Stormwater Design Guide

11/25/2024
Revision 1.0
Atlanta, GA 30308

Georgia Department of Transportation

This document was developed as part of the continuing effort to provide guidance within the Georgia Department of Transportation in fulfilling its mission to provide a safe, efficient, and sustainable transportation system through dedicated teamwork and responsible leadership supporting economic development, environmental sensitivity and improved quality of life. This document is not intended to establish policy within the Department, but to provide guidance in adhering to the policies of the Department.

Your comments, suggestions, and ideas for improvements are welcomed.

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DISCLAIMER

The Georgia Department of Transportation maintains this printable document and is solely responsible for ensuring that it is equivalent to the approved Department guidelines.

Forward

Welcome to the new Georgia Department of Transportation (GDOT) Stormwater Design Guide. In 2024, GDOT revised the Drainage Design for Highways manual and moved Chapters 9 and 10 as well as Appendices H and J into this new design guide. This guide focuses on the analysis of stormwater during the design of projects for compliance with National Pollutant Discharge Elimination System (MS4 and erosion control) permits, water quality analysis for ecology purposes, and detention analysis. This guide discusses means and methods for the design of such Best Management Practices which treat or detain stormwater runoff. For this first edition, Chapters 9 and 10 have simply been moved to the new guide with no modifications other than reference updates and minor changes to local road guidelines.

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Revision Summary

Revision Number	Revision Date	Revision Summary
1.0	11/25/24	New manual

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List of Effective Chapters

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Chapter 1. Erosion and Sediment Control Guidelines - Contents

Chapter 1. Erosion and Sediment Control Guidelines

1.1 Introduction

Stormwater runoff can be a major cause of impaired water quality in Georgia's streams, rivers, and lakes. Runoff from disturbed lands can degrade surface water by increasing the concentration of TSS, which also raises the turbidity. Since many pollutants have the tendency to adhere to solids, suspended solids in stormwater runoff can add significant quantities of nutrients, metals, and toxins. Making the problem worse, paved surfaces and storm sewer systems decrease the amount of runoff that can be absorbed into the ground, where stormwater would otherwise be filtered and detained. This chapter is concerned primarily with erosion and sedimentation control during the construction of roadway and facilities for GDOT. For control of other pollutants such as nutrients (e.g. nitrogen and phosphorus), dissolved and total metals (most commonly copper, lead, and zinc), and trash, the designer should refer to chapter 2 of this manual and also the GSMM.

Sedimentation problems are the result of inadequate erosion and sedimentation controls on construction sites. To prevent these problems, vegetative and structural BMPs control the erosion of soil and the resulting sedimentation. Proper BMP erosion and sediment management along with sampling turbidity levels of the construction site stormwater discharge can greatly reduce stormwater pollution from construction site activities. Turbidity, commonly measured in nephelometric turbidity units (NTU), is a measurement based on the amount of light scattered and absorbed by fine particles in suspension.

1.2 NPDES Program

NPDES permits are one of two types: an individual permit or a general permit. Individual permits are unique to each facility and are required for large MS4s. General permits prescribe one set of requirements for similar facilities that meet the eligibility criteria. Small MS4s and construction site activities, such as roadway projects, are normally covered by a general permit.

Applying for a general permit is accomplished by submitting a Notice of Intent (NOI) to the Georgia Environmental Protection Division (EPD). The NOI includes the location and description of the construction activity and defines the erosion, sedimentation, and pollution control plan (ESPCP) goals to minimize impacts with BMPs and monitoring. The NOI process is considerably less complicated than the application required for an individual permit.

Typically, ESPCPs for GDOT's construction projects should be in compliance with the State of Georgia NPDES General Permit, the *Manual for Erosion and Sediment Control in Georgia* (Green Book), [(1-2)] the Georgia Soil and Water Conservation Commission (GSWCC), GDOT's own guidelines, and all other applicable federal and state laws and rules. The designer should read and understand the applicable NPDES general permit and the Green Book prior to beginning the design of an ESPCP. Section 1.6 discusses common BMPs used for ESPCPs.

The linear nature of GDOT's projects creates some difficulty regarding the appropriate methods used to comply with the permit and the Green Book. The current edition of GDOT's *Plan Presentation Guide* (PPG) [(1-1)] gives the designer checklist-style guidance to overcome this difficulty by informing the designer how to prepare an effective and uniform ESPCP. The PPG explains what information to include in an ESPCP and how to present the information, but it does not address the technical aspects of ESPCP design. This chapter is a technical resource for ESPCP design, provides clarification to the requirements of the NPDES general permit, and points to the guidance provided by the Green Book. It is assumed that the reader already has a good understanding of the general format of a complete ESPCP prepared by GDOT and that the reader is aware of the current EPD-GSWCC checklist, which is the checklist for ESPCP preparation effective on January 1 of the year in which the land-disturbing activity was permitted. The current checklist and an explanation of how to address the checklist are available on GDOT's website under ROADS/Design Policies and Guidelines (also accessible through the following web address): https://www.dot.ga.gov/GDOT/Pages/designmanualsguides.aspx.

1.3 Georgia NPDES General Permit Regulations and Requirements

Copies of the current NPDES permits, the Green Book, and other related technical documents may be downloaded from the Georgia Soil and Water Conservation Commission's website, http://gaswcc.georgia.gov/documents-list.

The NPDES permitting authority in Georgia is the EPD. The EPD issues three general permits that authorize the discharge of stormwater from three distinct types of construction projects that disturb 1 or more acres of land. These three general permits, which are reissued every 5 years, are:

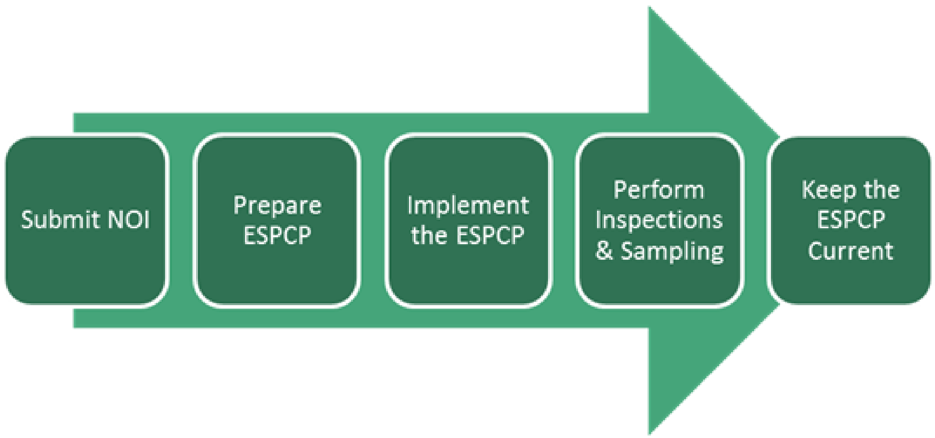
- **Stand-alone construction activity (GAR100001):** construction activities that are not part of a common development where the primary permittee chooses not to use secondary permittees.
- **Infrastructure construction sites (GAR100002):** construction activities that are not part of a common development that include the construction, installation, and maintenance of roadway and railway projects. These activities may also include all conduits, pipes, pipelines, substations, cables, wires, trenches, vaults, manholes, and other similar structures. Most all GDOT linear projects should be considered infrastructure construction sites.
- **Common development construction (GAR100003):** a contiguous area where multiple, separate, and distinct construction activities will be taking place at different times on different schedules under one plan.

Each of the permits is available on the EPD's website. Most GDOT-related projects fall under the general permit GAR100002. The major requirements of this permit are (also outlined in Figure 9.1):

- **Submission of an NOI** – A draft NOI is generally submitted by GDOT along with final plans, which is then completed by GDOT and given to EPD for review and comment. Two half-size ESPCP sets should be furnished for this submittal.
- **Preparation of an ESPCP** – The plan must detail the BMPs to be used at the site, and it must be prepared under the supervision of a GSWCC Level II Certified Design Professional whose professional license is issued by the State of Georgia in the field of: engineering, architecture, landscape architecture, forestry, geology, or land surveying; or a person that is a Certified Professional in Erosion and Sediment Control (CPESC) with a current certification by EnviroCert International, Inc. Design professionals shall practice in a manner that complies with applicable Georgia law governing professional licensure.
- **Implementation** – The plan must be implemented as designed.

- **Sampling** – For infrastructure projects, representative sampling may be utilized and is often performed. The permit requires that regulated sites be monitored by sampling the stormwater discharge quality with respect to turbidity.

Figure 1.1 - ESPCP flow chart



1.4 ESPCPs

1.4.1 Introduction

Preparation of the ESPCP requires an understanding of GDOT policy and appropriate construction general permit requirements. This section will discuss both the EPD-GSWCC requirements, as well as GDOT policy, to help guide the designer through plan production. Section 1.4.3 discusses in detail the submittal package that GDOT requires, but for any other information see the PPG [(1-1)] document.

The EPD-GSWCC checklist requires that all state waters within 200 feet and all ponds and lakes within 500 feet of the right-of-way be labeled on a *Drainage Area Map (53 Series)* and a *Watershed Map (55 Series)*. Additionally, these waterways should be shown on the BMP Location Detail Sheets (54 Series) if they are within the limits of the sheet, and they should be shown on the cover sheet if the scale allows. The plans should delineate all watersheds within the project limits. They should also show flow paths from the outfall discharge point to the receiving water. This is to assist personnel in identifying critical water features that can be affected by construction activities.

Where applicable, stream buffers for these streams must also be shown. All streams should be delineated by an ecologist and included in environmental documentation, including the Environmental Resource Impact Table (ERIT). Associated buffers that are on the right-of-way shall be described on the stream buffer table that is provided within the ESPCP General Notes Sheets. Georgia law restricts the amount and type of work that is permitted within the stream buffers, requiring the designer to describe the nature of work that is permitted within the buffer areas. Stream buffers begin at the point of wrested vegetation along the stream channel. Wrested vegetation is at the point of clear distinction between the flow of water and vegetation. This is caused by the normal movement of water where soil and vegetation are removed through naturally occurring erosion. The types of work qualifying for a buffer variance are listed in EPD Rule 391-3-7-.05, Buffer Variance Procedures and Criteria. However, even with an EPD buffer variance, a Section 404 nationwide permit from the U.S. Army Corps of Engineers would be required.

Georgia law permits work to be performed inside buffers without a variance on projects for the construction and maintenance of bridges and roadway drainage structures. GDOT and EPD interpret this law to mean that any work within 50 feet of either side of a culvert or other drainage structure (see examples 6 and 8 in the ESA Examples.pdf document on the ROADS website: http://www.dot.ga.gov/PartnerSmart/DesignManuals/ElectronicData/ESA%20Examples.pdf#search=ESA%20Examples%2Epdf) and any work within 100 feet of either side of a bridge (see ESA examples 9 and 10) will not require a buffer variance, provided the work within the buffer is associated with the structure. Occasionally, instances may arise that require areas beyond the 50-foot and 100-foot limits to be disturbed to build the structure. If projects require this additional area to construct a bridge or drainage structure, permission to work beyond the 50-foot and 100-foot limits without a variance may be granted by EPD on a case-by-case basis. Representatives of GDOT should consult with EPD to determine whether or not a particular project may warrant a buffer variance exemption. Buffer variances must be approved prior to the ESPCP and NOI being submitted to EPD for review.

Although work within the permitted 50-foot and 100-foot limits does not require a buffer variance, the construction activities will impact the stream buffer, and the stream buffer encroachment table should indicate that the buffer is impacted. The designer must also assume that the contractor should clear all the area within the right-of-way. Any area within the right-of-way where clearing is not permitted (buffer areas beyond the 50-foot or 100-foot limits mentioned above, habitat of any threatened or endangered species located on the right-of-way, etc.) should be marked with an orange barrier fence. Where there are instances that the right-of-way is not entirely cleared but purchased for future work, a plan note should be added to the plans indicating the new clearing limits. The buffer areas that have restricted access and are left undisturbed act as a BMP and should be labeled with the standard "Bf" symbol. A buffer cannot be thinned or trimmed of vegetation and must remain for water quality and the preservation of aquatic habitat.

1.4.2 Policy Guidelines

The design of the ESPCP is site specific and design elements will vary. However, the following guidelines provide assistance in the preparation of ESPCPs:

- Use approved sources for the proper design and location of BMPs, spacing, and application.
- Keep runoff velocities low by using use check dams, J hooks, earthen berms, and/or diversion ditches, for example.
- Do not place silt control gates in perennial or intermittent streams.
- Do not place sediment basins, ditches, or other structures in wetland areas.
- Be certain that sufficient right-of-way is available for BMPs.

Show the following background data on all ESPCP sheets: centerline with stationing, all edges of pavement, the construction limits, the right-of-way, all easements, and the location of all drainage structures, pipes, streams, lakes, and wetlands.

For staged projects, the ESPCP should correspond to the staged construction plans (19 Series) provided in the plan submission package. In certain cases, additional sub-stages must be shown to indicate the installation of perimeter BMPs and sediment storage BMPs. The construction plans should depict the final post-construction BMPs, which include ditch linings, riprap, vegetated swales, and stabilized drainage structures. See chapter 2 of this manual for additional information on post-construction stormwater BMPs. The title block shall show the normal project information, have the large letters "ESPCP", and indicate the particular stage of construction as "Stage 1", "Stage 2", etc.

ESPCPs are required for all projects regardless of the size of the disturbed area. ESPCPs for haul roads, borrow pits, excess material pits, etc., shall be prepared by the contractor. These plans shall be prepared for all stages of construction and should include the appropriate items and quantities.

For projects with less than 1 acre of disturbed area, an abbreviated ESPCP may be prepared, and only the Erosion and Sediment Control Legend and Uniform Code Sheets, the BMP Location Details, and any applicable Erosion and Sediment Control Construction Detail Sheets are required. All projects with 1 or more disturbed acres must have a complete stand-alone ESPCP. Abbreviated ESPCPs and complete stand-alone ESPCPs are placed in the back of the construction plans.

The complete ESPCP must include:

- an ESPCP Cover Sheet (50 Series)
- ESPCP General Notes Sheets (typically 2 or 3 sheets, 51 Series)
- ESPCP Legend and Uniform Code Sheets (52 Series)
- a Drainage Area Map (53 Series)
- BMP Location Details (54 Series)
- a Watershed and Monitoring Site Location Map (typically a USGS topographical sheet, 55 Series)
- Construction Details and Standards (for erosion and sedimentation control items only, 56 Series)

GDOT requires a Worksite Erosion Control Supervisor (WECS) to be on call 24 hours a day for all construction projects. The role of the WECS is primarily to oversee all erosion and sedimentation control related work throughout the project. They perform daily inspections on ESPCP BMPs to check performance and make adjustments as needed to comply with permit and contract requirements. The WECS works closely with the Field Project Engineer to prevent violations and reduce BMP failures. For more information on the WECS program, see GDOT Special Provision 161 or the GDOT Local Technical Assistance Program (LTAP) Office: https://www.dot.ga.gov/GDOT/Pages/LTAP.aspx.

1.4.3 Description of the Complete ESPCP

If the ESPCP is prepared by GDOT, it must be signed and stamped by GDOT's Chief Engineer. If the ESPCP is prepared by a consultant, it must be signed and stamped by a GSWCC Level II Certified Design Professional. Although the ESPCP preparation for a GDOT infrastructure project is discussed in detail in GDOT's PPG [(1-1)], a few important points are presented within this section. See section 1.4.4 of this chapter for additional information on signatory requirements.

A. BMP Location Detail Sheets:

BMP location detail sheets show the actual location of the BMPs for each stage of construction. These detail sheets should have the same drawing scale and orientation as the Construction Plan Sheets. Staged BMP installation must match the construction staging if the construction is staged. GAR100002 indicates that proposed contour lines and a BMP legend should be included on each BMP Location Detail Sheet. GDOT has found that adding these items to the BMP sheets causes confusion due to the excessive amount of line work. As a result, the BMP legend (54 Series) is placed directly in front of the BMP sheets, and the proposed contour lines are not shown on BMP sheets. The profile and cross-section views in the Construction Sheets provide information equivalent to proposed contour lines. However, existing contours should be shown during the Initial Phase to ensure adequate perimeter control and other initial BMPs. The direction of concentrated stormwater runoff should be shown with flow arrows. For plans that involve special grading (e.g., detention ponds or other post-construction stormwater design elements), proposed contour lines are shown on the BMP sheets for these areas.

On the BMP Location Detail sheets, show the information in the following bulleted list in bold format with the proper BMP symbol, line code and type for the item, as shown on the Erosion and Sediment Control Legend Uniform Code Sheet (see GDOT Construction Detail Sheets EC-L1 to EC-L6 for symbols, line codes, and patterns). When BMPs are shown as installed in later phases of construction, show those BMPs as faded, where retained. If any BMPs are no longer needed in later phases, the symbol and BMP should be removed.

All ditches that have protection of any type whether temporary or permanent must be shown with the width of the ditch and the depth of protection. The width and depth may be shown in tabular format, and can also be shown in the summary of quantities. Each type of ditch protection shall have a different code on the plan sheet.

- Perimeter silt fence Types NS (nonsensitive) and S (sensitive) as defined in the latest version of the Manual for Erosion and Sediment Control in Georgia, as required. They have their own line codes and symbols, which must be shown.
- Indicate which type of silt control gate is being used.
- All temporary sediment basins and skimmers should have the appropriate symbols.
- Show riprap slope protection with the pattern symbol. Any other form of slope protection must be shown by its symbol and pattern.
- All down-drain structures, temporary or permanent, should be labeled with their symbols and line codes.
- Silt retention barrier as recommended by the soil's lab by the symbol and line code
- Storm-drain outlet protection by the symbol and pattern

B. Watershed and Monitoring Location Map (scale no less than 1 inch = 2,000 feet):

Use a USGS 7.5-minute quadrangle map as the base topographic map, unless contours from a more accurate source can cover the entire area. If a quadrangle map is used, show the name, date published, scale, north arrow, and the contour interval.

This map differs from the Drainage Area Map in that it is prepared to a much larger scale to show the big picture. The most important items to show on this map are the receiving water(s), the delineation of the receiving-water SWDA(s), and the turbidity sampling location(s).

A site may have multiple receiving waters, each having a distinct SWDA. Delineate each receiving-water SWDA and indicate its area in square miles. In addition, the total project size must also be noted.

Note that the NPDES permit states, “When the permittee has chosen to use a USGS topographic map and the receiving water(s) is not shown on the USGS topographic map, the location of the receiving water(s) must be hand-drawn on the USGS topographic from where the storm water(s) enters the receiving water(s) to the point where the receiving water(s) combines with the first blue line stream shown on the USGS topographic map.”

C. ESPCP Construction Details and Standards:

Erosion and sediment control details and standard sheets are included as applicable and are obtained from GDOT’s ROADS website.

1.4.4 Signatory Requirements for ESPCPs

The education and certification requirements for individuals qualified to sign ESPCPs are established by the Official Code of Georgia O.C.G.A. § 12-7-19, and are defined by the GSWCC in Section 600-8-1 of the *RULES OF THE STATE SOIL AND WATER CONSERVATION COMMISSION*. Signatory requirements for ESPCP are defined by the Georgia EPD in Parts IV.B and C, and V.G of the general permit.

In accordance with the above regulations, the following protocol must be followed with regards to the signing of ESPCPs for GDOT projects:

- ESPCPs for projects requiring an NOI must be signed by a GSWCC Level II Certified Design Professional. A Design Professional means a professional licensed by the State of Georgia in the field of engineering, architecture, landscape architecture, forestry, geology, or land surveying or a person that is a Certified Professional in Erosion and Sediment Control (CPESC) and certified by the Certified Professional in Erosion and Sediment Control Inc.
- Consistent with agreement between GDOT and EPD, the signature, seal, and Level II certification number are required on the ESPCP Cover Sheet only.
- The GDOT Chief Engineer stamps, signs, and includes their Level II certification number on the completed ESPCP Cover Sheet prior to submission of final plans to the Office of Construction Bidding Administration (CBA). This includes in-house prepared and consultant prepared ESPCPs. Consultants must sign, seal and certify the ESPCP Cover Sheet prior to certification by the GDOT Chief Engineer.
- Subsequent revisions to the ESPCP must be certified (on the ESPCP Cover Sheet Revision Block) by the Level II Certified Design Professional in charge of the revision. The ESPCP must be amended whenever there is a change in the design, construction, operation, or maintenance that has a significant effect on BMPs with a hydraulic component. Refer to Part IV.C of the general permit. BMPs with a hydraulic component can be defined as requiring hydrologic analyses for design.
- The contractor is responsible for preparing supplemental ESPCPs for construction activities that are not defined in the ESPCP. In these cases, the contractor is required to have a Level II Certified Design Professional prepare, sign and certify the supplemental ESPCP.

1.4.5 Revisions to the ESPCP During the Life of a Project

If the contractor requests to alter the staged construction from that shown in the plans or to utilize construction techniques that render the original ESPCP ineffective, and if GDOT's construction project engineer approves the request, then the contractor has the responsibility of revising and recertifying the ESPCP to reflect all the changes. This should also include any revisions to erosion and sedimentation control pay item quantities.

The contractor may also wish to include several items that are not generally included on the original set of construction plans. These may include: haul roads, batch plants, staging areas, petroleum storage areas, and borrow or waste pits. If these items are not included in the original ESPCPs, the contractor must create a separate ESPCP and obtain all required permits pertaining to additional work that the contractor wishes to perform.

The WECS may authorize minor revisions to the ESPCPs with approval from the Field Project Engineer. Minor revisions only need to be "redlined" on the master set of erosion and sediment control plans kept at the project site and do not need the signature of a GSWCC Level II Certified Design Professional in the cover sheet revision block. Examples of minor revisions include adding silt fence, riprap, or check dams.

A major revision is the addition, deletion, or modification of a structural BMP with a hydraulic component (e.g., those BMPs on which the design is based on hydrological factors). Major revisions to the ESPCP are treated as formal Use on Construction revisions and require a recertification signature in the ESPCP cover sheet revision block by a GSWCC Level II Certified Design Professional. Copies of major revisions are submitted to the appropriate EPD district office.

1.5 Right-of-Way

Make certain that sufficient right-of-way or easement is available for the proper construction and maintenance of all structural BMPs. This concept also applies to post-construction stormwater BMPs, where required. Sufficient area is particularly important when using stream diversion channels and temporary sediment basins. To determine the required area, it is recommended that the designer prepare a preliminary ESPCP prior to right-of-way and easements being finalized.

1.6 BMP Location and Design Criteria

Many BMPs function by catching, filtering, and releasing stormwater runoff slowly. If BMPs are not installed properly or are misapplied, they will not perform effectively. They may even cause hazards such as the ponding of stormwater on the roadway. For example, inlet sediment traps along the roadway are not usually allowed because they tend to cause stormwater ponding. If the road is open to traffic, hydroplaning may result. Additionally, the impounded stormwater can leave behind a slick sediment residue once it drains. For these reasons, BMPs should never be installed to impound water on the roadway.

Once the construction site nears final stabilization, the project area begins to transition from construction stormwater BMPs (temporary controls) to post-construction stormwater BMPs (permanent controls). An example of this transition would be the conversion of a sediment basin into a dry detention pond after cleaning out sediment, or a temporary diversion channel converted to a vegetative swale. While functioning as a temporary sediment basin, the BMP shall meet retrofitting requirements stated in the Manual for Erosion and Sediment Control in Georgia, current edition. These requirements include but are not necessarily limited to volume requirements and length-to-width ratio requirements. After the BMP is converted to a permanent post-construction BMP, it shall meet the design guidelines for post-construction BMPs in this manual. See GDOT's R.O.A.D.S website for special construction details. See chapter 2 of this manual for more information on post-construction stormwater BMPs.

The information presented in this section is intended for use as a supplement to the Green Book and as an interpretive guide to the NPDES permit requirements. The information provides an overview on construction stormwater BMP implementation and special application of BMPs with respect to their use on GDOT's roadway construction projects. Refer to the Green Book [(1-2)] for a detailed treatment of BMP application, design, installation, and maintenance, as well as additional illustrations of BMPs.

