy Loam	0.25	1.02	B
Loam	0.19	0.52	B
Silt Loam	0.17	0.27	C
Sandy Clay Loam	0.14	0.17	C
Clay Loam	0.14	0.09	D
Silty Clay Loam	0.11	0.06	D
Sandy Clay	0.09	0.05	D
Silty Clay	0.09	0.04	D
Clay	0.08	0.02	D

Table 9.1. Hydrologic Soil Properties Classified by Soil Texture
(Virginia Stormwater Management Handbook, 1999, Et seq.)

9.2.5 Depth to Water Table

Infiltration basins should not be installed on sites with a high groundwater table. Inadequate separation between the basin bottom and the surface of the water table may result in contamination of the water table. This potential contamination arises from the inability of the soil surrounding the trench to filter pollutants prior to their entrance into the water table. Additionally, a high water table may flood an infiltration basin during periods of high precipitation and/or runoff. A minimum separation distance of no less than two feet is required between the bottom of an infiltration basin and the surface of the *seasonally* high water table, with four or more feet of separation preferred. Unique site conditions may arise which require an even greater separation distance. The separation distance provided should allow the basin to empty completely within a maximum of 48 hours following a runoff producing event.

9.2.6 Separation Distances

Infiltration basins should be located at least 20 feet down-slope and at least 100 feet up-slope from building foundations. Infiltration basins should not be located within 100 feet of any water supply well. Local health officials should be consulted when the implementation of an infiltration basin is proposed within the vicinity of a septic drainfield.

9.2.7 Bedrock

A minimum of two feet of separation is required between the bottom of an infiltration basin and bedrock, with four feet or greater recommended.

9.2.8 Placement on Fill Material

Infiltration basins should not be constructed on or nearby fill sections due to the possibility of creating an unstable subgrade. Fill areas are vulnerable to slope failure along the interface of the in-situ and fill material. The likelihood of this type of failure is increased when the fill material is frequently saturated, as anticipated when an infiltration BMP is proposed. Additionally, construction traffic and compaction activities will generally result in fill material exhibiting an infiltration rate below that which is desirable for an infiltration facility.

9.2.9 Karst

The concentration of runoff into an infiltration facility may result in the formation of flow channels. Such channels may lead to collapse in karst areas, and therefore the implementation of infiltration basins in known karst areas should be avoided.

9.2.10 Basin Location

When possible, infiltration basins should be placed in low visibility areas. When such a basin must be situated in a high profile area, care must be given to ensure that the facility empties completely within a 48 hour maximum. The location of an infiltration basin in a high visibility area places a great emphasis on the facility's ongoing maintenance.

9.2.11 Existing Utilities

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

9.2.12 Wetlands

When the construction of an infiltration basin is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation in their vicinity.

9.2.13 Floodplains

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

9.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing an infiltration basin for improvement of water quality.

9.3.1 Foundation and Embankment Material

Foundation data for the dam must be secured by the Materials Division to determine whether or not the native material is capable of supporting the dam while not allowing water to seep under the dam, as per Instructional and Informational Memorandum (IIM-LD-195) under *"Post Development Stormwater Management"*.

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

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

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

9.3.2 Outfall Piping

If the basin is equipped with a riser structure and outlet barrel, the pipe culvert under or through the basin embankment shall be reinforced concrete equipped with rubber gaskets. Pipe: Specifications Section 232 (AASHTO M170), Gasket: Specification Section 212 (ASTM C443).

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

9.3.3 Principal Spillway Design

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

The principal spillway should be equipped with a low flow orifice to permit draining of the facility in the event the infiltration surface becomes clogged and runoff cannot be infiltrated. This low flow orifice should remain plugged as long as the facility is infiltrating runoff at the rate for which it was designed.

9.3.4 Embankment

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

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

9.3.5 Embankment Height

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

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

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

9.3.6 Fencing

Per Instructional and Informational Memorandum (IIM-LD-195) under General Subject *"Post Development Stormwater Management,"* fencing is typically *not required or recommended* on most VDOT detention facilities. However, exceptions do arise, and the fencing of a dry extended detention facility may be needed. Such situations include:

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

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

9.3.7 Design Infiltration Rate

To provide a factor of safety, and to account for the decline in performance as the facility ages, the soil infiltration rate upon which a basin design is founded should be one-half the infiltration rate obtained from the geotechnical analysis (DCR, 1999, Et Seq.).

9.3.8 Maximum Storage Time

Infiltration basins should be designed to empty completely within 48 hours following a runoff producing event.

9.3.9 Runoff Pretreatment

Infiltration basins should be preceded by a pretreatment facility. Roadways and parking lots may produce runoff with high levels of sediment, grease, and oil. These pollutants can potentially clog the pore space in the basin floor, thus reducing its infiltration and pollutant removal performance. Suitable pretreatment practices include vegetated buffer strips, sediment forebays, and proprietary water quality inlets. At a minimum, each basin inflow point should be equipped with a sediment forebay. Individual forebay volumes should range between 0.1 and 0.25 inches over the outfall's contributing impervious area with the sum of all forebay volumes not less than 10 percent of the total WQV.

All infiltration basins that receive surface runoff as sheet flow should be equipped with a vegetated buffer strip at least 20 feet wide.

9.3.10 Discharge Flows

All basin outfalls must discharge into an adequate receiving channel as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR, 1992, Et seq.). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

9.4 Design Process

Many of the design elements in an infiltration basin are identical to those of a dry extended detention basin. These elements include estimation of flood control storage volumes, design of a multi-stage riser, storage indication (reservoir) routing, emergency spillway design, riser buoyancy calculations, and the design of sediment forebays. For those design items, the reader is referred to *Chapter 2 – Dry Extended Detention Basin*.

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

The following design example entails the construction of a small interchange and new section of two lane divided highway in Williamsburg. The total project site, including right-of-way and all permanent easements, consists of 24.8 acres. Pre and post-development hydrologic characteristics are summarized below in Table 9.2. Initial geotechnical investigations reveal a soil infiltration rate of 1.84 inches per hour with site soils classified as Hydrologic Soil Group B.

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

Table 9.2. Hydrologic Characteristics of Example Project Site

Step 1. Compute the Required Water Quality Volume

The project water quality volume is a function of the developed impervious area, and is computed as follows:

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

IA= Impervious Area (ft^2)

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

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

The impervious cover within the project site is less than 67 percent of the total project site. Therefore, the infiltration basin will be sized to treat the computed water quality volume of 20,328 cubic feet.

Step 2. Compute the Design Infiltration Rate

The design infiltration rate, f_d, is computed as one-half the infiltration rate obtained from the required geotechnical analysis. For the given site conditions, the design infiltration rate is computed as:

$$f_d = 0.5 f = (0.5)\left(1.84 \frac{in}{hr}\right) = 0.92 \frac{in}{hr}$$

Step 3. Compute the Maximum Ponded Depth of Infiltration Volume

The basin must be designed such that it is completely empty within a maximum of 48 hours following a runoff producing event. To ensure compliance with this requirement, the maximum ponding depth for the infiltration (treatment) volume is computed by the following equation:

$$d_{\max} = f_d \times T_{\max}$$

d_{max} = maximum allowable basin depth (ft)
f_d = design infiltration rate (in/hr)
T_{max} = maximum allowable drain time (48 hours)

The maximum allowable ponding depth is therefore computed as:

$$d_{\max} = \left(0.92 \frac{in}{hr}\right)\left(\frac{1 ft}{12 in}\right)(48 hr) = 3.68\, ft$$

Step 4. Compute the Minimum Allowable Basin Surface Area

Employing Darcy's Law, and assuming one-dimensional flow through the bottom of the basin, we can compute the minimum allowable surface area of the basin floor by the following equation:

$$SA_{\min} = \frac{WQV}{(f_d)(T_{\max})}$$

SA_{min} = minimum basin bottom surface area (ft^2)
WQV = treatment volume (ft^3)
f_d = design infiltration rate (in/hr)
T_{max} = maximum allowable drain time (48 hours)

The minimum allowable basin floor area is computed as follows:

$$SA_{\min} = \frac{20{,}328\, ft^3}{\left(0.92\,\frac{in}{hr}\right)\left(\frac{1\,ft}{12\,in}\right)(48\,hr)} = 5{,}524\, ft^2$$

Step 5. Size the Basin Based on Site-Specific Parameters

In order to reduce the amount of required right-of-way acquisition, the surface area of a structural BMP is minimized during the design process. However, minimization of surface area may require a BMP depth that is either impractical or, in the case of an infiltration facility, violates design parameters. The following design approach attempts to minimize the surface area of the basin while meeting restrictions on ponding depth.

The minimum allowable basin floor area was previously computed as 5,524 ft^2. This is the minimum basin area that, when considering a factor of safety, will ensure that the basin empties within a maximum of 48 hours. In practice, the actual configuration of an infiltration basin will be dictated largely by topography and other site-specific constraints. The final design may require multiple iterations to provide the required treatment volume. In this design, we will consider a basin of rectangular orientation, with a 2.5:1 length to width ratio. A schematic illustration of this basin configuration is shown in Figure 9.2.

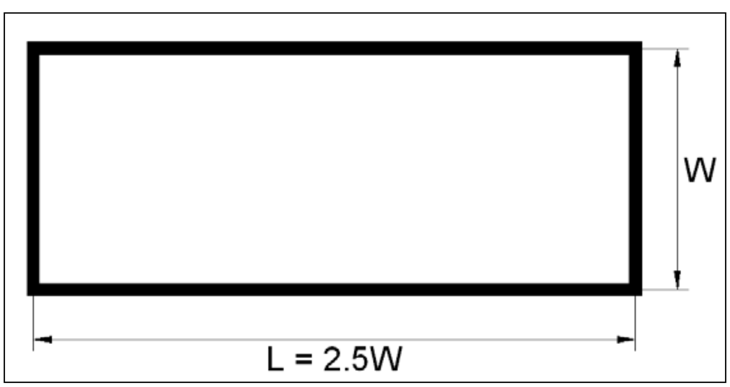


Figure 9.1. Schematic Basin Orientation

The dimensions of the basin floor can then be approximated by solving the following expression:

$$W \times 2.5W = 5{,}524\, ft^2$$
$$W = 47.0\, ft$$
$$L = 117.5\, ft$$

The volume above the basin floor that is allocated to infiltration can be approximated by the following equation:

$$V = \left(\frac{A_1 + A_2}{2} \right) d$$

V = infiltration (treatment) volume (ft^3)
A_1 = surface area of basin floor (5,524 ft^2)
A_2 = surface area above the basin floor allocated to infiltration
d = incremental depth between A_1 and A_2

Based on a trapezoidal approximation, the surface area, A_2, can be expressed as a function of depth, d:

$$A_2 = \left[47.0 + (2)(d)(Z) \right] \times \left[117.5 + (2)(d)(Z) \right]$$

Z = basin side slopes (ZH:1V)

In this example, we will consider that the basin side slopes are 3H:1V. The updated A_2 expression then becomes:

$$A_2 = \left[47.0 + (2)(d)(3) \right] \times \left[117.5 + (2)(d)(3) \right]$$

A total infiltration volume of 20,328 ft^3 must be provided above the surface of the basin floor. At this point, the designer can construct a plot of storage versus depth by employing the above equation for A_2 in the previous expression for volume, *V*. This plot is shown in Figure 9.2.

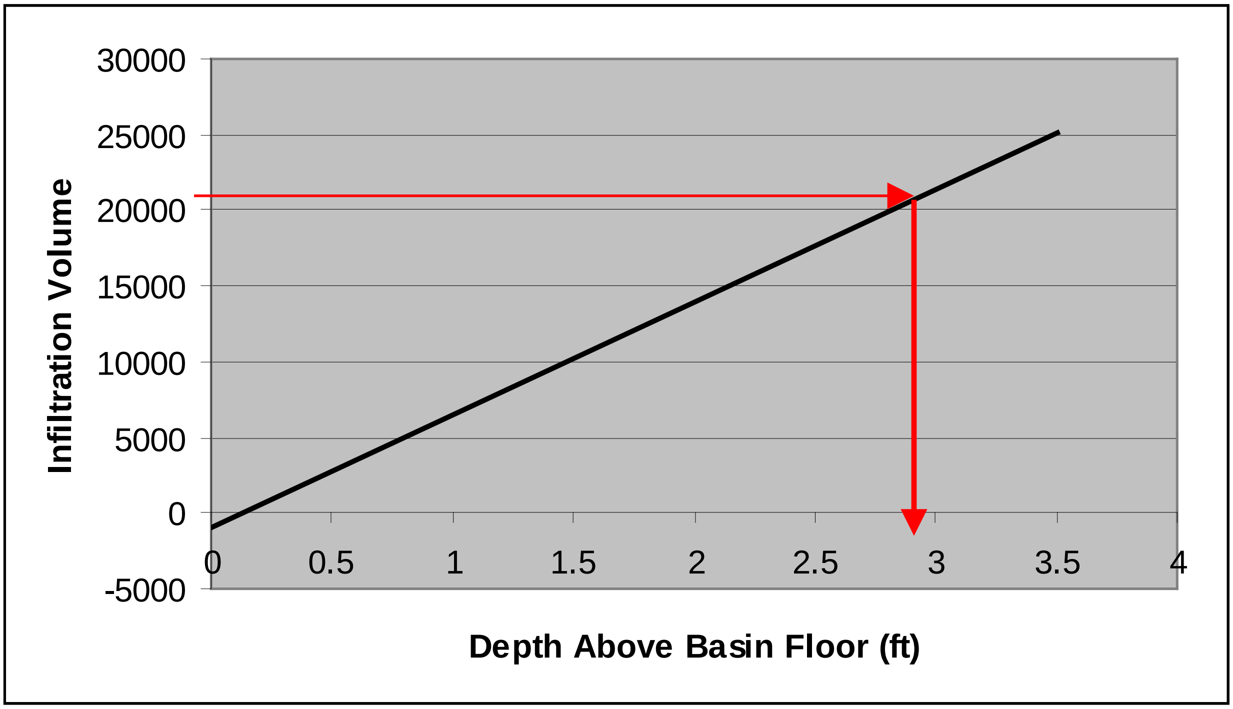


Figure 9.2. Plot of Infiltration Volume Versus Depth Above Basin Floor

The plot indicates that the infiltration volume of 20,328 ft^3 is provided at an approximate depth of 2.8 feet above the basin floor. This estimate can be verified as follows:

$$A_2 = [47.0 + (2)(2.8)(3)] \times [117.5 + (2)(2.8)(3)] = 8{,}568\, ft^2$$

The total storage volume provided above the permanent pool is then computed as:

$$V = \left(\frac{5{,}524 + 8{,}568}{2}\right) 2.8 = 19{,}729\, ft^3$$

The volume is less than the required storage volume of 20,328 ft^3, and therefore must be increased. The calculation is repeated for a ponded infiltration depth of 2.9 feet.

$$A_2 = [47.0 + (2)(2.9)(3)] \times [117.5 + (2)(2.9)(3)] = 8{,}688\, ft^2$$

The total storage volume provided above the permanent pool is then computed as:

$$V = \left(\frac{5{,}524 + 8{,}688}{2}\right) 2.9 = 20{,}607\, ft^3$$

The infiltration volume provided at a ponded depth of 2.9 feet exceeds (slightly) the minimum treatment volume of 20,328 ft^3 and is therefore acceptable. Additionally, the infiltration volume is provided at a depth that is less than the maximum allowable depth of 3.68 feet. Therefore, it can be anticipated that the basin will empty completely within the maximum allowable time of 48 hours.

At this point, the remaining design process largely mimics that of a Dry Extended Detention facility. Flood control storage can be provided in the facility beginning at 2.9 feet above the basin floor (the upper limit of the infiltration volume). The remaining design elements include estimation of flood control storage volumes, design of a multi-stage riser, storage indication (reservoir) routing, emergency spillway design, riser buoyancy calculations, and the design of sediment forebays. For those design items, the reader is referred to *Chapter 2 – Dry Extended Detention Basin*.

Step 6. Landscaping

Infiltration basins must exhibit a dense vegetative cover before any stormwater runoff is directed to the facility. Careful attention must be given to the types of vegetation selected for the basin floor and embankment. The vegetative species must be selected based on their inundation tolerance and the anticipated frequency and depth of inundation. The designer is referred to the Virginia Erosion and Sediment Control Handbook (DCR, 1992, Et seq.) for recommendations of specific vegetative species based on the facility's geographic location. Generally, low-growing stoloniferous grasses are good candidates for infiltration facilities as they permit long intervals between mowing, thus minimizing the frequency of traffic on the surface of the facility.

Maintenance of the facility's vegetative cover is essential to the long-term performance of the facility. A dense vegetative stand enhances infiltration, minimizes surface erosion, and deters invasive and detrimental vegetative species. Any bare spots on the surface of the facility should be re-seeded immediately.

The use of fertilizers should be minimized and avoided completely if practically possible. Excessive use of fertilizers on highly permeable soil may lead to groundwater contamination. Reference the Virginia Erosion and Sediment Control Handbook (DCR, 1992, Et seq.) for recommendations on appropriate fertilizer types and minimum effective application rates.

Chapter 10 – Porous Pavement

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

10.1 Overview of Practice

Porous pavement is a pervious traffic-bearing surface placed over a stone reservoir which is, in turn, underlain by highly permeable soil. The void space created by the stone reservoir provides storage for surface runoff generated on or diverted onto the porous surface. This runoff then infiltrates into the surrounding soil, through the bottom and sides of the stone reservoir. Porous pavement may substitute for conventional pavement on parking areas and areas with light traffic. Porous pavement is generally not suited for areas with high traffic volumes.

Porous pavement acts primarily as a water quality BMP. However, much like an infiltration trench (*Chapter 8 – Infiltration Trench*), when equipped with underground piping, the temporary storage volume of the reservoir may be increased to provide peak runoff reduction for the one and two year return frequency storms. Peak rate control of the 10-year and greater storm events is considered to be beyond the ability of the practice.

Studies have shown that particulates tend to settle to the bottom of a porous pavement system's stone reservoir while other pollutants often adsorb to the aggregate material. Consequently, the pollutant removal efficiency of a porous pavement system may not be as high as that of other types of infiltration practices. Per DCR recommendations, a porous pavement facility is considered to have a pollutant removal efficiency comparable to that of an extended dry detention facility (*Chapter 2 – Dry Extended Detention Basin*).

10.2 Site Constraints and Siting of the Facility

The implementation of a porous pavement system requires the designer to consider many of the same site constraints as with an infiltration basin or trench. These constraints are discussed as follows.

10.2.1 ***Drainage Area***

Porous pavement systems are generally not cost-effective for sites smaller than 0.25 acres in area. According to the FHWA (1996), the contributing drainage area to a porous pavement infiltration bed should be limited to a maximum of 10 acres in order to reduce the potential for excessive sediment loading. A primary cause of infiltration bed failure is clogging by sediment. The porous pavement system should not be located where runoff from adjacent areas introduces excessive sediment to the system. Additionally, for drainage areas of 10 acres and greater the cost effectiveness of porous paving systems is considered marginal compared to that of other BMPs.

10.2.2 ***Site Slopes***

Unlike other infiltration-based BMPs, which can be installed on slopes of up to 20 percent, porous pavement should not be installed when the traffic bearing surface of the system exceeds 3 percent in slope. Site topography should also permit the construction of a stone reservoir bed that is essentially level along its bottom surface. Porous pavement systems and their associated infiltration beds should be located a minimum of 50 feet away from any slope steeper than 15 percent. When site slopes do not permit the construction of a level infiltration bed, alternative BMP measures should be considered.

10.2.3 ***Site Soils***

The underlying soil infiltration rate is of critical importance in the design of a porous pavement system. *A subsurface analysis and permeability test is required* when such a facility is planned. The required subsurface analysis should include soil characteristics to a depth of no less than three feet below the proposed bottom of the stone reservoir. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils.

The soil infiltration rate should be measured when the soil is in a saturated condition. Soil infiltration rates which are deemed acceptable for porous pavement systems range between *0.52 and 8.27 inches per hour*. Soils with infiltration rates in this range are typically categorized as loam, sandy loam, and loamy sand.

Soils exhibiting a clay content of greater than 30 percent are unacceptable for infiltration facilities. Similarly, soils exhibiting extremely high infiltration rates, such as sand, should be avoided. Table 10.1 presents typical infiltration rates observed for a variety of soil types. This table is provided as a reference only, and does not replace the need for a detailed site soil survey.

Texture Class	Effective Water Capacity (C_w) (inch per inch)	Minimum Infiltration Rate (f) (inch per hour)	Hydrologic Soil Grouping
Sand	0.35	8.27	A
Loamy Sand	0.31	2.41	A
Sandy Loam	0.25	1.02	B
Loam	0.19	0.52	B
Silt Loam	0.17	0.27	C
Sandy Clay Loam	0.14	0.17	C
Clay Loam	0.14	0.09	D
Silty Clay Loam	0.11	0.06	D
Sandy Clay	0.09	0.05	D
Silty Clay	0.09	0.04	D
Clay	0.08	0.02	D

Table 10.1. Hydrologic Soil Properties Classified by Soil Texture
Source: (Virginia Stormwater Management Handbook, 1999, Et seq.)

10.2.4 *Depth to Water Table*

Porous pavement systems should not be installed on sites with a high groundwater table. Inadequate separation between the reservoir bottom and the surface of the water table may result in contamination of the water table. This potential contamination arises from the inability of the soil underlying the reservoir to filter pollutants prior to their entrance into the water table. Additionally, a high water table may flood the stone reservoir and render it inoperable during periods of high precipitation and/or runoff. A separation distance of no less than four feet is required between the bottom of the stone reservoir and the surface of the *seasonally* high water table. Unique site conditions may arise which require an even greater separation distance. The separation distance provided should allow the reservoir to empty completely within a maximum of 48 hours following a runoff producing event.

10.2.5 *Separation Distances*

Porous pavement systems should be located at least 20 feet down-slope and at least 100 feet up-slope from building foundations. Porous pavement systems should not be located within 100 feet of any water supply well. Local health officials should be consulted when the implementation of such a facility is proposed within the vicinity of a septic drainfield.

10.2.6 *Bedrock*

A minimum of four feet of separation is required between the bottom of a porous pavement's stone reservoir and bedrock, with four feet or greater recommended.

10.2.7 ***Placement on Fill Material***

Porous pavement systems should not be constructed on fill sections due to the possibility of creating an unstable subgrade. Fill areas are vulnerable to slope failure along the interface of the in-situ and fill material. The likelihood of this type of failure is increased when the fill material is frequently saturated, as anticipated when an infiltration BMP is proposed.

10.2.8 ***Implementation in Cold Weather Climates***

Porous pavement systems can be implemented in cold weather climates, provided that the reservoir layer extends to a depth beyond the frost line. During winter months, abrasives such as grit and/or sand and deicing chemicals *must not be used* on porous pavement. Plowing must be performed carefully, and as infrequently as possible.

10.2.9 ***Karst***

The concentration of runoff into a stone reservoir may lead to collapse in karst areas, and therefore the implementation of porous pavement in known karst areas should be avoided.

10.2.10 ***Existing Utilities***

Porous pavement systems may be constructed over existing easements, provided permission to construct the infiltration bed over these easements is obtained from the utility owner *prior* to design of the facility.

10.2.11 ***Wetlands***

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

10.3 General Design Guidelines

The following section presents a collection of design issues to be considered when designing a porous pavement *system* for improvement of water quality. The design steps discussed in this report are those exclusive to the water quality improvement function of a porous pavement system. Design of the porous pavement surface layer is beyond the scope of this report, and is a function of the anticipated traffic intensity, the California Bearing Ratio (CBR) of the site soils, the susceptibility of site soils to frost heave, and numerous other factors. The design of the porous surface layer should be performed by a qualified professional familiar with all VDOT standards and specifications governing asphalt design.

10.3.1 *System Storage Capacity*

Porous pavement systems can be designed as *full*, *partial*, or *water quality exfiltration* systems. Full exfiltration systems retain and infiltrate 100 percent of captured runoff. When the reservoir underlying the porous surface is full, runoff bypasses the system completely and is handled by a conventional stormwater capture and conveyance system. (FHWA, 1996)

Partial exfiltration systems are equipped with a bypass piping system. The bypass system routes runoff in excess of what can be infiltrated to a downstream conveyance system. Two types of bypass pipe configurations are shown in Figure 10.1 and Figure 10.2. The first configuration locates the perforated bypass pipe at the bottom of the aggregate reservoir layer. This configuration requires that the outlet manhole be equipped with a concrete weir such that water only discharges through the bypass system when the aggregate layer is in a saturated state. An alternative configuration locates the bypass pipe at the surface of the aggregate reservoir layer. This configuration is similar to the bypass configuration for an infiltration trench (see *Design Example Seven – Infiltration Trench*). When the bypass pipe is not located at the reservoir bottom, the pipe should have perforations on the underside only, else the bypass pipe shall be perforated as necessary to permit flow to freely enter the bypass system.

Water quality exfiltration systems function as partial exfiltration systems, but are designed only to hold and infiltrate the computed water quality volume.

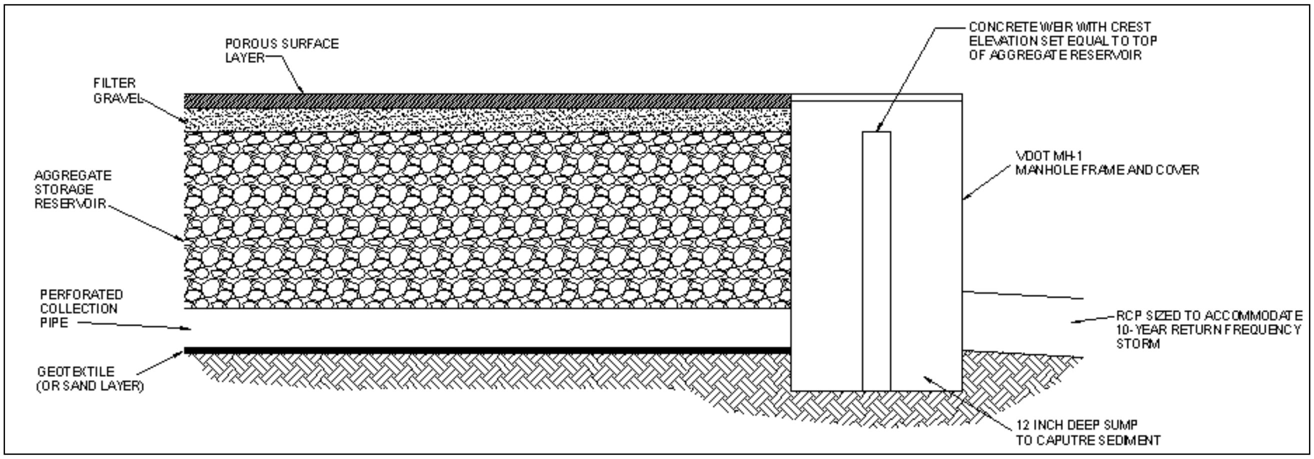


Figure 10.1. Common Bypass Pipe Configuration

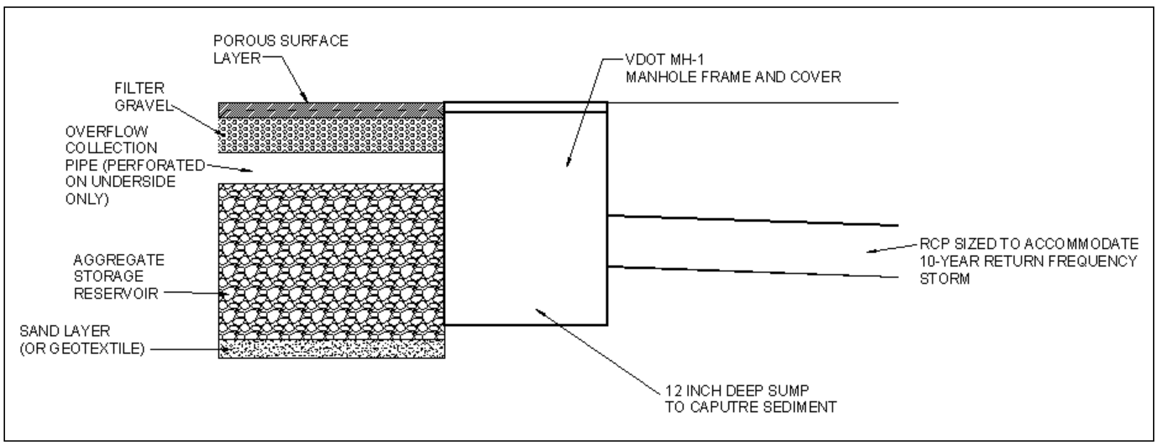


Figure 10.2. Alternative Bypass Pipe Configuration

10.3.2 *Design Infiltration Rate*

To provide a factor of safety, and to account for the decline in performance as the facility ages, the design infiltration rate used to size a porous pavement system should be one-half the infiltration rate obtained from the geotechnical analysis (DCR, 1999, Et seq.).

10.3.3 *Maximum Storage Time*

The stone reservoir of a porous pavement system should be designed to empty within 48 hours following a runoff producing event.

10.3.4 ***Stone Reservoir Sizing***

The reservoir's aggregate depth should extend to a depth of at least that of the local frost line as specified by the Virginia Uniform Statewide Building Code. The surface area of the reservoir is that area which, when multiplied by the trench depth and the aggregate porosity, provides the computed treatment volume.

10.3.5 ***Aggregate Material***

The porous pavement's reservoir layer should be overlain by a 2 inch thick filter layer comprised of VDOT Open-graded Course Aggregate #57. The reservoir should be comprised of 1 – 2 inch diameter clean aggregate (VDOT open-graded course aggregate No. 3). The reservoir layer should be underlain by an 8 inch layer of sand *or* filter fabric as approved by the Materials Division. This configuration is illustrated in Figure 10.3.

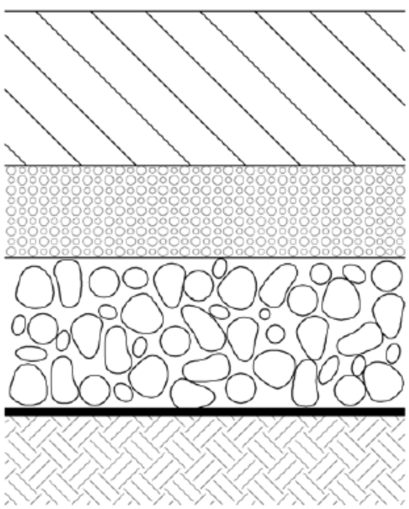


Figure 10.3. General Configuration of Porous Pavement Section
(Virginia Stormwater Management Handbook, 1999, Et seq.)