1.7 Chapter 1 References

1. Georgia Department of Transportation (GDOT). 2012. Plan Presentation Guide, Version 01.50.
2. Georgia Soil and Water Conservation Commission (GSWCC – Green Book), 2016. "Manual for Erosion and Sediment Control in Georgia," 2016 Edition.
3. Thompson, P.L., Kilgore, R.T. 2006, Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14, FHWA-NHI-06-086. Federal Highway Administration (FHWA), U.S. Department of Transportation, Washington, D.C.

Chapter 2. Post-Construction Stormwater - Contents

Chapter 2. Post-Construction Stormwater

2.1 Introduction

As of January 3, 2012, stormwater discharges from infrastructure owned and operated by GDOT are regulated by GADNR's EPD through GDOT's MS4 NPDES permit (permit number GAR041000). The permit was renewed on January 3, 2022. Post-construction stormwater management measures have been a part of GDOT policy, but the MS4 permit adds additional requirements. This chapter introduces post-construction stormwater management concepts, defines post-construction requirements of GDOT projects, and provides guidance on meeting these requirements and designing post-construction BMPs.

2.1.1 Chapter 2 Content Overview

The chapter is organized as follows:

2.1 Introduction

2.2 The Need for Post-Construction Stormwater Management

2.3 Project Applicability

2.4 MS4 Post-Construction Stormwater Management Minimum Standards

2.5 Post-Construction Stormwater BMP Selection Criteria

2.6 Post-Construction Stormwater BMP Design Criteria

2.7 Detention Design

2.8 Common BMP Components

2.9 Bridge Stormwater Quality Considerations

2.10 Safety Considerations for Stormwater BMPs

2.1.2 Additional Resources

This chapter provides post-construction stormwater management guidance for typical GDOT projects. Chapter 1 of the Drainage Design Policy Manual as well as the Plan Development Process Manual provides additional information regarding project milestone requirements, as well as other stormwater planning information. Each project has unique challenges related to stormwater management and the designer should consult GDOT for further guidance if necessary.

In addition to this manual, the GSMM (including the Coastal Stormwater Supplement) can be used as supplemental guidance for GDOT projects. However, this manual will serve as the primary design reference, and where guidance contained within may differ from the GSMM, GDOT policy will apply. GDOT post-construction BMP details and specifications should be reviewed and utilized during BMP design. Post-construction BMP and LID/GI checklists are available as part of the *MS4 Post-Construction Stormwater Report*, found on the GDOT Manuals & Guides website. This report should be used to document BMPs that were considered, excluded, and implemented on projects located in an MS4 area.

2.2 The Need for Post-Construction Stormwater Management

2.2.1 Introduction to Post-Construction Stormwater Management

Post-construction stormwater management (not to be confused with construction stormwater management and associated erosion and sediment controls, which are discussed in chapter 9) refers to the permanent practices and structures put in place to reduce, treat, or minimize stormwater pollution from stabilized, developed areas. BMPs for post-construction applications may include grass channels, filter strips, detention ponds, stormwater wetlands, or any other GDOT-approved BMPs for post-construction.

Pollutants in the roadway are generated from litter, vehicle wear (e.g., brake dust, tire wear), oil and antifreeze leaks, etc. Typical pollutants include suspended solids, dissolved and total metals (typically copper, lead, and zinc), nutrients (e.g., nitrogen, phosphorus), and trash. [(2-1)] While negative impacts associated with poor runoff quality are a major concern, runoff quantity can be equally troublesome. Increased runoff volume and peak flow as a result of development may cause indirect hydromodification to a stream system. Indirect hydromodification to a stream can include accelerated stream bank or shoreline erosion, changes in sediment transport and temperature, and reduced habitat.

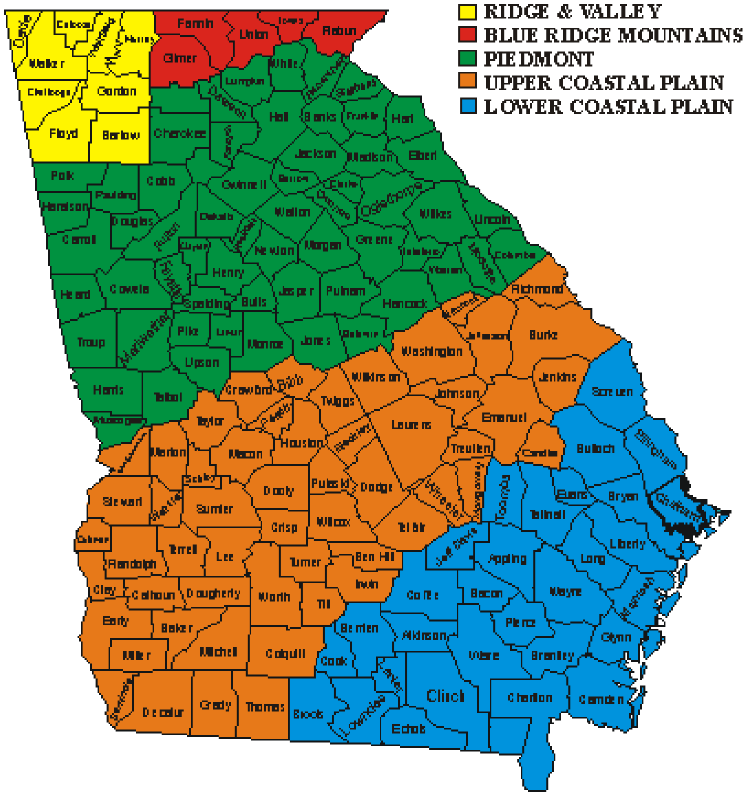
Stormwater pollution may result in a decrease of beneficial or desirable wildlife species and an increase in nuisance species. Stormwater pollution can also have the following negative effects: [(2-16)]

- Impairment of drinking water supplies
- Increased cost of treating drinking water
- Loss of, or decline in, recreational activities such as swimming and fishing
- Declining property values
- Economic loss related to commercial fishing, tourism, etc.

Georgia is divided into five physiographic regions, based on similarities in geomorphology, character, relief and environment. The regions, shown in Figure 2.2-1 are: Lower Coastal Plain, Upper Coastal Plain, Piedmont, Blue Ridge Mountains, and Ridge and Valley. The Georgia regions may have different stormwater concerns and stormwater solutions due to varying rainfall frequencies and distributions, geography, soil types, etc.

Communities in the northern part of the state are required to consider the effects of stormwater runoff on trout streams. As runoff flows over impervious surfaces, such as asphalt and concrete, the temperature increases and the heated water enters the receiving water. Temperature changes in receiving waterbodies can severely impact certain aquatic species, such as trout, which can survive only within a narrow temperature range. Communities in coastal areas are closely tied to the surrounding surface waters. Some coastal ecosystems are more sensitive to water quality issues. Poor water quality resulting from various sources (manufacturing, agriculture, etc.) can be harmful to the economy, health, and aesthetics of coastal areas. In addition, estuaries serve as nurseries for a significant amount of marine animals. Further, shellfish beds around the nation are often impacted by elevated bacteria levels found in runoff. For these reasons, coastal areas often have more stringent stormwater requirements that GDOT must also take into consideration. Additional information regarding stormwater management in coastal areas can be found in the *Coastal Stormwater Supplement to the Georgia Stormwater Management Manual*, First Edition, April 2009. (2-15)

Figure 2.2-1 – Physiographic Regions of Georgia
Reference: Georgia Department of Natural Resources



Post-construction stormwater management requires a comparison of post-developed conditions and flows to pre-developed conditions and flows. For GDOT projects, pre-development is defined as the condition of the site immediately prior to the implementation of the proposed project.

Post-construction stormwater management for roadway systems can present some unique challenges. Most entities that are required to manage stormwater are responsible for one discrete area, whereas GDOT roadways span the entire state, making maintenance of stormwater facilities challenging. GDOT right-of-way often limits the amount of space for BMP installation. In addition, GDOT right-of-way is extensively used as utility routes, leaving even less space for BMPs. Roadway safety requirements add additional constraints. All of these factors should be considered during the design of post-construction stormwater BMPs along with other limiting design constraints.

2.2.2 Stormwater Management for Special Environmental Concerns

Post-construction stormwater BMPs may be required for projects not located in an MS4 area due to flows, pollutant loads, increased runoff, or other environmental regulatory requirements. The need for BMPs separate from the MS4 program requirements are often required by regulatory agencies other than the GA EPD due to watershed-specific requirements to address impairments or threatened and endangered species. The GDOT Office of Environmental Services (OES) and the regulatory agency will determine the specific water quality and/or detention requirements and associated documentation on a case-by-case basis. An MS4 Post-Construction Stormwater Report is only required for projects located in a designated MS4 area. When addressing other environmental regulatory requirements, BMP designs need to follow the design guidance in this Manual and the Special Design Post-Construction Details to the extent possible. Priority should be given to cost-effective and low maintenance BMPs. Refer to Table 2.5-1 for relative cost of common BMPs approved for use on GDOT facilities. Use of BMPs and associated special details other than those shown in Table 2.5-1 or significant design deviations must be reviewed by ODPS prior to approval of plans. GDOT's exclusions and infeasibilities do not apply to post-construction stormwater BMPs required by OES.

2.2.3 Detention Analysis & Downstream Hydrologic Assessment [(2-18)]

A downstream hydrologic assessment is required for many projects with a post-developed flow increase or to evaluate effects of water quantity control facilities (detention) on peak discharge and timing downstream in the watershed. For maintenance (resurfacing, shoulder paving, bridge rehabilitation, culvert rehabilitation, ITB), safety (guardrail, cable barrier, signal upgrades, sign installation, ITS, single lane roundabouts, RCUTs with no added lanes, other safety projects which add less than 0.25 acres of net new impervious area), pedestrian improvements, and bridge replacement projects (over waterways), a detention report should not be submitted outside of MS4 requirements.

For other projects such as reconstruction and/or widening, the designer must evaluate post-development peak flows to determine if increased flows would cause flooding, spill outside of channel banks, overtop a road, result in some other have adverse effects on downstream properties or if detention will increase downstream flows.

If grading for the project will change the volume of any pond or if the outlet structure of any pond is to be modified, then the pond must be included in the detention and downstream hydrologic assessment.

Downstream analysis shall first be accepted by GDOT before specifying detention in the plans. If the project is preparing a Post Construction Stormwater Report, the downstream analysis shall be submitted as part of the Post Construction Stormwater Report. When the project does not need to have a Post Construction Stormwater Report, the detention report, if applicable, shall be submitted at least eight weeks prior to PFPR request. If a designer finds that detention is warranted later in the project, the designer shall coordinate with ODPS as soon as possible, but before purchasing ROW for the BMP and before specifying the BMP in the plans. If detention is recommended to be necessary, documentation of the adverse effects downstream shall be submitted to Design Policy and Support. GDOT will reserve final determination on the necessity of detention. The conveyance from the outfall should also be analyzed for capacity (water levels of 25-year, 24-hour storm events should stay within channels and pipes should not be in pressure flow).

An exception to this requirement occurs when discharging directly to channels or waterbodies that have drainage areas larger than five square miles (in which case downstream hydrologic assessment is not necessary).

Non-MS4 Detention Report

This section applies to outfalls on state routes.

The purpose of the Non-MS4 Detention Report (Report) is for the designer to provide information supporting its recommendation regarding detention. The information provided must be sufficient to allow GDOT to make an informed decision whether detention should be implemented. The goal of this decision-making process is both to protect downstream properties as much as practicable and to ensure that each proposed permanent BMP is definitively warranted. The Non-MS4 Detention Report template can be located on R.O.A.D.S.

The Non-MS4 Detention Report is not a permit-based report. Detention may only be added if detention is warranted and suitable.

In making recommendations regarding detention for a project, a designer must complete and provide ODPS with the relevant portions of Report and its Attachments A, A-1, and B (with Appendices) in the manner set forth hereinafter. After careful review and consideration of the contents of a Report submitted, the ultimate decision whether detention is warranted and suitable for a project shall be made by GDOT.

If it is being recommended that detention is not warranted for any outfalls, the designer shall submit the first page of the Report and its Attachment A-1 to stormreports@dot.ga.gov for widening projects, reconstruction projects, and new location projects. Since the rest of the Report addresses only those outfalls for which detention is recommended to be warranted and suitable, it is not required to be completed by the designer for submission when detention is not recommended to be warranted and suitable.

Attachment A-1 of the Report covers all outfalls on the project with drainage areas greater than 1 acre that add more than 0.1 acres of impervious area or at least 5% increase in overall drainage area. Outfalls which have contributing drainage areas of less than 1 acre do not need to be included in Attachment A-1. Drainage areas which add less than 0.1 acres of impervious area and less than 5% increase of overall drainage area pre-development to post-development also do not need to be included in Attachment A-1. The designer shall save any supporting documents or calculations within the Design Data Book. Supporting documents or calculations do not need to be submitted with the Non-MS4 Detention Report for outfalls where detention is not warranted.

If the designer recommends that detention may be both warranted and suitable, the designer shall fill out the entire Non-MS4 Detention Report, including all Attachments and Appendices, covering each outfall where detention is being recommended. Once completed, the entire Non-MS4 Detention Report (Report) shall be submitted to stormreports@dot.ga.gov. After being reviewed by ODPS, the Report shall be revised by the designer in any manner as may be directed by GDOT and must ultimately accepted by ODPS before detention is incorporated into any milestone plan submittal. It should be noted that Report **must** be accepted by ODPS before any purchase can be made of right-of-way for any BMP proposed for detention purposes outside of GDOT's MS4 areas.

Attachment A, GDOT's Post-Construction BMP Summary, must be completed by the designer and submitted with all detention reports recommending detention. The purpose of this documentation is to assist with the plan review process.

There are certain instances where it may not be suitable to implement post-construction BMPs. If detention is warranted but not suitable, contact the Office of Design Policy and Support. Criteria for when detention may not be suitable (for non-MS4 detention purposes) include the following:

1. Cases where the project would require an existing roadway alignment change solely to allow for a post-construction BMP. This applies only to existing roadway alignment changes or changes that would create a safety concern.
2. Instances where the installation of post-construction BMPs would require the re-alignment and/or piping of a stream.
3. Implementation of BMPs would cause loss of habitat for endangered or threatened species.
4. Implementation of BMPs would cause significant damage to cultural or community resource such as a historical site, archeological site, cemetery, park, wildlife refuge, nature trail, or school facilities.
5. Implementation of BMPs would result in violation of state or federal law, regulation, or policy. Examples include FEMA regulations and clear zone requirements.
6. Site limitations including shallow bedrock, contaminated soils, high groundwater, utilities, or underground facilities if avoidance or relocation is not suitable.
7. Site does not allow for gravity flow to the appropriate BMP.

GDOT currently allows two types of post-construction BMPs to be initially proposed for detention outside of GDOT's MS4 areas: dry detention basins and wet detention basins. If it is recommended that detention may be warranted, the designer shall first assess a dry detention basin for suitability. If the dry detention basin is considered not suitable, the designer shall next evaluate a wet detention basin for suitability. If both a dry detention basin and a wet detention basin are not suitable, ODPS should be contacted by the designer before proceeding to discuss whether other solutions may be suitable.

The Report's Attachment B, Detention BMP Documentation, is where the designer should include concise summaries for each drainage area evaluation. These summaries should include supporting information which BMPs may be warranted, which BMPs may be suitable, and, if wet detention is selected instead of dry detention, an analysis why dry detention is not recommended to be suitable.

Non-MS4 Detention Report Addendums

An addendum shall be submitted by the designer if there are significant project changes after the Non-MS4 Detention Report has been accepted by GDOT. An addendum shall be submitted if any of the following scenarios occurs:

1. An outfall not previously considered has been identified and the designer recommends detention is warranted and suitable for the outfall.
2. An outfall for which the designer previously recommended that detention was either not warranted or not suitable, detention is now recommended to be both warranted and suitable.
3. An outfall for which the designer previously recommended that detention was both warranted and suitable is now recommended to be either not warranted or not suitable.
4. The type of post-construction BMP recommended at an outfall has changed.

The following information shall be included in the addendum in PDF format and submitted to ODPS for review:

- Cover letter outlining the changes to the recommendations

- Revised cover page including signed, sealed, and dated PE stamp
- Revised Attachment A and Attachment A-1
- Revised sections and associated backup documentation in Attachment B
- Revised appendices, as relevant to the changes to the outfall or BMP
- Current construction plans

After reviewing the information submitted in the addendum, ODPS will make the final determination for whether detention is warranted and suitable.

Downstream Analysis Process

Detention BMPs are designed to attenuate flows, protect streams from bank erosion and hydromodification, and prevent flooding. However, attenuated peak flows from detention facilities can sometimes increase peak flow downstream due to the modified timing and increased overall volume of runoff. A downstream hydrologic analysis shall be performed for the 25-year storm to determine if combined flows from the project site and other properties have the potential to cause downstream problems.

Figure 2.2-2 illustrates the effect of peak discharge and timing. Detention can alter the peak flow timing so that the combined detained peak flow (the larger dashed triangle) is higher than if no detention is provided. In this case, detention shifts the peak flow to a later time so that, when combined with the flow from the rest of the drainage basin, downstream flooding is worse than if the post-development flow increases were not detained.

Figure 2.2-2 – Detention Timing Example (2-18)

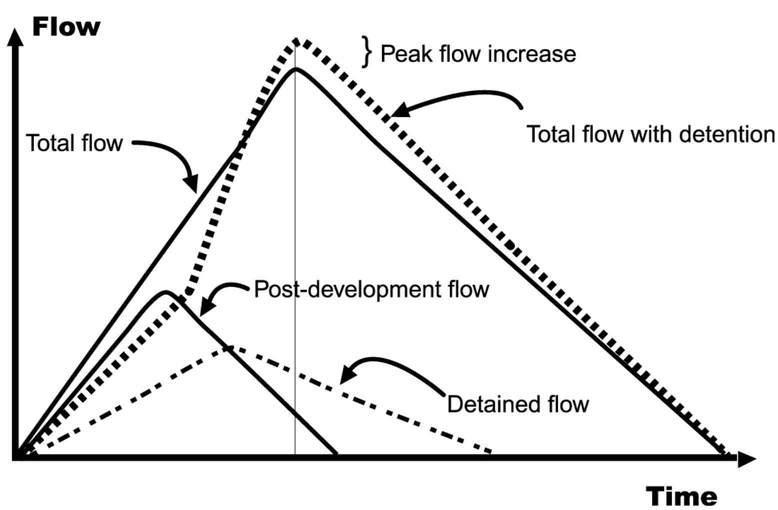
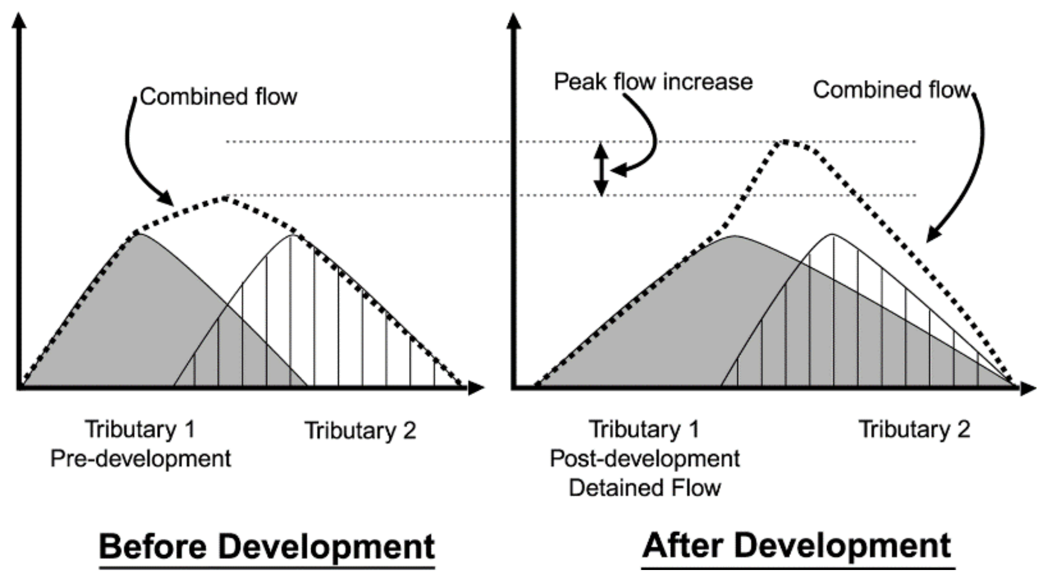


Figure 2.2-3 illustrates how even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased post-development runoff volume may combine with downstream tributaries to increase the downstream peak flows. The figure shows the pre-and post-development hydrographs from a development site (Tributary 1). The detention results in a post-development runoff hydrograph that meets the flood protection criteria (i.e., the site post-development peak flow is not greater than the pre-development peak flow). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than the pre-development combined flow. In this case,

the detention volume would have to have been increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

Figure 2.2-3 – Effect of Increased Post-Development Runoff Volume with Detention on a Downstream Hydrograph (2-18)



The downstream analysis should be performed by determining the existing conditions peak flow for the project site. Next, the zone influenced by the project development should be determined by identifying the point downstream at which the project site takes up approximately 9-11% of the total drainage area or where discharges from the project enter a stream or waterbody that is large enough for the site discharges to become negligible. For example, if the structural control (detention facility) drains 5 acres, the downstream analysis point should have a drainage area of about 50 acres or be the point where the discharge enters a large waterbody. Beyond this 10% area or large receiving waterbody, the detention discharge becomes relatively small and insignificant compared to the runoff from the total drainage area at that point. Selecting a downstream analysis point exactly at 10% may not be feasible, and engineering judgement may be required when defining the downstream analysis point. For example, if a point is identified where the project site takes up approximately 12% of the total drainage area, but the next tributary junction is significantly larger in area and would drop the project area to 1% of the total drainage area, choose the point that will more reasonably assess the impacts of the project. In this case that would be the 12% point. The typical steps in the application of the downstream hydrologic analysis are:

1. Determine the target peak flow for each project outfall for pre-development conditions.
2. Using aerial photography and a contour map or other topographic resources, determine the lower limit of the zone of influence (9-11% point or large receiving waterbody) and intermediate locations of concern such as downstream confluences, structures, and conveyances.
3. Obtain the basin characteristics (land use and soil type) for the zone of influence aerial photography, GIS datasets, NRCS web soil survey and other resources as necessary.

4. Calculate the downstream basin time of concentration.
5. Develop a hydrologic model using software (PondPack, TR-55, HydroCAD, Hydraflow), to determine the existing conditions peak flow rates and timing at each tributary junction beginning at the pond outlet and ending at the next tributary junction as close as possible to the 9-11% study point. For authorization to use other software in preparation of a Post Construction Stormwater Report, contact ODPS.
6. Run the model again using post-development conditions at the project site.

   Use the following criteria to determine if detention design is warranted.

   - Additional runoff would create (or increase) pressurized flow in downstream pipes for the design year storm.
   - Downstream local, private, or other infrastructure would no longer meet applicable design criteria.
   - Post-developed flows without detention would flood downstream property (one case of this would be if the unattenuated 25-year, 24-hour flows would exceed banks of the conveyance channel). Items to look for are elevations of buildings, basements, driveways, carports, etc.
   - Property downstream has history of flooding and analysis shows flooding would be increased with post-development flows without detention.
   - When the post development 25-year, 24-hour event flow without detention would be increased by more than 10% over the existing peak flow at the downstream point. This does not apply to very low flow increases of 3 cfs or less at the downstream point unless there is a specific risk to property.
   - Erosion and velocity impacts to downstream conveyances cannot be mitigated by armoring

   If none of the above conditions are met, detention design is most likely not warranted.
7. Design detention facilities such that the warranting criteria is resolved.
8. If the peak flow does increase, at the downstream hydrologic analysis point because of detention, one of the following must be completed:
   - Remove the detention pond (provided there is capacity to convey the flows harmlessly to the study point),
   - Redesign the detention storage and/or outlet control structure,
   - Receive approval from GDOT to waive detention requirements,
   - Provide infrastructure improvements downstream, or
   - Contact ODPS.

The need for detention facilities should be determined on a case-by-case basis and their use may not be required on certain projects. However, it is the designer's responsibility to provide all necessary supporting documentation for a detention analysis, per outfall, as to whether detention is necessary to prevent downstream impacts. A detention assessment may include numerous factors such as, but not limited to:

- Increase in peak flow rates
- Downstream conveyance capacity
- Environmental impacts

- Downstream detention facilities

When detention facilities are required, the supporting documentation provided by the designer will include the following information:

- Drainage area maps with topography and aerial photography showing existing and proposed drainage basins and flow paths
- Field notes, photographs, and any correspondence with local residents or other contacts
- FEMA or local flood maps (if available)
- Hydrologic & Hydraulic calculations (basin characteristics, routing reports, stage/storage/discharge, peak discharge)

Post-construction BMPs capable of providing detention will be designed in accordance with the BMP design requirements listed in section 2.6 of this manual. Additional information on detention design can be found in section 2.7 of this manual.

2.2.4 Water Balance Calculations [(2-18)]

Water balance calculations should be completed for post-construction stormwater BMPs that are designed to have a permanent pool of water. The calculations help determine if a drainage area is large enough, or has the appropriate characteristics, to support a permanent pool of water during average or extreme conditions. A simplified water balance procedure is described in the sections below.

2.2.4.1 Basic Equations

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

$$\Delta V = \Sigma I - \Sigma O$$

(2.2-1)

Where: Δ = "Change in"

V = Permanent pool volume

Σ = "Sum of"

I = Inflows

O = Outflows

The inflows consist of rainfall, runoff, and baseflow into the BMP. The outflows consist of infiltration, evaporation, evapotranspiration, and surface overflow out of the BMP. Therefore, Equation 2.2-1 can be expressed as follows:

$$\Delta V = \mathrm{P} + \mathrm{Ro} + \mathrm{Bf} - \mathrm{I} - \mathrm{E} - \mathrm{Et} - \mathrm{Of}$$

(2.2-2)

Where: P = Precipitation (ft)

Ro = Runoff (ac-ft)

Bf = Baseflow (ac-ft)

I = Infiltration (ft)

E = Evaporation (ft)

Et = Evapotranspiration (ft)

Of = Overflow (ac-ft)

Rainfall (P) ‐ Rainfall values can be obtained from NOAA Atlas 14. Monthly values are commonly used for calculations of values over a season. The rainfall used in this equation is the direct amount that falls on the permanent pool surface for the specified time period. When multiplied by the permanent pool surface area (in acres) it becomes acre-feet of volume.

Runoff (Ro) ‐ Runoff is equivalent to the rainfall for the period times the “efficiency” of the watershed, which is equal to the ratio of runoff to rainfall. In lieu of gage information, Runoff can be estimated one of several ways. One method has been proposed that uses the volumetric runoff coefficient (R_v), which gives a ratio of runoff to rainfall volume for a particular storm. If it can be assumed that the average storm that produces runoff has a similar ratio, then the R_v value can serve as the ratio of rainfall to runoff.

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

(2.2-3)

Where: I = Percent of impervious cover as a whole number (e.g., 80 for 80% rather than 0.8)

Not all storms produce runoff in an urban setting. Typical initial losses (often called “initial abstractions”) are normally taken between 0.1 and 0.2 inches. When compared to the rainfall records in Georgia, this is equivalent of about a 10% runoff volume loss. Thus, a factor of 0.9 should be applied to the calculated R_v value to account for storms that produce no runoff. Equation 2.2-4 reflects this approach.

$$Q = 0.9PR_v$$

(2.2-4)

Where: P = Precipitation (in)

Q = Runoff depth (in)

Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the BMP.

$$Ro = \frac{QA}{12}$$

(2.2-5)

Where: Ro = Runoff volume (acre-feet)

Q = Runoff depth (in)

A = Total drainage area minus pond area (ac)

Baseflow (Bf) - Most stormwater ponds and wetlands have little, if any, baseflow, as they are rarely placed in line with perennial streams due to environmental regulations. If so placed, baseflow must be estimated from observation or through theoretical estimates. Methods of estimation and baseflow separation can be found in most hydrology textbooks. Detention ponds located in coastal areas, however, often have groundwater baseflow during the wet season. For this situation, the analysis should incorporate estimated seasonal high groundwater level measurements from the project geotechnical investigation.

Infiltration (I) - Determination of the volume estimated to leave the facility by infiltration is complex and depends on many factors including soil type, water table depth, presence and location of rocklayers, surface disturbance and the presence or absence of a pond liner. The infiltration rate is governed by the Darcy equation as:

$$I = Ak_h G_h \tag{2.2-6}$$

Where: I = Infiltration (ac-ft/day)

A = Cross sectional area through which the water infiltrates (ac)

For the purposes of this analysis, use ponding area at the permanent pool

k_h = Saturated hydraulic conductivity or infiltration rate (ft/day)

G_h = Hydraulic gradient = pressure head/distance

G_h can be set equal to 1.0 for pond bottoms and 0.5 for pond sides steeper than about 4:1. The hydraulic conductivity values in Table 2.2-1 or other published resource can be used for planning level estimates including a water balance analysis. Refer to section 2.6 and appendix B for more information on when infiltration testing is required.

Table 2.2-1 Saturated Hydraulic Conductivity (2-11)

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

Table 2.2-1 Saturated Hydraulic Conductivity (2-11)		
Material	Hydraulic Conductivity	
	in/hr	ft/day
Clay loam	0.09	0.18
Silty clay loam	0.06	0.12
Sandy clay	0.05	0.1
Silty clay	0.04	0.08
Clay	0.02	0.04

Evaporation (E) ‐ Evaporation rates from an open water surface are dependent on differences in vapor pressure, which depend on temperature, wind, atmospheric pressure, water purity, and shape and depth of the pond. Most hydrology textbooks contain a number of methods for estimating and/or measuring evaporation. One common method is the pan evaporation method, though there are only two pan evaporation sites active in Georgia (Lake Allatoona and Griffin). A pan coefficient of 0.7 is commonly used to convert the higher pan value to the lower lake values.

Table 2.2-2 gives pan evaporation rate distributions for a typical 12-month period based on pan evaporation information from five stations in and around Georgia (including the two mentioned previously). Figure 2.2-4 depicts a map of annual free water surface (FWS) evaporation averages for Georgia based on a National Oceanic and Atmospheric Administration (NOAA) assessment completed in 1982. FWS evaporation differs from lake evaporation for larger and deeper lakes, but can be used as an estimate for the type of structural stormwater ponds and wetlands being designed in Georgia. Total annual values can be estimated from this map and distributed according to Figure 2.2-4.

Evapotranspiration (Et) ‐ Evapotranspiration consists of the combination of evaporation and transpiration by plants. The estimation of Et for crops in Georgia is well documented and has become standard practice. Estimates can be obtained from hydrology textbooks or from the NOAA website. However, there is little documented information related to evapotranspiration estimating methods for wetland plants, particularly in Georgia. Evapotranspiration rates are likely insignificant unless emergent vegetation covers a significant portion of the open water surface. In that case, the designer should compare estimates of lake evaporation with crop-based Et estimates and decide which value is most appropriate.

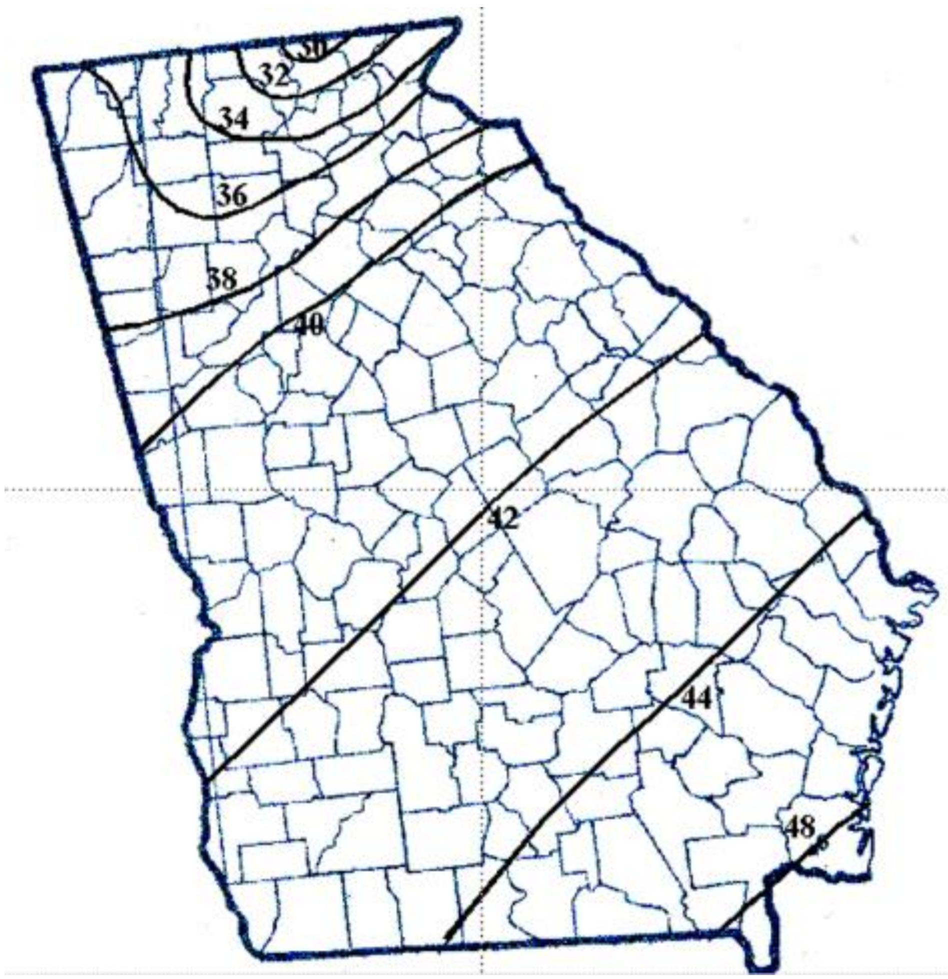
Overflow (Of) ‐ In the water balance calculations, overflow from the facility is either not considered at all, since the concern is for average values of precipitation, or is considered lost for all volumes above the maximum pond storage. When using long-term simulations of rainfall-runoff, large storms play an important part in pond design.

See the wet detention pond example water balance calculation in section 2.6.9.

Table 2.2-2 Evaporation Monthly Distribution (2-11)

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
3.2%	4.4%	7.4%	10.3%	12.3%	12.9%	13.4%	11.8%	9.3%	7.0%	4.7%	3.2%

Figure 2.2-4 – Average Annual Free Water Surface Evaporation (in inches)
Reference: NOAA, 1982

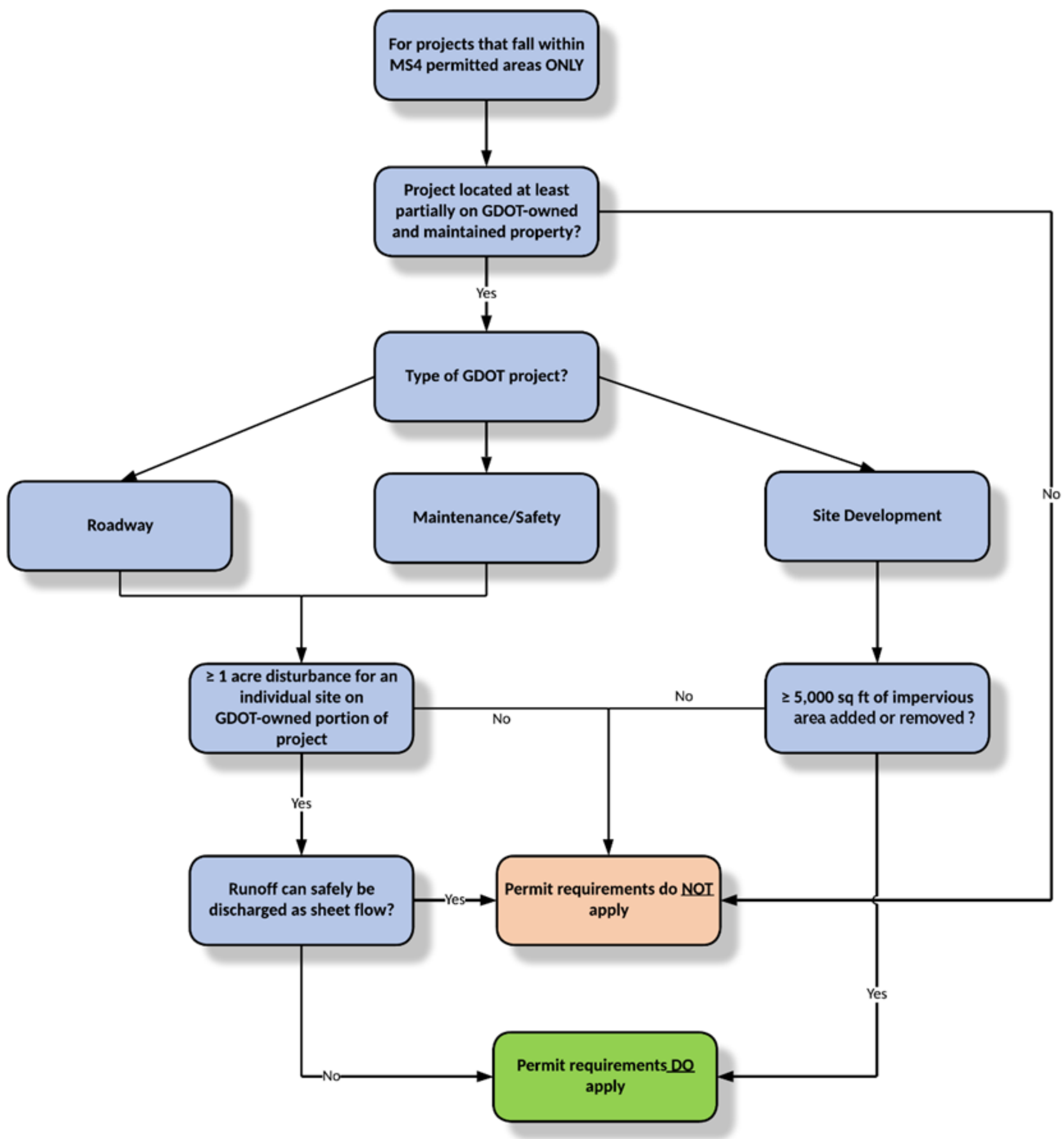


2.3 Project Applicability

Since January 3, 2012, GDOT's stormwater discharges have been regulated by Georgia EPD through GDOT's MS4 NPDES permit (permit number GAR041000), most recently renewed on January 3, 2022. The MS4 permit introduced additional stormwater requirements that apply to GDOT including implementation of post-construction stormwater practices to address water quality concerns and permit requirements.

The flowchart provided in Figure 2.3-1 is intended to aid in determining whether MS4 requirements apply to a project. For projects where MS4 requirements apply, section 2.4 summarizes the post-construction stormwater management requirements.

Figure 2.3-1 - MS4 applicability flowchart



GDOT has a three-tiered process to determine when post-construction stormwater practices are required for MS4 permit compliance.

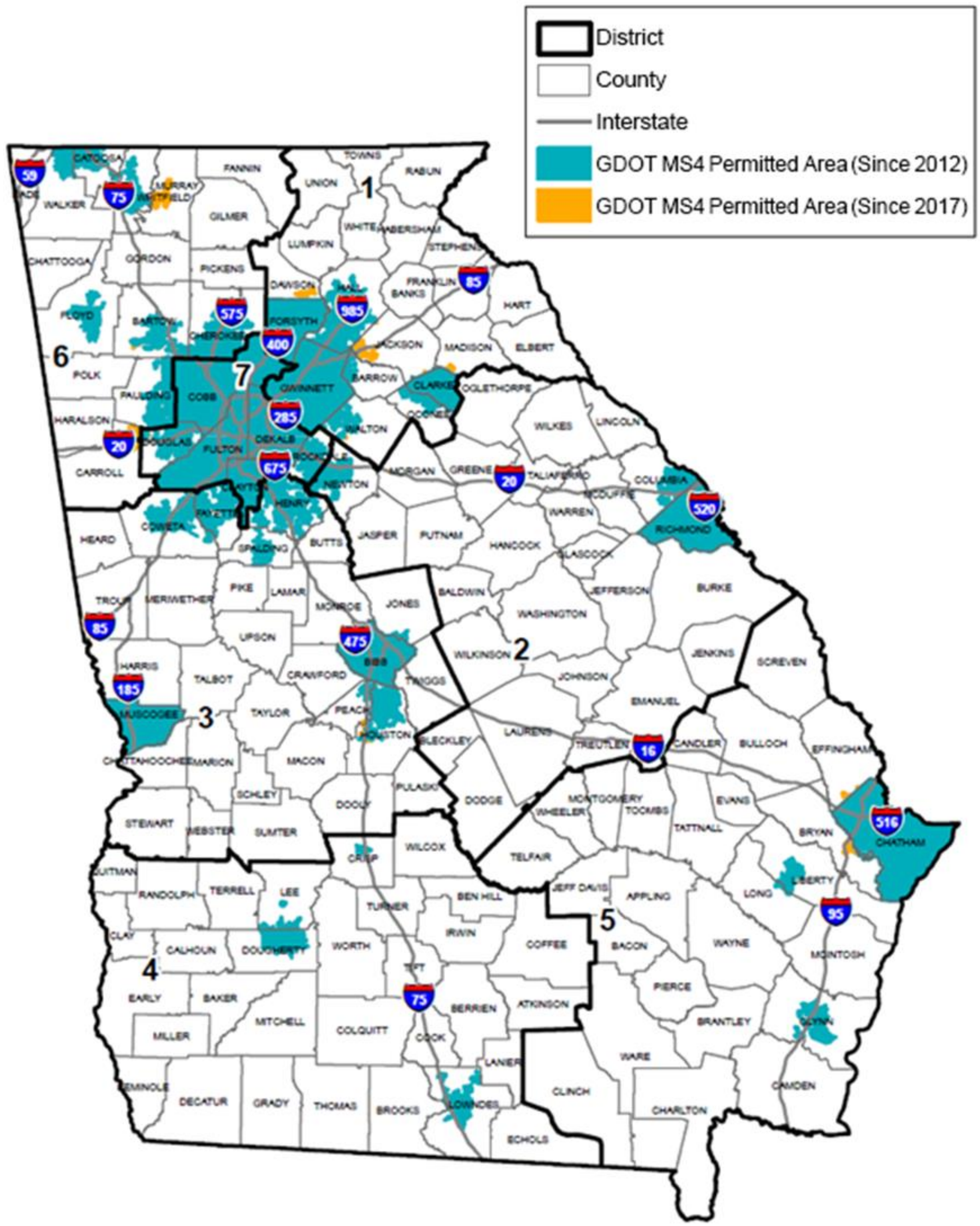
2.3.1 Project Level Exclusions

If a Project Level Exclusion applies, the entire project is exempt from complying with MS4-related post-construction stormwater requirements. Project Level Exclusions are defined below:

1. Roadways that are not owned or operated (maintained) by GDOT may not require post-construction BMPs. Coordinate with the appropriate local government or entity.
2. The project location is not within an MS4 area.
3. Maintenance and safety improvement projects such as resurfacing, maintenance projects that do not add impervious surface area, driveway access paving, shoulder paving and building, fiber optic line installation, sign addition, safety barrier installation, multi-use projects used solely for recreational purposes and separate from transportation projects (e.g. bike lanes on roads), and sound barrier installation.
4. Projects that have their environmental documents approved or right-of-way plans submitted for approval on or before June 30th, 2012.
5. Road projects that disturb less than 1 acre
6. Site development/redevelopment projects that create, add or replace less than 5,000 ft^2 of impervious area.
7. Projects in MS4 areas added to GDOT's 2017 MS4 permit with concept approval (start of preliminary engineering) before January 3, 2018.
8. Projects that discharged to a Combined Sewer Overflow area.

MS4 permitted areas include the counties and cities shown in Figure 2.3-2. A list of these cities and counties is provided in appendix A.

Figure 2.3-2 – GDOT MS4 permitted area



New development and redevelopment projects within MS4 areas must adhere to MS4 permit requirements if they meet one of the following descriptions:

- Linear roadway projects that disturb an area of 1 acre or more; or
- Site development and redevelopment projects that create, add, or replace 5,000 square feet or more of new impervious area

A land disturbance is defined as "any land change which may result in soil erosion from water or wind and the movement of sediments into state water or onto lands within the state, including, but not limited to, clearing, dredging, grading, excavating, transporting, and filling of land." (2-19)

Impervious area is defined as surface cover that has been affected by infrastructure or development activities such that infiltration of water into the underlying soil is not permitted. Typical examples include paved roads (except those paved with permeable pavement), paved parking, compacted aggregate base course surfaces, and rooftops.

The 2017 MS4 permit requirements apply to projects located in a new MS4 area (i.e. not listed in GDOT's 2012 MS4 permit) with concept approval on or after January 3, 2018. For projects in MS4 areas that were covered under the 2012 MS4 permit, there is a one-year transition phase to the 2017 permit requirements. The 2017 permit requirements apply for projects that receive Environmental Approval or submit right-of-way plans for GDOT review and approval or Design-Build and P3 projects that receive Environmental Approval or concept approval on or after January 3, 2018.

2.3.2 Outfall Level Exclusions

If a project does not qualify for a Project Level Exclusion, specific outfall drainage areas within a project should be evaluated for applicability of an Outfall Level Exclusion (specific only to an area of the project). Outfall Level Exclusions are defined below:

1. Where installation of post-construction BMPs on the project would require a roadway alignment change solely to allow for BMPs. This exclusion applies only to existing roadway alignment changes that would create a safety concern. A written explanation of the safety concern(s) must be included with the *MS4 Post-Construction Stormwater Report* when claiming this exclusion.
2. Where the installation of post-construction BMPs would require the re-alignment and/or piping of a stream.
3. Where installation of post-construction BMPs on a project would impact existing vegetated stream buffers or wetlands solely for the purposes of installing BMPs. See state stream buffer requirements for additional information.
4. Where stormwater discharges from the project site are designed to exit the right-of-way or enter a state water within the right-of-way as sheet flow. Sheet flow should be designed in a manner to ensure that the flow will not cause erosion or flooding. The designer should determine if this is possible by visiting the site prior to design and is required to provide a written explanation with supporting evidence when claiming this exclusion.

   GDOT approval is required to claim this exclusion for instances where stormwater discharges leave the right-of-way as sheet flow but channelize prior to discharging to a receiving stream or waterbody. If a ditch is visible in the cross-section, it is likely that this outfall level exclusion is not applicable.
5. As stated in section 4.2.5.1(a) of the GDOT MS4 permit, "Stormwater runoff that must be treated does not apply to flows that originate outside of GDOT's right-of-way or diverted flows from undisturbed areas." If feasible, direct all offsite stormwater around the project site to the cross drain or stream such that it does not combine with stormwater from the project's impervious surfaces or conveyance systems. This redirection allows the BMPs to only treat

or detain the stormwater that originates from GDOT's site, and stormwater that originates off-site to pass through the right-of-way unimpeded.

6. As stated in section 4.2.5.1(a) of the GDOT MS4 permit, for outfalls along linear roadway projects whereby the net impervious surface area within that outfall's drainage area has been reduced or remains the same as pre-developed conditions, post-construction stormwater requirements will not apply. Special consideration from GDOT may be given to those projects with a minimal increase in impervious area. In such cases, the designer will be required to provide supporting calculations showing that the increase in stormwater runoff and/or volume required to be treated for water quality is negligible with respect to the drainage area in question, and must also be agreed upon by GDOT. As a general rule increases over one tenth of an acre in impervious surface per basin are not considered negligible.

Note: Outfall Level Exclusions apply separately to each of the four major post-construction stormwater management requirements, which are discussed in detail in section 2.2.2.2.