10.3.6 ***Filter Fabric***

When the reservoir aggregate material is not underlain by a layer of sand, it must be underlain with filter fabric as shown in Figure 10.3. The filter fabric should be comprised of material approved by the VDOT Materials Division in accordance with all applicable DCR requirements.

10.3.7 ***Provision for Surface Clogging***

Porous pavement systems must have a backup method for water to enter the infiltration bed in the event that the porous surface fails or is altered. In parking lots without curbing, this can be accomplished by constructing an unpaved two foot wide stone drain along the downstream edge of the parking lot. The stone drain is then connected directly to the infiltration bed. When curbing is present, sump inlets with sediment traps can be installed in low-lying areas, and then connected directly to the infiltration bed.

10.4 Design Process

This section presents an example of the design process applicable to porous pavement systems serving as water quality BMPs. The pre and post-development runoff characteristics are intended to replicate stormwater management needs routinely encountered on VDOT *facilities* projects. The design steps discussed in this report are those exclusive to the water quality improvement function of a porous pavement system. Design of the porous pavement surface layer is beyond the scope of this report, and is a function of the anticipated traffic intensity, the California Bearing Ratio (CBR) of the site soils, the susceptibility of site soils to frost heave, and numerous other factors. The design of the porous surface layer should be performed by a qualified professional familiar with all VDOT standards and specifications governing asphalt design.

The hydrologic calculations and assumptions presented in this section serve only as input data for the detailed BMP design steps. Full hydrologic discussion is beyond the scope of this report, and the user is referred to Chapter 4 of the Virginia Stormwater Management Handbook (DCR, 1999, Et seq.) for expanded hydrologic methodology.

The porous pavement design will provide the technology-based water quality requirements arising from the parking lot of a VDOT-maintained interstate rest area facility located near Charlottesville. The total parking lot area consists of 4.8 acres, with no offsite drainage entering the parking facility. The total project site, including right-of-way and all permanent easements, consists of 6.2 acres. Geotechnical investigations reveal the site's saturated soil infiltration rate to be 2.7 inches per hour. The project site does not exhibit a high or seasonally high groundwater table. Table 10.2 presents the 10-year hydrologic characteristics of the parking facility.

		Albemarle County - 10 Year				
Acreage	**Rational C**	**A Constant**	**B Constant**	**t_c (min)**	**i_{10} (iph)**	**Q_{10} (cfs)**
4.8	0.9	161.6	18.73	5	6.81	29.4

Table 10.2. Peak Parking Lot Runoff Characteristics

Step 1. Compute the Required Water Quality Volume

The project site's water quality volume is calculated as one half inch over the developed impervious area. In this example, the total parking lot area will be considered impervious cover:

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

IA= Impervious Area (ft^2)

The project site in this example is comprised of a total drainage area of 4.8 acres. Therefore, the basic water quality volume is computed as follows:

$$WQV = \frac{4.8ac \times \frac{1}{2}in \times \frac{43{,}560\,ft^2}{ac}}{12\frac{in}{ft}} = 8{,}712\,ft^3$$

The parking lot area (4.8 acres) comprises 77 percent of the total project site area (6.2 acres). Therefore, adhering to the requirements for infiltration practices detailed in Table 1.1, we will set the design water quality volume as twice the basic water quality volume:

$$WQV_{Design} = 2 \times 8{,}712\,ft^3 = 17{,}424\,ft^3$$

Step 2. Compute the Design Infiltration Rate

The design infiltration rate, f_d, is computed as one-half the infiltration rate obtained from the required geotechnical analysis. For the given site conditions, the infiltration rate is computed as:

$$f_d = 0.5f = (0.5)\left(2.7\frac{in}{hr}\right) = 1.35\frac{in}{hr}$$

Step 3. Compute the Maximum Allowable Reservoir Depth

The aggregate reservoir must be designed such that it is completely empty within a maximum of 48 hours following a runoff producing event. To ensure compliance with this requirement, the maximum allowable trench depth is computed by the following equation:

$$d_{\max} = \frac{f_d \times T_{\max}}{V_r}$$

d_{max} = maximum allowable reservoir depth (ft)
f_d = design infiltration rate (in/hr)
T_{max} = maximum allowable drain time (48 hours)
V_r = void ratio of the stone trench (0.40 for VDOT Coarse-graded Aggregate)

The maximum allowable trench depth is therefore computed as:

$$d_{\max} = \frac{\left(1.35\frac{in}{hr}\right)\left(\frac{1\,ft}{12in}\right)(48hrs)}{0.40} = 13.5\,ft$$

Step 3b. Determine the Minimum Allowable Reservoir Depth

The bottom of the aggregate reservoir layer must be located below the frost line as specified by the Virginia Uniform Statewide Building Code. The frost line depth for the City of Charlottesville is 18 inches. Therefore, the bottom of the aggregate layer must extend to a depth of not less than 18 inches below the finished surface of the pavement.

Step 4. Compute the Required Reservoir Surface Area

The maximum loading ratio, defined as total drainage area to infiltration area is generally restricted to 6:1. The total parking lot area is 4.8 acres, therefore the minimum surface area of stone infiltration reservoir is computed as:

$$A_{\min} = \frac{4.8ac \times \dfrac{43{,}560\,ft^2}{ac}}{6} = 34{,}848\,ft^2$$

The surface area of the stone reservoir, along with its depth must provide storage for the computed water quality volume. Employing the minimum reservoir surface area, we compute the depth of the stone reservoir as:

$$d = \frac{WQV_{Design}}{(V_r)(A_{\min})} = \frac{17{,}424\,ft^3}{(0.40)(34{,}848\,ft^2)} = 1.25\,ft$$

The computed depth is less than the minimum allowable reservoir depth as stipulated by the local frost line depth (18 inches for the City of Charlottesville). Therefore, the reservoir depth is set at 18 inches.

Surface Area	34,848 ft^2
Depth	1.5 ft
Storage Volume*	20,909 ft^3

*Volume Based on Aggregate Porosity of 0.4

Table 10.3. Summary of Stone Reservoir Dimensions

Step 5. Provision for Overflow / Bypass

Because the design configuration presented in this example is a partial exfiltration system intended only to retain and infiltrate the water quality volume, provisions must be made for runoff events producing volumes in excess of this amount.

The overflow/bypass system will function as a conventional storm sewer system upon saturation of the stone reservoir layer. Therefore, the bypass system should be designed to carry a peak 10-year flow rate of 29.4 cfs (reference Table 10.2). The bypass system/storm sewer must discharge into an adequate receiving channel as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR, 1992, Et seq.). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

The bypass system may be constructed as shown in either Figure 10.1 or 10.2. In this example, the bypass will be designed as a PVC pipe placed on a 1.5 percent slope along the entire downstream edge of the stone reservoir. The pipe shall be perforated on its underside only. The bottom of the pipe will be placed at an elevation equal to the top surface of the stone reservoir layer (as shown in Figure 10.2). Therefore, flow will only enter the bypass system upon saturation of the stone reservoir layer Sizing of the underdrain pipe is accomplished by use of the Manning equation shown below:

$$Q = \frac{1.49}{n} \cdot AR_h^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$$

A typical Manning's *n* value for PVC pipe is 0.009 (Mays, 2001). For a fixed discharge, *Q*, the minimum required diameter, *D*, of a circular pipe flowing full can be computed by the following equation:

$$D = \left[\frac{(Q)(n)}{0.463} \times \frac{1}{\sqrt{s}}\right]^{3/8}$$

D= Minimum Pipe Diameter (ft)
Q= Pipe Discharge (cfs)
n= Manning's Roughness Coefficient
s= Pipe slope (ft/ft)

The minimum pipe diameter required to convey the facility's 10-year runoff is therefore computed as:

$$D = \left[\frac{(29.4)(0.009)}{0.463} \times \frac{1}{\sqrt{0.015}}\right]^{3/8} = 1.78 - ft = 21.4 - inches$$

The underdrain pipe shall be 24 inches in diameter.

The 24" perforated PVC underdrain shall connect to a conventional stormwater conveyance system and carry runoff volumes in excess of the water quality volume to an adequate receiving channel.

A cross section of this porous section is presented in Figure 10.4.

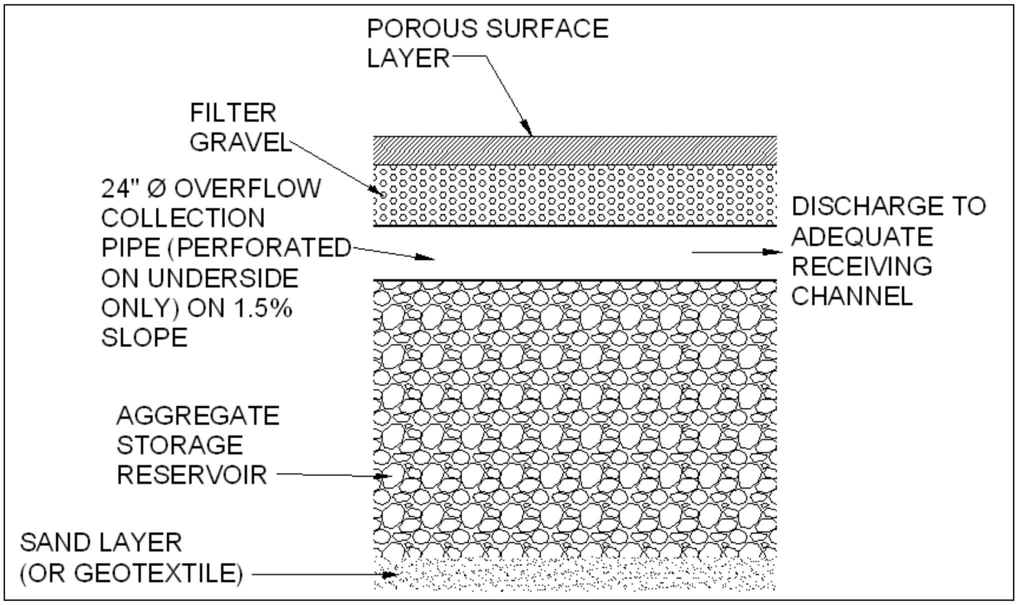


Figure 10.4. Profile Along Downstream Edge of Stone Reservoir

Course	Thickness (in)	Comments
Porous Surface	2.5-4	Permeability > 8 in/hr
Top Filter Course	1-2	1/2" diameter gravel
Underdrain Piping	24	Perforation on bottom side only
Stone Reservoir	17	Cleanly washed - 40% void space
Bottom Filter Course	2	1/2" diameter gravel
Filter Fabric*	N/A	MIRIFI #14 or equivalent
Undisturbed Soil	N/A	Min. Permeability 0.50 in/hr

* The filter fabric should be comprised of material approved by the VDOT Materials Division in accordance with all applicable DCR requirements.

Table 10.4. Summary of Porous Pavement Section

Chapter 11 – Bioretention

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

11.1 Overview of Practice

Bioretention practices form a class of BMP whose primary function is to improve the quality of stormwater runoff by means of adsorption, filtration, volitization, ion exchange, and microbial decomposition. However, some runoff rate and volume reduction is observed through the infiltration of runoff. In the most general sense, a bioretention BMP can be thought of as a modified infiltration area comprised of a *specific* mix of trees, plants, and shrubs intended to mimic the ecosystem of an upland (non-wetland) forest floor. There are two categories of bioretention BMP: *basins* and *filters.*

Bioretention *basins* are planting areas constructed as shallow basins in which stormwater inflow is treated by filtration through the surface plant material, biological and chemical reactions within the soil and basin vegetation, and the eventual infiltration into the underlying soil media. Bioretention *filters* function much the same as bioretention basins, but are used in locations where full infiltration is not feasible due to inadequate soil permeability or the proximity to wells, drainfields, or structural foundations. Bioretention filters are equipped with a connection to a local storm sewer system such that water enters the storm sewer after it has filtered through the bioretention cell. Figures 11.1 and 11.2 present the general configuration of a bioretention basin and filter. The designer is also referred to Figures 3.11-2 – 3.11-5 of the Virginia Stormwater Management Handbook (DCR, 1999, Et seq., Et seq.) for location and conceptual layout suggestions for bioretention facilities.

Yu (2004) states that bioretention units can be applied in treating stormwater runoff from VDOT facilities such as weigh stations, park-and-ride facilities, and welcome stations. Other possible application scenarios include rooftop runoff and runoff from short stretches of roadway. Because of their use of specific vegetative plantings and landscaping techniques, bioretention BMPs can provide significant aesthetic benefit to a developed site.

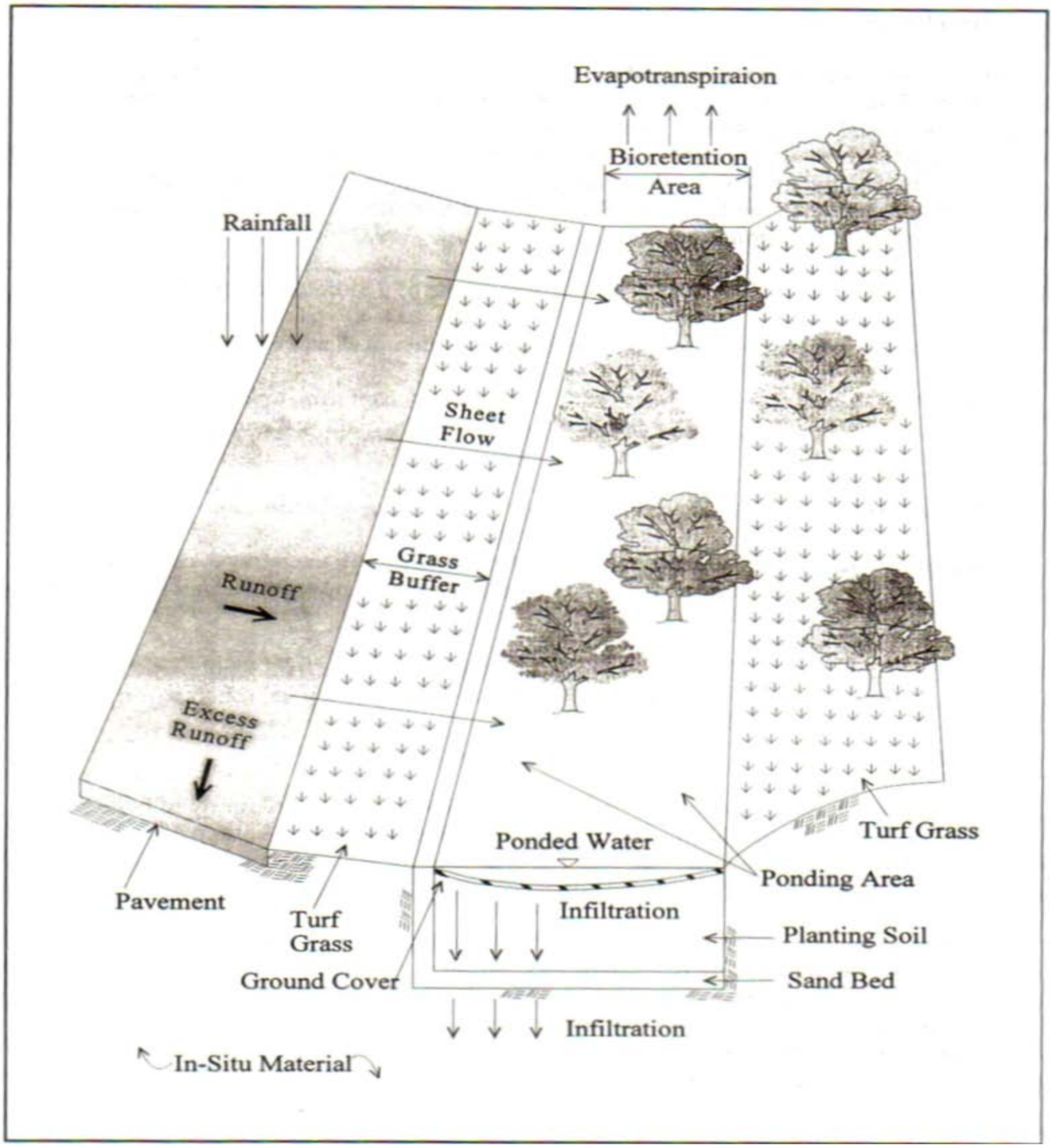


Figure 11.1. Schematic Bioretention Basin
(Virginia Stormwater Management Handbook, 1999, Et seq.)

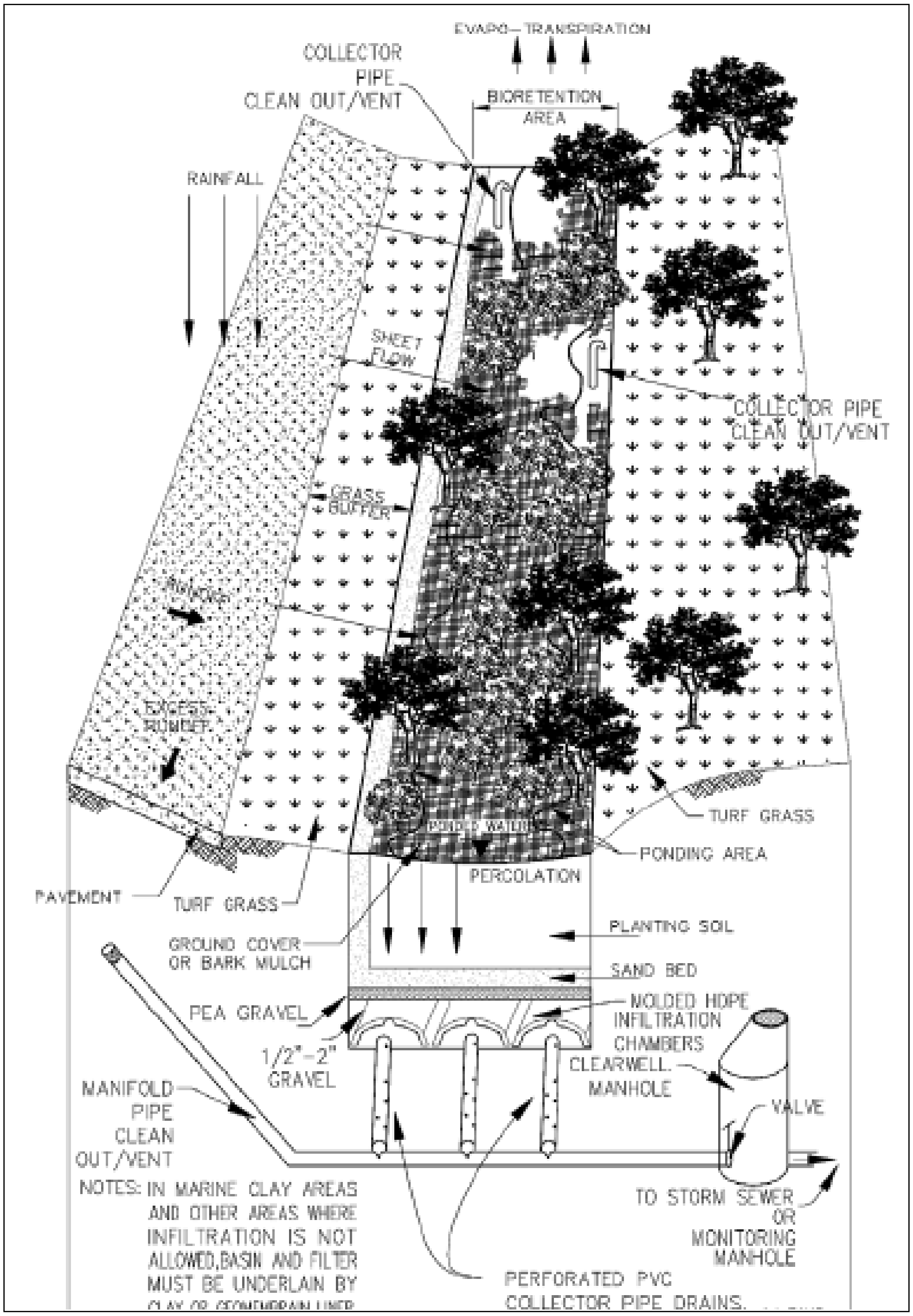


Figure 11.2. Schematic Bioretention Filter
(Virginia Stormwater Management Handbook, 1999, Et seq.)

11.2 Site Constraints and Siting of the Facility

When a bioretention facility is proposed the designer must consider a number of site constraints in addition to the contributing drainage area's impervious cover. These constraints are discussed as follows.

11.2.1 Minimum Drainage Area

The minimum drainage area contributing runoff to a bioretention cell is not restricted. However, the cost associated with constructing and maintaining a bioretention facility typically limits its use to drainage areas of at least 0.25 acres. Bioretention basins and filters are particularly well suited to small drainage areas.

11.2.2 Maximum Drainage Area

The maximum drainage area to a single bioretention facility should be restricted to no more than one acre.

11.2.3 Site Slopes

Bioretention facilities are suitable for installation on sites exhibiting average slopes less than 20 percent. Bioretention practices should be located a minimum of 50 feet away from any slope steeper than 15 percent. When average site slopes exceed 20 percent, alternative BMP measures should be considered.

11.2.4 Site Soils

This section refers to the native site soils underlying a bioretention facility. The planting soil mix of a bioretention facility is governed by specific guidelines discussed later in this chapter and also in the Virginia Stormwater Management Handbook (DCR, 1999, Et seq.).

Soil infiltration rate is a critical design element in a bioretention basin. When such a facility is proposed, *a subsurface analysis and permeability test is required*. The required subsurface analysis should investigate soil characteristics to a depth of no less than three feet below the proposed bottom of the engineered media. Data from the subsurface investigation should be provided to the Materials Division early in the project planning stages to evaluate the feasibility of such a facility on native site soils.

The soil infiltration rate should be measured when the soil is in a saturated condition. Soil infiltration rates which are deemed acceptable for bioretention facilities range between *0.52 and 8.27 inches per hour*. Infiltration rates falling within this range are typically exhibited by soils categorized as loam, sandy loam, and loamy sand.

Soils exhibiting a clay content of greater than 30 percent are unacceptable for bioretention facilities. Similarly, soils exhibiting extremely high infiltration rates, such as some types of sand, should also be avoided. Table 11.1 presents typical infiltration rates observed for a variety of soil types. This table is provided as a reference only, and does not replace the need for a detailed site soil survey.

Texture Class	Effective Water Capacity (C_w) (inch per inch)	Minimum Infiltration Rate (*f*) (inch per hour)	Hydrologic Soil Grouping
Sand	0.35	8.27	A
Loamy Sand	0.31	2.41	A
Sandy Loam	0.25	1.02	B
Loam	0.19	0.52	B
Silt Loam	0.17	0.27	C
Sandy Clay Loam	0.14	0.17	C
Clay Loam	0.14	0.09	D
Silty Clay Loam	0.11	0.06	D
Sandy Clay	0.09	0.05	D
Silty Clay	0.09	0.04	D
Clay	0.08	0.02	D

Table 11.1. Hydrologic Soil Properties Classified by Soil Texture

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

11.2.5 Depth to Water Table

Bioretention basins should not be installed on sites with a high groundwater table. Inadequate separation between the BMP bottom and the surface of the water table may result in contamination of the water table. This potential contamination arises from the inability of the soil underlying the BMP to filter pollutants prior to their entrance into the water table. Additionally, a high water table can flood the bioretention cell and render it inoperable during periods of high precipitation and/or runoff. A separation distance of no less than two feet is required between the bottom of a bioretention basin and the surface of the *seasonally* high water table. Unique site conditions may arise which require an even greater separation distance. Bioretention filters (Figure 11.2) may be considered for use on sites where a high groundwater table prohibits the use of a bioretention basin.

11.2.6 Separation Distances

Bioretention basins should be located at least 20 feet down-slope and at least 100 feet up-slope from building foundations. Bioretention basins should not be located within 100 feet of any water supply well. Local health officials should be consulted when the implementation of a bioretention basin is proposed within the vicinity of a septic drainfield. Generally, bioretention filters should be considered over bioretention basins for implementation in the vicinity of water supply wells, septic drainfields, and structural foundations. This is because bioretention filters provide conveyance of runoff by the local storm sewer upon percolation through the filter media, whereas bioretention basins infiltrate runoff to the surrounding subsoil.

11.2.7 Bedrock

A minimum of two feet of separation is required between the bottom of a bioretention basin and bedrock, with four feet or greater recommended.

11.2.8 Placement on Fill Material

Bioretention basins should not be constructed on or nearby fill sections due to the possibility of creating an unstable subgrade. Fill areas are vulnerable to slope failure along the interface of the in-situ and fill material. The likelihood of this type of failure is increased when the fill material is frequently saturated, as anticipated when a bioretention basin.

11.2.9 Karst

The concentration of runoff into a bioretention basin may result in the formation of flow channels. Such channels may lead to collapse in karst areas, and therefore the implementation of bioretention basins in known karst areas should be avoided.

11.2.10 Existing Utilities

Bioretention facilities can often be constructed over existing easements, provided permission to construct the strip over these easements is obtained from the utility owner *prior* to design of the strip.

11.2.11 Wetlands

When the construction of a bioretention facility is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation in their vicinity.

11.2.12 Perennial and Chlorinated Flows

Bioretention facilities must not be subjected to continuous or very frequent flows. Such conditions will lead to anaerobic conditions which support the export of previously captured pollutants from the facility. Additionally, bioretention facilities must not be subjected to chlorinated flows, such as those from swimming pools or saunas. The presence of elevated chlorine levels can kill the desirable bacteria responsible for the majority of nitrogen uptake in the facility.

11.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing a bioretention facility for improvement of water quality.

11.3.1 Facility Location

When the proposed bioretention facility is to receive runoff in the form of sheet flow, the overall grading of the site must direct all runoff to the facility prior to its leaving the site or entering a downstream conveyance system. Consequently, the proposed location of a bioretention facility must be established early in the project design phase and remain an integral component of the site design throughout.

11.3.2 Basin Size

The minimum floor area of a bioretention facility is a function of the water quality volume (WQV) to be treated from the facility's contributing drainage area. Table 11.2 shows the minimum bioretention floor areas as a function of WQV.

Bioretention Floor Area	WQV
2.5% of Contributing Impervious Area	0.5 Inches Over Impervious Area
4.0% of Contributing Impervious Area	1.0 Inches Over Impervious Area

Table 11.2. Minimum Bioretention Floor Area

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

The minimum size for any bioretention facility should be 10 feet wide (perpendicular to incoming sheet flow direction) and 15 feet long.

11.3.3 Basin Depth

The depth of the facility's planting soil (reference Figure 11.1) should be approximately 30 inches, *or* the diameter of the largest plant root ball plus 4 inches.

11.3.4 Surface Ponding Depth

The depth of ponding on the facility surface should be restricted to no more than 6 inches to preclude the development of anaerobic conditions within the planting soil.

11.3.5 Design Infiltration Rate

To provide a factor of safety, and to account for the decline in performance as the facility ages, the soil infiltration rate upon which a bioretention basin design is founded should be one-half the infiltration rate obtained from the geotechnical analysis.

11.3.6 Runoff Pretreatment

Bioretention facilities *must* be preceded upstream by some form of runoff pretreatment. Roadways and parking lots often produce runoff with high levels of sediment, grease, and oil. These pollutants can potentially clog the pore space in the facility, thus greatly reducing its pollutant removal performance. The selection of runoff pretreatment is primarily a function of the type of flow entering the facility, as disused below.

Runoff entering a bioretention basin or filter as *sheet flow* may be treated by a grass filter strip. The purpose of the grass buffer strip/energy dissipation area is to reduce the erosive capabilities of runoff prior to its entrance into the bioretention area. The recommended length of the grass buffer strip is a function of the land cover of the contributing drainage area and its slope. Under no circumstance should the grass buffer strip be less than 10 ft. The following table provides guidance in sizing the grass buffer strip leading to the bioretention area:

Parameter	Impervious Parking Lots				Residential Lawns				Notes
Maximum Inflow Approach Length (feet)	35		75		75		150		
Filter Strip Slope	≤ 2%	≥2%	≤2%	≥2%	≤2%	≥2%	≤2%	≥2%	Maximum = 6%
Filter Strip Minimum Length	10'	15'	20'	25'	10'	12'	15'	18'	

Table 11.3. Design Parameters for Grass Buffer Pretreatment

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

Flow may enter the bioretention facility in a concentrated flow regime. In such cases, a common pretreatment method is to pass the incoming flow through a grass-lined channel equipped with a pea gravel diaphragm prior to its entrance into the bioretention area. The recommended length of the grass swale is a function of the land cover of the contributing drainage area and its slope. When used as pre-treatment for bioretention facilities, grass swales should be at least 20 feet in length. The following table provides guidance in sizing the grass swale leading to the bioretention area:

Parameter	≤ 33% Impervious		Between 34% and 66% Impervious		≥ 67% Impervious		Notes
Slope	≤2%	≥2%	≤2%	≥2%	≤2%	≥2%	Maximum slope = 4%
Grassed channel minimum length (feet)	25	40	30	45	35	50	Assumes a 2' wide bottom width

Table 11.4. Design Parameters for Grass Swale Pretreatment

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

11.3.7 Offline Configurations

Whenever possible, bioretention facilities should be placed off-line so that flow is diverted onto it. This permits the facility to fill with only the desired treatment volume and bypass any remaining flow to the storm drainage system. Because offline bioretention BMPs are sized to accommodate only the designated water quality volume, a flow-splitter or diversion weir must be designed to restrict inflows to the bioretention area.