2.3.3 Infeasibilities

GDOT's MS4 permit requires treatment of stormwater runoff from GDOT property and right-of-way to the maximum extent practicable. Therefore, the requirements and minimum standards described in section 2.4 should be met to the maximum extent practicable. In some situations, site constraints and other factors make implementation of post-construction stormwater BMPs infeasible. The following criteria are used to define these situations (note: criteria should be applied to each outfall drainage basin individually):

1. The BMP costs equal or exceed 10% of the total project costs. If the BMP discharges within one linear mile upstream of and within the same watershed as a designated trout stream then the threshold for BMP cost infeasibility is 30% of the total project costs. Project costs should include:
   - right-of-way acquisition
   - roadway construction (not including Intelligent Transportation Systems (ITS) or toll related expenses)
   - utility relocation
   - mitigation costs

   BMP costs should only be compared to the portion of the project within the BMP's associated outfall drainage basin and should include:
   - additional right-of-way requirements
   - BMP construction and all other related design elements
2. Implementation of BMPs will cause 90 days or greater of delays to the project. This criteria does not apply to projects that discharge within one linear mile upstream of and within the same watershed as a designated trout stream.
3. Implementation of BMPs will cause loss of habitat for endangered or threatened species.
4. Implementation of BMPs will cause significant damage to a cultural or community resource such as an historical site, archeological site, cemetery, a park, wildlife refuge, nature trail, or school facility.

5. Implementation of BMPs would result in the displacement of a residence or business.
6. Implementation of BMPs would result in violation of state or federal law or regulation.
7. Site limitations including: shallow bedrock, contaminated soils, high groundwater, utilities, or underground facilities if avoidance or relocation is infeasible (cost of the relocation equals or exceeds the cost of the BMP).
8. Soil infiltration capacity is limited, where the soil hydraulic conductivity (K) is less than 0.5 in/hr ($3.5x10^{-4}$ cm/second).
9. Site is too small to infiltrate a significant volume.
10. Site does not allow for gravity flow to the appropriate BMP.

If it is determined infeasible to meet all of the minimum standards presented in this section based on the above criteria, the designer should strive to meet as many requirements as possible.

Consideration should be given for locating BMPs anywhere within the limits of the environmental study. Where there is a risk to life or property, the infeasibility criteria should be disregarded in favor of a prudent design.

2.3.4 MS4 Post-Construction Stormwater Management Documentation

MS4 post-construction stormwater requirements shall be considered during Concept Development. The MS4 Concept Report Summary must be submitted with the Concept Report. If it is a possibility that a BMP will be installed on a local route discuss with the local government during the Concept Team Meeting and document acceptance of maintenance responsibility.

The MS4 Post-Construction Stormwater Report, found on the GDOT Manuals & Guides website, is required at PFPR and FFPR for ALL projects located in an MS4 area and should be used to document the use or exclusion of post-construction BMPs. This document serves as a design aid and documentation for post-construction stormwater controls on GDOT projects. An *MS4 Post-Construction Stormwater Report Addendum* may be required if there are significant changes to the project after final GDOT approval of the Report is received. Refer to the *MS4 Post-Construction Stormwater Report* template and help files for detailed information on what is required in the Report. Refer to the Plan Development Process Manual and Flowcharts for detailed information on what is required at each project milestone.

2.4 MS4 Post-Construction Stormwater Management Minimum Standards

There are four major post-construction stormwater management requirements (referred to as "minimum standards" in the permit) that apply to GDOT projects meeting the criteria outlined in section 2.3:

- Stormwater runoff quality / reduction (retaining the runoff reduction volume, RR_v, and/or treating the water quality volume, WQ_v)
- Stream channel / aquatic resource protection (CP_v)
- Overbank flood protection (Q_{p25})
- Extreme flood protection (Q_f)

In cases where projects impact existing roadways and facilities, only the new proposed areas should be considered with respect to water quality treatment. The entire drainage area should be considered with respect to stormwater runoff quantity control measures. Existing or pre-developed conditions used in the determination of necessary stormwater runoff quantity control measures are defined as the conditions of the site immediately prior to the implementation of the proposed project. Section 2.5 of this chapter provides BMP selection guidance to aid in meeting the minimum standards.

The requirements associated with stream channel / aquatic resource protection, overbank flood protection, and extreme flood protection are waived for discharge points draining directly to channels or water bodies with drainage areas larger than 5 square miles. Runoff from GDOT right-of-way is not expected to significantly impact surface waters of this size. However, if discharging to a channel with a drainage area less than 5 square miles, the designer must conduct a downstream analysis (as described in section 2.2.3) to verify that proposed condition flows do not exceed existing condition flows causing an impact to life or property.

2.4.1 Stormwater Runoff Quality / Reduction

Small, frequent storms generate the majority of stormwater runoff. In addition, a significant portion of stormwater pollutants generated during large, less-frequent storms are discharged with the initial surface runoff of a rain event, known as the "first flush". For these reasons, GDOT is required to reduce pollutants in runoff from small storms by retaining runoff onsite (runoff reduction) and/or treating runoff before discharging it offsite.

Runoff reduction practices remove runoff, and therefore pollutants contained in the runoff, through a variety of processes including infiltration (most common and applicable to GDOT projects) evaporation, transpiration, and rainwater harvesting and reuse. Runoff reduction practices improve water quality and reduce the water quantity that must be managed for larger storm events. Designers shall first consider infiltration BMPs, where conditions permit. Preference should be given to BMPs that achieve 100 percent infiltration before others are considered.

Runoff reduction is not practicable for all sites and conditions. If the runoff reduction standard cannot be met, the remaining runoff must be treated. Georgia's water quality standard is the 85th percentile storm (equivalent to 1.2 inches of rainfall). Therefore, where MS4 requirements apply and runoff from the one-inch rainfall even cannot be retained onsite, BMPs must be sized to treat the remaining runoff from the first 1.2-inch rainfall event. See section 2.4.1.2 for additional information on calculating the remaining runoff that must be treated.

In addition to hydrologic benchmarks, requirements include a TSS reduction goal of 80%. Sediment causes aquatic habitat degradation and is a widespread cause of water quality impairment throughout Georgia. In addition, other stormwater pollutants are transported by TSS or are removed in amounts proportional to TSS. [(2-15)] This 80% reduction requirement is considered to be met if a BMP or system of BMPs, with a pollutant removal rate equal to or greater than 80% TSS, is sized to capture and treat the required water quality volume. If a runoff reduction practice is used but cannot remove the entire first inch of rainfall, the remaining volume (equal to the 1.2-inch rainfall minus the removed volume) must be treated to the 80% TSS removal standard.

2.4.1.1 Runoff Reduction Volume

The volume of runoff resulting from the first one inch of rainfall is known as the runoff reduction volume (RR_v) and is calculated for the new, or net new, impervious area using Equation 2.4-1:

$$RR_v = \frac{1\ in \times (R_v) \times A \times 43560\,\frac{ft^2}{acre}}{12\,\frac{in}{ft}} \tag{2.4-1}$$

Where: RR_v = runoff reduction volume (ft^3)

R_v = volumetric runoff coefficient, 0.05+0.009(I) (dimensionless)

I = percent imperviousness of onsite area (i.e., for 80% impervious area, use 80, not 0.8)

A = onsite drainage area of the post-condition basin (acres)

Since GDOT is only required to consider net new impervious area (proposed impervious area minus existing impervious area) in runoff reduction and water quality calculations, new construction projects (projects with no existing GDOT impervious area) and improvement projects (projects with existing GDOT impervious area such as road widenings and intersection improvements) require slightly different approaches for calculating the volumetric runoff coefficient. Improvement projects require that a net volumetric runoff coefficient be calculated. Example calculations for each scenario are provided below.

New Construction Example: 1.5-acre drainage area that is 80% impervious:

$$R_v = 0.05 + 0.009(80) = 0.77$$

$$RR_v = \frac{1 \times (0.77) \times 1.5 \times 43560}{12}$$

$RR_v = 4{,}193\ ft^3$

For new construction projects, the runoff reduction volume formula can be simplified to the following:

$$RR_v = \frac{1\ in \times (0.05A + 0.9A_{IMP}) \times\ 43560\,\frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

(2.4-2)

Where: RR_v = runoff reduction volume (ft^3)

A = onsite drainage area of the post-condition basin (acres)

A_{IMP} = impervious surface area in the post-condition basin (acres)

Improvement Project Example: 1.2-acre drainage area with 0.9 acres of existing impervious area. The proposed post-development drainage area is 1.5 acres with 1.2 acres of impervious area (Note: any use of the variable "A" refers to the post-basin size):

$$I_{(Pre)} = \frac{0.9}{1.5} = 60\%$$

$$I_{(Post)} = \frac{1.2}{1.5} = 80\%$$

$$R_{v(post)} = 0.05 + 0.009(80) = 0.77$$

$$R_{v(pre)} = 0.05 + 0.009(60) = 0.59$$

$$R_{v(post)} - R_{v(pre)} = 0.77 - 0.59 = 0.18$$

$$RR_v = \frac{1 \times (0.18) \times 1.5 \times 43560}{12}$$

RR_v = 980 ft^3

For construction improvement projects, the runoff reduction volume formula can be simplified to the following:

$$RR_v = 0.075 \times A_{NEW\ IMP} \times 43560\,\frac{ft^2}{acre}$$

(2.4-3)

Where: RR_v = runoff reduction volume (ft^3)

$A_{NEW\ IMP}$ = net increase in impervious area in the post-condition basin (acres)

Where: VP_{min} = minimum volume of the BMP (ft^3)

RR_v = runoff reduction volume (ft^3)

RR% = runoff reduction rate for the BMP (obtained from Table 2.5-1)

If the runoff retained onsite in a drainage area is less than the calculated RR_v, the water quality standard must be met for the remaining runoff from the 1.2-inch rainfall event.

2.4.1.2 Water Quality Volume

The volume of runoff resulting from the first 1.2 inches of rainfall is known as the water quality volume (WQ_v) and is calculated for the new, or net new, impervious area as is the case for the runoff reduction volume calculation. The water quality volume formula is:

$$WQ_v = \frac{1.2\ in \times (R_v) \times A \times 43560 \frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

(2.4-4)

Where: WQ_v = water quality volume (ft^3)

R_v = volumetric runoff coefficient, 0.05+0.009(I) (dimensionless)

I = percent imperviousness of onsite area (i.e., for 80% impervious area, use 80, not 0.8)

A = onsite drainage area of the post-condition basin (acres)

The process for calculating the water quality volume for new construction projects and for projects with additional proposed impervious area is identical to the runoff reduction calculations, with the exception that the rainfall value is 1.2 inches.

For new construction projects, the water quality volume formula can be simplified to the following:

$$WQ_v = \frac{1.2\ in \times (0.05A + 0.9A_{IMP}) \times\ 43560 \frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

(2.4-5)

Where: WQ_v = water quality volume (ft^3)

A = onsite drainage area of the post-condition basin (acres)

A_{IMP} = impervious surface area in the post-condition basin (acres)

For construction improvement projects, the water quality volume formula can be simplified to the following:

$$WQ_v = 0.09 \times A_{NEW\ IMP} \times 43560 \frac{ft^2}{acre}$$

(2.4-6)

Where: WQ_v = water quality volume (ft^3)

$A_{NEW\ IMP}$ = net increase in impervious area in the post-condition basin (acres)

For some designs, the drainage basin may substantially change from pre- to post-developed conditions. If this is the case, the post-developed drainage basin should be used in the water quality volume calculations. GDOT will review all drainage basins with proposed substantial changes.

The removal rates given for each BMP in section 2.6 may also be used to determine if the 80% TSS removal requirement has been met. If the TSS removal rate for a given BMP is less than 80%, BMPs may be installed in series (treatment train) to meet the requirement. Composite removal rates can be calculated by using the equation shown below:

$$Total\ TSS\ Removal = BMP1\ removal\ rate + [(remaining\ TSS)(BMP2\ removal\ rate)] + etc.$$

Common BMP treatment train options frequently used to meet the 80% TSS removal requirement include the following (BMPs are listed in order of upstream BMP to downstream BMP). BMPs are listed in order of upstream BMP to downstream BMP. The removal rate of the first BMP is added to the product of remaining TSS and the removal rate of the second BMP:

Filter Strip & Grass Channel:	60% + [(0.40)(50%)] = 80.0%
Grass Channel & Filter Strip:	50% + [(0.50)(60%)] = 80.0%
Dry Detention Basin & Grass Channel:	60% + [(0.40)(50%)] = 80.0%

As stated above, where MS4 requirements apply and runoff from the one-inch rainfall event cannot be retained onsite, BMPs must be sized to treat the remaining runoff from the first 1.2-inch rainfall event. The remaining runoff that must be treated can be calculated by subtracting the runoff reduction volume that was achieved in the basin from the target water quality volume. For example, assume a drainage basin has a target water quality volume of 5,000 cubic feet. A bioslope has 1,376 cubic feet of RRv credit. The runoff reduction achieved is 25% (obtained from Table 2.5-1) of 1,376 or 344 cubic feet. Therefore, the remaining volume that must be treated to remove 80% TSS is the target water quality volume of 5,000 cubic feet minus the runoff reduction volume achieved of 344 cubic feet which equals 4,656 cubic feet.

2.4.1.2.1 Calculating Water Quality Volume Peak Flow

Some BMPs, such as grass channels, enhanced swales, and bioslopes are designed to treat a given flowrate rather than volume. This flowrate is the peak discharge for the water quality storm and is referred to as the water quality volume peak flow, or Q_{wq}. In addition, BMPs are often designed in an offline configuration and use a flow bypass structure that allows flows from large storm events to bypass the system. Information regarding online and offline BMP applications can be found in section 2.5. Some flow bypass structures are sized based on flowrate. The Q_{wq} should typically be used for the sizing of these systems. Additional information on flow bypass structures can be found in section 2.8.2.

The following steps can be used to calculate Q_{wq}:

1. Calculate a CN (specific to the calculation of Q_{wq}) using Equation 2.4-7

$$CN = \frac{1{,}000}{10 + 5P + 10Q_{WV} - 10\sqrt{(Q_{WV}{}^2 + 1.25Q_{WV}P)}}$$

(2.4-7)

Where: Q_{WV} = water quality volume expressed in inches (use $1.2R_v$)

P = rainfall (inches) (use 1.2 inches)

2. The CN is used to determine I_a and subsequently, q_u. Note that guidance for determining q_u (and I_a and t_c) is shown as part of the description for calculating the channel protection volume in section 2.4.2. Use Equation 2.4-8 to calculate Q_{wq}.

$$Q_{wq} = q_u \times A \times Q_{WV}$$

(2.4-8)

Where: Q_{wq} = water quality volume peak flow (ft^3/s)

q_u = unit peak discharge ($ft^3/s/mi^2/inch$)

A = drainage area (mi^2)

Q_{WV} = water quality volume expressed in inches (use $1.2R_v$)

2.4.2 Stream Channel / Aquatic Resource Protection

Urbanization and development increase runoff volumes and velocities, potentially causing channel erosion and loss of aquatic habitat. In order to protect stream channels and aquatic resources, 24-hour extended detention should be provided for runoff from the 1-year, 24-hour storm, referred to as the channel protection volume (CP_V). Detention outlets are required to be protected with appropriate energy dissipation and velocity control measures as detailed in Chapters 6 and 7 of the Drainage Design Policy Manual. Also, applicable stream buffers should be preserved at the outlets. Note that CP_v control is not required where proposed discharges are less than 2.0 ft^3/s.

CP_v can be calculated using the NRCS TR-55 Method. [(2-34)] Methods presented in TR-55 and the associated WinTR-55 computer model can be used to calculate runoff volume and peak discharges and to develop hydrographs. A simplified peak discharge calculation method based on TR-55 is provided in Equation 2.4-18 in section 2.4.3.

In order to manually approximate channel protection volume, complete the following steps:

1. Calculate the direct runoff (Q) for the 1-year, 24-hour storm, (Equation 2.4-9)
2. Calculate the initial abstraction ratio, I_a/P (Equation 2.4-12)
3. Calculate the time of concentration
4. Determine the unit peak discharge, q_u (Figures 2.4-3, 2.4-4 and 2.4-5)
5. Determine the peak outflow to peak inflow discharge ratio, q_o/q (Figure 2.4-6)
6. Calculate the required storage volume (Equations 2.4-16 and 2.4-17)

Step 1: The SCS Curve Number Method uses Equation 2.4-9 to calculate direct runoff in inches (Q):

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

(2.4-9)

Where: Q = accumulated direct runoff (in)

P = accumulated rainfall (in)

S = potential maximum soil retention (in)

P is determined by using the National Oceanic and Atmospheric Administration (NOAA) Precipitation Frequency Data Server. (2-25) The NOAA data server can be found online by accessing the following website: http://hdsc.nws.noaa.gov/hdsc/pfds/index.html.

Using the interactive map and table for the location nearest the center point of the project site, identify the appropriate rainfall amount for the 1-year, 24-hour storm.

S can be expressed as a function of the SCS curve number, and is calculated using Equation 2.4-10:

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

(2.4-10)

Where: CN = SCS curve number (most drainage areas will require a composite CN)

A comprehensive list of curve numbers is provided in TR-55. A composite curve number should be calculated for multiple land uses using the following equation:

$$CN_{composite} = \frac{CN_1 A_1 + CN_2 A_2 + CN_3 A_3}{A_1 + A_2 + A_3}$$

(2.4-11)

Where: A = surface area

The curve numbers presented in TR-55 assume a prescribed amount of impervious area and can be adjusted for varying amounts of impervious area if needed. The curve number tables also assume that impervious areas are directly connected to the storm sewer system. Curve numbers can be adjusted for drainage areas where this is not the case. Refer to TR-55 for further guidance on adjusting the curve numbers to accommodate these scenarios.

Step 2: The initial abstraction ratio, I_a/P is determined by first calculating the initial abstraction using Equation 2.4-12. The initial abstraction (I_a) is the amount of water lost before runoff begins and includes water retained in surface depressions, water intercepted by vegetation, and evaporation.

$$I_a = 0.2 \times S$$

(2.4-12)

Where: I_a = initial abstraction (in)

The P value used in the initial abstraction ratio refers to the same P value used in Equation 2.4-9.

Step 3: The time of concentration, t_c is calculated using the Velocity Method. Alternatively, the Watershed Lag method may be used, see the NRCS National Engineering Handbook for reference. Further guidance for the Velocity Method can be found in the FHWA HDS-2. (2-7) In the Velocity Method time of concentration is found by summing the following three components of flow starting at the hydraulically most distant point in the drainage area:

1. Sheet flow
2. Shallow concentrated flow

3. Channel flow

Sheet flow is calculated using Equation 2.4-13:

$$T_o = \frac{\alpha}{P_2^{\ 0.5}} \left(\frac{nL}{\sqrt{S}}\right)^{0.8}$$

(2.4-13)

Where: T_o = sheet flow travel time (min)

n = Manning's roughness coefficient (dimensionless)

L = Length of sheet flow (ft) with a maximum of 100 ft

P_2 = 2-year, 24-hour rainfall depth (in)

S = Surface slope (ft/ft)

α = Unit conversion constant equal to 5.5 in SI units and 0.42 in CU units.

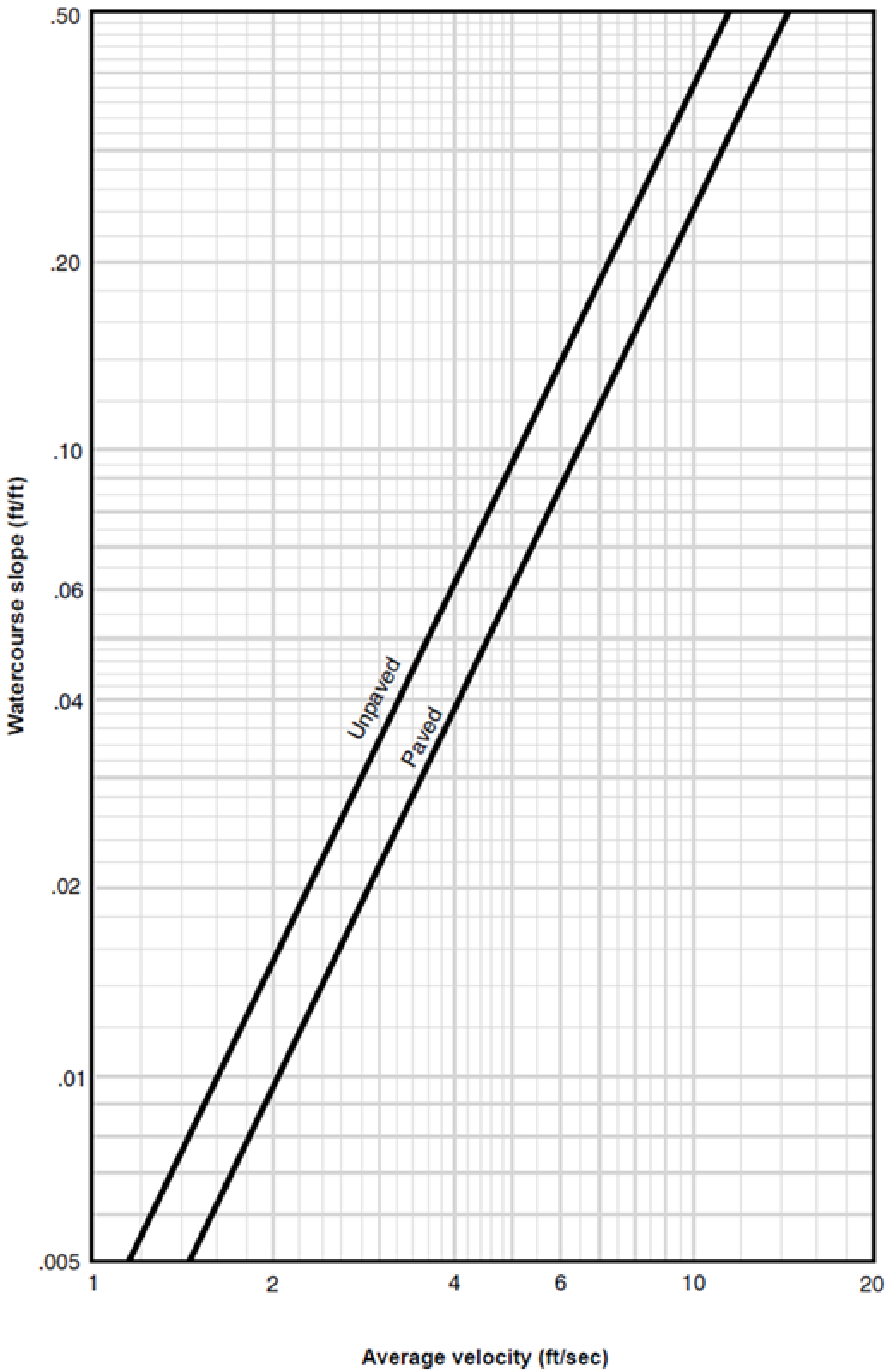
Since intensity depends on duration, the suggested solution procedure is to assume an initial value for the sheet flow travel time based on physical conditions. The corresponding intensity (i) is then obtained from the applicable intensity-duration-frequency relationship and the equation is solved. The computed T_o is compared to the assumed value for T_o and if they are not the same, the process is repeated until the assumed and computed values for T_o are the same. Note that the minimum time of concentration used for GDOT projects is 5 minutes.

Overland runoff or sheet flow typically collects into what is called shallow concentrated flow prior to flowing in a defined channel or constructed storm drainage facility. This type of flow should be treated separately from overland flow because velocities tend to be higher in these concentrated flow paths. Figure 2.4-1 defines shallow concentrated flow velocities as a function of slope. For water course slopes less than that plotted on Figure 2.4-1 (0.005), use the equations given in the figure to define velocity. It is not always apparent when overland flow changes to shallow concentrated flow and consequently, it is typical to assume a maximum overland flow length of 100 feet if shallow concentrated flow is not evident in the field. Given velocity, the travel time for shallow concentrated flow is computed as follows:

$$T_t = \frac{Flow\ length}{Velocity}$$

(2.4-14)

Figure 2.4-1 - TR-55 shallow concentrated flow nomograph



Following shallow concentrated flow, storm drainage flows into natural drainage channels or constructed drainage facilities. This can include flow into swales, ditches, stream channels, or closed conduit drainage facilities. If the flow concentrates in an open channel, the velocity may be estimated from the Manning's equation. For a discussion about Manning's equation refer to Chapter 6 of FHWA's HEC-22 manual and other FHWA publications.