The flow-splitter or diversion weir must be designed to admit a designated *volume* of runoff into the basin rather than to simply regulate the flow *rate* into the basin. The diversion structure may be prefabricated, or cast in place during construction. A schematic illustration of the flow-splitting weir is shown as follows:

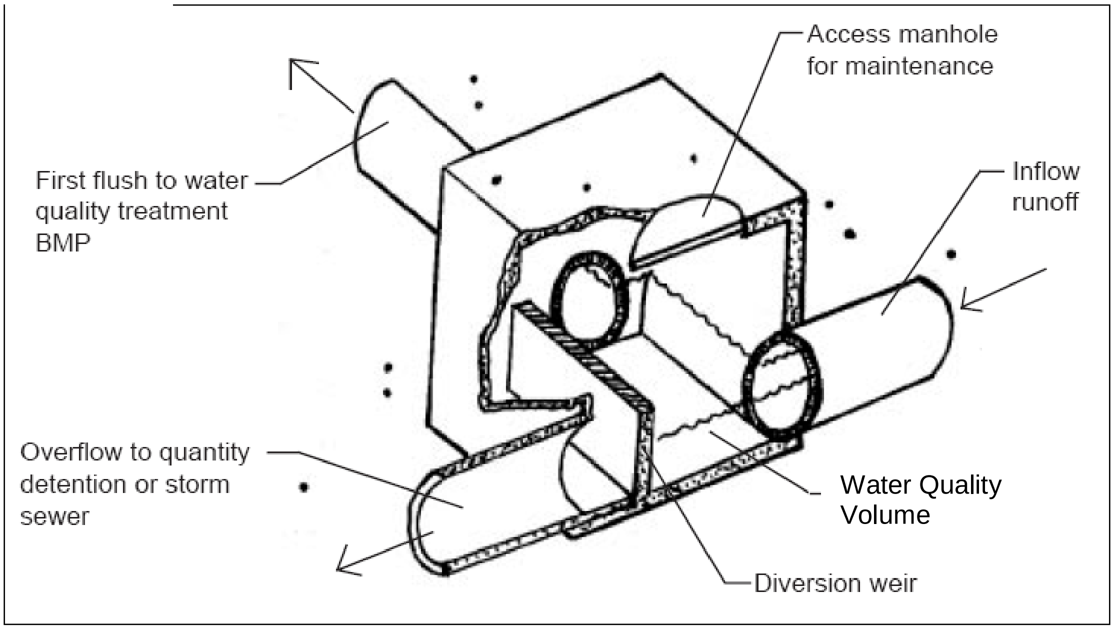


Figure 11.3. Flow-splitting Diversion Weir (Bell, Warren, 1993)

Typically, the construction of the diversion weir will place its crest elevation equal to the maximum allowable ponding depth in the bioretention area (6 inches for bioretention basins and 12 inches for bioretention filters). Flow over the diversion weir will occur when runoff volumes exceed the computed water quality volume. These overflows then enter the stormwater conveyance channel. This configuration results in minimal mixing of the held water quality volume with flows from large runoff producing events in excess of this volume. A modified design referred to as a *dual pond system* is characterized by a diversion weir which directs the computed water quality volume into the bioretention area, while conveying excess volumes downstream to a peak mitigation detention pond.

11.3.8 Overflow/Bypass Structure

When a bioretention facility is constructed online, or the maximum volume of flow entering the facility is not otherwise restricted, an overflow structure *must* be provided. This structure provides bypass for excess runoff when the bioretention subsurface and surface capacity is met. Common overflow structures include domed risers, grate or slot inlets, and weir structures. Budget, site aesthetics, and maintenance will govern the selection of the overflow structure. The sizing of the overflow structure must consider the flow rate for the design storm of interest, typically the 10-year runoff producing event. The crest or discharge elevation of the overflow structure should be set an elevation of 6 inches above the mulch layer of the bioretention bed. When designed as a bioretention *filter*, and equipped with an underdrain system, the crest of the overflow may be set at an elevation as much as one foot above the mulch layer of the facility. Typical domed riser overflow structures are shown in Figure 11.4.

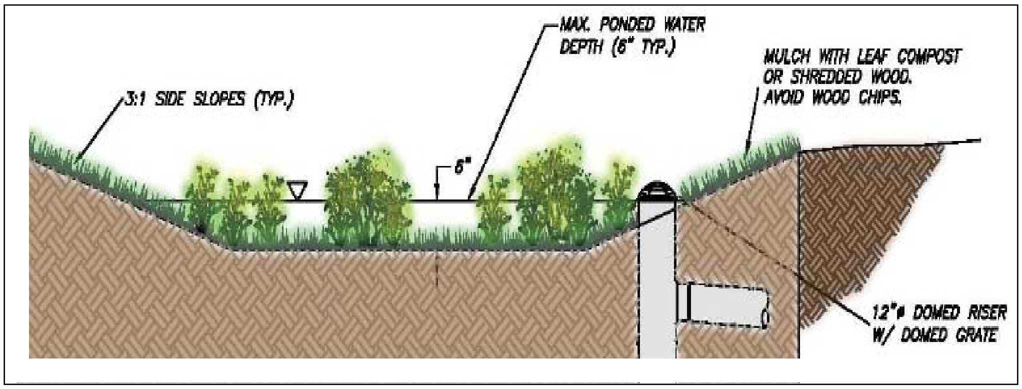


Figure 11.4. Typical Domed Riser Bypass Structure Configuration (PADEP, 2006)

11.3.9 Planting Considerations

The ultimate goal in the selection and location of vegetation within a bioretention facility is to, as closely as possible, mimic an upland (non-wetland) terrestrial forest ecosystem. This type of planting scheme is based on a natively-occurring forest's ability to effectively cycle and assimilate nutrients, metals, and other pollutants through the plant species, underlying soil, and also the system's organic matter. Of additional concern in the selection of vegetative planting species is aesthetics. Bioretention BMPS can often be incorporated into the stormwater management plans of high profile areas, providing a desirable site amenity in the form of landscaping. The design of bioretention facilities requires a working knowledge of indigenous horticultural practices, and it is recommended that a landscape architect or other qualified professional participate in the design process.

The Virginia Stormwater Management Handbook (DCR, 1999, Et seq.) provides a list of species suitable for inclusion in a bioretention facility. These species can be found in *Tables 3.11-7A – 3.11-7C* of the handbook. Species included have been deemed suitable based on their ability to tolerate pollutant loading, soil moisture fluctuations, and frequent inundation. Species not included in these tables *should not be selected* because they are not capable of surviving the conditions anticipated in a bioretention facility and/or they do not provide a desired level of pollutant uptake.

A minimum of three different species of trees and three different species of shrubs should be selected for *each* individual bioretention facility. Such diversity in species selection assists in reducing monoculture mortality concerns as well as providing a constant and predictable level of evapotranspiration and pollutant uptake. The ratio of shrubs to trees should range between 2:1 and 3:1.

A general guideline for determining the number of individual plantings required for a given bioretention area is 1,000 individual stems per planted acre. Table 11.5 provides average, maximum, and minimum planting guidelines as well as spacing recommendations.

	Tree Spacing (feet)	Shrub Spacing (feet)	Total Density (stems/acre)
Maximum	19	12	400
Average	12	8	1000
Minimum	11	7	1250

Table 11.5. Recommended Tree and Shrub Spacing

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

The Virginia Stormwater Management Handbook (DCR, 1999, Et seq.) provides a full discussion on the desirable planting soil and mulch layer characteristics of a bioretention facility in Minimum Standard 3.11. The planting soil of a bioretention facility should exhibit a pH ranging between 5.5 and 6.5 and a clay content of no greater than 5 percent.

11.4 Design Process

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

The bioretention basin design will meet the technology-based water quality requirements arising from construction of a *Park-and-Ride* facility located in York County. Site grading is such that runoff from the facility's parking lot is directed onto the bioretention area through a curb cut along the parking lot's downstream edge. This example is an *online* configuration, and therefore the facility must be equipped with a bypass for flows exceeding the storage capacity of the bioretention cell.

The total project site, including right-of-way and all permanent easements, consists of 1.34 acres. Pre and post-development land cover and hydrologic characteristics are summarized below in Tables 11.6 and 11.7. Geotechnical investigations reveal the saturated soil infiltration rate to be 1.8 inches per hour. The project site does not exhibit a high or seasonally high groundwater table.

	Pre-Development	Post-Development
Project Area (acres)	1.34	1.34
Land Cover	Unimproved Grass Cover	0.83 acres impervious cover
Impervious Percentage	0	62

Table 11.6. Hydrologic Characteristics of Example Project Site

		York County - 10 Year Rainfall Constants				
Acreage	**Rational C**	**A**	**B**	**t_c (min)**	**i_{10} (iph)**	**Q_{10} (cfs)**
0.83	0.9	186.78	21.22	8	6.39	4.8

Table 11.7. Peak Parking Lot Runoff

Step 1. Compute the Required Water Quality Volume

The project site' water quality volume is calculated as one half inch over the developed Impervious Area. This *basic* water quality volume is computed as follows:

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

IA= impervious area (ac.)

The project site in this example has a total drainage area of 1.34 acres. The total impervious area within the site is 0.83 acres. Therefore, the water quality volume is computed as follows:

$$WQV = \frac{0.83ac \times \frac{1}{2}in \times \frac{43{,}560\,ft^2}{ac}}{12\frac{in}{ft}} = 1{,}506\,ft^3$$

Step 2. Compute the Minimum Basin Floor Area

The minimum allowable bioretention surface area is a function of the site's water quality volume. The water quality volume in this example was based on one-half inch of runoff from the site's impervious cover. Therefore, referencing Table 11.2, the minimum floor area of the facility is 2.5 percent of the contributing impervious cover, computed as follows:

$$Area = 0.83ac \times \frac{43{,}560\,ft^2}{ac} \times 0.025 = 904\,ft^2$$

The minimum dimensions of a bioretention facility should be 10 feet wide (perpendicular to the incoming flow direction) and 15 feet long. The actual length to width ratio of the facility as well as its overall geometric configuration is determined by various site constraints such as topography and available area. In this example, we will employ a length to width ratio of 1.5:1. Therefore, the approximate dimensions of the facility are computed as follows:

$$\begin{aligned}
&L = 1.5W \\
&L \times W = 904\,ft^2 \\
&1.5W \times W = 904\,ft^2 \\
&W = 24.5\,ft \\
&L = 37\,ft
\end{aligned}$$

For bioretention areas with a preliminary computed length of greater than 20 feet, the actual design length should be twice that which ensures dispersal of incoming sheet flow. The following steps illustrate the process for evaluating whether or not the preliminary computed length must be increased to meet this requirement.

The bioretention area will be preceded upstream by pretreatment in the form of a grass filter strip. Runoff will leave the proposed parking lot through a curb cut, and then discharge onto the filter strip after passing over a level spreader. The size of the level spreader is a function of the 10-year flow from the contributing drainage area. The required level spreader dimensions are shown in Table 11.8.

Q10 (cfs)	Depth (ft)	Width of Lower Side Slope of Spreader (ft)	Length (ft)
0-10	0.5	6	10
20-10	0.6	6	20

Table 11.8. Minimum Level Spreader Dimensions

Source: Virginia Erosion and Sediment Control Handbook (DCR, 1992)

The 10-year peak rate of runoff from the roadway is 4.8 cfs (see Table 11.7). Therefore, the minimum level spreader "lip" length that will discharge runoff onto the strip is 10 feet. The chosen bioretention length of 37 feet is more than twice the level spreader length of 10 feet discharging sheet flow onto the grass filter strip, and is therefore acceptable.

Step 3. Specify Bioretention Depth

The depth of the facility's planting soil should be approximately 30 inches, *or* the diameter of the largest plant root ball plus 4 inches. Site grading and placement of the facility's overflow structure must ensure a maximum surface ponding depth of 6 inches.

Step 4. Design Overflow Structure

An overflow structure must be provided for large runoff producing events to bypass excess runoff when the bioretention surface and subsurface storage capacity is exceeded. The crest/outflow of the bypass system should be set at an elevation 6 inches above the surface of the bioretention floor. This will ensure discharge through the bypass system only when the design parameters of the bioretention area have been exceeded. Common overflow structures include domed risers, grate or slot inlets, and weir structures. The overflow/bypass system will function as a conventional storm sewer system when the facility's planting soil is saturated and a ponding depth of 6 inches is observed on the surface of the facility. Therefore, the bypass system should be designed to carry a peak 10-year flow rate of 4.8 cfs (reference Table 11.7). The bypass system must discharge into an adequate receiving channel as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR, 1992). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

Sizing of the bypass pipe is accomplished by use of the Manning equation shown below:

$$Q = \frac{1.49}{n} \cdot AR_h^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$$

A typical Manning's *n* value for reinforced concrete pipe is 0.013. For a fixed discharge, *Q*, the minimum required diameter, *D*, of a circular pipe flowing full can be computed by the following equation:

$$D = \left[\frac{2.16(Q)(n)}{S^{1/2}} \right]^{0.375}$$

D = minimum pipe diameter (ft)
Q = pipe discharge (cfs)
N = Manning's roughness coefficient
S = pipe slope (ft/ft)

Assuming a slope of 1.5 percent on the overflow pipe, we compute the minimum pipe diameter required to convey the facility's 10-year runoff as:

$$D = \left[\frac{(2.16)(4.8)(0.013)}{0.015^{1/2}} \right]^{0.375} = 1.04\,ft = 12.5\,inches$$

The bypass pipe shall be 15 inches in diameter.

The 15" bypass pipe shall connect to a conventional stormwater conveyance system and/or carry runoff volumes in excess of the water quality volume to an adequate receiving channel.

Step 5. Specify Number of Vegetative Plantings

A typical bioretention facility should be planted with approximately 1,000 stems per acre. This vegetation should be comprised of both shrubs and trees, with a shrub to tree ratio ranging between 2:1 and 3:1. A minimum of three different species of trees and three different species of shrubs should be specified, with specific plant species determined from Tables 3.11-7A – 3.11-7C of the Virginia Stormwater Management Handbook (DCR, 1999, Et seq.).

Employing a 2.5:1 shrub to tree ratio, the number of shrubs and trees for the proposed bioretention area is determined as follows:

Total bioretention area: $$24.5\,ft \times 37\,ft \times \frac{1ac}{43{,}560\,ft^2} = 0.02ac$$

Total number of stems: $$0.02ac \times 1{,}000\frac{stems}{ac} = 20$$

Total number of shrubs (s): s = 2.5 x # trees

Total number of trees (t): $$2.5t + t = 20 \Rightarrow t = 5.7$$

The bioretention area should be planted with 6 trees, 2 each from three different species. Additionally, a total of 15 shrubs should be planted, 5 each from three different species.

Step 6. Provide for Runoff Pretreatment

Runoff entering the proposed bioretention cell will pass through an upstream grass filter strip serving the purpose of pretreating the incoming runoff. Sizing of this filter strip is based on Table 11.3. The slope of the filter strip will be approximately 1.5 percent and the maximum flow path across the impervious parking lot is 75 feet. Obtained from Table 11.3, these parameters require a filter strip length of 20 feet.

Alternative Design – Bioretention Filter

Bioretention filters provide water quality improvement in essentially the same manner as bioretention basins, but are used in locations where full infiltration is not feasible either due to inadequate soil permeability or the proximity to wells, drainfields, or structural foundations. Bioretention filters are equipped with a connection to the site's storm sewer system such that water enters the storm sewer after it has filtered through the bioretention cell (see Figure 11.2). The same sizing and design parameters apply to bioretention filters as apply to bioretention basins, with the exception of maximum surface ponding depth. Because runoff filters through a bioretention filter more quickly than through a bioretention basin, the maximum surface ponding depth may be increased to 12 inches.

When a bioretention filter is chosen due to the proximity of the facility to wells, structural foundations, or septic drainfields, *the entire basin must be underlain by a synthetic liner* as approved by the Materials Division. When the selection of a bioretention filter arises due to inadequately low percolation rates of the site's native soils, the synthetic membrane may be omitted.

Chapter 12 –Stormwater Sand Filters

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

12.1 Overview of Practice

Stormwater sand filters are practices employed when the runoff from a site is expected to contain very high pollutant levels. These sand filters function by first pre-treating and temporarily storing runoff to remove the bulk of the large particle sediment, then percolating the runoff through the filter's sand media. As runoff filters through the sand media, water quality is improved through physical, chemical, and biological mechanisms. Various types of stormwater sand filters exist, and their application can be tailored to meet individual site needs. The most common types of stormwater sand filters are the Washington D.C. underground vault sand filter, the Delaware sand filter, and the Austin surface sand filter.

Stormwater sand filters act primarily as water quality BMPs; however, the water quality volume entering the filter is detained and released at a rate potentially capable of providing downstream channel erosion control. Peak rate control of the 10-year and greater storm events is typically beyond the capacity of a stormwater filtering system, and may require the use of a separate structural peak rate reduction facility.

Stormwater sand filters are commonly used in urbanized settings where entering runoff is generated from areas whose imperviousness ranges from 67 – 100 percent. The primary cause of failure in stormwater filtering systems is the clogging of the sand media through excessive sediment loading. The filters described in this document should not be used on sites having an impervious cover of less than 65 percent.

The Virginia Stormwater Management Handbook, (DCR, 1999, Et seq., Et seq.) identifies three types of stormwater stand filters appropriate for use in the state. These are the Washington D.C. Underground Vault Sand Filter, the Delaware Sand Filter, and the Austin Surface Sand Filter. Each filter type is described briefly in the following section.

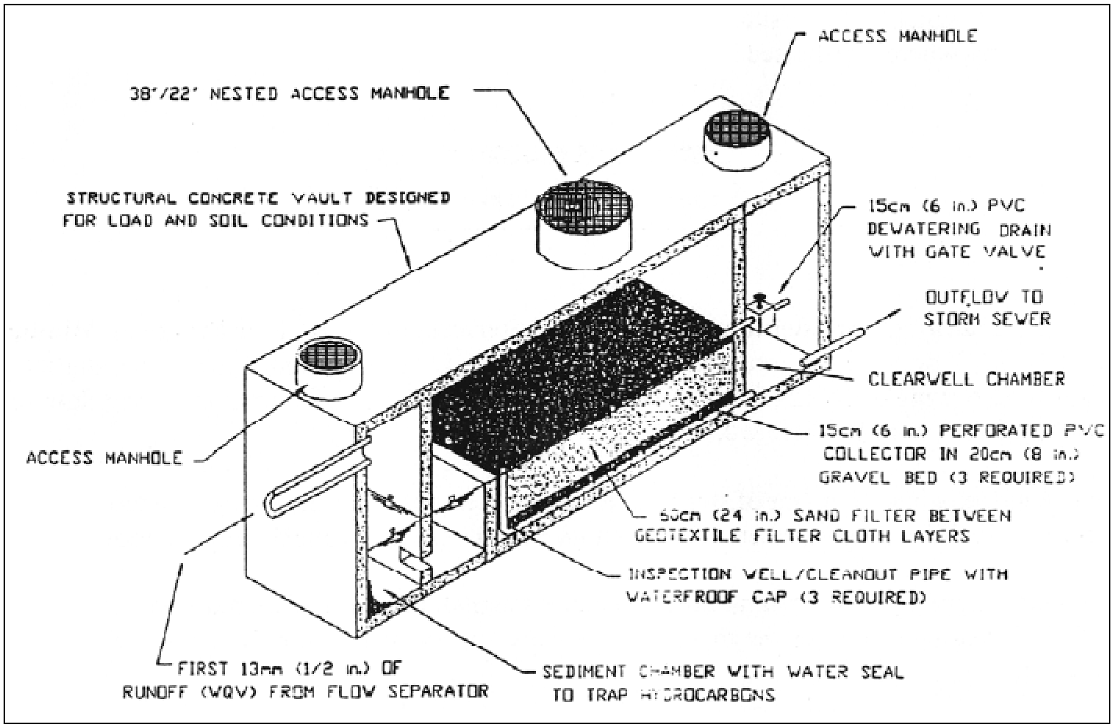


Figure 12.1. Washington D.C. Underground Vault Sand Filter
(Virginia Stormwater Management Handbook, 1999, Et seq.)

The Washington D.C. underground vault sand filter shown in Figure 12.1 can be either precast or cast in place and is composed of three chambers. The first chamber is a three foot deep "plunge pool" which absorbs energy and pre-treats runoff by trapping sediment and floating organic matter. The first chamber is hydraulically connected to the second chamber containing the sand filter media. Finally, the third chamber serves as a collection point for filtered runoff, where it is then directed to the downstream storm sewer. This type of filter is typically constructed *offline*, with only the site water quality volume directed to the structure.

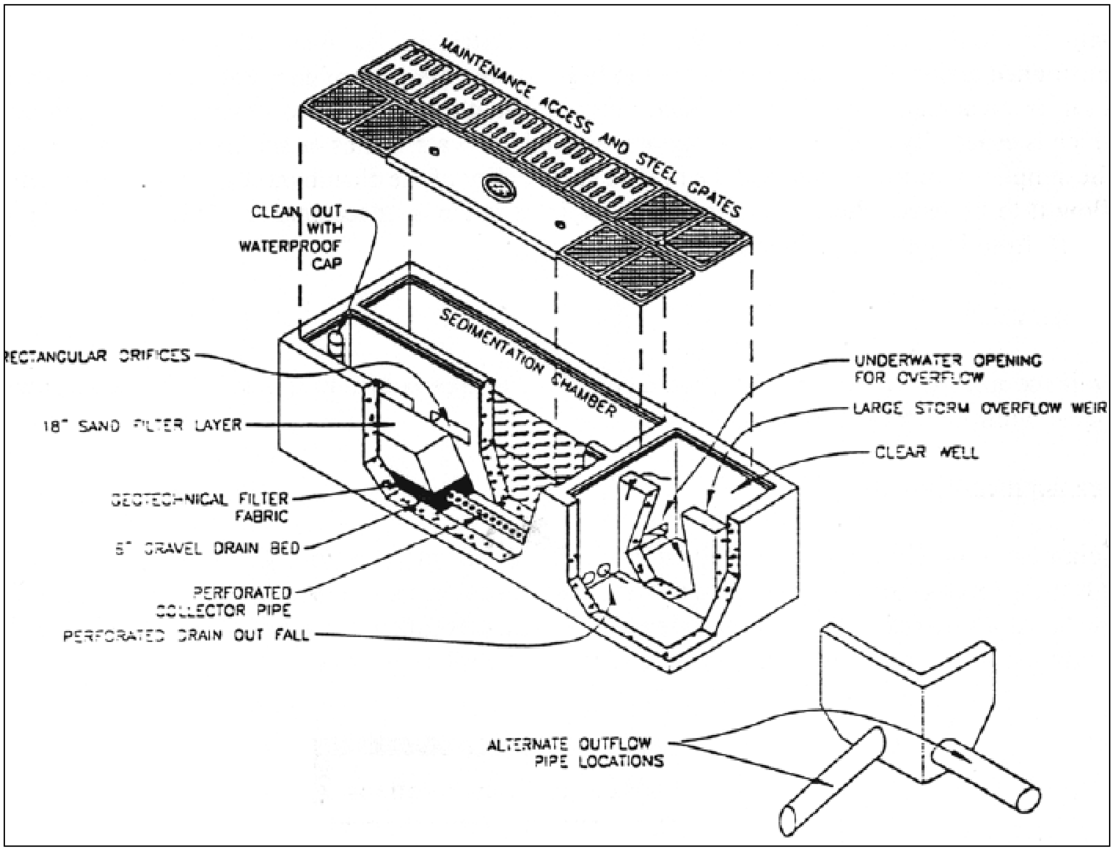


Figure 12.2. Delaware Sand Filter
(Virginia Stormwater Management Handbook, 1999, Et seq.)

The Delaware sand filter shown in Figure 12.2 was originally conceived as an *online* facility (unlike the Washington D.C. sand filter), processing all runoff leaving its contributing drainage shed up to the point that overflow is reached. When applied on VDOT projects, the Delaware sand filter should be equipped with a flow-splitting device such that only the site water quality volume is treated by the filter. The Delaware sand filter is characterized by two parallel chambers, one serving as pre-treatment sedimentation chamber and the other holding the sand filter media. The pre-treatment chamber holds a permanent pool analogous to that of a septic tank. Flow entering the pre-treatment chamber causes the water level in the chamber to rise and eventually spill into the filter chamber where full treatment occurs. Upon filtering through the sand media, treated runoff is collected in the clearwell located at the lower end of the structure. From there, the treated runoff is directed to the receiving storm sewer.

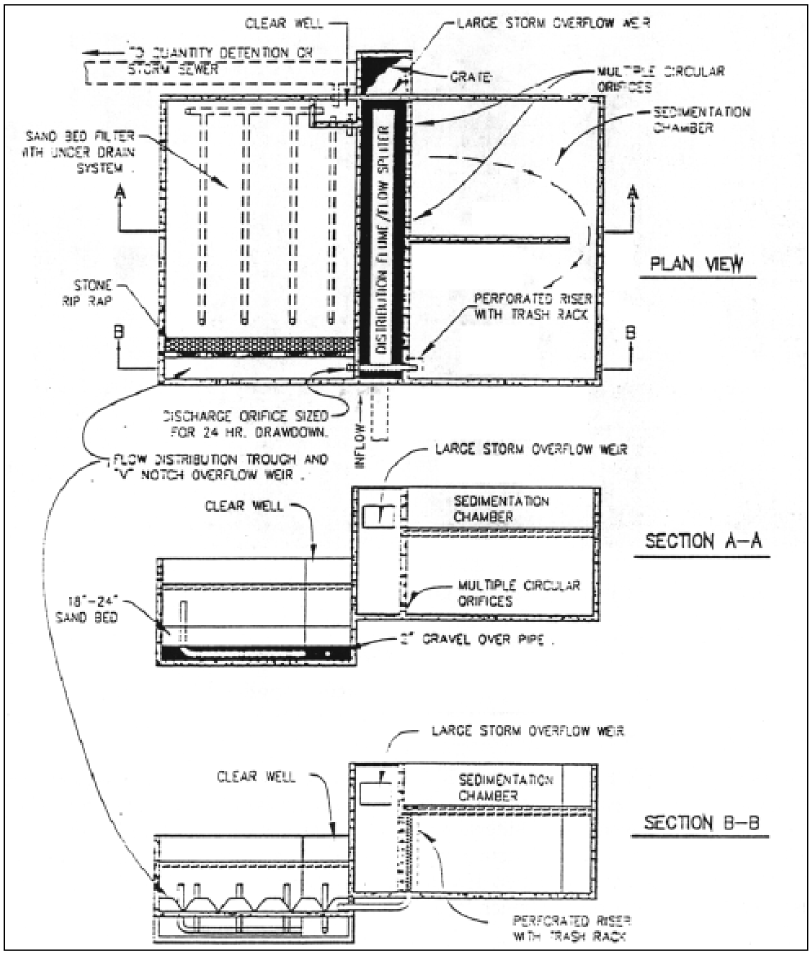


Figure 12.3. Austin Surface Sand Filter
(Virginia Stormwater Management Handbook, 1999, Et seq.)

The Austin surface sand filter, as shown in Figure 12.3, is composed of an open basin characterized by a pre-treatment sedimentation basin that is often large enough to hold the entire water quality volume from the contributing drainage shed. This volume is then released into the sand bed filtration chamber over a period of 24 hours. Alternative designs employ a much smaller sedimentation chamber, and compensate for the increased clogging potential by increasing the surface area of the filtration chamber. Typically, both chambers of the Austin filter are constructed of concrete; however, when soil conditions and/or the application of a geomembrane liner permit, the pre-treatment sedimentation chamber may be constructed into the ground.

12.2 Site Constraints and Siting of the Filter

The designer must consider a number of site constraints in addition to the contributing drainage area's impervious cover when a stormwater sand filter is proposed. These constraints are discussed as follows.

12.2.1 Minimum Drainage Area

The minimum drainage area contributing to an intermittent stormwater sand filter is not restricted. These types of filters are best suited to small drainage areas.

12.2.2 Maximum Drainage Area

The maximum drainage area to a single stormwater sand filter varies by filter type. Table 12.1 shows the impervious acreage which may be directed to a single filter, as a function of filter type.

Filter Type	Appropriate Drainage Shed (Impervious Acres)
D.C. Underground Vault	0.25 – 1.25
Delaware	1.25 Maximum
Austin Surface	Greater than 1.25

Table 12.1. Appropriate Drainage Area by Filter Type

Austin surface sand filters have been applied on sites with drainage areas as large as 30 acres; however on sites greater than 10 acres, despite a reduction in cost per volume of runoff treated arising from the economy of scale, the cost-effectiveness of an Austin sand filter is often poor when compared to alternative BMP options.

12.2.3 Elevation of Site Infrastructure

Whenever possible, stormwater filtering systems should be designed to operate exclusively by gravity flow. This requires close examination of the difference in elevation between the filter's discharge point (manhole, pipe, or receiving channel) and the storm sewer discharging runoff into the filter. This difference in elevation dictates the hydraulic head available on the filter while still remaining in a state of gravity flow. When the filter's clearwell discharge point is below the elevation of the downstream receiving point, an effluent pump is a viable alternative; however, this option requires routine scheduled maintenance by trained crews knowledgeable in the maintenance of such mechanical equipment.

12.2.4 Depth to Water Table and/or Bedrock

The liner or concrete shell of a sand filter should generally be located 2 to 4 feet above the site seasonally high water table. The presence of a high water table can flood the filter during construction. Additionally, placing a sand filter within the groundwater table may give rise to infiltration, thus flooding the filter and rendering it inoperable during periods of inflow. When it is deemed feasible and desirable to employ an intermittent sand filter on a site exhibiting a shallow groundwater table, the effects of infiltration and flotation must be accounted for. The liner or concrete shell of the filter must be waterproofed in accordance with the methods and materials specified by the Materials

Division. Additionally, buoyancy calculations must be performed and additional weight provided within the filter as necessary to prevent floatation.

12.2.5 Existing Utilities

Sand filters may be constructed over existing easements, provided permission to construct the facility over these easements is obtained from the utility owner *prior* to design.

12.2.6 Wetlands

When the construction of a sand filter is planned in the vicinity of known wetlands, the designer must coordinate with the appropriate local, state, and federal agencies to identify wetlands boundaries, their protected status, and the feasibility of BMP implementation in their vicinity.

12.2.7 Upstream Sediment Loading

The primary cause of filter failure is premature clogging arising from the presence of excessive sediment in the runoff directed to the filter. Therefore, runoff directed to stormwater filters should originate primarily from small impervious watersheds. In most applications, runoff flows through an open air “pretreatment” chamber prior to entering the filter chamber. This process allows large particles and debris to settle out. The filters described in this document should not be used on sites exhibiting an impervious cover of less than 65 percent.

12.2.8 Aesthetic Considerations

Stormwater sand filters provide an attractive BMP option on high profile sites where visually obtrusive BMPs such as extended dry detention facilities and other basins are undesirable. Typically, sand filtration BMPs are visually unobtrusive and may be located on sites where aesthetic considerations and/or the preservation of open space is deemed a priority.

12.2.9 Control of Surface Debris

Sand filters constructed as underground vaults often receive “Confined Space” designation under Occupational Safety and Health Administration (OSHA) regulations. Consequently, maintenance operations involving personnel entering the vault may become quite costly. In an effort to reduce the frequency of this type of maintenance operation, prevention of trash and other debris from entering the filter should be prioritized. This is accomplished through the use of trash racks and flow-splitting devices on offline facilities.

12.2.10 Hydrocarbon Loading

Sand filters are capable of receiving hydrocarbon-laden runoff; however, the facility owner must realize that such loading conditions will inevitably lead to rapid clogging of the filter media. When the presence of hydrocarbons is anticipated in the runoff entering a sand filter, the filter's pre-treatment chamber should be designed to remove unemulsified hydrocarbons prior to their entrance into the primary filter chamber. An alternative option is to provide an upstream "treatment train" composed of a BMP(s) capable of reducing the level of hydrocarbons present in the runoff entering the sand filter.

12.2.11 Perennial and Chlorinated Flows

Sand filters must not be subjected to continuous or very frequent flows. Such conditions will lead to anaerobic conditions which support the export of previously captured pollutants from the facility. Additionally, sand filters must not be subjected to chlorinated flows, such as those from swimming pools or saunas. The presence of elevated chlorine levels can potentially kill the desirable bacteria responsible for the majority of nitrogen uptake in the facility.

12.2.12 Surface Loading

Sand filters constructed as underground vaults must have their load-bearing capacity evaluated by a licensed structural engineer. This evaluation is of paramount importance when the filter is to be located under parking lots, driveways, roadways, or adjacent to highways.

12.3 General Design Guidelines

The following presents a collection of design issues to be considered when designing a sand filter for improvement of water quality.

12.3.1 Isolation of the Water Quality Volume (WQV)

Sand filters should have only the site water quality volume directed to them. In Virginia, this is also true for the Delaware sand filter which has traditionally been installed online with stormwater conveyance systems. The most popular means of isolating the water quality volume is through the use of a diversion weir in the manhole, channel, or pipe conveying runoff to the BMP. Typically, the elevation of this weir is set equal with the water surface elevation in the BMP when the water quality volume is present. This approach ensures that flows beyond the water quality volume bypass the filter and are conveyed downstream by the storm drainage system. It is noted that the flow-splitter or diversion weir is used to convey a designated *volume* of runoff into the filter rather than to simply regulate the flow *rate* into the filter. The diversion structure may be prefabricated, or cast in place during construction. A schematic illustration of the flow-splitting weir is shown as follows:

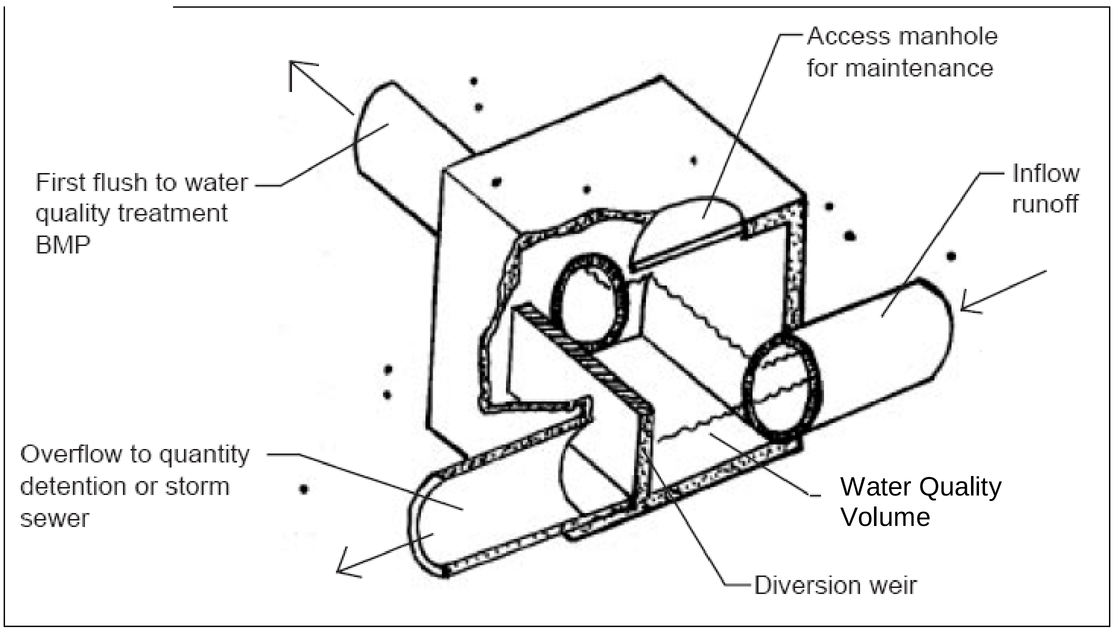


Figure 12.4. Flow-splitting Diversion Weir (Bell, Warren, 1993)

Typically, the construction of the diversion weir will place its crest elevation equal to the maximum allowable ponding depth on the sand filter. This results in flow over the diversion weir when runoff volumes greater than the computed water quality volume enter the stormwater conveyance channel. This configuration results in minimal mixing between the held water quality volume and flows from large runoff producing events in excess of this volume.

An alternative approach is to provide a “low flow” pipe leading directly from the upstream structure to the sand filter. Water enters the BMP through this low-flow conduit, and once the water level rises to that equal with the allowable ponding depth on the filter, flow is conveyed downstream by a bypass pipe located at a higher elevation. A schematic illustration of this configuration is shown as follows:

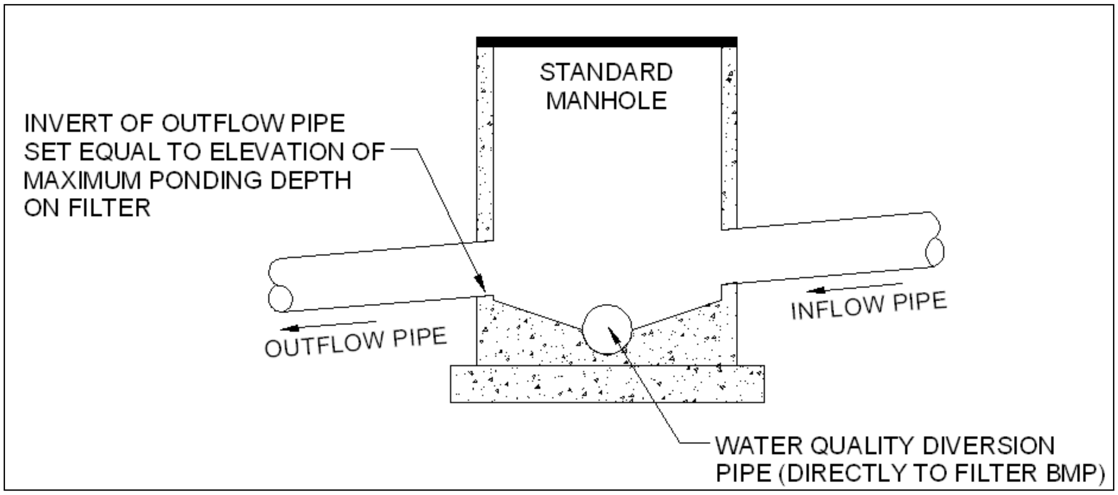


Figure 12.5. Flow-Splitting Manhole Structure

12.3.2 Sand Filter Media

The sand filter media of an intermittent sand filter should meet the specifications of VDOT Grade A Fine Aggregate or as otherwise approved by the Materials Division.

12.3.3 Discharge Flows

All filter outfalls must discharge into an adequate receiving channel as defined by Regulation MS-19 in the Virginia Erosion and Sediment Control Handbook, (DCR, 1992, Et seq.). Existing natural channels conveying pre-development flows may be considered receiving channels if they satisfactorily meet the standards outlined in the VESCH MS-19. Unless unique site conditions mandate otherwise, receiving channels should be analyzed for overtopping during conveyance of the 10-year runoff producing event and for erosive potential under the 2-year event.

12.3.4 Filter Sizing

Sand filters should be sized using a Darcy's Law approach, ensuring that the site water quality volume is filtered completely through the sand media within a maximum of 40 hours. Sizing the filter such that full drawdown of the water quality volume occurs within 40 hours ensures that aerobic conditions are maintained in the filter between storm events.

The coefficient of permeability of a filter's sand media may range as high as 3.0 feet/hour upon installation; however, due to filter clogging after only a few runoff producing events, the rate of permeability through the media has been observed to decrease considerably. Therefore, the coefficient of permeability employed in filter sizing calculations is a function of the degree to which pre-treatment is planned for the facility (full pre-treatment or partial pre-treatment). The following section presents specific sizing guidelines for each of the previously described types of sand filters in the context of a design scenario

.

12.4 Design Process

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

A design example is presented for each of the three aforementioned types of sand filter recommended for use in Virginia. The filter designs will meet the technology-based water quality requirements arising from a one-acre VDOT maintenance yard. The site water quality volume is directed into the filter by means of a diversion weir situated in the storm sewer. This example is an *offline* configuration. The design will include a Washington D.C. sand filter, a Delaware sand filter, and an Austin sand filter.

The total project site, including right-of-way and all permanent easements, consists of 1.0 acre. Pre and post-development land cover and hydrologic characteristics are summarized below in Table 12.2.

	Pre-Development	Post-Development
Project Area (acres)	1.0	1.0
Land Cover	Unimproved Grass Cover	1.0 acres impervious cover
Impervious Percentage	0	100

Table 12.2. Hydrologic Characteristics of Example Project Site

Site topography is such that the invert of the pipe exiting the sand filter from its clearwell chamber is 4.5 feet lower than the invert of the storm sewer pipe discharging runoff into the filter's pre-treatment chamber.

Step 1. Compute the Required Water Quality Volume

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

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

IA= Impervious Area (ac.)

The project site in this example is composed of a total drainage area of 1.0 acres. The total impervious area within the site is 1.0 acres. Therefore, the *basic* water quality volume is computed as follows:

$$WQV = \frac{1.0ac \times \frac{1}{2}in}{12\frac{in}{ft}} = 0.042\,\mathrm{ac}\cdot ft \times \frac{43{,}560\,ft^2}{ac} = 1{,}830\,ft^3$$

Referencing Table 1.1, sand filters treating drainage sheds whose impervious fraction ranges between 67 and 100 percent should be sized for *twice* the basic water quality volume. Therefore, the filters in this example will be sized to treat a volume of 3,660 ft^3.

Upon evaluating various site constraints, cost, and maintenance considerations the designer will select which of the aforementioned types of sand filter best meets the site water quality needs. The following section demonstrates the sizing procedure for each of three types of intermittent sand filter.

Step 2A. Size Filter and Pre-Treatment Sedimentation Chamber – Washington D.C. Underground Vault Sand Filter

The variables expressed in the D.C. sand filter sizing equations are related to the following figure.

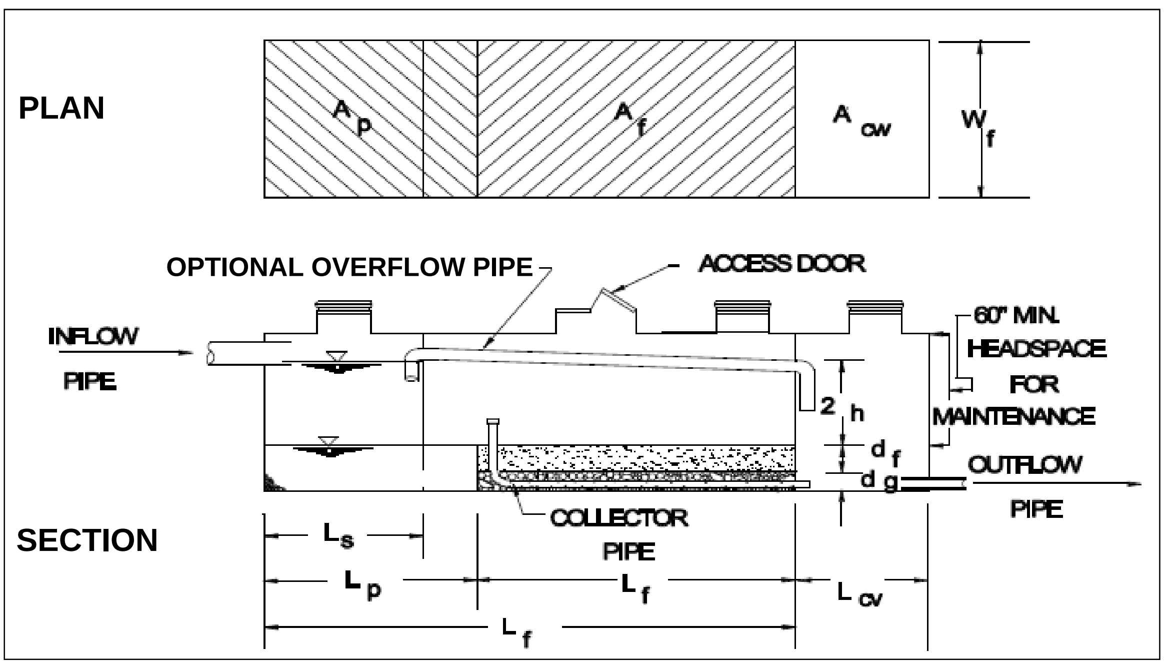


Figure 12.6. D.C. Sand Filter – Cross Section
(Virginia Stormwater Management Handbook, 1999, Et seq.)

The D.C. sand filter is a *partial pre-treatment* intermittent sand filter. The total surface area of the sand media is computed by the following equation:

$$A_f = \frac{545 I_a d_f}{\left(h + d_f\right)}$$

A_f= Minimum surface area of sand bed (square feet)
I_a= Impervious fraction of contributing drainage shed (acres)
d_f= Sand bed depth (typically 1.5 to 2.0 feet)
h= Average depth of water above surface of sand media (ft)

In this application, we will select a sand media depth of 2 feet. The sand filter media must be wrapped in a filter cloth approved by the Materials Division. Additionally, the sand layer is then underlain by a layer of ½ - 2 inch diameter washed gravel (10 inches thick) and overlain by a layer of 1 – 2 inch diameter washed gravel (1 – 2 inches thick).

The overall depth of all filter media is the sum of the sand media and the gravel underlay and overlay. This depth calculation is as follows:

$$d_m = d_f + d_g = 24in + 10in + 2in = 36in = 3\,ft$$

It was previously determined that the total elevation difference between the pipe discharging runoff into the filter and the pipe carrying effluent from the filter is 4.5 feet. Therefore, as shown in Figure 12.5, the *maximum* possible ponding depth, *2h*, on the filter is calculated by subtracting the total filter media depth from this total elevation difference:

$$2h = 4.5\,ft - 3\,ft = 1.5\,ft$$

Therefore, the average ponding depth on the filter, *h*, is determined to be *0.75 feet*.

The required surface area of the sand filter media is then computed as:

$$A_f = \frac{545(1.0ac)(2\,ft)}{(0.75\,ft + 2\,ft)} = 396.4\,ft^2$$

Next, the length and width of the filter are computed. This design will employ a rectangular configuration with at 2:1 length-to-width ratio.

$$L_f = 2W_f$$
$$2W_f^{\,2} = 396.4\,ft^2 \Rightarrow W_f = 14.1\,ft$$
$$L_f = 28.2\,ft$$

Rounding the computed dimensions to nominal values yields the following filter surface parameters:

L_f (ft)	W_f (ft)	A_f (ft2)
28.5	14	399

Table 12.3. D.C. Filter Surface Dimensions

The next step is to compute the maximum available storage volume on the surface of the filter, V_{Tf}. This is computed based on the filter surface area and the maximum possible ponding depth, *2h* (1.5 feet):

$$V_{Tf} = 399\,ft \times 1.5\,ft = 598.5\,ft^3$$

Next, the total storage volume provided in the void space of the gravel and sand media is computed. The porosity of the sand and gravel filter media is typically taken to be 40 percent.

$$V_V = 0.4 \times A_f \times (d_f + d_g)$$
$$V_V = 0.4 \times 399\,ft^2 \times (2\,ft + 1\,ft) = 478.8\,ft^3$$

The next step is to compute the volume of inflow that passes through the filter media while the total water quality volume is accumulating in the BMP. This calculation is based on a coefficient of permeability, *k*, of 2 ft/day (0.0833 ft/hr) for the sand media and a total filling time of one hour. The pass-through volume during filling is computed by the following equation:

$$V_Q = \frac{kA_f(d_f + h)}{d_f}$$

For the design parameters previously established, the pass-through volume is computed as:

$$V_Q = \frac{0.0833\frac{ft}{hr}(399\,ft^2)(2\,ft + 0.75\,ft)}{2\,ft} = 45.7\,ft^3$$

The volume which must be stored awaiting filtration is computed from the following equation:

$$V_{st} = WQV - V_{Tf} - V_V - V_Q$$

For the design parameters previously established, the required storage volume, V_{st}, is computed as:

$$V_{st} = 3{,}660\,ft^3 - 598.5\,ft^3 - 478.8\,ft^3 - 45.7\,ft^3 = 2{,}537\,ft^3$$

The volume to be stored awaiting filtration dictates sizing of the filter's permanent pool volume. The length of this pool is defined as L_p (see Figure 12.6), and is computed as follows:

$$L_p = \frac{V_{st}}{(2h \times W_f)}$$

For the design parameters previously established, the permanent pool length, is computed as:

$$L_p = \frac{2{,}537\, ft^3}{(1.5\, ft \times 14\, ft)} = 120.8\, ft$$

The next design step is to compute the length of the sedimentation chamber, L_s, to provide storage for 20 percent of the site water quality volume (standard for a partial pre-treatment practice). The length of the sedimentation chamber is computed by the following equation:

$$L_s = \frac{0.2WQV}{(2h \times W_f)}$$

For the design parameters previously established, the length of the filter's sedimentation chamber is computed as:

$$L_s = \frac{0.2 \times 3{,}660\, ft^3}{(1.5\, ft \times 14\, ft)} = 34.9\, ft$$

The final design step is to adjust the length of the permanent pool. If the computed length of the permanent pool is greater than the length of the sedimentation chamber plus 2 feet, then the permanent pool length is not adjusted; however, if the computed length of the permanent pool is less than the length of the sedimentation chamber plus 2 feet, the permanent pool length should be increased to dimensions of L_s + *2 feet*. In this example no adjustment is necessary.

Table 12.4 presents the final design summary of the Washington D.C. sand filter, with variables as defined in Figure 12.6.

Filter Length (L_f) ft	**Filter Width (W_f) ft**	**Filter Area (A_f) ft^2**	**Permanent Pool Length (L_p) ft**	**Sedimentation Chamber Length (L_s) ft**
28.5	14	399	120.8	34.9

Table 12.4. Design Summary – D.C. Sand Filter

Special Considerations for Implementation of a Washington D.C. Intermittent Sand Filter

- For maintenance access, a minimum of 60 inches of headroom is required in the sedimentation and filter chambers. In the filtration chamber, this headroom should be measured from the top of the filter media.

- Passage of flow from the sedimentation chamber to the filter chamber should occur through an opening located a minimum of 18 inches below the depth of the weir dividing the two chambers. The cross-sectional area of this opening should, at a minimum, be 1.5 times the area of the pipe(s) discharging into the BMP.

- The total depth of the filter media must at least equal the height of weir separating the sedimentation and filtration chambers

- The filtration bed's underdrain piping should consist of three 6-inch diameter schedule 40 perforated PVC pipes placed on 1 percent slope. Perforations should be 3/8 inch diameter with maximum spacing between perforated rows of 6 inches. The underdrain piping should be placed within the gravel filter media with a minimum of 2 inches of cover over the pipes.

- When the filter is placed underground, a dewatering drain controlled by a gate valve must be located between the filter chamber and the clearwell chamber.

- Access should be provided to each filter chamber through manholes of at least 22 inches in diameter.

Step 2B. Size Filter and Pre-Treatment Sedimentation Chamber – Delaware Sand Filter

The variables expressed in the Delaware sand filter sizing equations are related to the following figure:

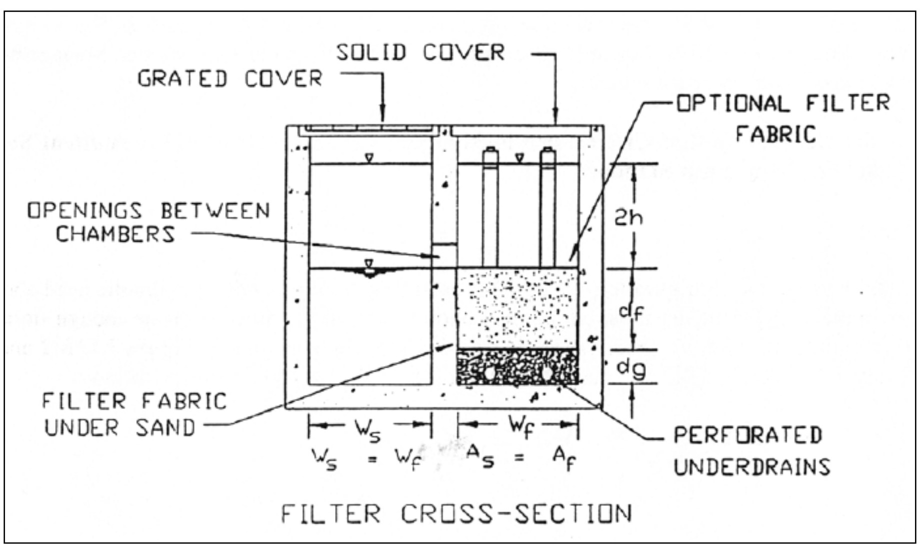


Figure 12.7. Delaware Sand Filter – Cross Section
(Virginia Stormwater Management Handbook, 1999, Et seq.)

The Delaware sand filter's shallow configuration typically results in minimal hydraulic head acting on the filter. This configuration makes the Delaware filter ideal on sites with limited elevation difference between filter inflow and outflow points. Depending on site-specific constraints, and the maximum available hydraulic head, one of two different equations governs sizing of the filter surface area.

If the maximum hydraulic head acting on the filter (*2h* as shown in Figure 12.7) is less than 2'-8", the following equation should be used to compute the minimum filter surface area:

$$A_f = \frac{WQV}{\left(4.1h + d_f\right)}$$

WQV= Water quality volume
A_f = Minimum surface area of sand bed (square feet)
d_f = Sand bed depth (typically 1.5 to 2.0 feet)
h = Average depth of water above surface of sand media (ft)

When the maximum available head is greater than 2'-8", the following equation governs sizing of the filter surface area:

$$A_f = \frac{545 I_a d_f}{\left(h + d_f\right)}$$

I_a = Impervious fraction of contributing drainage shed (acres)

It was previously determined that the total elevation difference between the pipe discharging runoff into the filter and the pipe carrying effluent from the filter is 4.5 feet. Therefore, the *maximum* possible ponding depth, *2h*, on the filter is calculated by subtracting the total filter media depth from this total elevation difference:

$$2h = 4.5\,ft - 3\,ft = 1.5\,ft$$

Therefore, the first equation applies as the available head on the filter is less than 2'-8". In this application, we will select a sand media depth of 2 feet. The average ponding depth on the filter, *h*, is determined to be *0.75 feet* and the filter surface area is computed as:

$$A_f = \frac{3{,}660\,ft^3}{\left(\left(4.1\right)\left(0.75\,ft\right) + 2\,ft\right)} = 721.2\,ft^2$$

Next, the length and width of the filter are computed. This design will employ a rectangular configuration with a 2:1 length-to-width ratio.

$$L_f = 2W_f$$
$$2W_f^{\,2} = 721.2\,ft^2 \Rightarrow W_f = 19.0\,ft$$
$$L_f = 38.0\,ft$$

Rounding the computed dimensions to nominal values yields the following filter surface parameters:

L_f (ft)	W_f (ft)	A_f (ft2)
38	19	722

Table 12.5. Delaware Filter Surface Dimensions

The Delaware sand filter is characterized by two parallel chambers, one serving as a pre-treatment sedimentation chamber and the other holding the sand filter media. The dimensions of the sedimentation chamber (L_s, W_s, and A_s) are identical to those of the filtration chamber shown in Table 12.5.

Special Considerations for Implementation of a Delaware Intermittent Sand Filter

- The filtration bed's underdrain piping should consist of two 4-inch diameter schedule 40 perforated PVC pipes placed on 1 percent slope. Perforations should be 3/8 inch diameter, minimum 4 holes per row, and row spacing a maximum of 6 inches. The underdrain piping should be placed within the gravel filter media with a minimum of 2 inches of cover over the pipes.

- Weepholes are recommended between the filter chamber and the clearwell to permit draining if the underdrain piping should fail or become clogged.

- It is recommended that the sand filter media be wrapped in a filter cloth approved by the Materials Division. Additionally, the sand layer should be underlain by a layer of ½ - 2 inch diameter washed gravel (10 inches thick) and overlain by a layer of 1 – 2 inch diameter washed gravel (1 – 2 inches thick).

Step 2C. Size Filter and Pre-Treatment Sedimentation Chamber – Austin Surface Sand Filter

The Austin sand filter can be designed for full or partial pre-treatment of sediment. Full pre-treatment of inflow is characterized by capturing and detaining the entire WQV and releasing it into the filtration chamber over a period of not less than 24 hours. Partial pre-treatment of sediment entails providing pre-treatment storage for 20 percent of the WQV in a sedimentation chamber hydraulically connected to the filtration chamber (as with the D.C. and Delaware sand filters). Sizing of the sand media is a direct function of the volume of pre-treatment. The following equations govern filter sizing:

Filters equipped with full pre-treatment of inflow: $A_f = \dfrac{100\,ft^2}{\text{Acre Treated}}$

Filters equipped with partial pre-treatment of inflow: $A_f = \dfrac{545 I_a d_f}{\left(h + d_f\right)}$

This design example will employ full pre-treatment of inflow. Therefore, the required filter area is computed as:

$$A_f = \frac{100\,ft^2}{acre} \times 1acre = 100\,ft^2$$

Austin sand filters should be sized with a minimum length-to-width ratio of 2:1. Employing this ratio, the following dimensions are computed for the filter:

$$L_f = 2W_f$$
$$2W_f^{\,2} = 100\,ft^2 \Rightarrow W_f = 7.1\,ft$$
$$L_f = 14.2\,ft$$

Rounding the computed dimensions to nominal values yields the following filter surface parameters:

L_f (ft)	W_f (ft)	A_f (ft2)
14.5	7	101.5

Table 12.6. Austin Filter Surface Dimensions

The next step is to size the pre-treatment sedimentation chamber. The surface area of the sedimentation basin is calculated from the Camp-Hazen equation as shown:

$$A_s = \frac{Q_o}{W} \times \left[-\,ln\left(1 - E\right)\right]$$

With: A_s = sedimentation basin surface area (ft^2)

Q_o = discharge rate from basin (WQV / 24hr)

$= \dfrac{ft^3}{24hr} x \dfrac{1hr}{3600s} = cfs$; where WQV = water quality volume in ft^3

W = particle settling velocity (ft/sec)

E = ediment trapping efficiency of suspended solids (90 percent)

The particle settling velocity is a function of the impervious area contributing to the filtering practice. The following values are used in sizing the pretreatment basin:

Impervious Percentage	Particle Settling Velocity (ft/sec)
≤75	0.0004
>75	0.0033

Table 12.7. Particle Settling Velocities (MDE, 2000)

The filter under design will serve a site with 100 percent impervious cover. Therefore, the filter area is computed as:

$$A_s = \frac{3{,}660\,ft^3}{24 hour} \times \frac{1hr}{3{,}600\sec} \times \frac{1}{0.0033} \times \left[-\ln(1-0.9)\right] = 29.6\,ft^2$$

Pre-treatment must be provided for the entire WQV. Therefore, the depth of the sedimentation chamber is computed as:

$$d_s = \frac{3{,}660\,ft^3}{29.6\,ft^2} = 123.6\,ft$$

The depth of a sedimentation chamber should not exceed 10 feet. When the Camp-Hazen approach yields depths exceeding 10 feet, the following equation should be used to size the filter's pre-treatment chamber:

$$A_s = \frac{WQV}{10\,ft}$$

$$A_s = \frac{3{,}660}{10\,ft} = 366\,ft^2$$

The filter pre-treatment chamber will be located parallel to the filter sedimentation chamber as shown in Figure 12.3. Therefore, the length of the pre-treatment chamber is set equal to the length of the sedimentation chamber, 14.5 feet. The width of the pre-treatment chamber is then computed as follows:

$$W_s = \frac{366\,ft^2}{14.5\,ft} = 25.2\,ft$$

Table 12.8 presents a design summary of the Austin sand filter.

Filter Length (L_f) ft	Filter Width (W_f) ft	Filter Area (A_f) ft^2	Sedimentation Chamber Length (L_s) ft	Sedimentation Chamber Width (W_s) ft
14.5	7	101.5	14.5	25.2

Table 12.8. Design Summary – Austin Sand Filter

The next step is to design an outlet configuration that will discharge the WQV from the pre-treatment chamber to the sedimentation chamber over a period of not less than 24 hours. Typically this conveyance occurs through a perforated stand pipe as shown in Figure 12.3. Control of flow should be dictated by a throttle plate or other flow-restricting mechanism, *not* the perforations in the stand pipe. The following steps illustrate sizing of the orifice.

Discharge of the water quality volume from the pre-treatment chamber to the filter chamber must occur over a period of not less than 24 hours. The Virginia Stormwater Management Handbook identifies two methods for sizing a water quality release orifice. The VDOT preferred method is METHOD 2, “average head/average discharge.”

The water quality volume is attained at a ponded depth of 10 feet in the pre-treatment chamber, therefore the average head associated with this volume is computed as:

$$h_{avg} = \frac{10\,ft}{2} = 5\,ft$$

$$Q_{avg} = \frac{WQV}{(24hr)(3{,}600\sec/hr)} = \frac{3{,}660\,ft^3}{(24hr)(3{,}600\sec/hr)} = 0.04cfs$$

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

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

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

The head is estimated as that acting upon the *invert* of the water quality orifice when the total water quality volume of 1,830 ft^3 is present in the chamber. While the orifice equation should employ the head acting upon the center of the orifice, the orifice diameter is presently unknown. Therefore, the head acting upon the orifice invert is used. The small error incurred from this assumption does not compromise the usefulness of the results.

Rearranging the orifice equation, the orifice area is computed as

$$a = \frac{Q_{avg}}{C\sqrt{2gh}} = \frac{0.04}{0.6\sqrt{(2)(32.2)(5)}} = 0.004\,ft^2$$

The diameter is then computed as:

$$d = \sqrt{\frac{4a}{\pi}} = \sqrt{\frac{(4)(0.004)}{3.14}} = 0.071\,ft = 0.852in$$

An orifice with an outlet diameter of 0.75 inches will be employed to release the water quality volume into the filter chamber over the minimum 24-hour period.

Special Considerations for Implementation of an Austin Intermittent Sand Filter

- The depth of the sand filter media should range between 18 and 24 inches
- When constructed as an underground vault, a minimum of 60 inches of headroom is required in the sedimentation and filter chambers. In the filtration chamber, this headroom should be measured from the top of the filter media.
- The minimum length-to-width ratio of the filter chamber is 2:1.
- The pre-treatment sedimentation chamber should include a sediment sump for accumulation and subsequent removal of filtered sediment.