The time of concentration is the sum of overland flow time, shallow concentrated flow time, and concentrated flow time:

$$t_c = T_o + T_{t(shallow\ concentrated)} + T_{t(concentrated)}$$

(2.4-15)

As previously noted, the minimum t_c that should be used on GDOT projects is 5 minutes.

Step 4: Use the calculated t_c (or the minimum t_c of 5 minutes) and the I_a/P value to compute the unit peak discharge (q_u) from Figures 2.4-3 to 2.4-5 below. If I_a/P falls outside of the ranges provided in the figures, either the limiting values or another peak discharge calculation method should be used.

These figures are specific to an SCS rainfall distribution, which for Georgia is either a Type II or Type III time distribution. They are also specific to peaking factors. Peaking factors may vary from 600 in mountainous regions, to 300 for flat (coastal) areas. A peaking factor of 484 represents rolling hills and is representative of the majority of north Georgia. A peaking factor of 300 should be used for South Georgia, which is characterized by <2% slopes and significant storage (standing water) during storm events. Refer to Figure 2.4.-2 for approximate NRCS TR-55 rainfall distribution and peaking factor geographic boundaries.

Figure 2.4-2 – Approximate rainfall distribution and peaking factor geographic boundaries

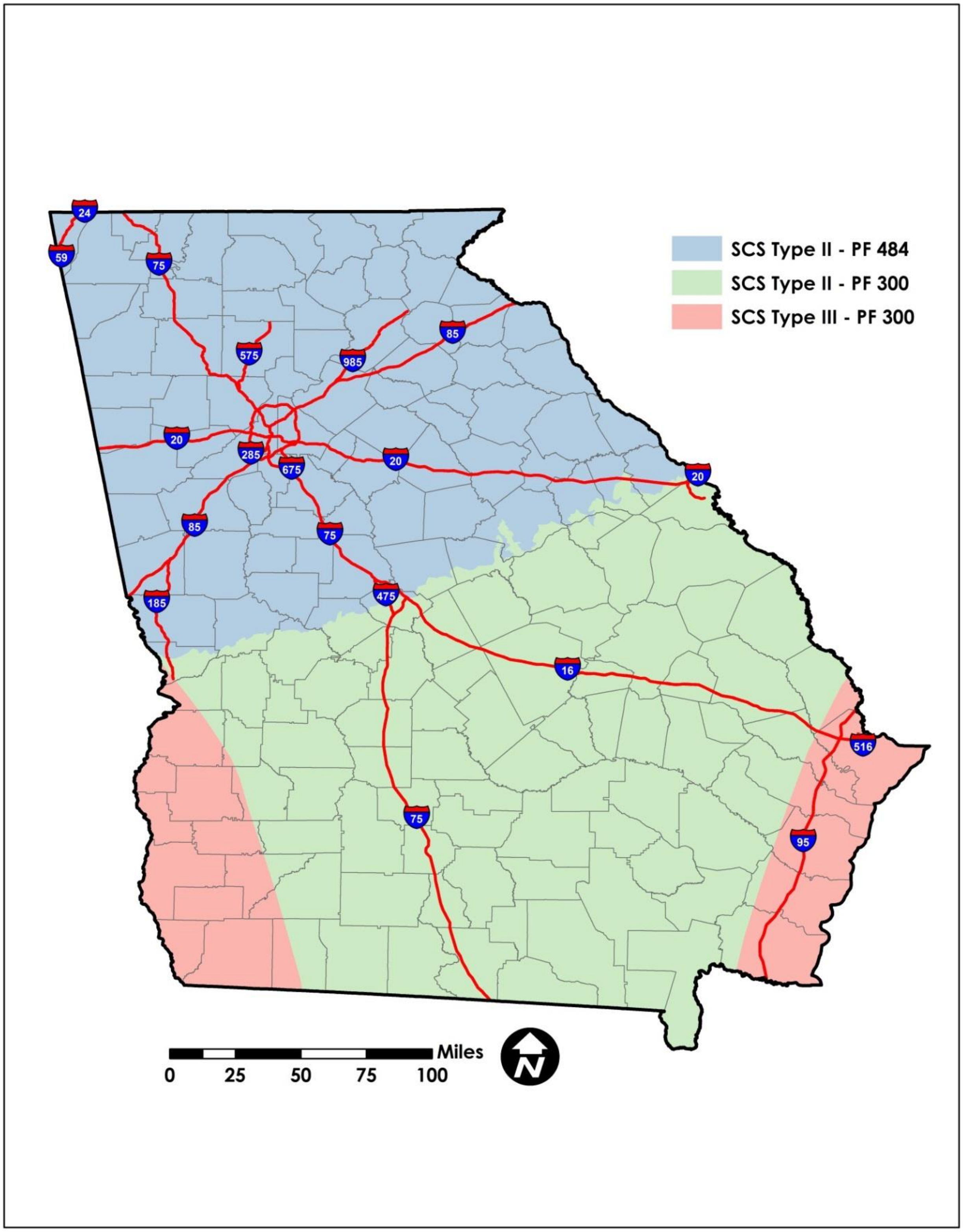


Figure 2.4-3 - Unit peak discharge (q_u) for SCS Type II rainfall distribution and 484 peaking factor (2-34)

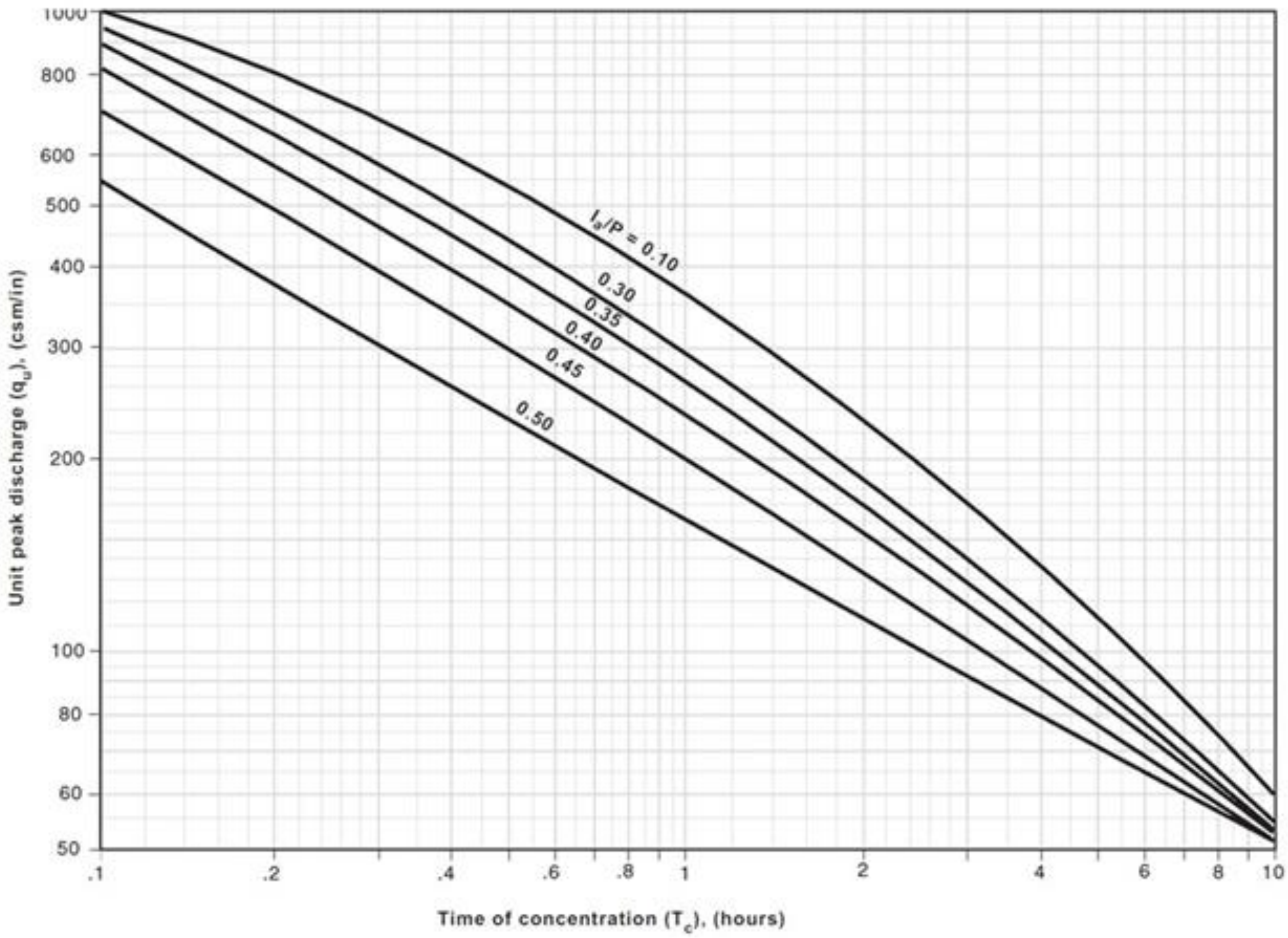


Figure 2.4-4 - Unit peak discharge (q_u) for SCS Type II rainfall distribution and 300 peaking factor

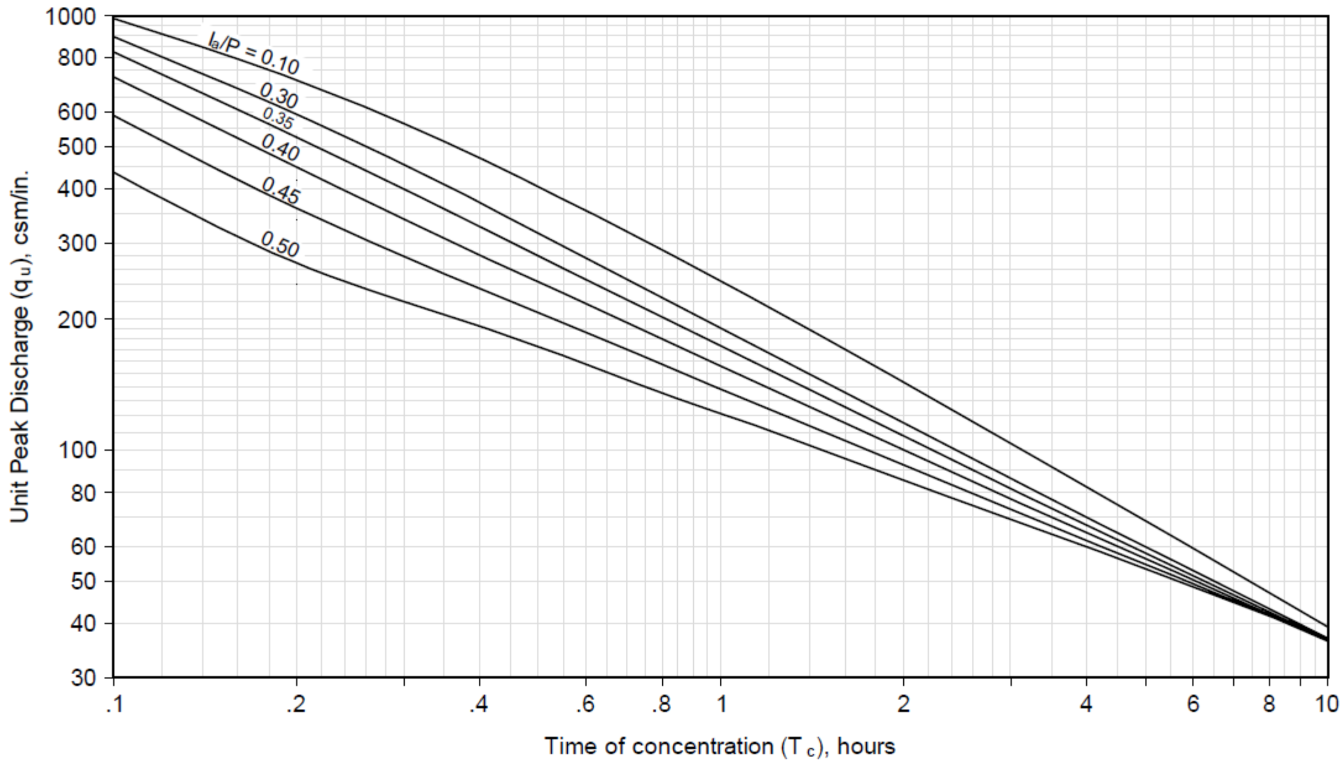
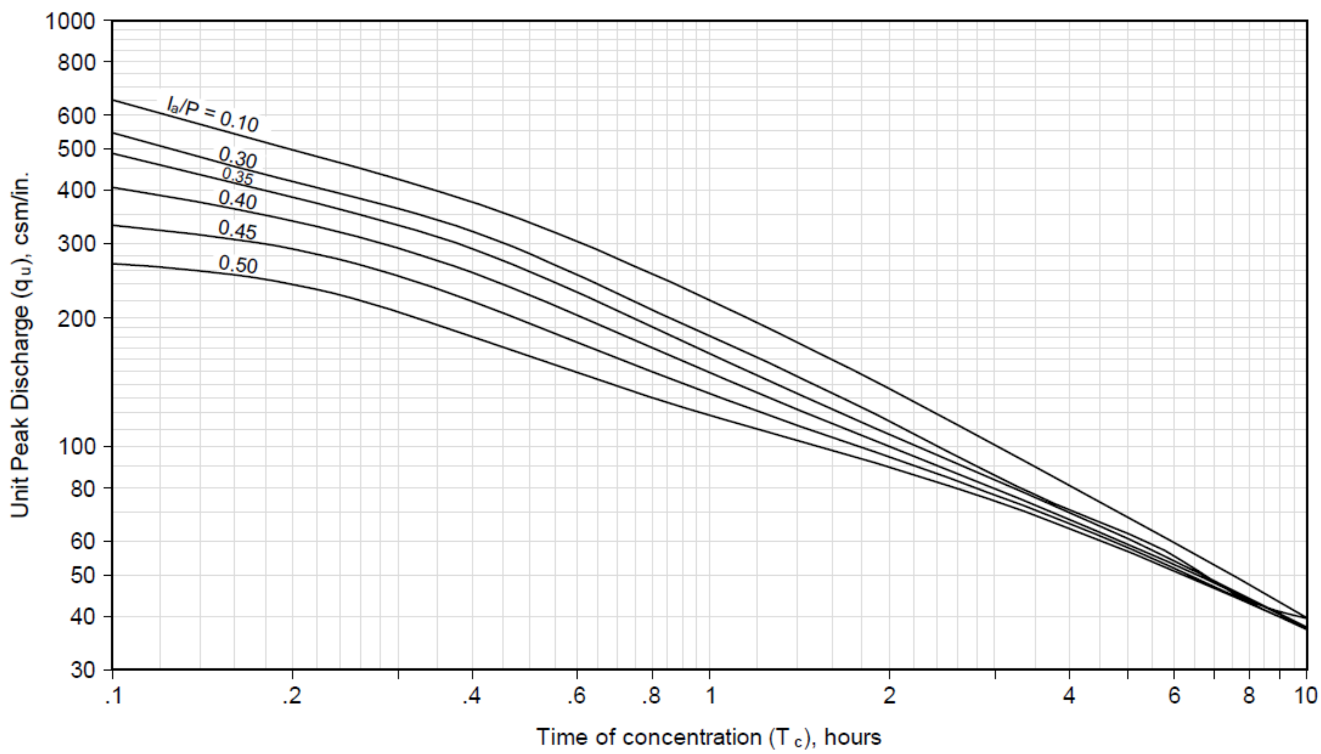
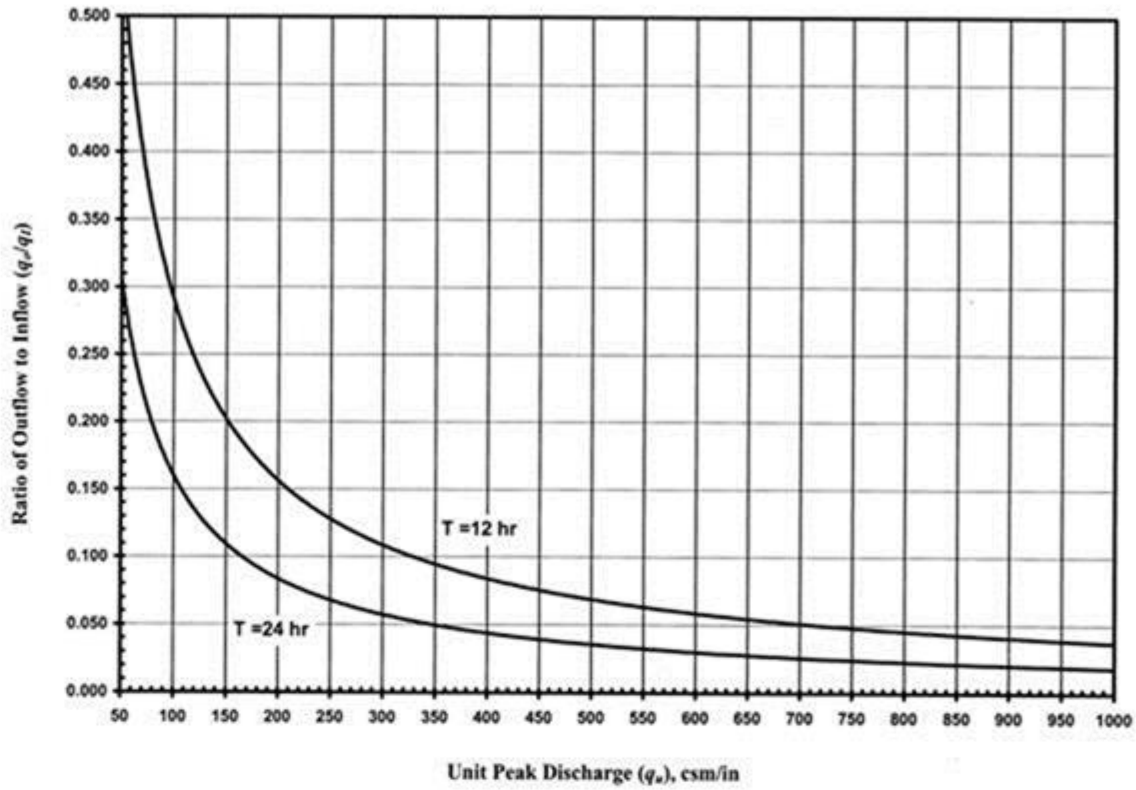


Figure 2.4-5 - Unit peak discharge (q_u) for SCS Type III rainfall distribution and 300 peaking factor



Step 5: Use the unit peak discharge and the T=24 hr curve to determine the ratio of outflow to inflow (q_o/q_i) from Figure 2.4-6.

Figure 2.4-6 - SCS ratio of outflow to inflow curves (2-24)



Step 6: Using the q_o/q_i ratio value calculated from Figure 2.4-6, use Equation 2.4-16 to calculate the required storage volume to runoff volume ratio (V_s/V_r).

$$\frac{V_s}{V_r} = 0.682 - 1.43\left(\frac{q_o}{q_i}\right) + 1.64\left(\frac{q_o}{q_i}\right)^2 - 0.804\left(\frac{q_o}{q_i}\right)^3$$

(2.4-16)

Where: V_s = required storage volume (acre-feet)

V_r = runoff volume (acre-feet)

q_o/q_i = peak outflow discharge to peak inflow discharge ratio

Using the V_s/V_r ratio value calculated above, use Equation 2.4-17 to calculate the required storage volume (V_s).

$$V_s = \left({}^{V_s}/_{V_r}\right) \times Q \times A \times 3630$$

(2.4-17)

Where: V_s = required storage volume - CP_v (ft^3)

Q = direct runoff (inches – 1-year, 24-hour storm for CP_v)

A = total drainage area (acres)

Erosion prevention measures such as energy dissipation and velocity control (i.e., riprap aprons/basins and baffled outlets) should also be employed at outlets to provide stream channel protection. These concepts are discussed in chapter 7 of the Drainage Design Policy Manual.

Riparian stream buffers also play an important role in protecting stream channels. Vegetative root systems provide soil structure benefits that prevent erosion. Riparian buffers provide additional stormwater benefits such as runoff velocity reduction, infiltration, and nutrient uptake. Other environmental benefits provided by buffers include wildlife habitat and surface water temperature moderation. A 25-foot buffer applies to all state waters and a 50-foot buffer applies to state waters designated as Trout Streams. Buffers are measured horizontally, starting at "the point where vegetation has been wrested by normal stream flow or wave action." [(2-14)] If stream buffer disturbances cannot be avoided, consult the Official Code of Georgia Annotated (O.C.G.A.) 391-3-7-.05 [(2-12)] and the GA EPD, *Stream Buffer Mitigation Guidance*, April 2011 [(2-14)] for mitigation requirements and guidance.

2.4.3 Overbank Flood Protection

Overbank flood protection should be provided to protect against flooding from middle-frequency storm events. To meet this standard, the proposed peak flow rate for the 25-year, 24-hour storm (Q_{p25}) must not exceed the existing peak flow rate. This requirement may be waived by the local jurisdiction if the downstream system has adequate capacity to convey the 25-year storm at ultimate build-out. Again, the CP_v, Q_{p25}, and Q_f requirements may be waived for drainage areas that flow directly into surface waters that have a drainage area greater than 5 square miles. The designer must still conduct a downstream analysis (as described in section 2.2.3) to verify that proposed condition flows do not exceed existing condition flows causing an impact to life or property.

The NRCS TR-55 (for drainage areas less than 2,000 acres) or USGS Hydrograph (for drainage areas 25 acres – 25 square miles) methods may be used to calculate Q_{p25}. The TR-55 method is presented here as a majority of roadway drainage basin areas are less than 25 acres. For guidance on the USGS Hydrograph method, refer to the GSMM or chapter 3 of Drainage Design Policy Manual. The GSMM also includes an example calculation for the USGS approach. For full TR-55 procedures, documentation, and example calculations, refer to the TR-55 *Urban Hydrology for Small Watersheds* [(2-34)] document or the WinTR-55 computer model.

A simplified peak discharge calculation method taken from TR-55 is provided in Equation 2.4-18.

$$Q_p = q_u A Q F_p$$

(2.4-18)

Where: Q_p = peak discharge (ft^3/s) ($Q_p = Q_{p25}$ for overbank flood protection)

q_u = unit peak discharge (ft^3/s /mi^2/in)

A = drainage area (mi^2)

Q = runoff (in)

F_p = pond and swamp adjustment factor, see Table 2.4-1

Table 2.4-1 Pond and Swamp Adjustment Factors (2-34)	
Percentage of pond and swamp areas	F_p
0	1
0.2	0.97
1	0.87
3	0.75
5	0.72

Complete the simplified NRCS peak runoff rate calculation using the following steps:

1. Determine the rainfall depth (P) for the 25-year, 24-hour storm using the NOAA Precipitation Frequency Data Server (Atlas 14: https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=pa.
2. Determine the CN and direct runoff (Q) in inches using the guidance previously provided for determining CP_v. (Equation 2.4-9 and Equation 2.4-11)
3. Use CN to determine initial abstraction (I_a) (Equation 2.4-12) and compute I_a/P.
4. Determine time of concentration (t_c) using the guidance provided in the CP_v section.
5. Use Figures 2.4-3, 2.4-4, and 2.4-5 and guidance in the CP_v section to determine q_u.
6. Determine F_p using Table 2.4-1.
7. Use Equation 2.4-18 to calculate Q_p.

Verify if detention is required by completing a downstream analysis as discussed in section 2.2.3.