Step 3. Establish the Crest Elevation of the Water Quality Diversion Weir

The intermittent sand filters presented in this design should have *only* the site water quality volume directed to them. The most popular means of isolating the water quality volume is through the use of a diversion weir in the manhole, channel, or pipe conveying runoff to the BMP. The crest elevation of the weir should be set equal with the water surface elevation corresponding to the maximum available ponding depth on the filter(s), *2h*, as previously defined. This approach ensures that flows beyond the water quality volume bypass the filter and are conveyed downstream by the storm drainage system with minimal mixing of the water quality volume held in the BMP. The weir and downstream receiving structures should typically be sized to accommodate the 10-year return frequency storm.

Chapter 13 – Vegetated Roofs

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

13.1 Overview of Practice

The following example presents design guidance for Vegetated Roof applications serving runoff quality and quantity needs on VDOT facilities buildings. A vegetated roof cover is a veneer of vegetation that is grown on and completely covers an otherwise conventional roof, thus more closely matching surface vegetation than that of the impervious roof. (PADEP, January 2005)

The vegetated roof veneer may range between two and six inches in thickness, and may be comprised of multiple layers including waterproofing membranes, synthetic insulation, engineered and non-engineered soil media. With proper installation and selection of materials, even thin vegetated covers are capable of providing significant rainfall retention, runoff reduction, and water quality improvement.

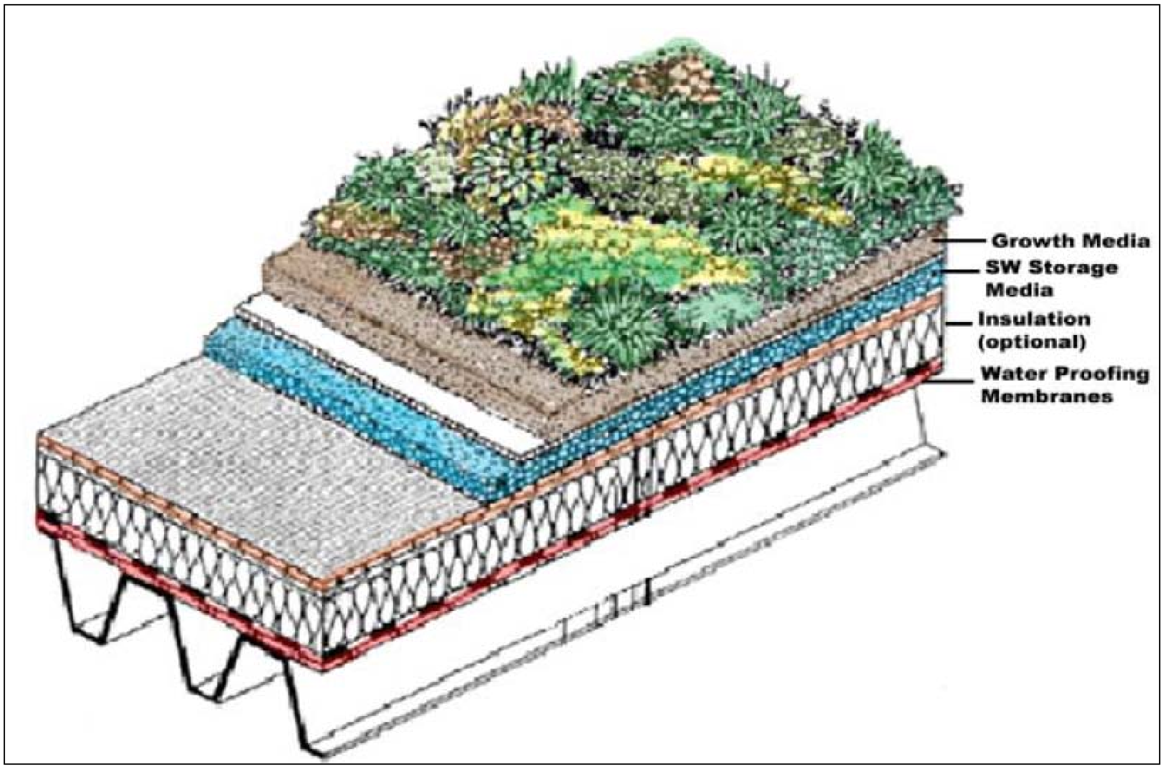


Figure 13.1. Vegetated Roof Schematic (Roofscapes, Inc.)

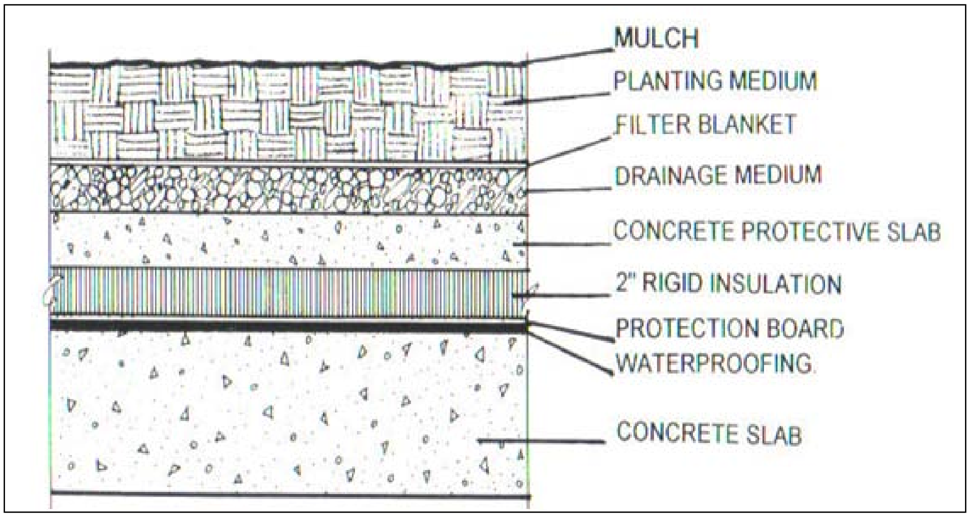


Figure 13.2. Typical Vegetated Roof Section
(Osmundson, 1999)

13.2 General Application Considerations

- Vegetated roofs may be applied as part of new construction or in retrofit applications.
- Vegetated assemblies on roofs with pitches steeper than 2V:12H must be supplemented with additional structural measures to protect against sliding.
- The roof structure of the building for which a vegetated roof practice is planned must be evaluated for compatibility with the anticipated maximum dead and live loads. Typical dead loads for wet vegetated covers range from 8 to 36 pounds per square foot. Live loading values can vary considerably and are a function of rainfall retention. Actual design weights should be established using a standardized laboratory procedure.
- The application of a vegetated roof system, in all application scenarios, requires a premium waterproofing system.
- The chosen vegetation must create a vigorous, drought-tolerant cover. The most successful and commonly used ground covers for un-irrigated roof installations are varieties of Sedum and Delosperma. Vegetated roof designs deeper than four to six inches are able to incorporate a wider array of vegetation, including Dianthus, Phlox, Antennaria, and Carex.
- Roof access must be provided to ensure proper maintenance and replanting of vegetative cover as necessary.

Source: Pennsylvania DEP Stormwater Best Management Practices Manual, December 2006.

13.3 Design Guidelines

- Vegetated roof installations intended to serve as water quality BMPs must not be fertilized. Generally, non-irrigated assemblies are strongly preferred, even though they preclude the use of certain, otherwise acceptable, plant species.

- Internal building drainage, including provisions to cover and protect deck drains or scuppers (small openings to permit the drainage of water from a floor or rooftop), must anticipate the need to manage large rainfall events without inundating the vegetated cover.

- When the selected waterproofing membrane is not root-fast, a supplemental root-barrier must be installed.

- National Roofing Contractors Association (NRCA) and American Society for the Testing of Materials (ASTM) standards should be employed when choosing and testing the roof's waterproofing membrane.

- Roof flashing should extend 6 inches higher than the top of the growth media surface and be protected by counter-flashings.

- Care must be taken during installation of the vegetated cover to ensure that the waterproofing membrane is not damaged.

- The vegetated layer should provide an internal drainage capacity capable of accommodating the two-year return frequency event without generating surface runoff.

- Deck drains and scuppers serving to discharge water from the roof area should be equipped with access chambers. These enclosures should include removable lids to allow ready access for inspection.

- A vegetated roof's engineered soil media should contain no clay particles and should contain no more than 15% organic matter.

- The engineered media employed in vegetated roof applications should have a maximum moisture capacity ranging between 30 and 40 percent.

- If insulation is included in the roof covering system, it may be located above or below the primary waterproofing membrane.

- The International Code Council (ICC) and all other applicable standards should be considered for ballasted roofs.

Source: Pennsylvania DEP Stormwater Best Management Practices Manual. December 2006.

13.4 Types of Vegetated Roofs

Vegetated roof systems that exceed 10 inches in depth are considered *intensive* roof covers. Intensive assemblies are intended primarily to achieve aesthetic and architectural objectives, with only secondary consideration of stormwater management function. These deep intensive systems may be called "roof gardens." *Extensive* roof covers, by contrast, are usually 6 inches or less in depth and have a well-defined stormwater management objective as their primary function. The focus in this example is on the design of an extensive vegetated roof BMP.

Vegetated roof BMPs generally fall into three design categories:

- Single media with synthetic underdrain layer
- Dual media
- Dual media with synthetic retention/detention layer

13.4.1 Single Media Assemblies

Single media assemblies are most often used in pitched roof applications, and for thin and lightweight applications. The plants are selected from very drought-tolerant species, and the engineered media is of very high permeability. The profile of a single media vegetated roof assembly is typically as follows:

- Waterproofing membrane
- Root barrier (optional, depending upon the root resistance properties of the waterproofing membrane)
- Semi-rigid plastic geotextile drain or mat
- Separation geotextile
- Engineered growth media
- Foliage layer

Single media vegetated roof assemblies installed on pitched roofs may require the use of slope bars, rigid slope stabilization panels, cribbing, reinforcing mesh, or other provisions to prevent sliding and instability.

Single media assemblies used on flat roofs typically require a network of perforated internal drainage conduits to effectively convey percolated rainfall to deck drains and scuppers.

Assemblies with rigid geotextile drains or mats can be irrigated from beneath, while assemblies with drainage composites will require direct watering.

13.4.2 Dual Media Assemblies

In contrast to single media assemblies, dual media vegetated roof assemblies utilize two types of non-soil media. Fine-grained media with some organic content is placed over a basal layer of coarse lightweight mineral aggregate. Dual media assemblies do not include a geocomposite drain. The objective of a dual media assembly is to improve the drought resistance of the system by attempting to replicate a natural growth environment in which sandy topsoil overlies gravelly subsoil. These assemblies are typically 4 to 6 inches thick and are comprised of the following layers:

- Waterproofing membrane
- Protection layer
- Coarse-grained drainage media
- Root-permeable non-woven separation geotextile
- Fine-grained engineered growth media layer
- Foliage layer

Dual media assemblies are less versatile than their single media counterparts, and their implementation is restricted to roof pitches of 1.5:12 or less.

Large dual media assemblies should incorporate a network of perforated internal drainage piping to convey percolated rainfall.

Dual media assemblies are optimally suited to base irrigation methods.

13.4.3 Dual Media with Synthetic Retention / Detention Layer

Dual media assemblies employ plastic panels (geocomposite drain sheets) with cup-like receptacles on their upper surfaces. These sheets are then filled with coarse lightweight mineral aggregate. The cups trap and retain precipitation. The profile of a dual media system implementing a synthetic holding layer is as follows:

- Waterproofing membrane
- Felt fabric
- Retention / detention panel
- Coarse-grained drainage media
- Separation geotextile
- Fine-grained growth media layer
- Foliage layer

The complexity of the dual media synthetic assembly typically results in a total BMP depth of five inches or greater. These assemblies should only be considered for roof pitches less than or equal to 1:12.

Dual media assemblies equipped with synthetic retention / detention layers are best irrigated by surface spraying or mid-level drip.

13.5 Drainage Provisions

Adequate drainage is essential to the proper functioning of a vegetated roof. Failure of the roof drainage system can lead to loss of vegetation as well as penetration of water into surrounding structures. (Osmundson, 1999) Adequate drainage is a product of two key elements of the vegetated roof – the drainage medium and the drainage piping.

The drainage medium must consist of rot-proof material through which water can percolate and eventually enter the roof drains. In the United States, as early as the 1930's, pebbles and broken rock were being applied in rooftop gardens as a drainage medium.

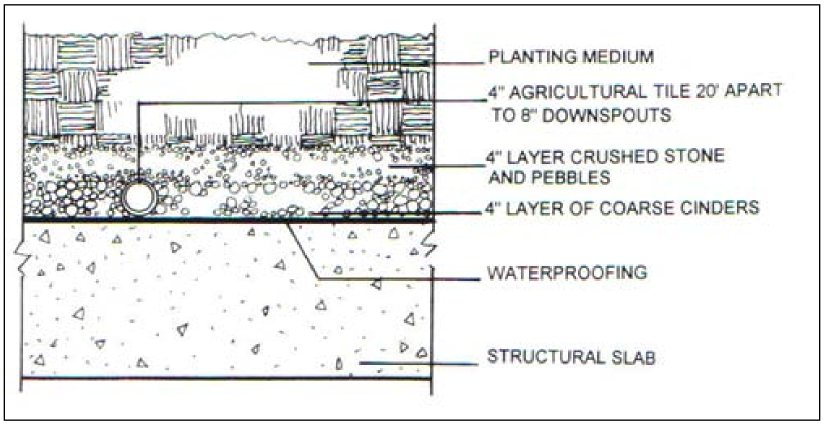


Figure 13.3. Crushed Stone Drainage Medium
(Osmundson, 1999)

The most notable shortcoming of the crushed stone drainage medium shown in Figure 13.3 is its weight. Modern proprietary materials have been developed to provide superior drainage function without the excessive weight of aggregate material with comparable void space. Today, crushed stone drainage mediums are considered obsolete.

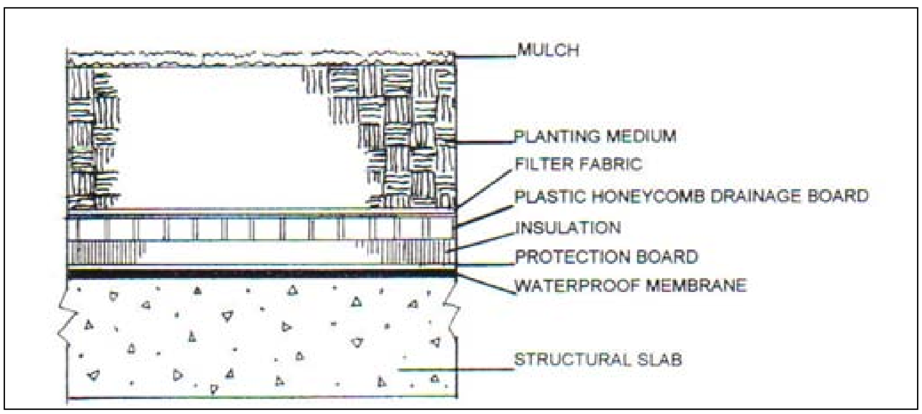


Figure 13.4. Proprietary Structural Drainage Medium
(Osmundson, 1999)

One popular proprietary drainage device is the Grass-Cel system. When topped with a layer of plastic filter fabric (necessary to prevent clogging by the fines contained in overlying planting media), the Grass Cel system provides a strong, easily handled and cut, lightweight drainage layer. Other varieties of proprietary drainage medium are Enkadrain and Geotech.

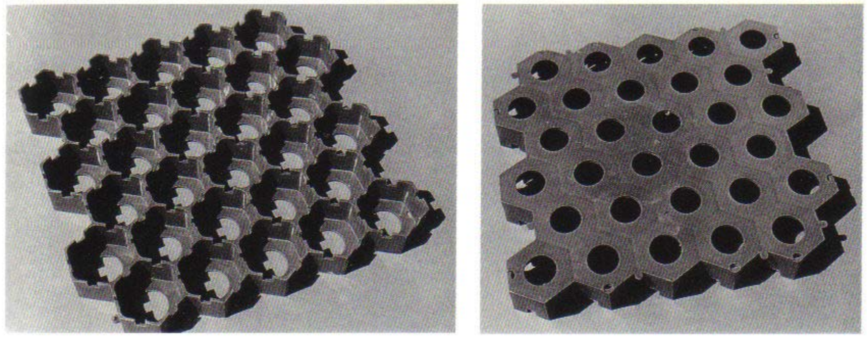


Figure 13.5. Two Types of Grass Cel Drainage Medium
(Osmundson, 1999)

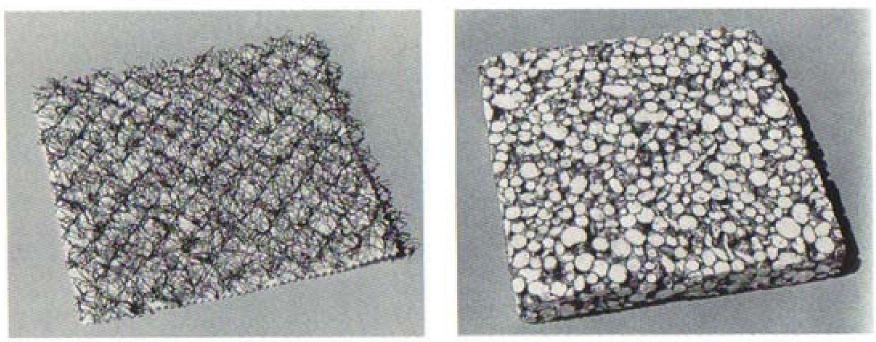


Figure 13.6. Enkadrain (left) and Geotech (right)
(Osmundson, 1999)

Typically, the drainage piping for a vegetated roof assembly will be plastic, cast iron, or brass. A number of different drain types exist.

One type of vegetated roof drain is the *round* or *deck* drain. The round drain is characterized by a grated horizontal top surface and perforated side surfaces. They are useful because their design allows flow to enter at the ground surface level as well as through the sides.

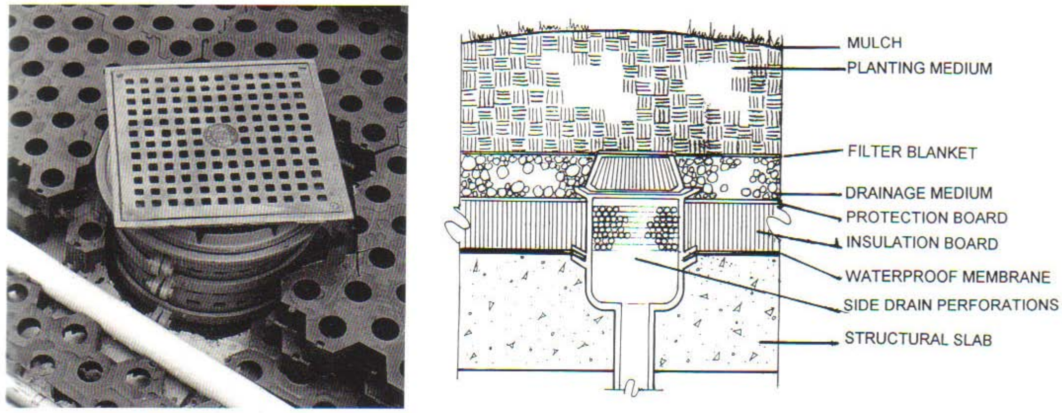


Figure 13.7. Round Drain Situated in Grass Cel Drainage Medium
(Osmundson, 1999)

Another type of vegetated roof drain is the *dome* drain. The dome drain is characterized by its raised dome-shaped surface. It is particularly useful because its elevated surface permits water to enter even when the lower perforations become clogged by leaves and other debris.

A type of drain popular in Europe consists of a combination of sloping concrete trough or gutter in the concrete protective slab covered by a “half-section” of perforated plastic pipe covered in filter fabric. Water entering the system flows through the protective slab, into the gutter, eventually reaching the building downspouts.

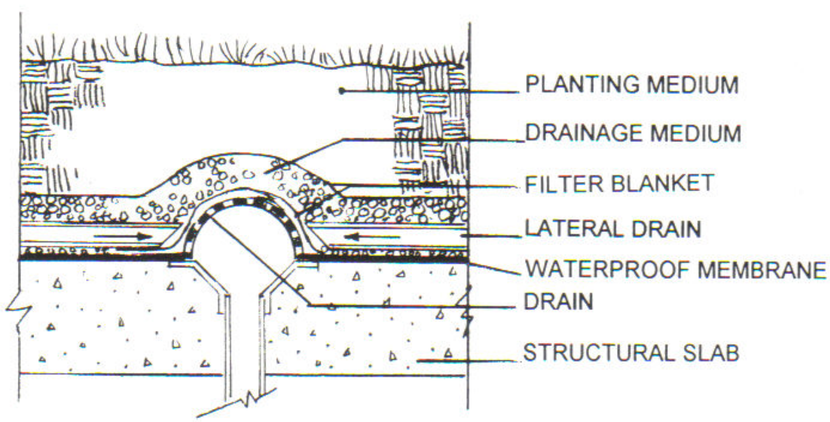


Figure 13.8. Perforated Half Pipe Drain
(Osmundson, 1999)

The filter fabric/blanket chosen to prevent clogging of the drainage medium should meet the following specifications:

- Grab Tensile Strength (ASTM-D4632) 120lbs
- Mullen Burst Strength (ASTM-D3786) 225psi
- Flow Rate (ASTM-D4491) 95 gal/min/ft^2
- UV Resistance after 500 hours (ASTM-D4355)
- Heat-set or heat –calendared fabrics are not permitted.

(Pennsylvania DEP Stormwater Best Management Practices Manual – December 2006)

The following is a non-exhaustive list of filter fabric manufacturers:

- Mirafi
- Supac
- Typar
- AMOCO
- EXXON
- TerraTex

U.S. Department of Transportation. Federal Highway Administration. Evaluation and Management of Highway Runoff Water Quality. Washington, D.C., 1996

Regardless of the type of drain employed, the system should be equipped with debris-collection basins to avoid clogging of the drainage piping by the inherent presence of debris and fine soil matter. (Osmundson, 1999) The pipes to which the drainage system connects are part of the building drainage system. Therefore, design of the vegetated roof drainage system will require an iterative design approach, working closely with the architect and structural engineer.

13.6 Growth / Planting Media

It is nearly impossible to classify a given soil mixture as optimal for all vegetated roof applications. Detailed performance data for a particular growth media requires long-term, controlled monitoring. In general, however, the growing media should adhere to certain guidelines, described as follows (Source: Osmundson, 1999):

- The optimum planting media consists of 45% sand, 45% soil and 10% humus.

- The presence of silt should be kept to a minimum. Silt possesses the ability to clog the system's filter fabric.

- Mulching should be avoided, as wash-off is likely during severe rainfall producing events.

- The growth media must provide a permanent means of supplying internal aeration to prevent compaction of the mix.

- The selected media must drain completely and efficiently over a 24 hour period.

- The media must be suitable for the plant species chosen. It must be able to supply or absorb water and nutrients for the vegetation to use over time.

- The media should exhibit very little shrink / swell phenomena, retaining its original volume over time.

13.7 Stormwater Peak Rate and Volume Mitigation

While conventional hydrologic methods are used to estimate the runoff from a vegetated roof system, one must consider that the runoff released from the system is not surface runoff, but rather percolated water. The rate and quantity of water released from a vegetated roof assembly during a particular return frequency storm is dependant upon the following physical properties of the assembly.

- Maximum media water retention
- Field capacity
- Plant cover type
- Saturated hydraulic conductivity
- Non-capillary porosity

The assembly's maximum water retention is a product of the quantity of water that the media can hold against gravity in a drained condition.

In the absence of continuous simulation modeling or detailed laboratory performance data, a reasonable approach to assessing peak mitigation performance of a vegetated roof assembly is to compare its performance to that of a conventional impervious roof.

A general rule of thumb when computing runoff from vegetated roof systems is that *for storm events in which the total rainfall depth is no more than three times the maximum media water retention for the assembly, the rate of runoff from the roof will be less than or equal to that of open space*. (PADEP, 2005)

The maximum moisture content of a vegetated roof drainage media is 40 percent. In the following tables, the required depth of a vegetated roof drainage media layer located in Henrico County is shown by return frequency storm. Vegetated roof assemblies whose drainage media depth and maximum moisture content achieve the target values shown will exhibit runoff patterns similar to undeveloped, open cover conditions.

Return Frequency (yrs)	24-Hr. Rainfall (in)
2	2.8
10	4.5
25	6.0
100	7.8

Table 13.1. Twenty four Hour Rainfall Depths, Henrico County
Virginia Stormwater Management Handbook, (DCR, 1999)

For runoff patterns to behave similarly to those of undeveloped open space, the available water retention within the drainage media of a vegetated roof assembly must be greater than or equal to *one third* of the rainfall depth for the return frequency storm for which peak mitigation is desired. These equivalent depths are presented as follows.

Return Frequency (yrs)	Required Media Moisture Retention (in)
2	0.9
10	1.5
25	2.0
100	2.6

Table 13.2. Required Media Moisture Retention Depth for Roof Assembly to Behave as Open Space (Henrico County)

The physical depth of a vegetated roof assembly drainage media needed to achieve the moisture retention depths presented in Table 13.2 is a function of the maximum moisture content available within the media. Below are the required media depths for drainage medium exhibiting moisture contents of 30 and 40 percent respectively.

30 Percent Maximum Moisture Retention	
Return Frequency (yrs)	**Required Drainage Media Depth (in)**
2	3.0
10	5.0
25	6.7
100	8.7

Table 13.3a. Required Drainage Media Depth for Roof Assembly to Behave as Open Space (30% moisture content)

40 Percent Maximum Moisture Retention	
Return Frequency (yrs)	**Required Drainage Media Depth (in)**
2	2.3
10	3.8
25	5.0
100	6.5

Table 13.3b. Required Drainage Media Depth for Roof Assembly to Behave as Open Space (40% moisture content)

13.8 Pollutant Removal Performance

While various claims for pollutant removal performance of rooftop gardens have been made, it is not clear at this point that there is a sufficient database to support them. What is clear is that the opportunity of this BMP to intercept overland flow with its associated load of suspended sediment, phosphorous and nitrogen is non-existent. The only true source of pollutants on the rooftop garden will be atmospheric deposition, assuming there is no fertilizer application, as recommended in virtually all guidance documents. We can only surmise there has been little to no investigation of the removal process in the case of atmospheric deposition.

13.9 Vendor Websites

The book by Theodore Osmundson (1999) provides an excellent reference on the landscaping details of rooftop gardens, with many photographs of outstanding installations. However, this reference provides little guidance on the engineering aspects of rooftop drainage and structural design so critical to the success of the rooftop garden. Therefore, we believe it is imperative that the drainage engineer contact various vendors regarding engineered roof top systems, together with the architect and structural engineer for the site development well before the design of any roof top garden system. We have provided a partial list of vendors and their website addresses to assist in this process, recognizing that this list is not exhaustive and that there are other proprietary systems. Our list of vendors does not in any way constitute an endorsement of any one product.

American Hydrotech, Inc
www.hydrotechusa.com

Building Logics
www.buildinglogics.com

Elevated Landscape Technologies Inc. (ELT)
www.eltgreenroofs.com

Green Grid
www.greengridroofs.com

Henry Company
www.henry-bes.com/greenroofing.asp .

Prairie Technologies
www.prairie-tech.com

Roofscapes, Inc.
www.roofscapes.com

Xero Flor America, LLC
www.xeroflora.com

Chapter 14 – Rainwater Capturing Systems

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

14.1 Overview of Practice

Capture and Reuse BMP measures include a number of devices intended to intercept precipitation, store it for a period of time, and provide a means for reuse of the water. These capture devices include cisterns, rain barrels, and vertical storage or “fat downspouts.” The capture and reuse approach to stormwater management can be applied in both site development and retrofit applications. Use as a BMP for highway runoff is limited. Generally, use of stored rainwater in potable applications is not advised in the absence of treatment; however, in addition to reducing stormwater runoff, the intercepted water is ideal for fire protection and irrigation.

14.1.1 Types of Capture and Storage Devices

Cisterns are containers designed to hold large volumes of water (by definition, cistern volumes are typically 500 gallons or more). Cisterns may be located underground or on the surface. Cisterns are available in a variety of sizes and materials, including fiberglass, concrete, plastic, and brick.

Figure 14.1. Various Size Cisterns (PADEP, January 2005)

Rain Barrels are containers designed exclusively to capture runoff from roof leaders and downspouts. Rain barrels vary in volume, and are sized based on the roof area from which they are receiving runoff or as a minimum volume computed by a water budget approach, as discussed later in this document.

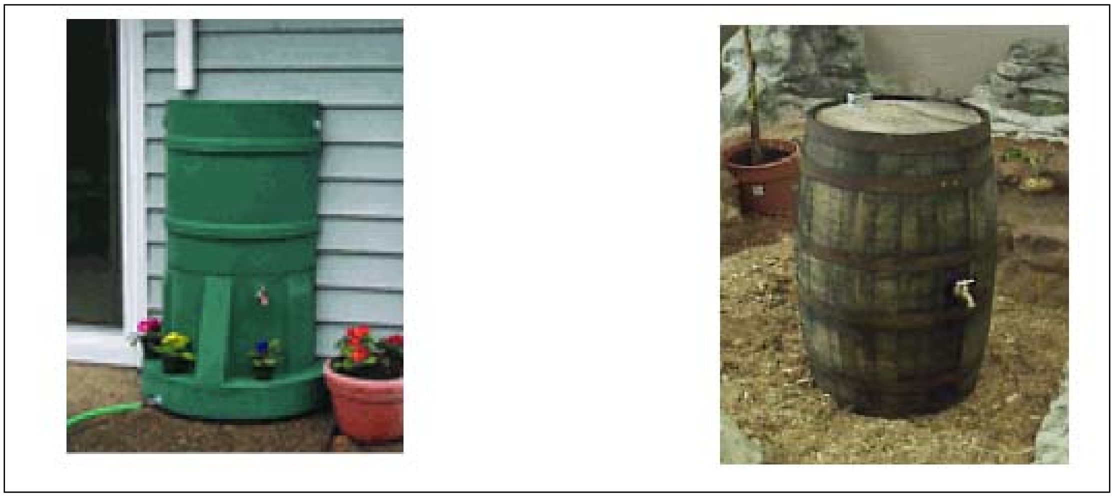


Figure 14.2. Rain Barrels (PADEP, January 2005)

Vertical Storage units or "fat downspouts" function in the same manner as cisterns and rain barrels, but are typically much larger and usually rest against the building from which they are intercepting runoff. Often, the water stored in these vertical storage units is used to provide fire protection. When employed as storage for fire protection, the storage volume is dictated by applicable codes. The design and sizing of vertical storage units and fat downspouts must be accomplished by working closely with both the architect and structural engineer.

Vertical Storage Unit

Figure 14.3. Vertical Storage (Fat Downspouts)
(PADEP, January 2005)

Proprietary storage units, such as RainStore, may be located beneath paths and walkways. These storage devices often provide a supplemental irrigation supply.

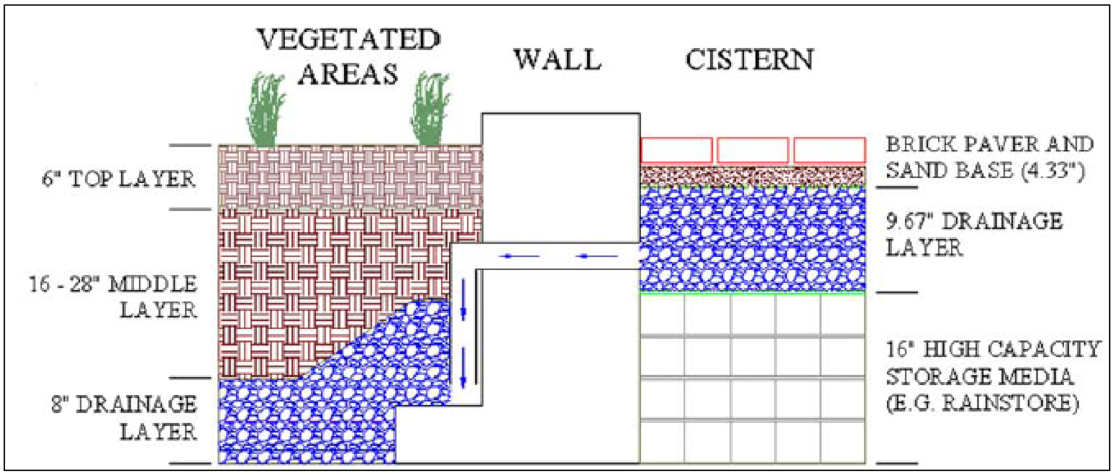


Figure 14.4. Storage of Runoff Beneath Brick Walkway
(specifications from PADEP, January 2005)

14.1.2 Application of Stored Rainwater

While the use of stored rainwater as a potable supply is not recommended, a number of non-potable needs may be addressed by a capture and reuse approach. These include:

- Irrigation of landscaped areas and gardens
- Storage for fire protection needs
- “Greywater” needs such as flushing toilets
- Athletic field irrigation

In addition to satisfying non-potable water needs, rainwater capture devices can serve to reduce runoff volume and the frequency of surcharge events in urban combined sewer systems.

14.2 Design Considerations

- The first step in the consideration of a capture and reuse system is to determine the water demand for the proposed reuse application. The demand is critical in determining the feasibility and size of the harvesting system. The volume of water harvested and stored, at a minimum, must equal the computed demand.

- The capture and storage system must provide drawdown between storm events such that the required stormwater storage volume is available.

- The conveyance system that delivers reused stormwater or greywater from the storage system must not cross connect with domestic or commercial potable water systems.

- Storage units and conveyance systems must be clearly marked as non-potable water.
- Screens may be used as a means to filter debris from capture and storage units.
- Rainfall storage units should be protected from direct sunlight by positioning and landscaping.
- When providing an overflow outlet for the storage unit, the proximity to building foundations must be considered.
- In cold climates, capture and reuse systems should be disconnected during the winter months to prevent freezing.
- Underground cisterns must be watertight.
- Rain barrels and surface cisterns should have a cover with a tight fit capable of keeping out unwanted surface water, animals, dust, and light.
- Cisterns, rain barrels, and vertical storage systems should be equipped with a means for overflow in the event of heavy runoff producing events.
- Buried cisterns should possess observation risers extending to at least 6 inches above grade.
- Re-use applications may require that the stored rainwater be pressurized. Stored water will exhibit a pressure of 0.43 psi per foot of elevation. Irrigation systems will usually require a minimum of 15 psi.

Source: PADEP, January 2005

14.3 Stormwater Performance

The employment of capture and reuse systems exhibits a positive impact on the volume, peak rate, and quality of stormwater runoff from a site.

The volume reduction is simply the volume of runoff from a single storm event that is captured and stored by the harvesting system. If the cistern or barrel is empty at the start of the precipitation event, the maximum potential volume reduction is the actual volume of the capture device.

Because capture and reuse devices take a volume of water out of the total site runoff, the reduced volume may result in a reduced rate of runoff from the site.

The removal of pollutants from stormwater entering a capture device takes place through filtration of the recycled primary storage, and natural filtration through soil and vegetation of any overflow discharge. A number of factors influence the pollutant removal performance of a rainwater harvesting system. These include the volume below the outlet of the system allocated to sediment accumulation, the hydraulic residence time, and the frequency of maintenance.

14.4 Design Approach

The first design element to consider in the installation of a capture and reuse system is that of a first flush diverter. Rooftops can collect dust, leaves, twigs, insect bodies, animal feces, pesticides, and other airborne residue. A first flush diverter routes the first flush of stormwater from the catchment surface away from the storage tank. A number of factors influence the recommended volume of water that should be diverted. These include the frequency of dry days, amount of accumulated debris, and the catchment area. One rule of thumb for first flush diversion is to divert a minimum of 10 gallons for every 1,000 square feet of collection surface. (Texas Water Development Board, 2005)

The most basic first flush diverter is a 6 or 8 inch PVC standpipe. The diverter fills with the first-flush volume, backs up, and then allows water to enter the conveyance and storage system. A pinhole drilled at the bottom of the pipe or a hose bib fixture left slightly open permits the gradual leakage of the first-flush volume (TWDB, 2005). The following lengths of PVC piping are required for first flush storage.

Diameter (in)	Length (inches) per Gallon of Storage
3	33
4	18
6	8
8	5

Table 14.1. Length of Piping Per Gallon of Storage (TWDB, 2005)

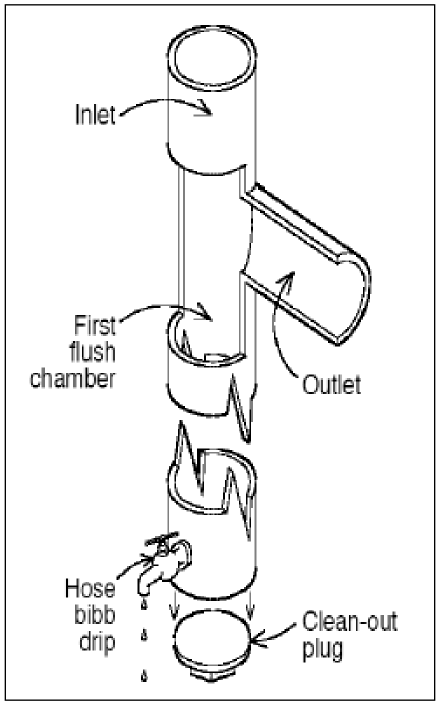


Figure 14.5. Simple Standpipe First Flush Diverter
(TWDB, 2005)

Another variation of first flow capture devices is the standpipe equipped with a ball valve. In this configuration, as the chamber fills, the ball floats up and seals on the seat, trapping the first flush water and routing additional inflow into the storage tank.

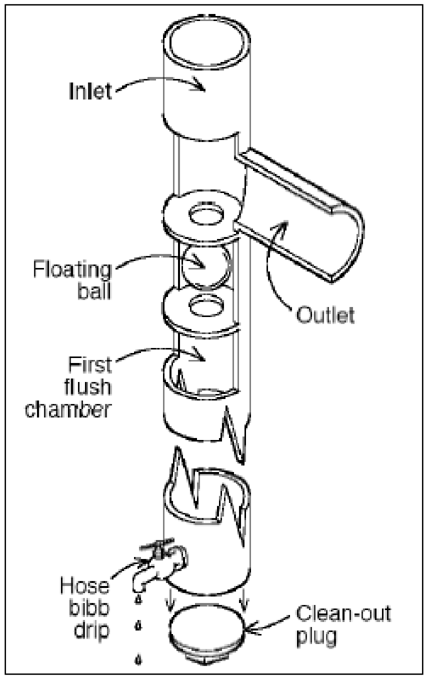


Figure 14.6. Standpipe With Ball Valve
(TWDB, 2005)

The next step in the design process is to size the capture system. Typically, the system must be designed such that the volume of water captured and stored equals or exceeds the volume of water for which anticipated use is planned (demand). The first consideration is that of how much water can be collected. Theoretically, about 0.62 gallons of water per square foot per inch of rainfall can be collected; however, in practice, some precipitation is lost to the first-flush bypass, evaporation, splash-out, and leakage. Rough catchment surfaces are less efficient at conveying water, as water trapped in pore spaces tends to be lost to evaporation. Additionally, intense rainfall events often result in the inability of the system to capture the entire volume of water landing on the catchment surface. Obviously, once storage cisterns or barrels are full, rainwater is lost as overflow. For design purposes, collection efficiencies of 75 to 90 percent should be considered. The catchment area is the "footprint" of the roof. Regardless of the roof pitch, the total area covered by the collection surface should be considered in estimating the supply of captured water. Only catchment areas whose runoff is collected by a conveyance system (roof gutter) should be considered. (TWDB, 2005)

One popular method for sizing a rainwater harvesting and storage system is to employ the monthly water balance method. This method begins by assuming a volume of rainwater already in storage, adding the volume of water captured each month, and subtracting the demand. Two different methods of estimating monthly rainfall are commonly used; the *average rainfall method*, and the *median rainfall method*. The Virginia State Climatology Office maintains an online database with monthly climate information from various stations across the state. This information can be obtained at:

http://climate.virginia.edu/online_data.htm#monthly

Average rainfall is computed by summing historical rainfall and dividing it by the period of record. Median rainfall is the amount of rainfall that occurs in the midpoint of all historic rainfall totals for any given month. When the data is available, employing the median rainfall provides for the most conservative approach to sizing rainfall harvesting systems. The following example shows a typical water budget approach to determining the feasibility and sizing of a rainfall harvesting system.

Given Data: Average monthly rainfall for Louisa County
2,500 square-foot catchment area
85% assumed catchment efficiency
Demand as shown in Table 13.2 on the following page

The supply of monthly rainfall is computed as the product of average rainfall, catchment area, catchment efficiency, and the 0.62 gallons per square foot per inch of rainfall constant. The calculation of monthly supply is shown below for January with an average precipitation of 3.14 inches:

Monthly Supply = (Catchment Area)(Average Rainfall)(Rainfall Constant)(Catchment Efficiency)

$$2{,}500\,ft^2 \times 3.14 in \times 0.62 \frac{gal/ft^2}{in} \times 0.85 = 4{,}137\,gal$$

This value is added to the initial storage volume at the beginning of the month (1,000 gallons for this example), and then the monthly demand is subtracted. The result becomes the initial volume for the month of February, and the calculation is repeated. The monthly budget calculation is presented in the following table with column (A) water demand is in gallons; (B) average rainfall is in inches; (C) rainfall collected is in gallons; and (D) end-of-month storage is in gallons.

Month	A Water Demand (gal)	B Average Rainfall (in)	C Rainfall Collected (gal)	D End of Month Storage (1,000 gal to start)
January	4,500	3.14	4,137	637
February	4,500	3.04	4,005	142
March	4,500	3.80	5,007	649
April	4,500	3.06	4,032	180
May	4,500	3.68	4,848	529
June	4,500	3.69	4,862	890
July	4,500	4.36	5,744	2,134
August	4,500	4.26	5,613	3,247
September	4,500	3.65	4,809	3,556
October	4,500	3.57	4,703	3,759
November	4,500	3.58	4,717	3,976
December	4,500	3.32	4,374	3,850

Table 14.2. Monthly Water Budget

Employing the average monthly rainfall and the monthly water budget approach, we see from Table 13.2 that the storage unit(s) in this scenario would be sized to hold a *maximum* of 3,976 gallons (observed at the end of November) in order to retain all excess rainwater and meet the demand for each month. Alternatively, the *minimum* size storage would only have to be 1,126 gallons [3,976 - (3,850 – 1000)] if the goal is to meet all monthly demands *and* have 1,000 gallons in storage at the end of December each year. In this scenario we must be willing to spill some water during heavy rainfall months.

Chapter 15 – Catch Basin Inserts

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

15.1 Overview of Practice

The following design example provides guidance for the implementation of manufactured water quality inlets and catch basin inserts for purposes of runoff quality management on VDOT facilities projects.

Catch basins are chambers or sumps which provide the entrance point for surface runoff into a stormwater conveyance system. Catch basin inserts are employed to intercept coarse sediments, oils, grease, litter, and debris from the runoff prior to its entrance into the storm sewer. Catch basin inserts are well suited to parking lots, maintenance yards, and other locations where runoff travels directly from an impervious surface into the stormwater conveyance system. (VTRC, 2004)

Water quality inlets encompass a broad spectrum of BMPs designed to remove non point source pollutants from runoff. These structural BMPs vary in size and treatment capacity, but typically employ some form of settling and filtration to remove particulate pollutants. Water quality inlets may exist as hydrodynamic separator systems (see Design Example 15), multi-chambered treatment trains, and a wide array of proprietary products discussed later in this design example.

Many types of catch basin inserts/water quality inlets exist; however, these different configurations generally exhibit similar strengths and shortcomings. The following presents the most common variations of water quality inlet filtering systems.

15.1.1 Tray Type

Tray type filters function by passing stormwater through a filter media situated in a tray located around the perimeter of the inlet. Runoff enters the tray and exits via weir flow under design conditions. Runoff from large storms simply passes over the tray into the inlet unobstructed.

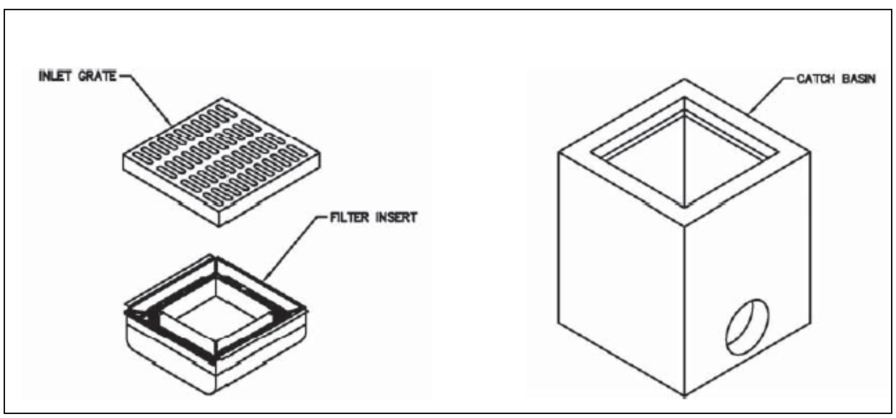


Figure 15.1. Water Quality Inset Tray (PADEP, 2005)

15.1.2 Bag Type

Bag type inserts are made of fabric and placed in the drain inlet around the perimeter of the grate. Runoff entering the drain must pass through the bag prior to exiting through the drain pipe outlet. The system is usually equipped with overflow holes to prevent backwater conditions during heavy runoff producing events.

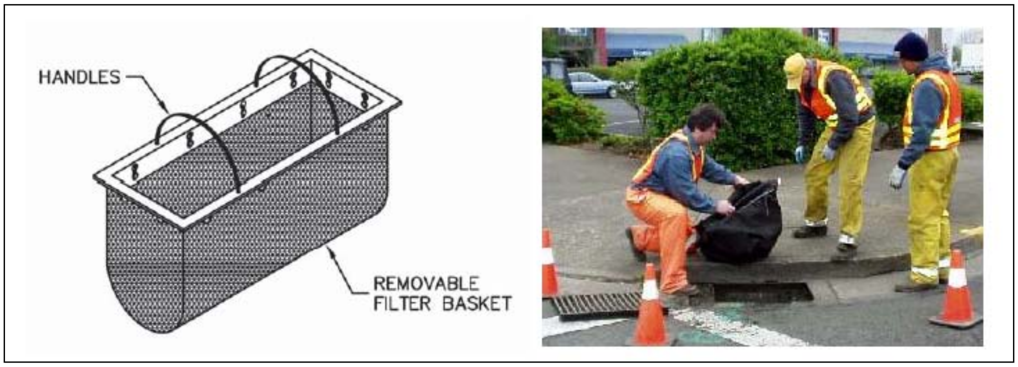


Figure 15.2. Bag Type Inlet Filter and Installation (PADEP, 2005)

15.1.3 Basket Type

Basket type inserts set into the inlet and can be removed for periodic maintenance. Small orifices permit small storm events to weep through, while larger storms overflow the basket. Basket type inserts are useful for filtering trash, debris, and large sediment, but require consistent maintenance.

Figure 15.3. Basket Type Inlet Filter (PADEP, 2005)

15.1.4 Sumps in Inlets

Inlets can be designed such that space is created below the invert of the outlet pipe(s) for sediment and debris to deposit. Generally, this space will be 6 to 12 inches deep. Small weep holes should be drilled into the bottom of the inlet to prevent standing water for long periods of time. Note that if weep holes are used to drain a sumped inlet, the inlet must conform to applicable design requirements for infiltration facilities. Inlets equipped with a sump require regular maintenance and sediment removal.

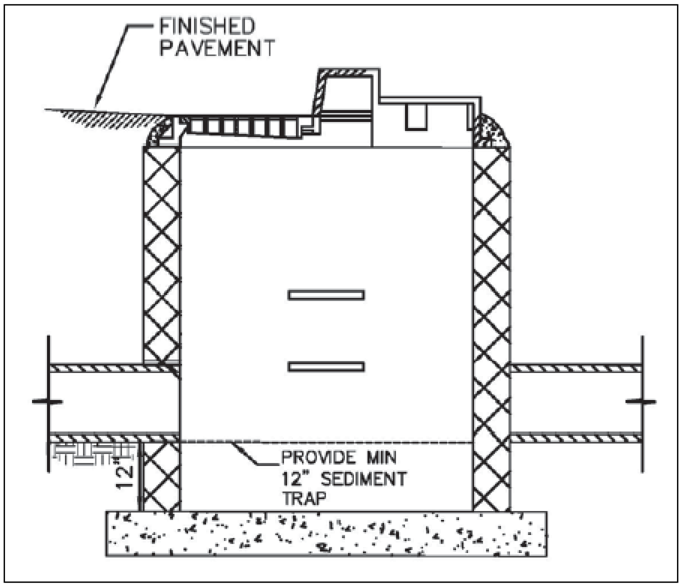


Figure 15.4. Catch Basin Equipped With Sediment Sump (PADEP, 2005)

15.2 Design Considerations

The design process for a specific installation of a water quality inlet or catch basin insert usually begins with a review of various vendor publications and use of preliminary sizing guidelines provided by the vendor. The specific design criteria for the proprietary system being considered should be obtained from the manufacturer or vendor to ensure that the latest design and sizing criteria are used. At the very least, the design for a particular site should be reviewed by the manufacturer to ensure that the system is adequately sized and located.

15.2.1 Key Considerations Unique to Manufactured Products

- Independent performance data must be available to prove a demonstrated capability of meeting stormwater management goals.

- The chosen system or device must be appropriate for use in the geographic region for which implementation is planned.

- Installation and operations/maintenance requirements must be understood by all parties approving and using the system or device in question.

15.2.2 General Design Guidance

- Specific site conditions must be matched with the manufacturer/vendor guidelines and specifications. Geographic location and land use will determine the specific pollutants and their associated loading rates.
- The re-suspension of particles and sediment is of concern. To avoid such re-suspension, the drainage area to each water quality inlet or catch basin should

be restricted to no more than one acre of impervious cover. Regular maintenance and removal of accumulated debris is essential.

- Retrofits should be designed specifically for the existing inlet.

- Location of the water quality inlet or catch basin should provide ease of maintenance, and be at the forefront of the design process.

- If the inlet is used during construction operations for erosion and sedimentation control, the insert should be reconfigured and cleaned per manufacturer guidelines prior to its implementation in the final site design.

- Overflow should be provided such that storms in excess of the device capacity (typically the computed water quality volume) are bypassed.

Source: PADEP, 2005

15.3 Maintenance

The manufacturer's guidelines for maintenance should be followed for any proprietary system. The expected pollutant type and loading rate for the specific site of interest must also be considered. During construction operations, water quality inlets should be inspected a minimum of once per week, and cleaned as needed. Post-construction, they should be emptied when full of sediment and trash / debris. Thorough cleaning should occur at least twice per year. Water quality inlets and catch basins equipped with filtering devices should also be inspected after all heavy runoff producing events. Regular maintenance is critical to ensuring the continued functioning of water quality inlet systems. Studies have shown that water quality inlets storing in excess of 60 percent of their total sediment capacity may resuspend the stored sediments into the runoff entering the inlet. (PADEP, 2005)

15.4 Manufactured Products

The following discussion of manufactured water quality filters is intended only to serve as a description of the most widely used proprietary systems. The products discussed in this design example are not intended to constitute an exhaustive list of all catch basin / inlet filtering systems available. Presentation of the following products does not preclude the use of other available systems, nor does it constitute an endorsement of any one system.

The Virginia Transportation Research Council, via contract with University of Virginia, has constructed the following information matrices for the most widely used catch basin inserts and water quality inlets, as of 2004. The user is referred to the following for the originally published matrices:

Virginia Transportation Research Council. VDOT Manual of Practice for Stormwater Management. Charlottesville, Virginia, 2004.

System Type	Manufacturer	Operation	Sizing and/or Area Treated	Maintenance	Cost	General Performance	Comments
Catch Basin Inserts							
Sorbant™	Sorbant Environmental Corp. Aventura, FL.	Flow cascades over 3 tiers of sorbent pads. Primarily for hydrocarbon removal	Structure drops into standard inlets.	ND	ND	Sorbs 16 to 22 times its weight in hydrocarbons. Does not leach in flooded conditions.* (Corcoran and Rich, 1995)	Catch basin or curb inlet design.
BMP Filter "CB" Series Catch Basin Insert	StormWater Compliance International Oroville, CA (www. stormwatercompintl.com/)	Insert directs flow through mesh screens for sediment removal, then through proprietary media filters.	Applied to catch basins or curb inlets. Overflow allows up to 0.63 cfs through the system.	Hydrocarbon media changes color when saturated. Replacement of other media filters every 6 months. More frequent cleaning of debris.	$900	Oil and grease removal to less than 5 mg/L. Neutral pH: 6-8, BOD & COD reduced to less than 50 mg/L; TSS removal over 90%.*	Company also manufactures oil/water separators, curb inlet filters, inline filters.
Hydro-Kleen™ Filtration System	Hydro Compliance Management, Inc. Brighton, MI (www. hydrocompliance.com/)	Multi-chambered system. Flow through sedimentation chamber to 2 media filters: proprietary material for hydrocarbon removal then activated carbon for final polishing.	Treats first-flush, with bypass available.	Filter change every 4-6 months. More frequent sediment cleanout by vacuum truck.	$1,200 - $2,500 per unit. Filter change: $400 including labor. Low installation cost.	Reduces hydrocarbons, pesticides, herbicides, VOCs to below detection limits.*	Can customize media for site-specific loads. Can be catch basin or burn inlet system. Vendor claims product satisfies structural BMP requirements for NPDES compliance.
Aqua-Guard™	AquaShield, Inc.	Flow through sedimentation chamber and filter media.	ND	Sediment removal by shop-vac or vacuum truck. Filter media changes color to black when replacement is needed.	ND	Effective removal of TSS, soluble and insoluble O&G, phosphorus, nitrogen, VOCs, sulfides, heavy metals. Certified by CA EPA 90-95% removal of dissolved petroleum and oils.*	Standard sizing for drop-in application.
StreamGuard™	Bowhead Manufacturing Co. Address: P.O. Box 80327 Seattle, WA 98108	The insert's universal skirt adapter is installed under a storm drain grate and provides water quality treatment through filtration.	Size based on flow rates from 20 to 40 gpm.	Remove trash and debris when accumulation becomes significant.	$56 to $93 each, depending on size.	Independent testing by King County Surface Water Management Division of	Installed at the U.S. Coast Guard Station in Chesapeake, VA.

Table 15.1. Catch Basin Inserts Information Matrix (VTRC, 2004)

System Type	Manufacturer	Operation	Sizing and/or Area Treated	Maintenance	Cost	General Performance	Comments
		gravity settling and absorption.				Washington State demonstrated oil removal efficiencies of 88% when tested in a park-and-ride lot catch basin. Catch basin inserts installed at SeaTac International Airport's passenger pick-up area show average removal efficiencies for Total Suspended Solids of 80%, and for oil & grease of 94%.	
The SNOUT™	Best Management Products, Inc.	Simple hood covers outlet structure. Bottom of hood sites below static water level. Keeps floatables (including trash) above outlet.	ND	SNOUT itself does not require maintenance. Remove trash and debris when accumulation becomes significant.	Low hundreds	Inspections show significant accumulation of gross pollutants.*	Suitable for use with catch basins or water quality inlets. Can be equipped with flow restriction and/or odor control filter.
Filter bag inserts – general	*Multiple Vendors:* DrainPac™ by Drain Works; Drainguards by Ultra Tech; Ultra-Urban Filters by AbTech Industries.	Heavy filter fabric held in place by inlet grate.	Standard sizes for drop-in installation	Frequent inspection and cleanout	ND	Mainly designed to capture trash and sediment. Some also claim sorption of O&G. Can be effective if frequently maintained.	Improper installation causes leaks/bypass of runoff around filter media.

Table 15.1 Cont'd. – Catch Basin Inserts Information Matrix (VTRC, 2004)

System Type	Manufacturer	Operation	Sizing and/or Area Treated	Maintenance	Cost	General Performance	Comments
Water Quality Inlets							
Oil/Water Separator (OWS)	*Multiple Vendors:* Areo-Power®; Flo-Trends, Inc.; PSI International, Inc.	Coalescing plate or tube separator. Flow-through system.	Usually designed for specific applications.	ND	ND	Low to negative removal of TSS, TPH, and O&G. (Othmer et al., 2001)	General inability to reduce low levels of hydrocarbons. Not generally recommended.
MCTT (Multi-Chambered Treatment Train)	Developed at the University of Alabama-Birmingham. Specifications are given for cast-in-place construction	Flow through 3 chambers: screening, tube settling, media filtration. Provides some detention. Customize	Surface area of unit typically 0.5 – 1.5% of the drainage area	Six-month inspections. Replace sorbent pillows & clean catch basin	$10,000 - $20,000 per 0.25 acre. (Schueler, 1994)	Treats 95% of annual rainfall. Toxicity reduced by filtration. Flow restrictions can	May be able to customize system depending on site pollutant
		with aerators, sorbent pads, multi-media filters	Criteria can be expanded to include storm characteristics and anticipated loads.	every 6 – 12 months. Media replacement after 3 – 5 years. Ensure mosquito control.		provide up to 24 hrs settling (US EPA, 1999c)	characteristics.
BaffleBox	Multiple Vendors: Suntree Technologies, Inc., or Cast-in-place construction	Large sediment trap comprised of multiple concrete or fiberglass chambers separated by weirs. Usually with trash screens and skimmers.	Usually 10 – 15 ft. long by 6 – 8 ft. wide. (2 ft. wider than inlet pipe)	Monthly during wet season, 2 – 3 months during dry season.	Installation: $20,000 - $30,000 Maintenance: $0.24/kg removed (avg. $450 per event)	Approx. 2,500 – 3,800 kg/yr sediment removal but highly site-specific. Model performance: removed at least 90% sand or sandy clay, but reduced to only 28% for fly ash. Differences in accumulated material noted between chambers.	Better performance with larger boxes. Systems become septic and odorous without base flow. Many systems installed in Florida. Wash-out can be a problem with larger events.
Oil/Grit Separators (OGS)	Usually cast-in-place construction.	On-line system. Flow through three chambers: sediment & trash, oil containment, energy dissipation. Inverted elbow in oil chamber retains floatables.	Treat 0.1" runoff. Recommended as a last resort for treatment area less than 1 acre.	Quarterly	$5,000 - $16,000; average $8,500 (US EPA, 1999d)	Of 109 systems investigated, the average residence time was less than 30 minutes. Poor retainment of trash and debris. 10 – 40% solids removal with 1	Used mainly at gas stations, fast food restaurants and other small, but highly-developed sites. Hundreds installed in the DC metro area. Better performance

Table 15.2. Water Quality Inlets Information Matrix (VTRC, 2004)

Figures 14.5 through 14.9 are representative of many vendor products which can be viewed at the following EPA Region 1 New England website:

http://www.epa.gov/NE/assistance/ceitts/stormwater/techs.html

Additional vendor products and preliminary design information can be found at the US EPA NPDES/STORMWATER/BMPMENU website:

http://cfpub.epa.gov/npdes/stormwater/menuofbmps/post_7.cfm

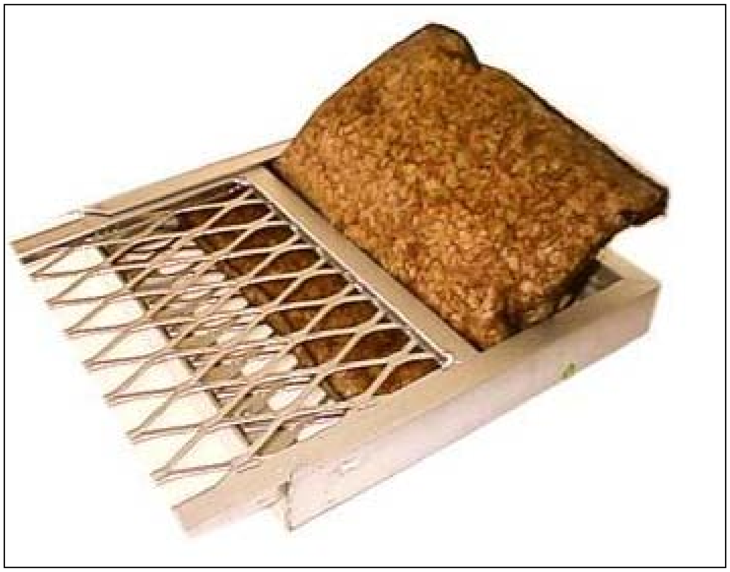


Figure 15.5. Sorbant Filter Pillow System

Source**: Sorbant Environmental Corp**
P.O. Box 80-2505 • Aventura, FL 33280
305-655-9911 - Fax: 305-655-0470