To estimate the required storage volume:

1. Complete the above steps to determine the peak runoff rate under pre-developed conditions and post-developed conditions.
2. Determine the peak outflow to inflow ratio (q_o/q_i) by dividing the pre-development peak runoff rate by the post-developed peak runoff rate.
3. Use Equation 2.4-16 to calculate the required storage volume to runoff volume ratio (V_s/V_r).
4. Use Equation 2.4-17 to calculate the required storage volume (V_s).

2.4.4 Extreme Flood Protection

Finally, extreme flood protection should be provided to prevent flood damage from large storms, maintain existing 100-year floodplain boundaries, and to protect the structural integrity of stormwater infrastructure. Extreme flood protection is achieved by controlling the 100-year, 24-hour event (Q_f) so that flooding is not exacerbated by the project. Q_f should be calculated using the same methodologies previously presented for Q_{p25} (NRCS TR-55 or USGS Hydrograph). Q_f must be controlled on-site or by regional structures to maintain the existing 100-year floodplain where structures have already been constructed within the 100-year floodplain fringe area. Refer to the GSMM for additional guidance. Where the full build-out floodplain is sufficiently sized to account for extreme flow increases, designers may simply size on-site conveyance systems to safely pass Q_f. If detention is used to control Q_f, the same downstream analysis should be performed as described for the Q_{p25} for the 10% zone of influence. As previously stated, the CP_v, Q_{p25}, and Q_f requirements may be waived for drainage areas that flow directly into surface waters that have a drainage area greater than 5 square miles.

2.4.5 How the Sizing Criteria Volumes Work Together

The Runoff Reduction volume (RRv) / Water Quality volume (WQ_v) and the Channel Protection volume (CP_v) are calculated individually with their respective volumes determining the elevation/invert of the volume above. For instance, calculations for the RRv / WQ_v (computed using the basin area and percent of new impervious area) and CP_v (computed by detaining the 1-year, 24-hour runoff over a period of 24 hours) result in two specific volumes that are contained within the BMP.

Please note that the NPDES MS4 permit requires that the designer first analyze whether the 1-inch rainfall event can be infiltrated. If this is achievable, then no consideration of the WQ_v is required. If the metric cannot be achieved, then the WQ_v minus any RRv infiltration achieved in the design must be treated. For example, a designer calculates that for an outfall 1-inch of rainfall generates a RR_v of 10,000 ft^3 and 1.2-inches generate a WQ_v of 12,000 ft^3. If the project achieves 10,000 ft^3 of infiltration (RR_v), no treatment of WQ_v is required. If the designer is able to infiltrate 5,000 ft^3 then the full RR_V is not infiltrated. The designer must then provide water quality treatment for 7,000 ft^3 (12,000 ft^3 – 5,000 ft^3). For the remainder of this section, WQ_v shall be used interchangeably as RR_v, WQ_v, or a combination thereof.

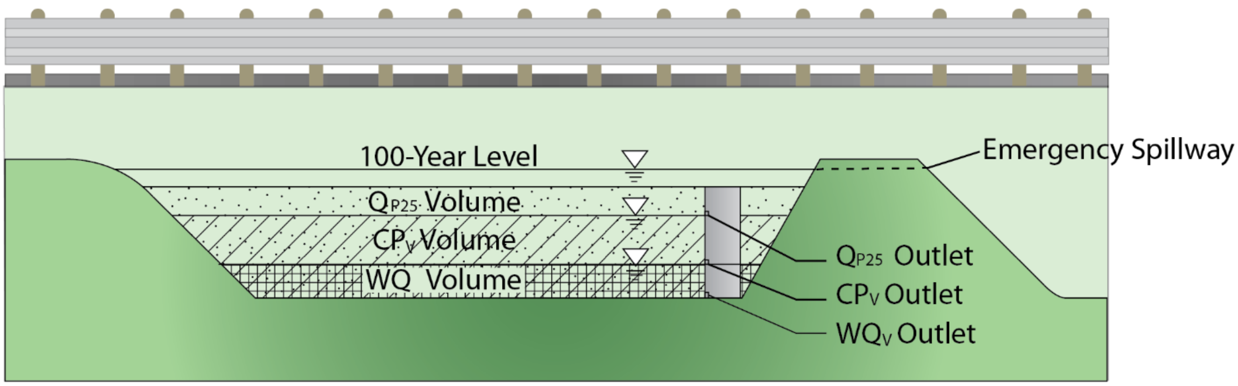
If BMPs incidental to the design of the roadway provide treatment to remove at least 80% of the calculated average annual post development total suspended solids load for a 1.2-inch rainfall event, then runoff reduction would be waived. For the purposes of the permit, incidental BMPs shall include filter strips, grass channels, and porous asphalt (OGFC). This waiver is not applicable to projects that discharge within one linear mile upstream of and within the same watershed as a designated trout stream.

The volumes are “nested” when you look at calculating the overall storage requirements of the BMP. While each sizing criteria volume is a separate calculation, the smaller volume(s) are inherent to the larger volume(s). For example, if a BMP is being designed for both the WQ_v and the CP_v, the first step is to calculate both volumes individually. Assume that the WQ_v was calculated to be 6,500 ft^3, and the CP_v was calculated to be 45,000 ft^3. The additional volume over the WQ_v that the BMP must be designed for in order to meet the CP_v criteria would be 45,000 ft^3 – 6,500 ft^3 or 38,500 ft^3. Since WQ_v is the smallest of the volumes, it is located at the bottom of the BMP along with its appropriately sized orifice. The top of this volume is the invert elevation of the CP_v orifice. Comparing the water quality volume to the stage/storage information of the proposed basin, the invert elevation of the CP_v outlet can be determined. Similarly, the top of the CP_v volume determines the elevation of the subsequent Q_{p25} outlet(s). In the case where these volumes are part of an extended detention basin, these volumes must completely drain within a 24-hour period (Other water quality BMPs may require a longer draw down time. Please refer to subsequent BMP sections in chapter 2 for specific guidance). Consequently, each volume has an individually sized orifice to ensure this requirement is met. The WQ_v orifice is sized so that the water quality volume drains over 24 hours, and the CP_v orifice is sized so that the difference between the CP_v and WQ_v drains over 24 hours.

In addition, the Q_{p25} criterion specifies that the post development 25-year, 24-hour storm peak flow rate not exceed the predeveloped flow rate. The NRCS TR-55 Methodology described in section 2.4.3 provides a process for estimating this volume to provide a starting point for storage design. However, the 1-year, 25-year and 100-year storm events must be routed through the pond and outlet structure to obtain an accurate analysis of BMP performance. The pond outflow characteristics are the result of all outlet devices working in conjunction with each other.

See Figure 2.4-7 for a visual representation of how the sizing criteria volumes and orifice locations work together.

Figure 2.4-7 - Sizing Criteria Volumes Illustration



2.4.6 LID and GI Considerations

In addition to the post-construction requirements discussed in this section, the MS4 permit requires the consideration of LID and GI during the design of GDOT facilities. LID and GI definitions can vary according to the source of the definition. The following are definitions provided from USEPA documents.

Low Impact Development:

> "…A management approach and set of practices that can reduce runoff and pollutant loadings by managing runoff as close to its source(s) as possible." LID practices promote the use of natural systems as part of a holistic approach to incorporate infiltration, evapotranspiration, and rainwater harvesting practices. (2-36)

Green Infrastructure:

> "An adaptable term used to describe an array of products, technologies, and practices that use natural systems – or engineered systems that mimic natural processes – to enhance overall environmental quality and provide utility services. As a general principle, Green Infrastructure techniques use soils and vegetation to infiltrate, evapotranspirate, and/or recycle stormwater runoff."
> (https://ofmpub.epa.gov/sor_internet/registry/termreg/searchandretrieve/glossariesandkeywordlists/search.do?details=&glossaryName=Greening%20EPA%20Glossary) (2-35)

Although definitions for LID and GI can vary, an important underlying concept includes integrating stormwater BMPs early in the design that promote infiltration, reuse, and evapotranspiration to reduce runoff volume. In addition to removing common stormwater pollutants such as nutrients and TSS, BMPs that reduce runoff volume help recharge aquifers and protect against hydromodification and stream channel erosion. The following LID/GI BMPs should be considered for GDOT projects:

- Reduced roadway footprint
- Porous pavements such as open graded friction course (OGFC) and porous European mix (PEM) on interstate and state route resurfacing and new construction.
- Using rural shoulder in lieu of urban curb and gutter
- Landscaping areas outside of clear-zones with trees
- Post-construction BMPs that allow for infiltration, evapotranspiration, and stormwater harvesting
- Minimize siting on porous soils, erodible soils, or steep slopes (>15%)
- Fitting the design to the terrain
- Following *Better Site Design* principles as presented in the GSMM to reduce post-construction stormwater runoff

Current GI practices already implemented and encouraged by GDOT include the following:

- Using recycled materials such as asphalt and concrete
- Environmental planning to avoid impacting wetlands and surface waters
- Including water quality considerations early in the planning process

In addition, some of the BMPs presented in section 2.6 are considered to be LID/GI practices. LID/GI practices for site development (non-linear) projects can be found in the GSMM. GDOT is required to track the LID/GI practices that were considered during the design of facilities where MS4 requirements apply and report the practices that were implemented. The LID/GI Checklist, an attachment to the *MS4 Post-Construction Stormwater Repor*t on the GDOT Manuals & Guides website, is used to document this and should be included with each set of plans for projects located in an MS4 area.

2.5 Post-Construction Stormwater BMP Selection Criteria

A multitude of BMP selection methods have been developed with varying degrees of complexity. The selection process outlined in this section is aimed at meeting the post-construction stormwater minimum standards outlined in GDOT's MS4 NPDES permit using the most cost-effective and viable BMPs.

2.5.1 Overview/Introduction of Selection Criteria

There are many factors to consider during the BMP selection process. Some criteria such as the BMP's cost and ability to meet requirements are weighted more heavily in the decision-making process. However, any one factor can render a BMP infeasible. Refer to section 2.3.3 for information on post-construction BMP infeasibility determination. The most common BMP selection criteria are listed as follows:

- Stormwater management and treatment requirements
- Safety
  - Motorist
  - GDOT maintenance staff
  - General public
- Site constraints
  - Available right-of-way
  - Soils (e.g., infiltration rate)
  - Bedrock and water table
  - Topography (adequate slope for gravity flow as well as excessive slopes)
  - Setback requirements
  - Environmentally or socially-sensitive areas (e.g., stream buffers, endangered species, historic landmarks)
- Cost
  - Capital
  - Operating (maintenance)
  - Service life
- Special watershed or stream considerations
- Maintenance challenges

2.5.2 Information Required

A significant amount of data gathering for selection criteria is needed prior to BMP selection. Designers should familiarize themselves with the project area, receiving water body, and watershed. The following information will aid in the decision-making process:

- Topography
  - Low-relief areas need special consideration because many BMPs require a hydraulic head to move stormwater runoff through the facility
  - High-relief areas may limit the use of practices that need flat or gently sloping areas to reduce sediment and/or runoff flow velocities. High-relief terrain may impact dam heights to the point that the use of a practice becomes infeasible.
- Existing site conditions and land cover
- Anticipated post-construction conditions

- Soils and groundwater data
  - Key evaluation factors are based on an initial investigation of the NRCS hydrologic soil groups at the site. More detailed geotechnical tests are required for infiltration trenches to confirm permeability and feasibility. See appendix B for more information. For bioretention and enhanced dry swales, the design should utilize infiltration estimates from NRCS soils survey information. Systems will be tested and be adapted, if needed, during construction.
- Underground utilities on site as well as nearby septic systems and water supply wells
- Drainage area characteristics
- Receiving water body and watershed
  - Determine if the project is subject to additional BMP criteria as a result of an adopted local watershed plan or special provision.
  - Cold and cool water streams have habitat qualities capable of supporting trout and other sensitive aquatic organisms. Therefore, the design objective for these streams is to maintain habitat quality by preventing stream warming, maintaining natural recharge, preventing bank and channel erosion, and preserving the natural riparian corridor. Table 2.5-1 shows which BMPs can potentially reduce thermal pollution. If a BMP does not provide the possibility of temperature reduction, it is not an appropriate option when discharging to a trout stream.

If the site is considered a hotspot or is located over a water supply aquifer, additional requirements may apply. A hotspot is a land use or activity that has the potential to generate relatively high contaminated stormwater runoff, such as a fueling station or de-icing facility. Refer to chapter 1 of Drainage Design Policy Manual for guidance on agency coordination and regulations.

2.5.3 BMP Menu

Table 2.5-1 lists BMPs that have been pre-approved for use at GDOT facilities in order of cost effectiveness. Each BMP's runoff reduction and pollutant removal capabilities are also included.

Typically, OGFC will be one of the most cost effective BMPs since it is a material substitution for conventional asphalt pavement. The use of OGFC as a BMP will depend on roadway characteristics rather than site constraints and requires approval for use from OMAT. Therefore, it has been listed last in the list of most cost effective BMPs.

In some cases, additional BMPs presented in the 2016 GSMM may be considered. However, underground detention and proprietary devices will generally not be allowed. Exceptions may be made by GDOT representatives where site constraints prevent the use of other BMPs and when water quality measures are required for environmental reasons other than MS4. For example, stormwater planters/tree boxes may be a suitable alternative in a highly urbanized area. Designers must receive approval from ODPS if a BMP other than one on the pre-approved list is proposed for a project. The following items must be included in the submittal:

- Why the deviation is necessary
- What is the benefit to the Department
- What are the proposed BMP maintenance requirements

- Who will be responsible for BMP maintenance
- What is the anticipated design life of the BMP
- BMP design details
- BMP cost estimate including installation

If approved, refer to the 2016 GSMM for design guidance.

Designers should familiarize themselves with GDOT-approved BMPs prior to beginning the selection process. Section 2.6 includes summary sheets and diagrams providing overviews of each of the approved BMPs.

2.5.4 Selection Process

Online BMP applications provide stormwater treatment or detention within the primary flowpath of runoff. Therefore, this type of application must consider the design volume and control higher design storm flow rates and volumes.

Offline BMP applications provide stormwater treatment or detention away from the primary flowpath of runoff. The simplest method of designing an offline BMP is to place the BMP at a location upstream of the outfall where an area of impervious surface equivalent to the net impervious acreage drains to the BMP. If there is no feasible location to place the BMP where the area of impervious surface is equal to or no more than 1.0 acre greater than the net impervious acreage, proceed with analyzing diversion pipes and bypass structures.

The flow bypass structure is designed to divert only the required treatment of stormwater runoff away from the main conveyance system to the BMP. This reduces the volume and velocity of flow entering the BMP, which often helps limit the amount of erosion or scour near the inlet of the BMP. See section 2.8.2 for more information on the design of flow bypass structures. In order to use a bypass structure, prior approval from the Office of Design Policy and Support shall be required before incorporating a bypass structure into the design.

If using a separate pipe system to bypass flow, be sure to maintain existing drainage patterns to the greatest extent practical. Where feasible, avoid using parallel or crossing pipes. If parallel or crossing pipes cannot be avoided, provide sufficient clearance between pipes.

In order to combine pre-development outfalls in the post-development condition, prior approval from the Office of Design Policy and Support shall be required before the combined outfalls are incorporated into the design.

LID/GI BMPs must be considered on all applicable GDOT projects and applied where budget and schedule will not be negatively impacted. These are BMPs that reduce impervious area, treat stormwater at the source, replace “grey infrastructure” with natural systems, and utilize infiltration, evapotranspiration, and reuse. Grass channels and filter strips would be examples of LID/GI BMPs and could be used in lieu of curb and gutter. Refer to section 2.4.6 for more information on LID/GI principles.

Other environmental or water quality concerns or issues should be considered where applicable. For example, threatened and endangered species may be present and require consideration during drainage design. Also, appropriate velocity control and energy dissipation should be provided at all outlets to prevent erosion. Refer to chapter 7 within Drainage Design Policy Manual for further guidance. Finally, flood control practices must be implemented where there is a potential impact to life and property. Further information regarding flood control can be found in chapters 1, 7, and 8 of the Drainage Design Policy Manual.

The following is a stepwise approach for selecting stormwater BMPs for GDOT facilities. Although the procedures are presented in sequential order, the process will likely be iterative and multiple factors may need to be considered concurrently to arrive at the best solution. Figure 2.5-1 illustrates the BMP selection process. In addition, Table 2.5-1 can be used as an initial screening tool to rule out BMPs that may be infeasible.

Stormwater Treatment Requirements – Using the guidance provided in section 2.4, determine all stormwater treatment requirements, including $RR_{v,}$ WQ_v, CP_v, Q_{p25}, Q_f, and additional requirements if impaired waters or trout stream protection apply. Eliminate any BMPs that will not achieve treatment goals, keeping in mind that BMPs can be used in series as illustrated in the previous section.

Site Consideration – Review the site for constraints that may preclude the use of certain BMP types and develop a list of appropriate BMPs for use at the site. The first site factor to consider is the soil type. Evaluate the soil type in the drainage areas to determine if infiltration may be feasible. At Concept, use the NRCS web soil survey, historical geotechnical investigation reports, or other published documentation to determine the soil types. Infiltration should only be considered if the results of the investigation indicate HSG A or B soils in a drainage area. Other potential site constraints include available space, topography, and safety and hazardous concerns presented by the post-construction stormwater BMP. Due to the regular maintenance requirements, all BMPs must be built on right-of-way and not a permanent easement with the exception of filter strips and grass channels. Based on the site constraints, determine which BMPs are appropriate. It is the obligation of the designer to determine if guardrails are warranted and exercise a standard of care that ensures public safety for each BMP design. Refer to section 2.10 for post-construction stormwater BMP safety considerations.

Preliminary Design and Feasibility – Start the feasibility evaluation with any infiltration BMPs included on the list of appropriate BMPs. Next, prioritize BMPs that are the most cost-effective according to Table 2.5-1. Review the guidance provided in section 2.6 for the applicable BMP, determine an estimated size and identify any additional requirements affecting feasibility of the BMP for the site.

BMP Design – If the BMP is deemed feasible, proceed with the design using the guidance provided in section 2.6. If the BMP is not feasible, repeat the process for another appropriate BMP or state why no BMPs are feasible for the site.

Figure 2.5-1 - BMP selection process flowchart

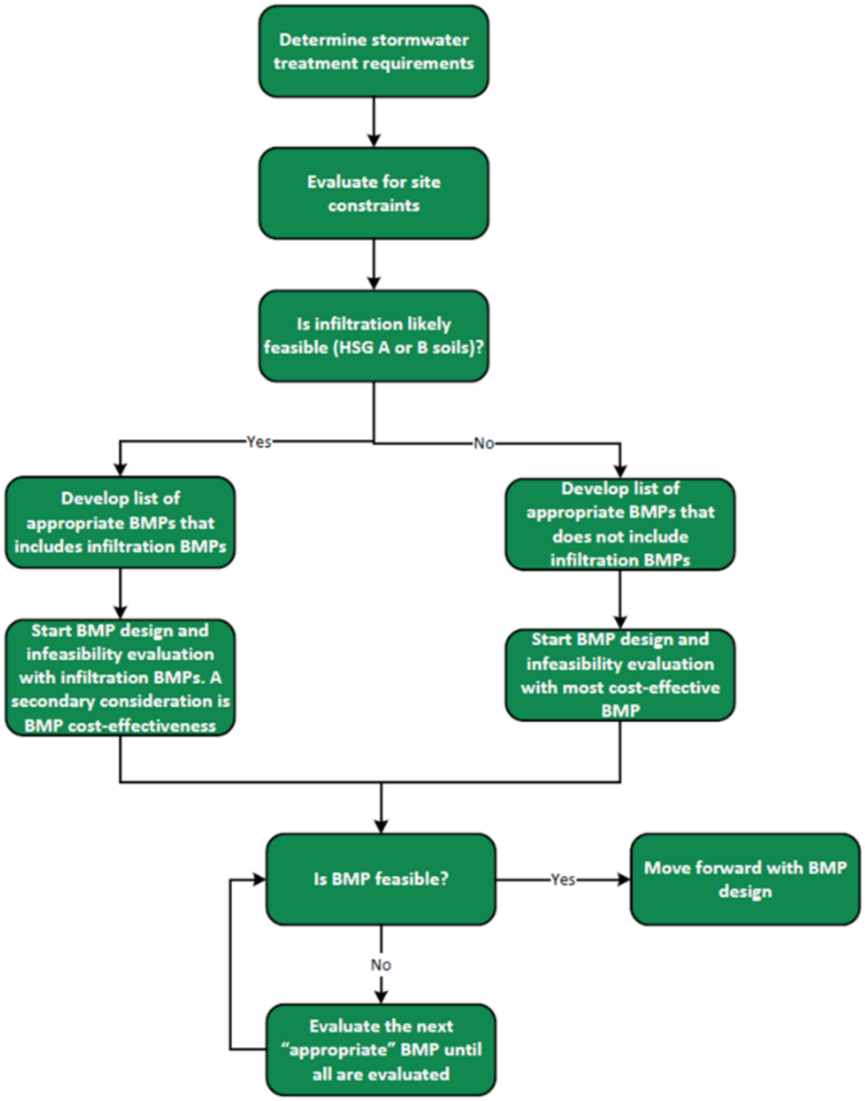


Table 2.5-1 BMP Screening Criteria (adapted from the GSMM)

BMP		Stormwater Treatment										Site Applicability							Cost Considerations	
BMP		RR(%)	WQ_v / TSS (%)	CP_v	Q_{p25} / Q_f	TP (%)	TN (%)	Fecal Coliform (%)	Metals (%)	Detention	Temperature Reduction	Roadway Applicability	LID/GI	Drainage Area (ac)	Space Req'd (% of Imperv. Area)	Max Site Slope	Minimum Head	Depth to Water Table	Construction Cost	Maintenance Burden
Filter Strips		25%	60%	X	X	20%	20%	X	40%	No	Yes	High	Yes	N/A	20%	25%	<1 ft	1-2 ft	Low	Low
Grass Channels	A/B HSG	25%	50%	X	X	25%	20%	X	30%	No	Yes	High	Yes	5 max	10%	4%	<1 ft	2 ft	Low	Low
	C/D HSG	10%																		
Bioslopes		25%	85%	X	X	60%	25%	60%	75%	No	Yes	High	Yes	N/A	N/A	5%	N/A	2 ft	Med	Med
Enhanced Dry Swales		50%	80%	?	X	50%	50%	X	40%	Low	Yes	High	Yes	5 max	10-20%	4%	NA	2 ft	Med	Med
Bioretention Basins	w/ open underdrain	50%	85%	?	X	0%	60%	90%	95%	Low	Yes	Med	Yes	5 max	3-6%	20%	3 ft	2 ft	Med-High	Med
	w/ upturned underdrain	75%	100%																	
	w/ capped underdrain	100%				0%	100%	100%	100%											
Enhanced Wet Swales		0%	80%	?	X	25%	40%	X	20%	Low	No	High	Yes	5 max	10-20%	4%	1 ft	Below	High	Low
Infiltration Trenches		100%	100%	?	X	100%	100%	100%	100%	Low	Yes	High	Yes	5 max	2-3%	6%	1 ft	4 ft	High	High
Sand Filters		0%	80%	?	X	50%	25%	40%	50%	Low	Yes	Med	Yes	10 max	2-3%	6%	5 ft	2 ft	High	High
Dry Detention Basins		0%	60%	✓	✓	10%	30%	X	50%	Yes	No	Med		75 max	N/A	15%	3 ft	2 ft	Low	Low
Wet Detention Ponds		0%	80%	✓	✓	50%	30%	70%	50%	Yes	No	Low	Yes	10 min*	2-3%	15%	6-8 ft	2 ft (if aquifer)	Low	Low
Stormwater Wetlands – Level 2		0%	85%	?	X	75%	55%	85%	60%	No	No	Low	Yes	5 min	3-5%	Flat	2-3 ft	2 ft (if aquifer)	Med	Med
Stormwater Wetlands – Level 1		0%	80%	✓	✓	40%	30%	70%	50%	Yes	No	Low	Yes	5 min	3-5%	8%	2-3 ft	2 ft (if aquifer)	Med	Med
OGFC		0%	80%	X	X	X	X	X	X	No	No	High	Yes	N/A	0%	N/A	N/A	N/A	Low	Low
Regenerative Stormwater Conveyance		0%	80%	X	X	70%	70%	0%	0%	No	Yes	High	Yes	50 max	Varies	10%	Varies	Above	High	Med

✓ -BMP meets the stormwater treatment requirement
? - BMP may meet the stormwater treatment requirement depending on size, configuration, and site constraints
X -BMP does not meet the stormwater treatment requirement
* - Minimum drainage area of ten acres is required to maintain the permanent pool (unless groundwater is present).

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2.6 Post-Construction Stormwater BMP Design Criteria

This section presents design criteria for BMPs that are pre-approved for use on GDOT projects. The list of pre-approved BMPs currently includes:

1.Filter strips	7.Bioretention basins
2.Grass channels	8.Dry detention basins
3.Enhanced swales (dry & wet)	9.Wet detention ponds
4.Infiltration trenches	10.Stormwater wetlands
5.Bioslopes	11. *Open Graded Friction Course (OGFC)
6.Sand filters	12. Regenerative Stormwater Conveyance

*Typically, OGFC will be one of the most cost effective BMPs since it is a material substitution for conventional asphalt pavement. The use of OGFC as a BMP will depend on roadway characteristics rather than site constraints and has therefore been listed last in the list of most cost effective BMPs.

Each of the BMP subsections is organized to provide important information needed for the successful design of the BMP. First, the BMP overview page summarizes important considerations associated with each BMP and provides general introductory information. Next, a more detailed description of the function and configuration of the BMP is provided to further familiarize designers with each BMP. An applications/feasibility section is then presented which discusses the site conditions and locations where the BMP may be favorable or should be avoided. Site constraints that may render a BMP infeasible are also presented in this section. Finally, the design section presents the overall BMP sizing procedure and the design process involved for each component. Note that methods and calculations needed for some design elements are presented in various sections of chapter 2 and various chapters of the Drainage Design Policy Manual.

All BMP information is focused on the linear application of the BMP. Non-linear applications of these BMPs, such as site development applications or other unique scenarios, will require additional design considerations. The designer is not limited to the pre-approved list of BMPs. The use of any other type of BMP not on the pre-approved list requires following the design deviation procedure and receiving approval from ODPS. Refer to section 2.5.3 for more information.

Following the design discussion, a maintenance section describes design aspects and strategies to facilitate maintenance procedures, help reduce long-term costs, extend the life of the BMP, and improve safety for maintenance personnel. For detailed maintenance information regarding each BMP, see GDOT's *Stormwater System Inspection and Maintenance Manual*, specifically the section on *Post-Construction Structures and Controls*.

Summary

2.6.1 Filter Strip

Advantages	Disadvantages
• Minimal construction effort and change to existing landscape • Effective for highway runoff pollution • Adaptable to a variety of site conditions • Flexible in design and layout • Lower cost alternative • Able to be used alone or as a combined measure	• Sensitive to erosion and concentrated flow • Provides less volume control than most BMPs • Large land requirement

Description: A filter strip is a uniformly sloped and vegetated area designed to treat sheet stormwater flow by filtering, slowing, and infiltrating runoff.

Design Considerations:

- Slopes should be between 2% and 25% (perpendicular to the roadway)
- Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion

Maintenance Considerations:

- Ensure that runoff is entering strips as sheet flow. Consider installing a level spreader or similar device.

Construction Considerations:

- Before grass has established in the filter strip, bare soil within the area is susceptible to erosion and scour. Any bare earth should be protected with a temporary Type 1 Turf Reinforcement Matting (TRM1).

Applicability for Roadway Projects:

- Highly suitable for roadway projects though they do require considerable right-of-way compared to some other stormwater BMPs.

Stormwater Management Suitability:

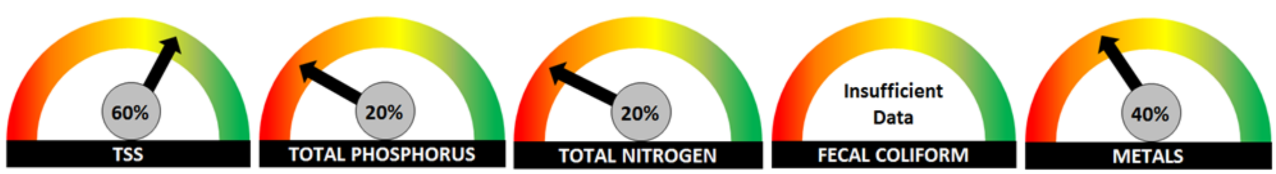
○ Runoff Reduction
○ Water Quality
○ Channel Protection
X Overbank Flood Protection
X Extreme Flood Protection
○ Temperature Reduction

✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

Filtration is the primary treatment mechanism though infiltration is possible where permeable soils exist. Filter strips provide excellent pretreatment when used in combination with other types of structural stormwater BMPs.

Treatment Capabilities

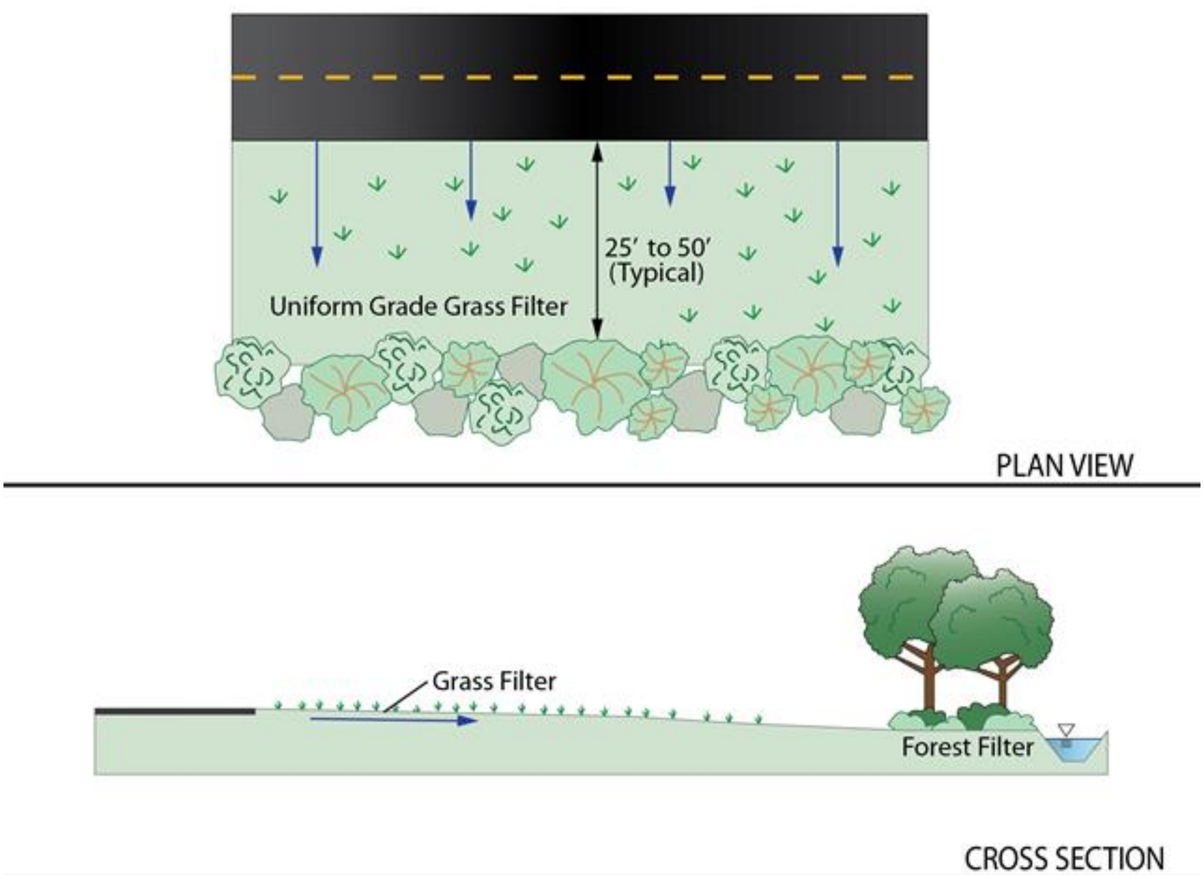


2.6.1 Filter Strip

Description

A filter strip is a uniformly graded and densely vegetated BMP that provides sheet flow, resulting in pollutant removal from stormwater runoff through increased sedimentation, vegetative filtering, and infiltration. Filter strips can be comprised of a variety of shrubs, grasses, and native vegetation to facilitate filtration, increase roughness, and benefit water quality. Filter strips are best suited for treating runoff from roads and highways, roof downspouts, small parking lots, and pervious surfaces. They are also ideal components of the outer or most upland zone of a stream buffer or as pretreatment for another BMP in a treatment train application. Filter strips are most often used in conjunction with rural roadway sections (curb and gutter not present) allowing the shoulder of the roadway to create sheet flow across the filter strip. Filter strips are considered a preferential BMP as they are adaptable in a linear setting, highly cost-effective, and are also considered an LID/GI measure. Figure 2.6.1-1 shows a typical filter strip.

Figure 2.6.1-1 - Typical filter strip configuration



Stormwater Management Suitability

- Runoff Reduction – Vegetated filter strips provide partial runoff reduction benefits. They become more effective with increased infiltration rate of the native soils. A filter strip provides 25% of the runoff reduction volume. Performance is dependent on vegetation density and contact time for settling, filtration, and infiltration.
- Water Quality – A filter strip is a stormwater treatment practice that can remove a variety of pollutants through several removal mechanisms. Vegetated filter strips are typically used as a pre-treatment component to reduce incoming runoff velocity, filter particulates, and uptake pollutants from the runoff. When sized correctly, they provide a 60% TSS removal efficiency. Either another BMP should be used in a treatment train with the filter strip or the filter strip can be sized for over-conveyance to provide the additional required water quality treatment. Because filter strips are typically situated along the length of a roadway, they may intercept additional drainage area and not just the new impervious surface. GDOT's water quality volume, however, is typically calculated based on the new impervious area only. If there is sufficient existing impervious area in the basin, the filter strip can be sized to for a larger impervious surface area, increasing the overall BMP TSS removal, and meeting GDOT's water quality requirement of 80% TSS removal from new impervious area. This can be achieved if the impervious area treated is at least 1.33 times the new impervious area (in width or additional filter strip length). For example, if a new lane was being constructed with an additional impervious width of 12 feet, the filter strip could be sized for at least a 16 feet pavement width and intercept the flow from the existing impervious area and meet the 80% TSS removal requirement.
- Channel Protection – Another control will be required in conjunction with a filter strip to provide the required detention or other controls necessary.
- Overbank Flood Protection – Another control will be required in conjunction with a filter strip to reduce the post-development peak flow of the 25-year storm (Q_{p25}) to pre-development levels (detention).
- Extreme Flood Protection – Filter strips must be designed to safely pass extreme storm or provide flow diversion.
- Temperature Reduction – Filter strips can provide for temperature reduction.

Pollutant Removal Capabilities

Filter strips improve stormwater quality by reducing suspended solids, metals, and nutrients in stormwater runoff through sedimentation and interception, vegetated filtration, and biological uptake. Performance is dependent on vegetation density and contact time for settling, filtration and infiltration. Research on fecal coliform removal has been inconclusive but suggests that filter strips are generally not considered to be effective BMPs for treating bacterial loads. The following average pollutant removal rates may be utilized for design purposes:

- TSS – 60%
- Total phosphorus (TP) – 20%
- Total nitrogen (TN) – 20%

- Fecal coliform – insufficient data
- Heavy metals – 40%
- Temperature – Temperature reduction is provided.

Application and Site Feasibility

Vegetated filter strips are best suited to treat smaller drainage areas. Flow must enter the filter strip as sheet flow spread out over the width (long dimension normal to flow) of the strip, generally no deeper than 1 to 2 inches. If flows will not enter the filter strip as sheet flow, special provision must be made to ensure design flows spread evenly across the filter strip.

Please note that flows that discharge from a filter strip across right-of-way boundaries and do not immediately concentrate may qualify for an outfall level exclusion and would not need to meet the filter strip criteria outlined in this section.

Siting information and constraints follow:

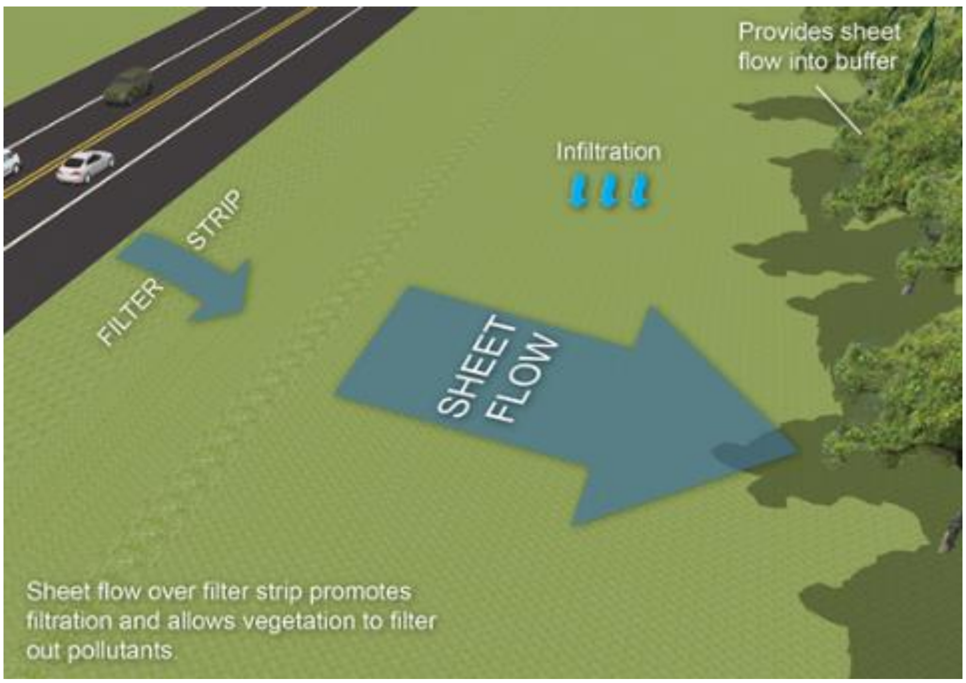
- **Drainage Area** – Filter strips generally have a maximum drainage area of 5 acres, but 2 acres is preferred.
- **Space Required** – Filter strip surface area is dependent on contributing drainage area and the slope of the filter strip. A drainage area to filter strip surface area ratio of 2:1 is recommended. Utilize available vegetated roadway shoulder as a roadside filter strip when possible. Locate the filter strip on the right-of-way or in a permanent drainage easement with appropriate access.
- **Site Slope** – Filter strips should be designed with slopes between 2% and 25% (perpendicular to the roadway). Greater slopes would encourage the formation of concentrated flow. Flatter slopes would encourage standing water. The sheet flow depth through the filter strip should be no more than 2 inches.
- **Depth to Water Table** – The seasonal high water table should be at least 1 foot lower than the ground at any point along the filter strip.
- **Soils** – Verify there are no bare spots present on existing slopes.
- **Vegetation** – Vegetation should be specified per Section 700 – Grassing.
- **Flow Velocity and Depth** – Use Table 2.6.1-1 to ensure velocity and depth requirements are met.
- **Other Constraints/Considerations –**
    - Riparian buffer should not be cleared for filter strip construction. Pedestrian traffic across the filter strip should be limited through channeling onto sidewalks.
    - The filter strip should be at least 15 feet long (25 feet preferred) to provide filtration and contact time for water quality treatment. The recommended maximum strip length is 48 feet.
    - Where flows become concentrated, using a level spreader to redistribute flow may be warranted near slope transitions, ESAs, adjacent properties, or areas exceeding an

overland flow length of 75 feet for impervious surfaces and 100 feet for pervious surfaces (see Additional Design Considerations and section 2.8.4 of this chapter for level spreader guidance).

- Filter strips are typically an on-line practice. On-line practices provide stormwater control within the flowpath of the runoff, conversely off-line practices provide stormwater control away from the flowpath. Both the top and toe of the slope, immediately preceding and following the filter strip, should be designed to encourage sheet flow and prevent erosion by minimizing slope in these areas.

Figure 2.6.1-2 shows a filter strip in a typical on-line application.

Figure 2.6.1-2 - Typical filter strip components and treatment processes



Data for Design

The data needed for filter strip design may include the following:

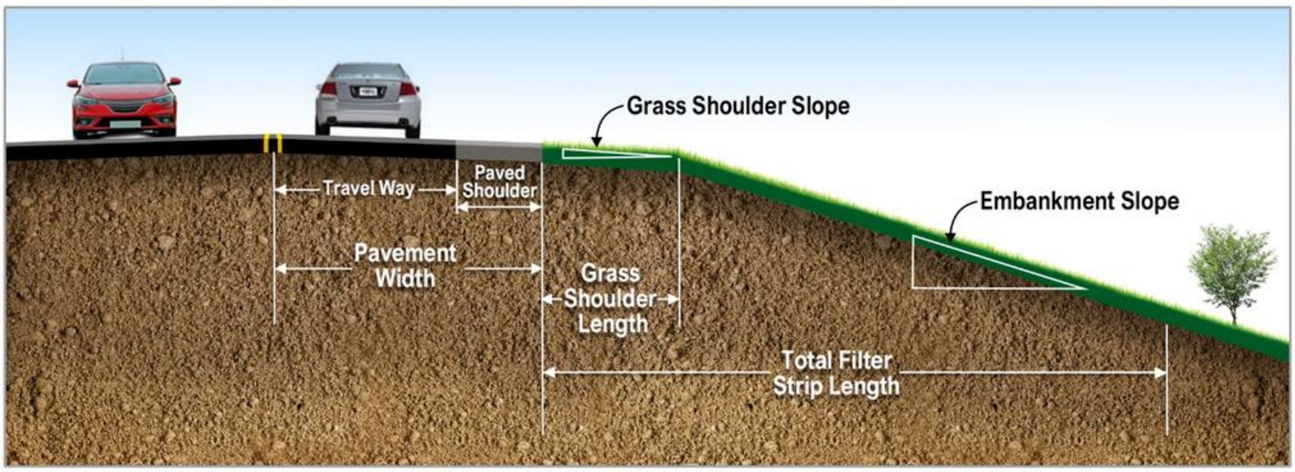
- Existing and proposed site, topographic and location maps, and field reviews
- Field measured topography or digital terrain model (DTM)
- Aerial/site photographs
- Drainage basin characteristics (slope, shape, size, soils, and land use)
- Preliminary plans including plan view, roadway and drainage profiles, cross sections, utility plans, and soil report
- Environmental constraints
- Design data of nearby structures (storm sewer as built information)
- Additional survey information

After initial data gathering, the contributing drainage area should be delineated and water quality volume and/or associated peak flow draining to the most downstream segment of the filter strip should be calculated based on post-project land use conditions (refer to section 2.4.1.2 of this chapter).

Next, preliminary dimensions for the filter strip area, roughness coefficient, and design slope should be determined. Location and general configuration for the filter strip should be set based on the above siting information.

The slope of the filter strip parallel with the roadway should be as flat as possible; however, this is usually influenced by the roadway profile, or shoulder slope. [(2-26)] A typical filter strip design for a roadway application is depicted in Figure 2.6.1-3 below.

Figure 2.6.1-3 - Typical filter strip design for a roadway application



Vegetation

A variety of shrubs, grasses, and native vegetation can also be used to facilitate filtration, increase roughness and benefit water quality. A list of grasses and vegetation appropriate for use in Georgia can be found in GDOT specification section 700.

Pretreatment

A number of other BMPs, including bioretention areas and infiltration trenches, may employ a filter strip as a pretreatment measure in a treatment train application.

Filter Strip Sizing

Table 2-6.1-1 has been developed based on research conducted by the Georgia Institute of Technology. The table assumes that drainage from the pavement will sheet flow across the filter strip. Please note that the filter strip length is measured from the edge of pavement and includes the 6% unpaved shoulder adjacent to the roadway. For instance, many shoulders have a 6% slope of grassing coming off of a paved shoulder going to a 4:1 slope. The designer would use the roadway unpaved shoulder width, the 4:1 slope and the pavement width to determine the length of filter strip.

Table 2.6.1-1 Total Filter Strip Length for Select Applications												
Pavement Width (ft)	**Embankment Slope 4:1**				**Embankment Slope 6:1**				**Embankment Slope 8:1**			
	Grass Shoulder (ft)				**Grass Shoulder (ft)**				**Grass Shoulder (ft)**			
	2	**4**	**6**	**8**	**2**	**4**	**6**	**8**	**2**	**4**	**6**	**8**
12	24	23	22	21	21	21	20	19	20	19	19	18
14	25	24	23	22	23	22	21	21	21	21	20	20
16	27	26	25	24	24	23	23	22	22	22	21	21
18	28	27	26	25	25	25	24	23	23	23	22	22
20	30	28	27	26	26	26	25	24	24	24	23	23
22	31	30	29	28	27	27	26	25	25	25	24	24
24	32	31	30	29	28	28	27	26	26	26	25	25
26	33	32	31	30	29	29	28	27	27	27	26	26
28	34	33	32	31	30	30	29	28	28	28	27	27
30	35	34	33	32	31	31	30	29	29	28	28	27
32	36	35	34	33	32	31	31	30	30	29	29	28
34	37	36	35	34	33	32	31	31	30	30	29	29
36	38	37	36	35	34	33	32	32	31	31	30	30
38	39	38	37	35	34	34	33	32	32	31	31	30
40	39	38	37	36	35	34	34	33	32	32	31	31
42	40	39	38	37	36	35	34	34	33	33	32	32
44	41	40	39	38	37	36	35	34	34	33	33	32
46	42	41	40	39	37	37	36	35	34	34	33	33
48	43	42	40	39	38	37	37	36	35	35	34	34
50	43	42	41	40	39	38	37	36	36	35	35	34
52	44	43	42	41	39	39	38	37	36	36	35	35
54	45	44	43	42	40	39	38	38	37	36	36	35
56	45	44	43	42	41	40	39	38	37	37	36	36
58	46	45	44	43	41	40	40	39	38	37	37	36
60	47	46	45	44	42	41	40	40	38	38	37	37

** The table above is adapted from Georgia DOT Research Project 17-22, Final Report - Optimizing Design of GDOT Post Construction Stormwater BMPs for Performance while minimizing Right-of-Way Acquisition and Peak Flows, August 2021, Table 3, pg 28.*

Maintenance Considerations

Without proper maintenance, BMPs will function at a reduced capacity and may cease to function altogether. Maintaining the vegetative cover and sheet flow over the filter strip is essential to the proper operation of the filter strip. A properly designed BMP includes access considerations for maintenance:

- Provide adequate right-of-way or easement.

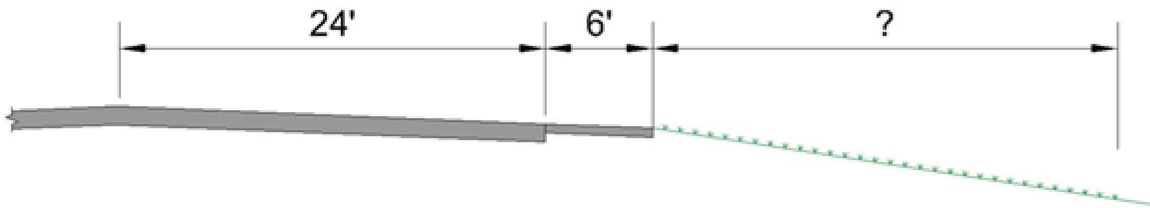
- Follow normal maintenance activities for grassed slopes including inspecting for erosion and ensuring dense vegetation

Refer to GDOT's *Stormwater System Inspection and Maintenance Manual*, for specific maintenance requirements.