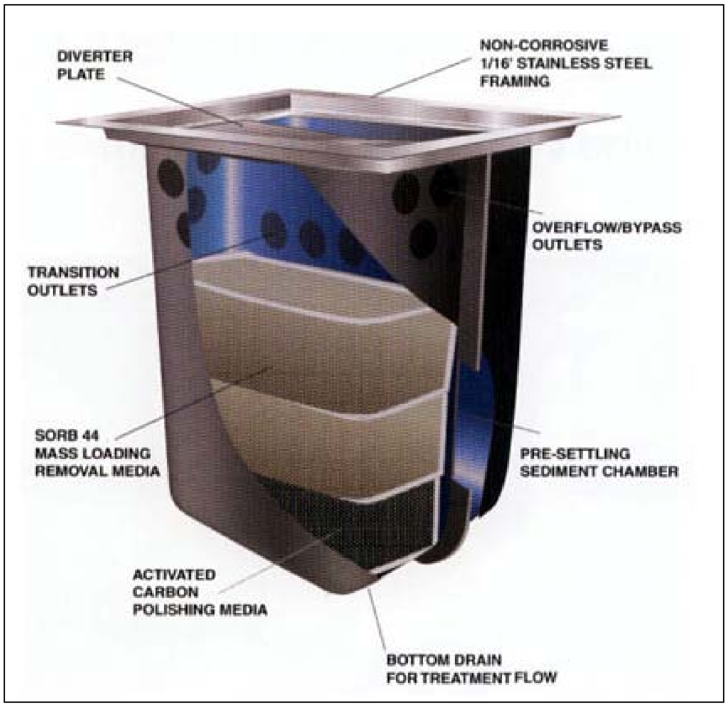


Figure 15.6. Hydro-Kleen Filtration System

Source: Hydro Compliance Management, Inc. Brighton, MI

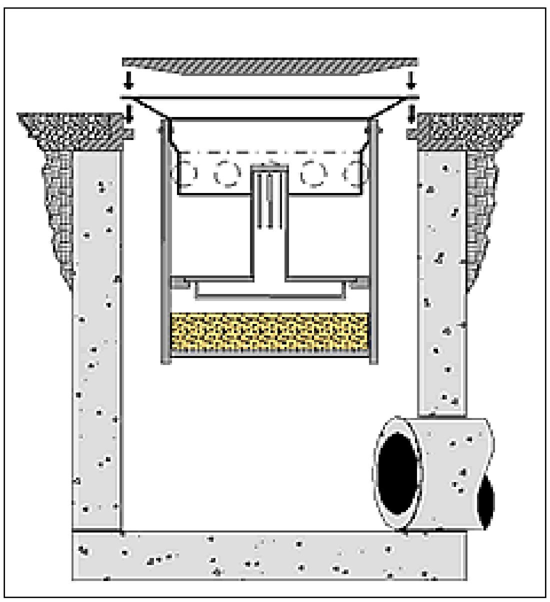


Figure 15.7. Aqua-Guard Catch Basin Insert

Source: Aquashield, Inc.;Water Services Inc. 1102 C. Montalona Rd.
Dunbarton, NH 03046

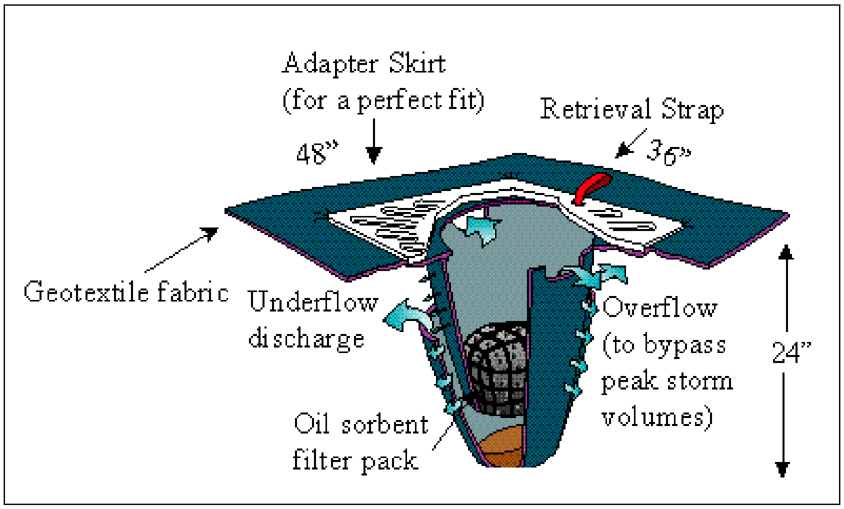


Figure 15.8. StreamGuard Catch Basin Insert

Source: Bowhead Manufacturing Co.
P.O. Box 80327
Seattle, WA 98108

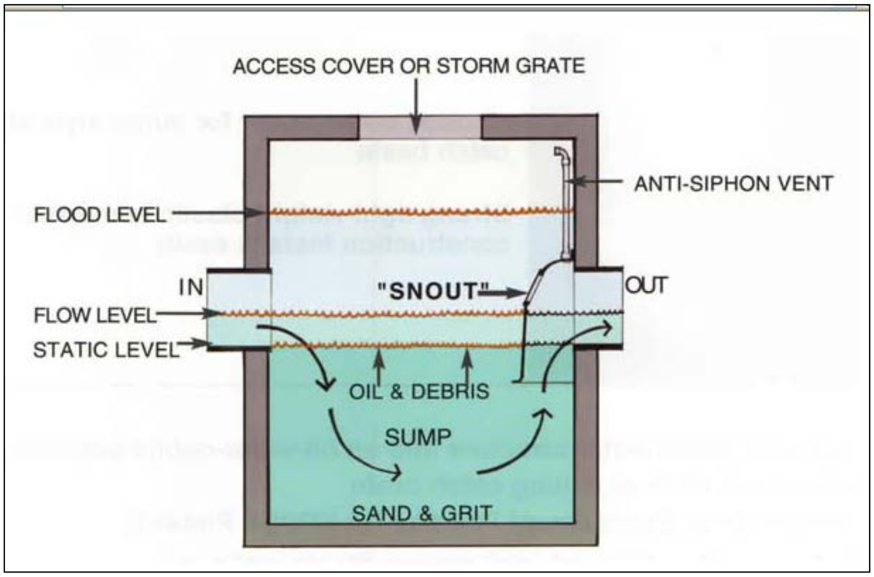


Figure 15.9. The SNOUT Catch Basin Insert

Source: Best Management Products, Inc., 53 Mount Archer Road, Lyme, CT 06371

Chapter 16 – Hydrodynamic Separators

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

16.1 Overview of Practice

The following design example provides guidance for the implementation of manufactured oil / water hydrodynamic separation devices for purposes of runoff quality management on VDOT facilities projects.

Hydrodynamic separation devices are designed to remove settleable solids, oil and grease, debris, and floatables from stormwater runoff through gravitational settling. Oil / water separation devices are not intended to mitigate the peak rate of runoff from their contributing watershed. Their implementation is solely for water quality enhancement in urban and ultra-urban areas where surface BMPs are not feasible. These manufactured systems are designed as flow-through structures. In contrast to conventional BMP measures capable of storing a designated water quality volume, flow into a manufactured hydrodynamic separator is regulated by its inflow pipe or other structural hydraulic devices. When the maximum design inflow is exceeded, the inflow may be regulated by a pipe restrictor, causing stormwater to back up into the upstream conveyance system or associated storage facility. When structural devices are employed to regulate flow into the hydrodynamic separator, flows in excess of the desired treatment volume either bypass the structure completely or bypass the separator's treatment chamber (VADCR, 2000).

Hydrodynamic separators are often employed as pretreatment measures for high-density or ultra urban sites, or for use in hydrocarbon hotspots, such as gas stations and areas with high vehicular traffic. Hydrodynamic separators *cannot be used for the removal of dissolved or emulsified oils and pollutants such as coolants, soluble lubricants, glycols and alcohol* (Georgia Stormwater Manual 2001). Hydrodynamic separators are limited in application by the following:

- Hydrodynamic separators are not capable of removing more than 80 percent of total suspended solids TSS.
- Dissolved pollutants are not effectively removed by these BMPs.
- Frequent maintenance is required to maintain desired pollutant removal performance levels.
- Hydrodynamic separators do not reduce peak rates of runoff to pre-developed levels.

Hydrodynamic separation devices are generally categorized as *Chambered Separation Structures* or *Swirl Concentration Structures*.

Chambered separation devices rely on gravitational settling of particles and, to a lesser degree, centrifugal forces to remove pollutants from stormwater. Chambered systems exhibit an upper bypass chamber and a lower storage / separation chamber. Runoff enters the structure in the upper bypass chamber and is channeled through a downpipe into the lower storage / separation, or treatment chamber. The system is designed such that when inflow exceeds the operating capacity, flow "jumps" the downpipe and completely bypasses the lower treatment chamber (VADCR, 1999).

Swirl separation structures are characterized by an internal mechanism that creates a swirling motion. This motion results in the settling of solids to the bottom of the chamber. Additional chambers serve to trap oil and other floating pollutants. Swirl separators do not exhibit a means for bypassing large runoff producing events. Larger flows simply pass through the structure untreated; however, due to the swirling motion within the structure, large flow events do not re-suspend previously trapped particulates. (VADCR, 1999)

16.2 Design Considerations

The design process for a specific installation of a hydrodynamic separator usually begins with a review of various vendor publications and use of preliminary sizing guidelines provided by the vendor. The specific design criteria for the hydrodynamic separator being considered should be obtained from the manufacturer or vendor to ensure that the latest design and sizing criteria are used. At the very least, the design for a particular site should be reviewed by the manufacturer to ensure that the system is adequately sized and located. The following criteria are intended to serve only as general guidelines.

- The use of oil-grit hydrodynamic separators should be limited to the following applications:
  - Pretreatment for other structural controls.
  - High-density, ultra urban or other space-limited development sites.
  - Hotspot areas where the control of grit, floatables, and/or oil and grease is required.

- Hydrodynamic separators are typically limited in use to drainage areas less than five acres. It is recommended that the contributing drainage area to any single separator be limited to one acre or less of impervious cover.

- Manufactured separation systems can be used in almost any soil or terrain. Additionally, since located underground, aesthetic and public safety issues are rarely encountered.

- Separation devices are sized based on *rate* of runoff. This design criteria contrasts with most BMPs, which are sized for a designated runoff *volume*.

- Hydrodynamic separators are typically designed to bypass runoff flows in excess of the design flow rate. This bypass may be accomplished by a built in bypass mechanism or a diversion weir or flow splitter located upstream of the separator in the runoff conveyance system. As with all runoff control structures, an adequately stabilized outfall must be provided at the separator's discharge point.

- The separator units should be watertight to prevent possible groundwater contamination.

- The separation chamber must provide three distinct storage volumes:
  - Volume for separated oil storage at the chamber top
  - Volume for settleable solids at the chamber bottom

   - Volume to provide adequate flow-through detention time (volume to ensure maximum horizontal velocity of 3 ft/min through the chamber)
- The total wet storage of the gravity separator unit should be at least 400 cubic feet per contributing impervious acre.

- The minimum depth of the permanent pools should be four feet.

- Hydrodynamic separators require a much more intensive maintenance schedule than other BMP measures. A typical maintenance schedule is shown as follows:

Activity	Schedule
Inspect the gravity separator unit.	Quarterly
Clean out sediment, oil and grease, and floatables, using catch basin cleaning equipment (vacuum pumps). Manual removal of pollutants may be necessary.	As needed

Table 16.1. Typical Maintenance Activities for Gravity Separators

- All specific design criteria should be obtained from the manufacturer.

Source: Georgia Stormwater Management Manual, published by the Atlanta Regional Commission, Atlanta, Georgia, 2001

16.3 Manufactured Products

The following discussion of manufactured hydrodynamic separators is intended only to serve as a description of the most widely used proprietary systems. The products discussed in this design example do not constitute an exhaustive list of all hydrodynamic separation devices available. Presentation of the following products does not preclude the use of other available systems, nor does it constitute an endorsement of any one system.

16.3.1 Stormceptor

Stormceptor is a precast, modular, vertical cylindrical tank divided into an upper bypass and lower storage chamber. The Stormceptor functions by diverting flow through a downpipe into the lower storage / separation chamber. Flow is then routed horizontally around the circular walls of the separation chamber. The circular flow motion, along with gravitational settling, traps sediments and other particulate pollutants. Flow then exits the Stormceptor through an outlet riser pipe. The outlet pipe is submerged, thus preventing trapped floatables from exiting the structure. The configuration also prevents turbulent flow in the storage / separation chamber, thus preventing resuspension of trapped particulates. The Stormceptor has no moving parts, and requires no external power source. (VADCR, 1999)

During large runoff producing events, flow entering the Stormceptor floods over the diversion weir and through the bypass chamber into the downstream conveyance system. The overflow of the system is controlled by the incoming stormwater velocity and the hydraulics of the diversion weir. The bypass configuration does result in a backwater condition in the upstream conveyance system. (VADCR, 1999)

It is generally recommended that Stormceptor systems be fully pumped a minimum of once per year. This frequency must be increased if high levels of sediment loading are observed. Schematic details of the Stormceptor system are presented as follows.

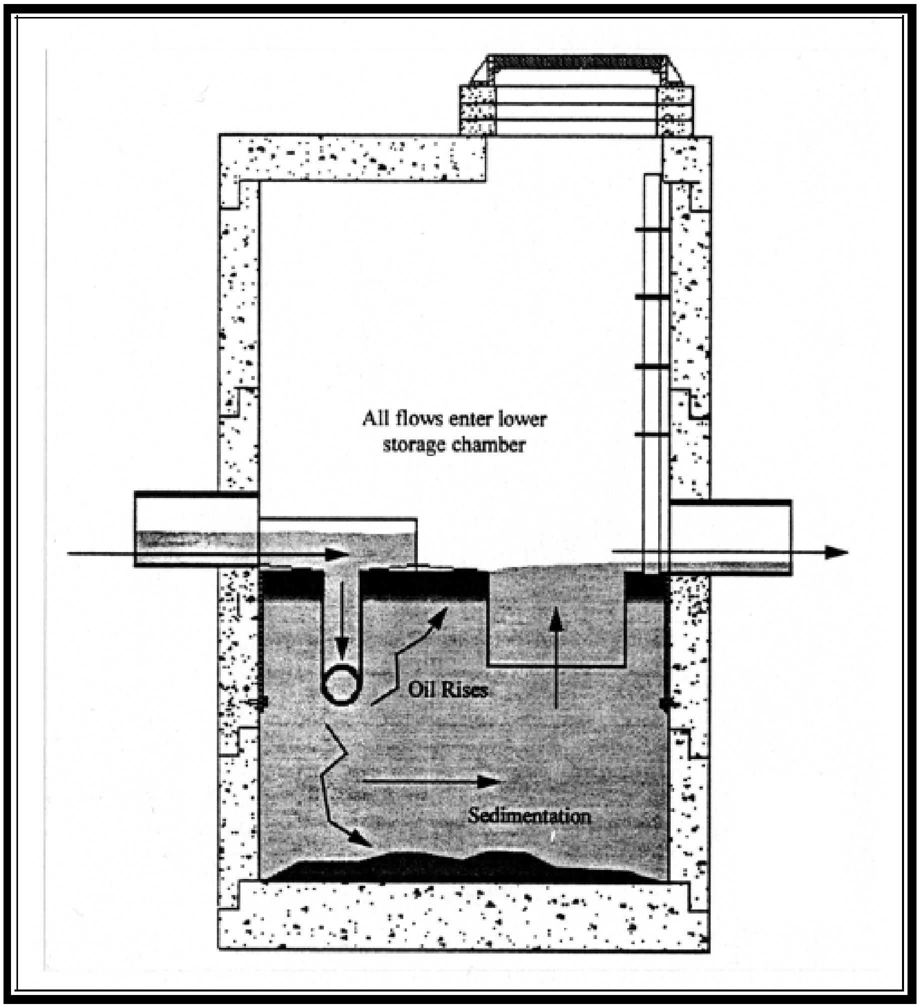


Figure 16.1. Stormceptor During Normal Flow Conditions

Source: Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

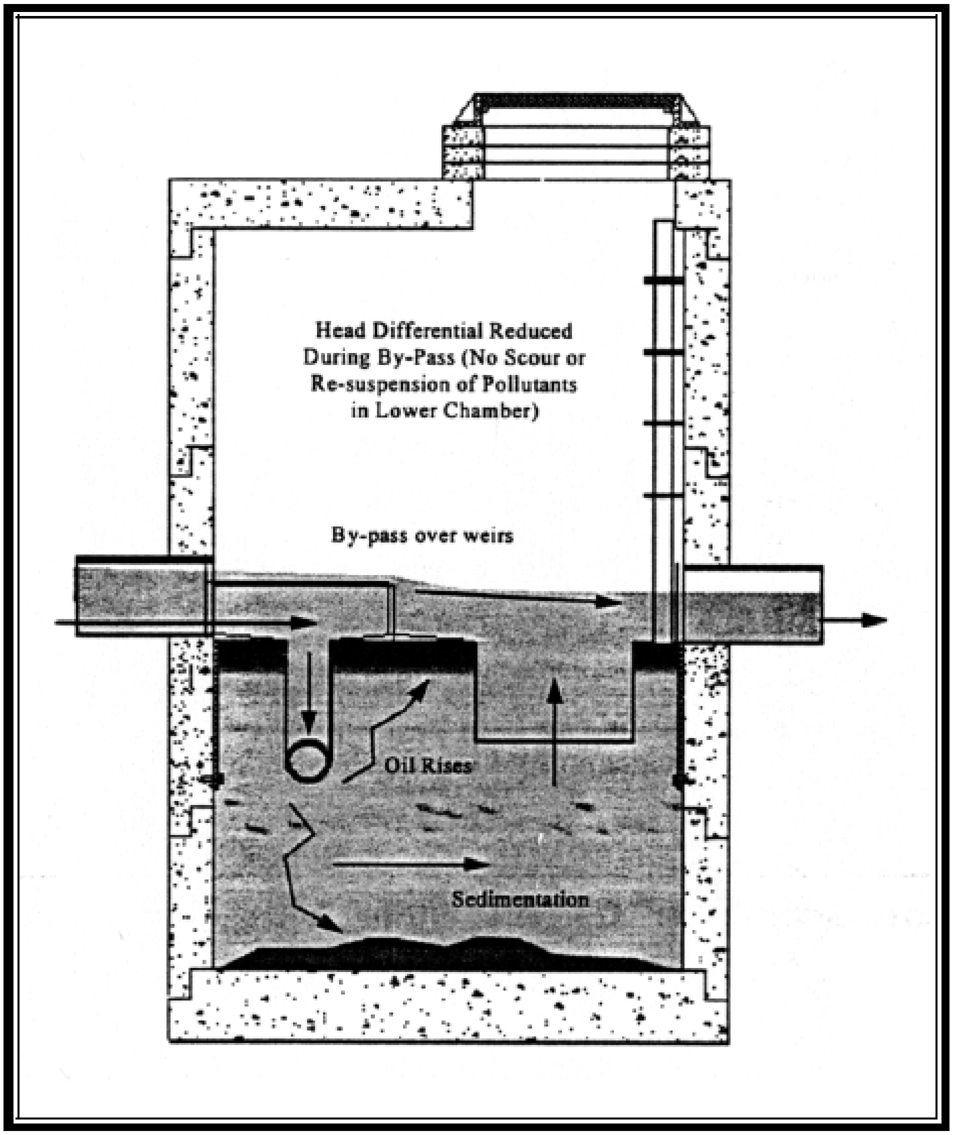


Figure 16.2. Stormceptor During High Flow Conditions

Source: Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Current Stormceptor product information and vendor contacts can be obtained at: http://www.stormceptor.com

16.3.2 Vortechs Stormwater Treatment System

The Vortechs Stormwater Treatment System is a precast rectangular unit composed of three chambers. The first chamber serves as a grit chamber, and creates a swirling motion that directs settleable solids toward the center where they become trapped. The Vortechs system is an all-inclusive proprietary system, with the swirl-inducing mechanism self contained within the unit. Flow is then slowly released from this chamber into the oil chamber. The oil chamber contains a barrier which traps oil and grease and other floatable pollutants. The final chamber is the flow control chamber, which forces water to back up, thus reducing velocities and turbulence. The Vortechs Stormwater Treatment System contains no moving parts and requires no external power source. (VADCR, 1999)

During large runoff producing events, the flow control chamber of the Vortechs system forces runoff to fill the structure. As this occurs, the swirling action in the grit chamber increases, keeping sediment concentrated at the center of the chamber. Because the swirling action of the system increases as the volume of runoff entering the structure increases, the resuspension of previously deposited material is eliminated. The Vortechs system is capable of providing limited flow attenuation within its storage capacity. When the volume of runoff entering the structure exceeds the capacity of the three chambers, the conveyance system leading to the Vortechs system will experience a backwater condition.

To ensure proper performance, the Vortechs system must be cleaned when it becomes full of pollutant material. During the first year of operation, the manufacturer recommends monthly inspections since contaminant loading rates vary greatly. Cleaning of the system is most readily accomplished by use of a vacuum truck.

Schematic details of the Vortechs system are presented as follows.

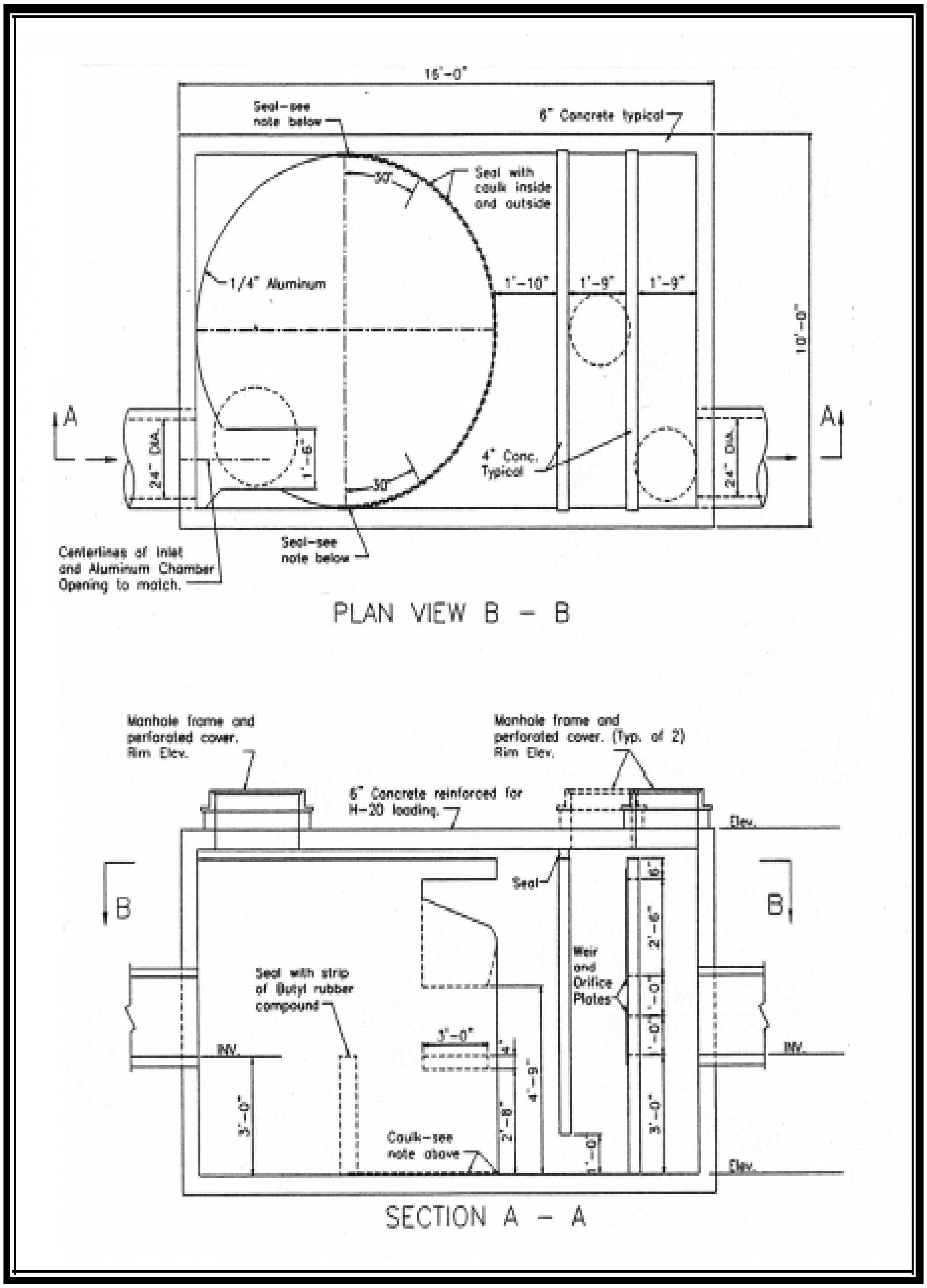


Figure 16.3. Vortechs Stormwater Treatment System

Source: Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Current Vortechs product information and vendor contacts can be obtained at: www.vortechnics.com

16.3.3 Downstream Defender

The Downstream Defender system is adaptable to all types of land uses. Additionally, the Downstream Defender can be installed in existing pipe systems as a retrofit.

The Downstream Defender is characterized by a concrete cylindrical structure with stainless steel components, and an internal 30^o sloping base. Runoff entering the structure passes through a tangential inlet pipe, resulting in a swirling motion. The flow then spirals downward along the perimeter of the structure. During this downward path, heavier particles settle out by gravity and by drag forces exerted along the wall and base of the structure. As flow rotates about the vertical axis, these solids are directed toward the base of the structure, where they are stored. The system's internal components direct the main flow away from the structure's perimeter and back up the middle of the vessel as a narrower spiraling column rotating at a slower velocity than the outer downward flow. When this upward flow reaches the top of the structure, it is virtually free of solids, and is then discharged through the outlet pipe. The Downstream Defender has no moving parts and requires no external power source.

During the first 12 months of operation, inspections should be conducted frequently following runoff-producing events in order to determine the sediment loading rate. After this time, a probe may be used after storm events to determine a maintenance schedule. H.I.L. Technology, Inc. recommends inspection and clean-out of the Downstream Defender system a minimum of twice per year.

Schematic details of the Downstream Defender system are presented as follows:

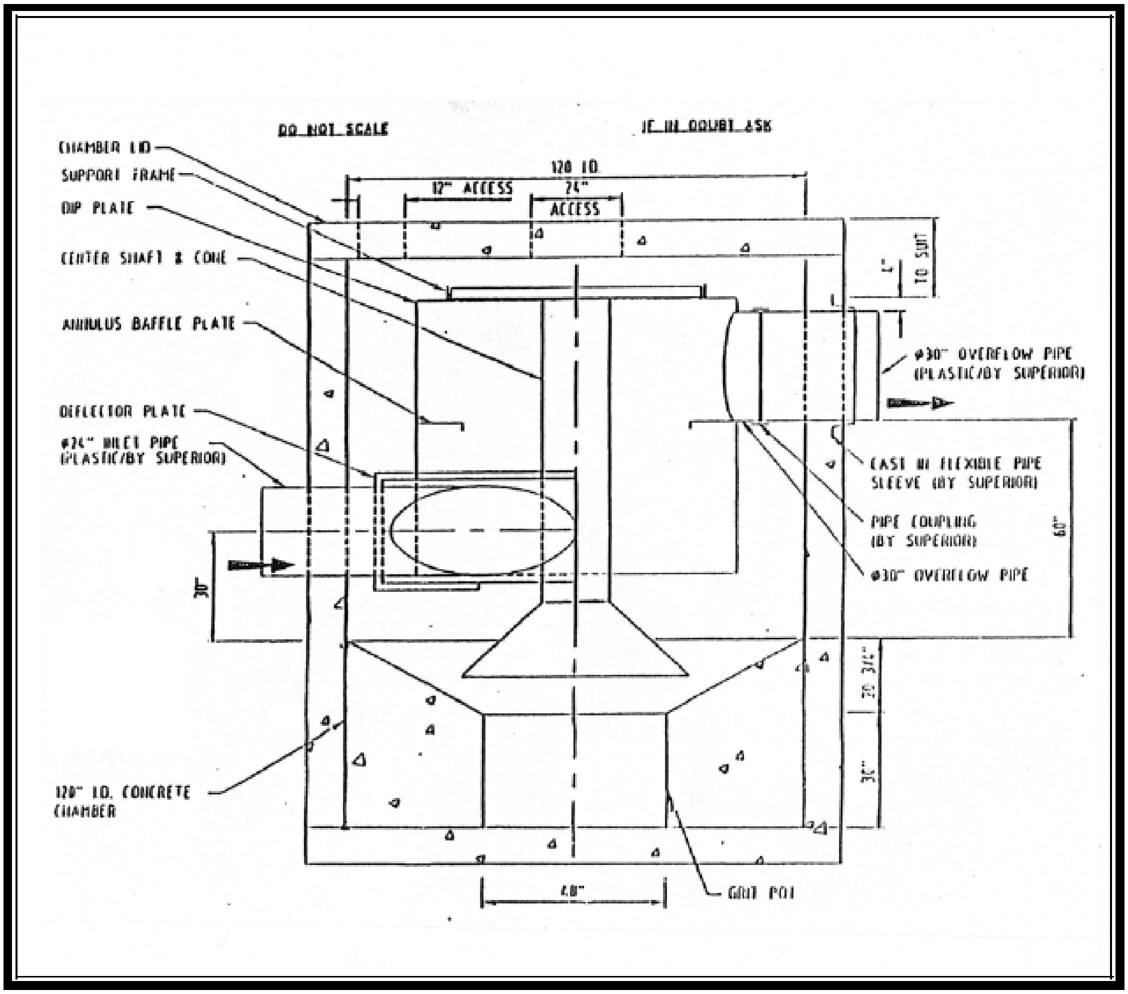


Figure 16.4. Section View of Downstream Defender System

Source: Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

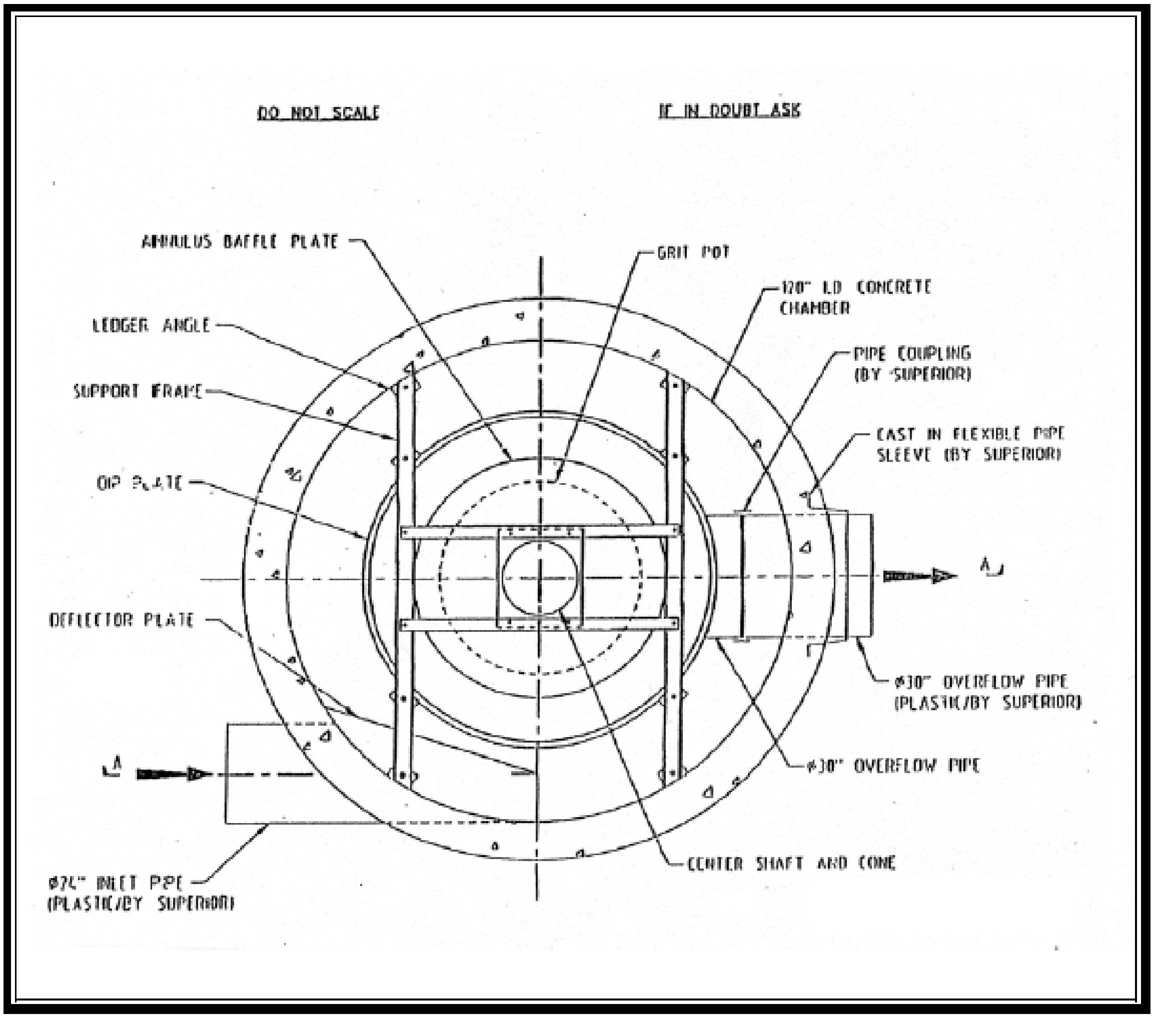


Figure 16.5. Plan View of Downstream Defender System

Source: Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Current Downstream Defender product information and vendor contacts can be obtained at:
www.hil-tech.com

16.3.4 BaySaver

The BaySaver system is composed of three main components: the primary separation manhole, the secondary storage manhole, and the BaySaver Separator Unit. Runoff enters the system through the primary separation manhole. The larger sediments contained in the runoff settle into the primary separation manhole whose flow exits through a trapezoidal weir. The runoff leaving the primary separation manhole carries with it floating contaminants, debris, and fine sediment which are then treated in the secondary storage manhole. The BaySaver system employs three potential flowpaths for runoff entering the system. First flush and low flows are diverted into the second manhole for the most efficient treatment. As the water level rises in the primary separation manhole, more water flows over the skimming weir and into the secondary manhole. The majority of oils and fine sediments are removed by this flow path. During more intense storms, water can flow through 90-degree elbow pipes located in the primary separation manhole. Because the elbows are situated below the surface, the water entering the secondary storage manhole is free from floating contaminants. During large, infrequent storm events, the BaySaver system bypasses the treatment stages, conveying water directly from inlet to outlet. Bypassed flows are prevented from entering the sedimentation manholes, and thus resuspension of contaminants does not occur. The BaySaver system contains no moving parts and requires no external power source. (VADCR, 1999)

It is generally recommended that BaySaver systems be fully pumped a minimum of once per year. This frequency may be increased if high levels of sediment loading are observed.

Schematic details of the BaySaver system are presented as follows.

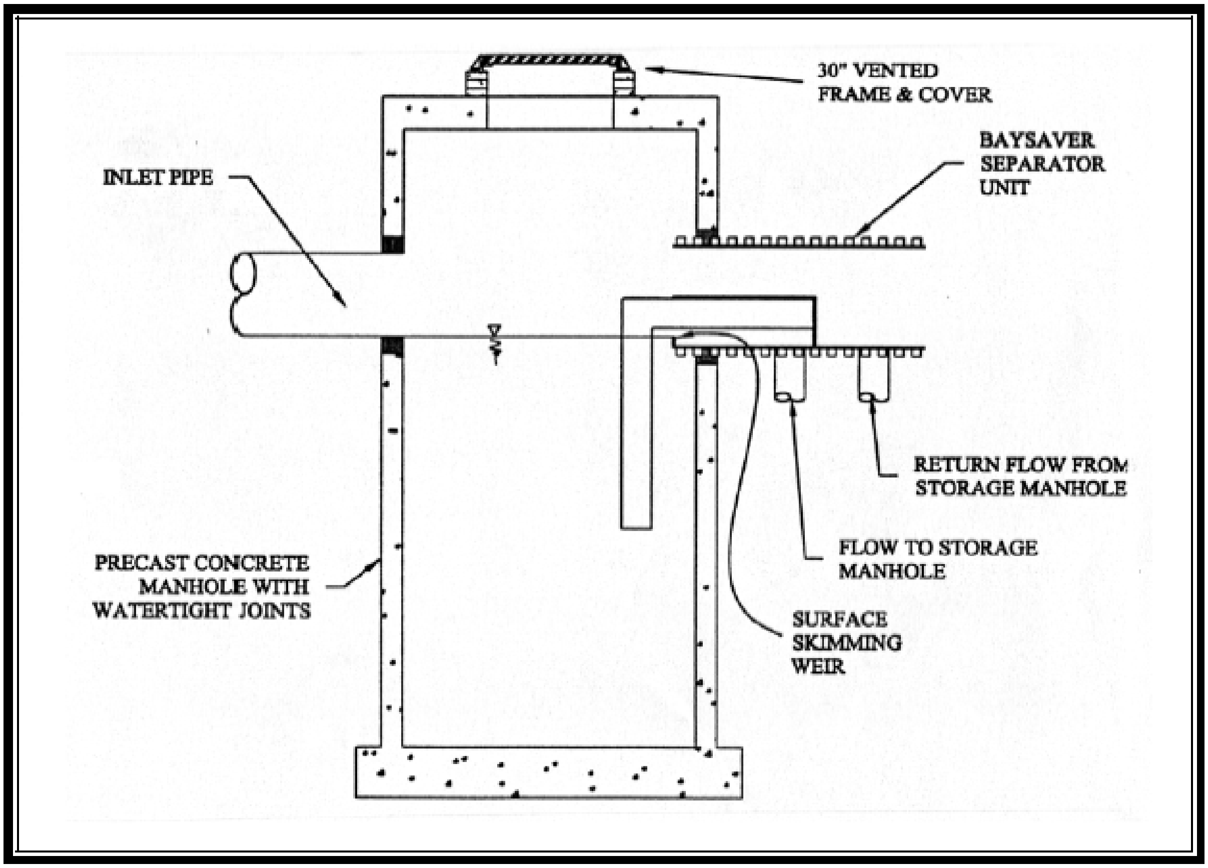


Figure 16.6. BaySaver Primary Separation Manhole

Source: Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

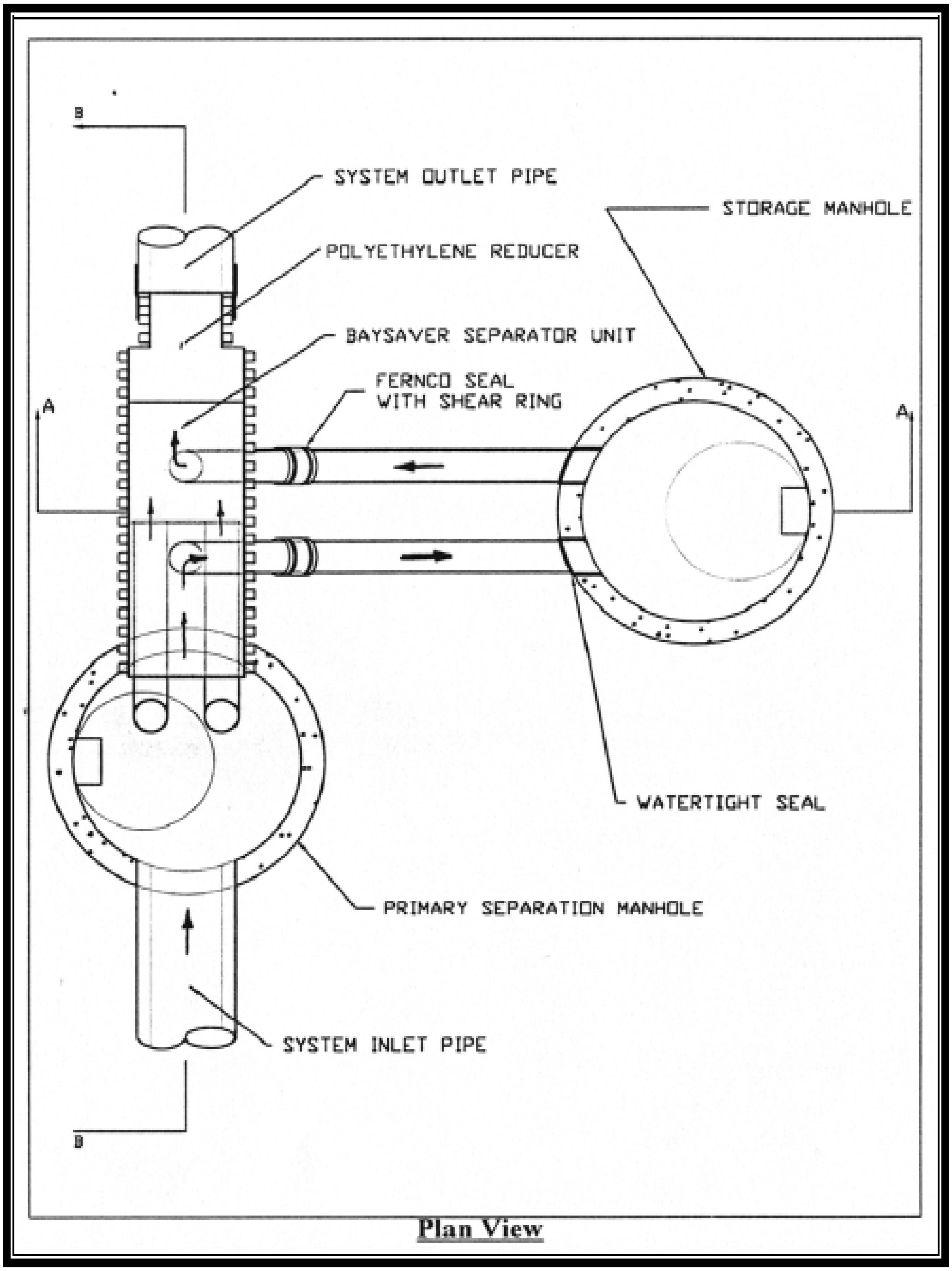


Figure 16.7. Plan View of BaySaver System

Source: Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

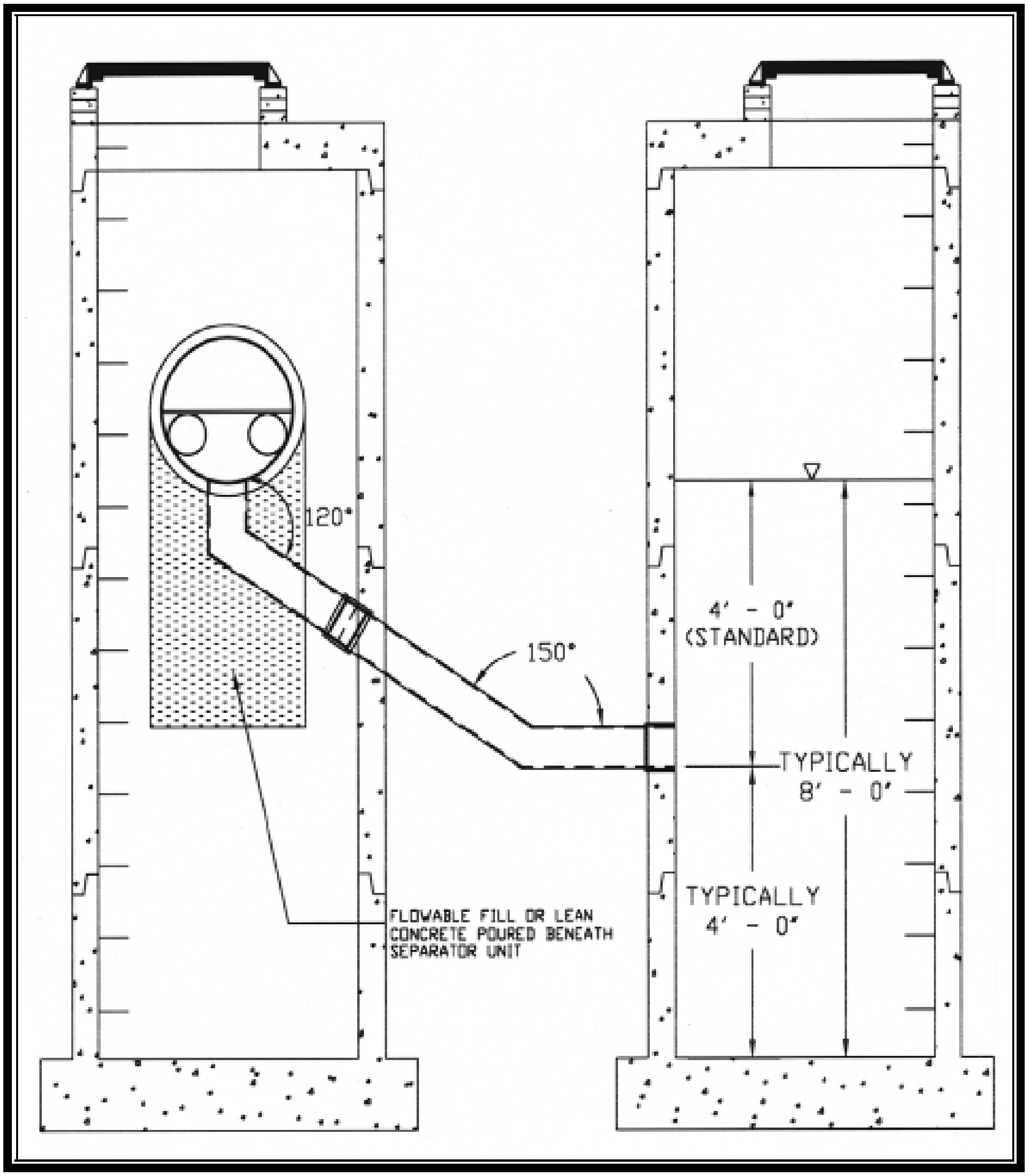


Figure 16.8. Section through BaySaver Storage Manhole

Source: Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999

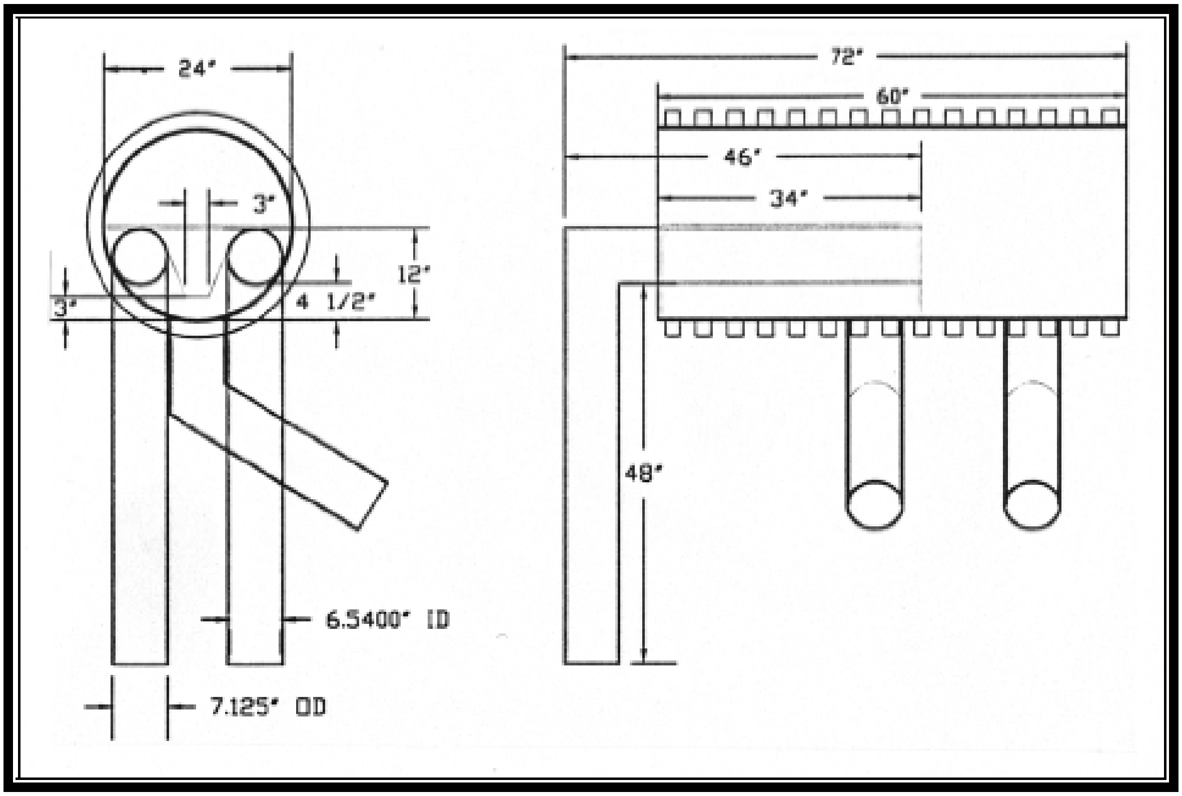


Figure 16.9. BaySaver Separation Unit

Source: Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999

Current Baysaver product information and vendor contacts can be obtained at: http://www.baysaver.com/

The Virginia Transportation Research Council, via contract with University of Virginia, has constructed the following information matrices for the most widely used hydrodynamic separators, as of 2004. The user is referred to the following for the originally published matrices:

Virginia Transportation Research Council. VDOT Manual of Practice for Stormwater Management. Charlottesville, Virginia, 2004.

System Type	Manufacturer	Operation	Sizing and/or Area Treated	Maintenance	Cost	General Performance	Comments
Hydrodynamic Separators							
V2B1	Kistner, Inc.	Swirl concentrator in 2 chambers. Second chamber collects floatables and has outlet. Maintains wet pool. Treats only first flush.	1 – 25 cfs treatment capability. Sized for local 2-month storm. Flows greater than first flush diverted directly to outlet.	Required only in first chamber if regular maintenance. Residuals removed by vacuum truck.	ND	80% TSS removal for first flush.	Floating pollutants isolated from peak storm flows.
Bay Saver®	Bay Saver, Inc.	Gravity treatment in 2 manholes connected by HDPE separator. Primary manhole in-line with the	Either according to flow rate or impervious area. Three units	Required in either chamber when accumulation reaches 2 ft	$7,000 - $18,000 (materials only)	Designed to remove TSS, O&G, and debris,*	
		storm drain. First-flush or low-flow diverted to storage chamber for settling and O&G removal. Outflow from center of static wter column to retain floatables back to primary manhole. Maintains wet pool in storage chamber.	available correspond to range of treatable areas: 1.2 – 8.0 acres impervious area. Largest systems treats maximum up to 11 cfs.				
Stormceptor®	CSR America	Manhole-shaped device. First-flush or low flows divered beneath high-flow platform to settling chamber. Outflow from center of static water column to retain floatables. Maintains wet pool.	8 units available: 900 – 7,200 gal.; 0.55 – 6.7 acres of impervious area. Sized to treat 90% of annual rainfall.	Perform maintenance when stored material reaches 15% total system volume. Recommend quarterly inspections during first year to establish schedule.	Typical installation is $9,000 for 1 acre drainage area. Unit cost: $7,600 - $33,560 per unit. (US EPA, 1999e)	Varying reports. Vendor claims 50 – 80% removal of TSS based on field testing by contracted agencies. Canada ETV reports 81-94% TSS removal; 42-67% TKN removal.	Improper installation compromises system performance. Also, available with inflow configured for curb inlet or submerged application. Over 4,000 installations.
Stormvault™	Jensen Precast	Rectangular footprint. Interior baffles minimize horizontal velocity to enhance settling and prevent resuspension. Bypass available.	Variable sizes afforded by adding modular sections. Sized to treat 85% annual rainfall or runoff. Variable outlet structure allows extended detention.	Large footprint allows extended periods between maintenance. Recommended inspections to establish schedule.	ND	Laboratory testing indicates low horizontal velocity near vault bottom to minimize resuspension. Extensive evaluation provided in Brisbane et al., 2000	Several field monitoring studies are being performed.
Vortechs™	Vortechnics, Inc.	Rectangular footprint comprised of 3 chambers; swirl concentrator, O&G removal, underflow to energy dissipator. Maintains wet pool.	10 units available to treat maximum 10-yr design storms of 1.6 – 25 cfs without bypassing. On-line system sizing criteria based on 1 ft^2 grit chamber surface area per 100 gpm peak flow rate.	Monthly inspection during first year after installation or whenever loading have been high.	$10,000 - $40,000 per unit, not including shipping or installation (US EPA, 1999e)	Vendor claims 80% TSS removal for flow less than or equal to design events. Sediment storage capacity 0.75 – 7.0 yd^3 depending on model.*	Improper installation compromises system performance. 1998 US EPA Environmental Technology Innovator Award.
CDS®	CDS Technologies, Inc.	Non-mechanical screening system. Circular flow maintained within unit	Treats first-flush with bypass option. Precast systems	3 – 4 times per year. Frequent inspection is required	$2,300 - $7,200 per cfs capacity (including	100% of particle size of mesh opening; Over 90% for	Vendor has won several engineering awards in Australia

Table 16.2. Hydrodynamic Separators Information Matrix (VTRC, 2004)

System Type	Manufacturer	Operation	Sizing and/or Area Treated	Maintenance	Cost	General Performance	Comments
		Pollutants settle to sump or remain floating and trapped in center column. Radial flow cleans screens. Maintains wet pool.	available up to 62 cfs. Cast-in-place options can treat up to 300 cfs. Screen size and unit diameter determined for specific applications.	especially during first month after installment. Maintenance includes inspection of screens for damage and measurement of sediment depth.	installation)	particles ½ the size of opening; over 85% for particles 1/3 size of opening; 80-90% O&G using sorbent materials. Complete trash removal*	Installations in the US, Australia and New Zealand.
Downstream Defender™	H.I.L. Technology, Inc.	Swirl concentrator creates a 3D flow path. Sediment settles to bottom of storage area. O&G also stored outside treatment path to prevent re-entrainment. Maintains a wet pool.	4 units range from 0.74 to 13 cfs design flows with corresponding 3 – 25 ft^3 capacity.	Clean-out after 1 – 2.5 ft of sediment accumulates – or annually.	\$10,000 - \$35,000 per unit (including installation)	PSD trapped sediments 0.001 – 0.01 mm (over 95% measured less than 75 µm). Estimate total solids removal was over 80% for theoretical design flows. Oil storage capacity 70 – 1050 gal.; sediment storage capacity of 0.7 – 8.7 yd^3.	ND

Table 16.2 Cont'd. – Hydrodynamic Separators Information Matrix (VTRC, 2004)

BIBIOGRAPHY

Chapter 2

Virginia Department of Conservation and Recreation. Virginia Erosion and Sediment Control Handbook. Richmond, Virginia, 1992.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Virginia Department of Transportation. Virginia Department of Transportation Road and Bridge Standards. Richmond, Virginia, 2006.

Virginia Tech & Penn State Urban Hydrology Model. T.A. Seybert, D.F. Kibler, A.S. Chamberlain, T.F. Smith. Version 6.2. June 2005.

U.S. Army Corps of Engineers. Technical Memorandum Number 9. Washington, D.C., 1987.

U.S. Department of Transportation. Federal Highway Administration. Hydraulic Design of Highway Culverts. Washington, D.C.: 1985 (revised 2001).

Chapter 3

Virginia Department of Conservation and Recreation. Virginia Erosion and Sediment Control Handbook. Richmond, Virginia, 1992.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Virginia Department of Transportation. Virginia Department of Transportation Road and Bridge Standards. Richmond, Virginia, 2006.

Chapter 4

Federal Highway Administration. Evaluation and Management of Highway Runoff Water Quality, FHWA-PD-96-032, Washington, D.C., 1996.

Virginia Department of Conservation and Recreation. Virginia Erosion and Sediment Control Handbook. Richmond, Virginia, 1992.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Virginia Department of Transportation. Virginia Department of Transportation Road and Bridge Standards. Richmond, Virginia, 2006.

U.S. Bureau of Reclamation. Design of Small Dams. 1960 (Revised 1977).

Chapter 5
Federal Highway Administration. "Evaluation and Management of Highway Runoff Water Quality, FHWA-PD-96-032, Washington, D.C., 1996.

Virginia Department of Conservation and Recreation. Virginia Erosion and Sediment Control Handbook. Richmond, Virginia, 1992.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Virginia Transportation Research Council (Yu, Et al.). VDOT Manual of Practice for Stormwater Management. Charlottesville, Virginia, 2004.

Chapter 6
Akan, A. O. Open Channel Hydraulics. Elsevier/Butterworth-Heinemann, 2006.

Federal Highway Administration (Chen and Cotton). Design of Roadside Channels with Flexible Linings, Hydraulic Engineering Circular 15, FHWA-IP-87-7, 1988.

Federal Highway Administration. Evaluation and Management of Highway Runoff Water Quality, FHWA-PD-96-032, Washington, D.C., 1996.

Mays, L. W. Water Resources Engineering. John Wiley & Sons, Inc., 2005.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Virginia Tech & Penn State Urban Hydrology Model. T.A. Seybert, D.F. Kibler, A.S. Chamberlain, T.F. Smith. Version 6.2. June 2005.

Chapter 7
Federal Highway Administration. Evaluation and Management of Highway Runoff Water Quality, FHWA-PD-96-032, Washington, D.C., 1996.

Mays, L. W. Water Resources Engineering. John Wiley & Sons, Inc., 2005.

Pennsylvania Department of Environmental Protection. Stormwater Best Management Practices Manual. Harrisburg, PA, 2006.

Virginia Department of Conservation and Recreation. Virginia Erosion and Sediment Control Handbook. Richmond, Virginia, 1992.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Virginia Transportation Research Council (Yu, Et al.). VDOT Manual of Practice for Stormwater Management. Charlottesville, Virginia, 2004.

Chapter 8
Virginia Department of Conservation and Recreation. Virginia Erosion and Sediment Control Handbook. Richmond, Virginia, 1992.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Chapter 9
Virginia Department of Conservation and Recreation. Virginia Erosion and Sediment Control Handbook. Richmond, Virginia, 1992.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Chapter 10
Federal Highway Administration. Evaluation and Management of Highway Runoff Water Quality, FHWA-PD-96-032, Washington, D.C., 1996.

Mays, L. W. Water Resources Engineering. John Wiley & Sons, Inc., 2005.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Chapter 11
Bell, Warren. A Catalog of Stormwater Quality Best Management Practices for Heavily Urbanized Watersheds. National Conference on Urban Runoff Management: Enhancing Urban Watershed Management at the Local, County, and State Levels, 1993.

Pennsylvania Department of Environmental Protection. Stormwater Best Management Practices Manual. Harrisburg, PA, 2006.

Virginia Department of Conservation and Recreation. Virginia Erosion and Sediment Control Handbook. Richmond, Virginia, 1992.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Virginia Transportation Research Council (Yu, Et al.). VDOT Manual of Practice for Stormwater Management. Charlottesville, Virginia, 2004.

Chapter 12
Bell, Warren. A Catalog of Stormwater Quality Best Management Practices for Heavily Urbanized Watersheds. National Conference on Urban Runoff Management: Enhancing Urban Watershed Management at the Local, County, and State Levels, 1993.

Maryland Department of the Environment. Maryland Stormwater Design Manual. Baltimore, MD 2000.

Virginia Department of Conservation and Recreation. Virginia Erosion and Sediment Control Handbook. Richmond, Virginia, 1992.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Chapter 13
Osmundson, Theodore. Roof Gardens History, Design, and Construction. New York: W.W. Norton & Company, 1999.

Pennsylvania Department of Environmental Protection. Stormwater Best Management Practices Manual. Harrisburg, PA, 2006.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Chapter 14
Pennsylvania Department of Environmental Protection. Stormwater Best Management Practices Manual. Harrisburg, PA, 2006.

Texas Water Development Board. The Texas Manual on Rainwater Harvesting. Austin, Texas. Third Edition, 2005.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Chapter 15
Pennsylvania Department of Environmental Protection. Stormwater Best Management Practices Manual. Harrisburg, PA, 2006.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Virginia Transportation Research Council (Yu, Et al.). VDOT Manual of Practice for Stormwater Management. Charlottesville, Virginia, 2004.

Chapter 16
AMEC Earth and Environmental, Center for Watershed Protection, Debo and Associates, Jordan Jones and Goulding, Atlanta Regional Commission. Georgia Stormwater Management Manual. Atlanta, Georgia, 2001.

Pennsylvania Department of Environmental Protection. Stormwater Best Management Practices Manual. Harrisburg, PA, 2006.

Virginia Department of Conservation and Recreation. Virginia Stormwater Management Handbook. Richmond, Virginia, 1999.

Virginia Transportation Research Council (Yu, Et al.). VDOT Manual of Practice for Stormwater Management. Charlottesville, Virginia, 2004.