Filter Strip Example Calculation

GIVEN:

- A new roadway project located in Atlanta, Georgia.
- The proposed project includes 200 feet of roadway (in length).
- The drainage area that discharges to the filter strip includes the following: two 12-foot lanes and a 6-foot paved shoulder that will drain via sheet flow directly into the filter strip.
- Assume no stormwater is collected as "off-site" or "bypass" runoff.
- Assume that the existing ground and available right-of-way is sufficient for a filter strip with an unpaved shoulder width of 4 feet, a slope of 4:1 and length of 200 feet.
- Assume a dense grass will be used.
- WQ_v = 571 ft^3; Q_{wq} = 0.245 ft^3/s



FIND:

An appropriate filter strip to treat runoff from the proposed roadway.

SOLUTION:

Due to the fact that stormwater runoff drained via sheet flow directly to the filter strip and the filter strip was the same width as the roadway segment length. Table 2.6.1-1 could be utilized. By looking up the pavement width and slope, the designer comes the solution of 34 feet.

Table 2.6.1-1 Total Filter Strip Length for Select Applications												
Pavement Width (ft)	Embankment Slope 4:1				Embankment Slope 6:1				Embankment Slope 8:1			
	Grass Shoulder (ft)				Grass Shoulder (ft)				Grass Shoulder (ft)			
	2	4	6	8	2	4	6	8	2	4	6	8
12	24	23	22	21	21	21	20	19	20	19	19	18
30	35	34	33	32	31	31	30	29	29	28	28	27
60	47	46	45	44	42	41	40	40	38	38	37	37

Summary

2.6.2 Grass Channel

Advantages	Disadvantages
• Lower cost • Reduction of impervious area • Well suited for linear environment • Stormwater collection and conveyance • Aesthetic benefits	• Drainage area, flow velocity, and flow depth limitations • Must be used in series with other BMPs for removal credit • Design heavily dependent on existing site conditions and topography

Description: A vegetated channel designed to enhance water quality through the settling of suspended solids.

Design Considerations:

- Contributing drainage area less than 5 acres
- Water quality rainfall event flow velocity less than 1.0 ft/s and flow depth less than 4 inches
- Minimum residence time of 5 minutes
- Recommended slope between 1% and 2% with a maximum of 4%.
- Side slopes 3:1 or flatter
- Minimum 2-foot clearance from groundwater

Construction Considerations

- Before permanent grass has been established in the channel, bare soil within the channel is susceptible to erosion and scour. Any bare earth should be protected with TRM.

Maintenance Considerations:

- Provide adequate access to the BMP and appropriate components

Applicability for Roadway Projects:

- Well suited for linear environments, interchanges, and facilities
- May be contained within the roadway right-of-way

Stormwater Management Suitability:

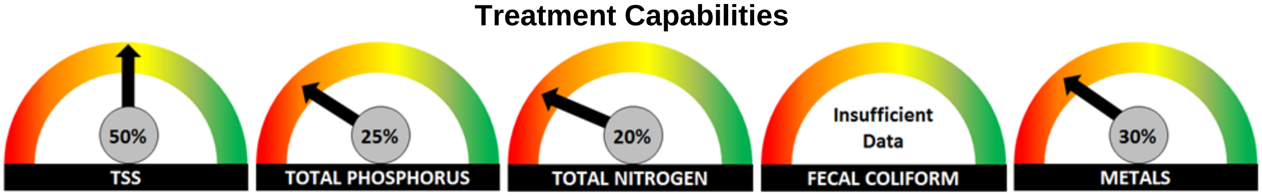
○ Runoff Reduction
○ Water Quality
○ Channel Protection
X Overbank Flood Protection
X Extreme Flood Protection
○ Temperature Reduction

✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

When properly incorporated into overall site design, grass channels may reduce impervious cover, partially infiltrate runoff with pervious soils, complement the natural landscape, and render aesthetic benefits.

Treatment Capabilities



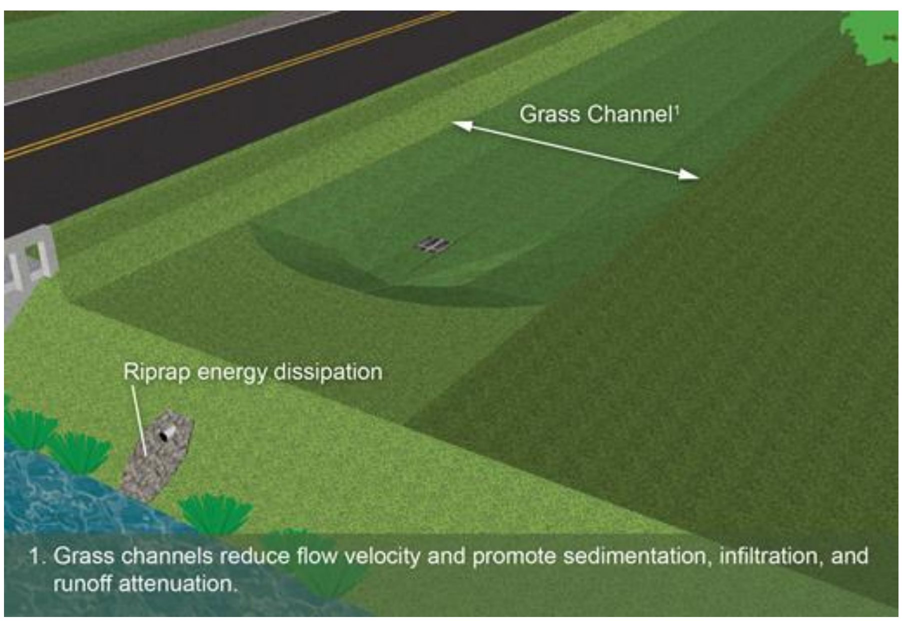
2.6.2 Grass Channel

Description

Grass channels, a form of a “biofilter,” are trapezoidal or parabolic shaped vegetated channels that work as a vegetative filter designed to enhance water quality through the settling of suspended solids through filtration, infiltration, and biofiltration. Grass channels also assist in meeting runoff velocity targets for the water-quality design storm of small drainage areas.(2-17) By reducing flow velocity, grass channels promote sedimentation, infiltration, and runoff attenuation. (2-26) Only vegetative filters provide an acceptable pollution management measure while conveying stormwater runoff. These vegetative filters include: waterways, ditches or swales, filter strips, and grass channels. A grass channel may serve as a runoff collection and conveyance system by acting as a single BMP, pretreatment BMP to another BMP, and/or as a link between other measures. Grass channels are limited to small drainage areas (less than 5 acres) and are well suited for incorporation into many applications and land uses, including linear roadway environments. They are considered an LID and GI practice and may also provide aesthetic benefits by accenting the natural landscape.

Grass channels differ from traditional roadside ditches in that grass channels, designed for water quality purposes, promote increased residence time and decreased conveyance velocity for the water-quality design storm. Grass channel design should provide a sufficient channel length to attain a minimum residence time of 5 minutes, while runoff velocity within the channel should not exceed 1.0 ft/s for the water quality design rainfall event peak discharge. (2-17) Water quality benefits are typically achieved by broadening base widths, lowering slopes, and creating dense vegetation. In areas with permeable soil, grass channels may also partially infiltrate runoff from small storm events, reducing runoff volume. A typical grass channel configuration is illustrated in Figure 2.6.2-1.

Figure 2.6.2-1 - Typical grass channel configuration



Stormwater Management Suitability

- Runoff Reduction – Grass channels provide partial runoff reduction benefits. They become more effective the higher the infiltration rate of the native soils. A grass channel can be designed to provide 25% of the runoff reduction volume for type A and B hydrologic soils or 10% of the runoff reduction volume for type C and D hydrologic soils. Performance is dependent on vegetation density and contact time for settling, filtration, and infiltration.
- Water Quality – Grass channels can be used to remove a variety of pollutants from stormwater runoff. They are typically used as the pre-treatment component of a larger "treatment train" to reduce incoming runoff velocities and filter out particulates. A grass channel provides 50% TSS removal if designed if designed, constructed, and maintained correctly Either another BMP should be used in a treatment train with the grass channel or the grass channel can be sized for over-conveyance to provide the additional required water quality treatment. Because grass channels are typically situated along the length of a roadway, they may intercept additional drainage area and not just the new impervious surface. GDOT's water quality volume, however, is calculated based on the new impervious area only. If there is sufficient existing impervious area in the basin, the grass channel can be sized to for a larger impervious surface area, increasing the overall BMP TSS removal, and meeting GDOT's water quality requirement of 80% TSS removal from new impervious area. This can be achieved if the impervious area treated is at least 1.6 times the new impervious area. For example, if the calculated target water quality volume is 5,000 ft^3, the grass channel can be sized to include 8,000 ft^3 of water quality volume and meet the 80% TSS removal requirement (8,000 ft^3 * 50% TSS removal = 5,000 ft^3 * 80% TSS removal).
- Channel Protection – For smaller sites, a grass channel may be designed to capture the entire channel protection volume (CP_v). Given that a grass channel is typically designed to completely drain over 48-72 hours, the requirement of extended detention for the 1-year, 24-hour storm runoff volume will be met. For larger sites, or where only the WQ_v is diverted to the grass channel, another control must be used to provide CP_v extended detention.
- Overbank Flood Protection – Another control will likely be required in conjunction with a grass channel to reduce the post-development peak flow of the 25-year storm (Q_{p25}) to pre-development levels (detention).
- Extreme Flood Protection – Grass channels must provide flow diversion and/or be designed to safely pass extreme storm flows while protecting vegetation.
- Temperature Reduction – Grass channels can provide for temperature reduction.

Pollutant Removal Capabilities

The following average pollutant removal rates may be utilized for design purposes: (2-17)

- TSS – 50%
- TP – 25%
- TN – 20%
- Fecal Coliform – Insufficient Data

- Heavy Metals – 30%
- Temperature – Temperature reduction is provided.

Water quality benefits may be maximized when the channels are designed in series with other structural stormwater controls.

Application and Site Suitability

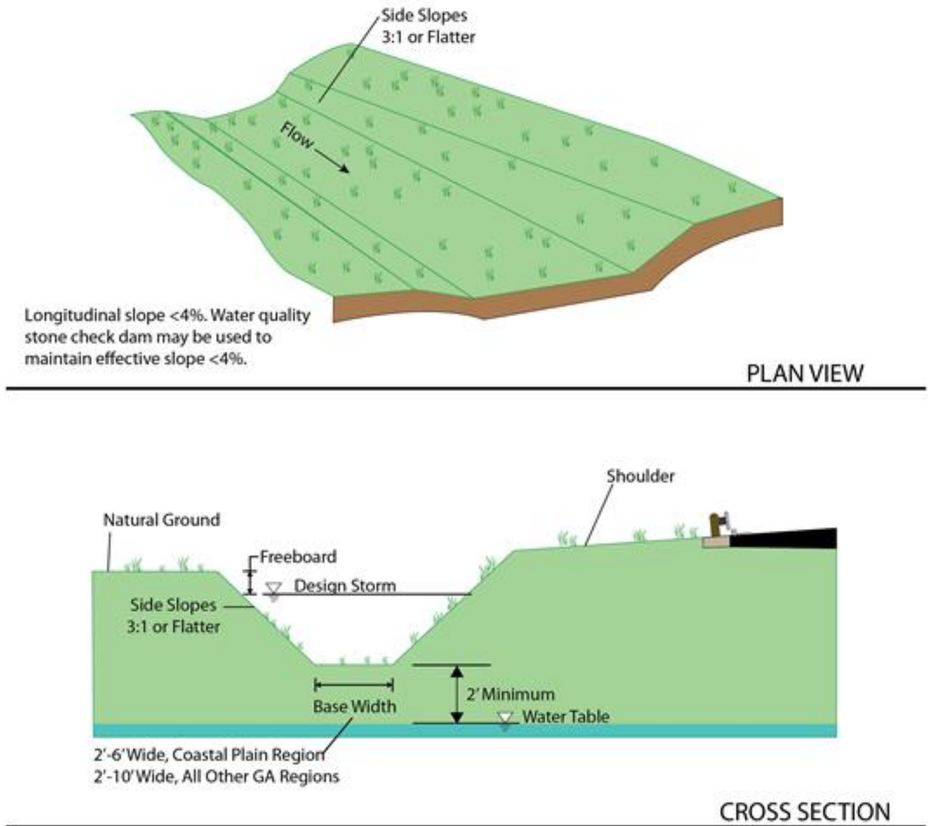
Grass channels are a low-cost option appropriate for various transportation applications, including linear roadways, interchanges, and facilities. Grass channels are not intended for runoff attenuation and should not act as a singular BMP when flooding is a concern.

Channel location and configuration will be largely dependent upon the contours of the land adjacent to a roadway alignment, the available right-of-way, and the results of the hydraulic analysis. Channels should not be placed within the limits of delineated wetlands. The final location should be coordinated with the project environmentalist to ensure compliance with the approved environmental document. Siting information and constraints include the following:

- **Drainage Area –** Maximum contributing drainage area of 5 acres. If the practice is used on larger drainage areas, the flows and volumes through the channel become too large to allow for filtering and infiltration of runoff.
- **Side Slope** – Slopes of the channel should be 3:1 or flatter.
- **Longitudinal Slope –** Between 1-4%; slopes between 1-2% are recommended.
- **Base Width –** The maximum width of a grass channel is a function of the regional geology in order to control stream braiding within channels. Braiding develops more easily in loose, incoherent soils (sands and glacial till, etc.). Cohesive soils (e.g., saprolite) are more resistant to braiding, so the maximum channel width may be greater. Therefore, the maximum channel width is 6 feet for the Georgia Coastal Plain (Upper and Lower Coastal Plain) and 10 feet for all other regions of Georgia.
- **Minimum Depth to Water Table** – A minimum of 2 feet is required between the channel bottom and the seasonal high groundwater table.
- **Runoff Velocities** – Must not be erosive. The maximum velocity of the water quality peak flow is 1.0 ft/s.
- **Flow Depth –** The maximum flow depth of the water quality peak flow is 4 inches.
- **Residence Time –** A minimum 5-minute residence time is required for the water quality peak flow. Residence time may be increased by reducing the slope of the channel, increasing the wetted perimeter, planting a denser grass, or installing check dams.
- **Soils** – No restrictions, although grass channels located on permeable soils (i.e., hydrologic soil group A or B soils) provide greater stormwater management benefits.

A stable channel is the ultimate goal for all channels located within a highway right-of-way or that impact highway facilities. A stable channel is a densely vegetated channel capable of withstanding erosion from the stormwater runoff. In addition to water quality design specifications, grass channel design is also required to comply with hydraulic design and freeboard requirements of the open channel design policy, as outlined in chapter 6 of the Drainage Design Policy Manual.

Figure 2.6.2-2 - Typical grass channel configuration



Data for Design

The initial data for grass channel design may include the following:

- Existing and proposed site, topographic and location maps, and field reviews
- Field measured topography or digital terrain model (DTM)
- Soils data from the NRCS Web Soil Survey or other source
- Aerial/site photographs
- Drainage basin characteristics
- Preliminary plans including plan view, roadway and drainage profiles, cross sections, utility plans, and soil report
- Environmental constraints
- Design data of nearby structures (storm sewer as built information)
- Additional survey information

Vegetation

The type of grass selected should be a dense variety that can withstand relatively high velocity flows and both wet and dry periods. To maximize water quality benefits, the grass should be as dense as possible. A list of grass varieties appropriate for use in Georgia can be found in GDOT specification section 700.

Additional Design Considerations

Water quality benefits may be enhanced through the use of permanent check dams at pipe inflow points and at various other points along the grass channel. Refer to the Check Dams Special Construction Detail for more information, including clear zone considerations.

The grass channel design must also adequately convey runoff from design storms as established based on roadway, traffic, site, and safety parameters and stated in the GDOT open channel design policy. Additional design considerations include compliance with regulatory agencies, freeboard, channel lining, energy dissipation, and outlet protection. Refer to chapter 6 of the Drainage Design Policy Manual for the Open Channel Design Policy.

Grass Channel Sizing

1. Determine the goals and primary function of the grass channel.

   The goals and primary function of the BMP must take into account any restrictions or site-specific constraints. Also take into consideration any special surface water or watershed requirements.

   - A grass channel must be designed for the water quality volume. The grass channel, however, can provide some runoff reduction benefit and reduce the required detention volume downstream. To calculate the RR_v credited for the practice (sized for WQ_v), Steps 2 – 8 have to be met, then proceed to Step 9. Otherwise the design process ends with Step 8.

2. Determine if the development site and conditions are appropriate for the use of a grass channel.

   Consider the application and site feasibility criteria in this section to determine if site conditions are suitable for a grass channel.

3. Calculate the Target Water Quality Volume.

   Calculate the water quality volume formula using the following formula:

$$WQ_v = \frac{1.2\,in \times (R_v) \times A \times 43560\,\frac{ft^2}{acre}}{12\,\frac{in}{ft}}$$

   Where:

   WQ_v = water quality volume (ft^3)
   R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.
   A = the contributing onsite drainage area with proposed land use classifications draining to the most downstream segment of the grass channel (acres)

4. Calculate the water quality volume peak flow.
   Calculate the water quality volume peak flow using the following formula and guidance in section 2.4.1.2.1.

$$Q_{wq} = q_u \times A \times Q_{WV}$$

   Where:

   Q_{wq} = water quality volume peak flow (ft^3/s)

   q_u = unit peak discharge (ft^3/s /mi^2/inch)

   A = drainage area (mi^2)

   Q_{WV} = water quality volume expressed in inches (use $1.2R_v$)

5. Determine channel geometry that meets the design requirements for the WQ_v storm event.
6. If calculating manually, use minimum channel geometry requirements, Manning's Equation, the Continuity Equation, and channel design charts found in HDS-3 (5-4) to begin the iterative computation process (see chapter 6 of the Drainage Design Policy Manual for more information). An alternative solution is to utilize computer software to design the channel that meets design requirements for the WQ_v storm event. Modify base width value and channel slope until the flow depth is less than 4 inches and the flow velocity is less than 1 ft/s.

7. Calculate the minimum length of the grass channel using a 5-minute residence time.
   To calculate the minimum length (feet) of the grass channel using a 5-minute residence time, use Equation 2.6.2-1, shown below. In situations where the minimum length required is longer than what the site allows, consider using stone check dams to increase the residence time over a shorter length. For more information on this type of design, see Volume II of the GSMM. (2-17)

$$L = V \times (5\ minutes) \times \left(\frac{60\ seconds}{1\ minute}\right)$$

(2.6.2- 1)

   Where:

   L = minimum length of channel (ft)
   V = velocity through the channel using the water quality volume peak flow (Q_{wq}) (ft/s)

8. Confirm the channel can pass all design requirements with required freeboard.
   Refer to chapter 6 of the Drainage Design Policy Manual for freeboard requirements.

9. Calculate the runoff reduction volume conveyed to the practice.

$$RR_v = \frac{1\ in \times (R_v) \times A \times 43560 \frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

   Where:

   RR_v = runoff reduction volume (ft^3)
   A = area draining to this practice (acres)
   R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.

10. Calculate the runoff reduction volume credited.
    Using Table 2.5-1 - *GDOT BMPs and Associated Pollutant Removals*, lookup the appropriate runoff reduction percentage (or credit) provided by the practice:

$$RR_v(credited) = RR_v(RR\%)$$

Where:
RR_v (credited) = runoff reduction volume provided by this practice (ft^3)
RR_v = runoff reduction volume conveyed to this practice (ft^3)
RR% = runoff reduction percentage, or credit, assigned to the specific practice

Maintenance Considerations

Without proper maintenance, BMPs will function at a reduced capacity and may cease to function altogether. A properly designed BMP takes into account access for maintenance:

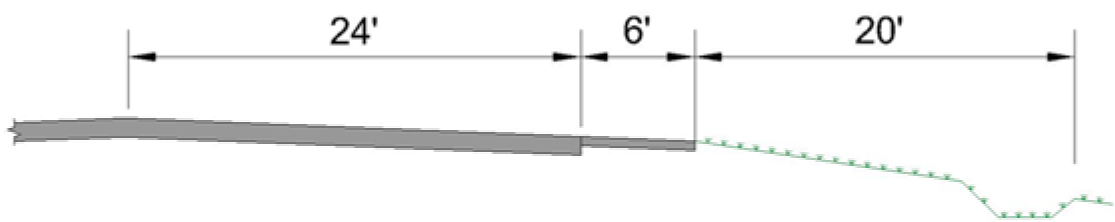
- Provide adequate right-of-way or easement.
- Provide access roads and ramps for appropriate equipment to all applicable BMP components.
- Provide space to turn around if necessary.
- Check for sufficient area to safely exit and enter the highway, if applicable.

Refer to GDOT's *Stormwater System Inspection and Maintenance Manual*, for specific maintenance requirements.

Grass Channel Example Calculation

GIVEN:

- A new roadway project located in Dallas, Georgia.
- The proposed project includes 1,300 feet of roadway (in length).
- The drainage area that discharges to the grass channel includes the following: two 12-foot lanes, a 6-foot paved shoulder, and a 20-foot wide grassed area, draining via sheet flow.
- Assume no stormwater is collected as “off-site” or “bypass” runoff.
- Assume that the existing ground and available right-of-way is sufficient for a grass channel with a longitudinal slope of 1% and length of 1,300 feet.
- The designer has previously calculated the following hydrologic information:
  - RR_v = 3,195 ft^3
  - WQ_v = 3,835 ft^3
  - Q_{wq} = 1.10 ft^3/s
  - Q_{p25} = 6.77 ft^3/s



FIND:

- Size the grass channel to meet design requirements for the WQ_v flow and to safely convey the peak flow from the design storm event (25-year).

SOLUTION:

1. Determine if it is necessary to calculate the runoff reduction volume credited for the practice in order to reduce the detention volume requirements downstream. For this example, the runoff reduction volume credited will be calculated.
2. Based on the existing ground geometry, a grass channel with a longitudinal slope of 1.0% is appropriate for the site.
3. The water quality volume was already calculated to be 3,835 ft^3.
4. The water quality volume peak flow was already calculated to be 1.10 ft^3/s.
5. Based on the existing ground geometry, the grass channel will utilize a longitudinal slope of 1.0%. If calculating manually, use minimum channel geometry requirements, Manning’s Equation, the Continuity Equation, and channel design charts found in HDS-3 (5-4) to begin the iterative computation process (see chapter 6 of the Drainage Design Policy Manual for more

information). An alternative solution is to utilize computer software to design the channel that meets design requirements for the WQ_v storm event.

Q_{wq} = 1.10 ft^3/s

Manning's n = 0.24 (densely vegetated grass swale)

Computer Software Iteration 1:

- Given: Longitudinal slope = 1.0% (0.01 ft/ft)
- Assume: Base width = 2 ft
- Assume: Side slopes = 3:1

  Flow depth = 8.64 inches (0.72 ft) **> 4 inches Too High**

  Flow velocity = 0.4 ft/sec **< 1 ft/sec. OK**

Adjust channel dimensions and longitudinal slope as needed until flow depth and velocity are satisfactory.

Computer Software Iteration 2:

- Given: Longitudinal slope = 1.0% (0.01 ft/ft)
- Assume: Base width = 8 ft
- Assume: Side Slopes = 6:1
  - Flow depth = 4.56 inches (0.38 ft) **> 4 inches Too High**
  - Flow velocity = 0.3 ft/sec **< 1 ft/sec. OK**

Computer Software Iteration 3:

- Given: Longitudinal slope = 1.0% (0.01 ft/ft)
- Assume: Base width = 10 ft
- Assume: Side Slopes = 8:1
  - Flow depth = 4 inches (0.33 ft) **≤ 4 inches OK**
  - Flow velocity = 0.26 ft/sec **≤ 1 ft/sec. OK**

6. Verify the length available meets the minimum length of the grass channel calculated using a 5-minute residence time.

$$L = V \times (5\ minutes) \times \left(\frac{60\ seconds}{1\ minute}\right) = 0.26 \times 5 \times 60 = 78\ ft$$

Therefore, the 1,300 feet available for the length of the grass channel is sufficient.

7. Next, verify that channel design meets all design requirements as outlined in the design requirements of this section. Use Q_{p25} in the same manner as above to determine channel depth and verify stable channel design for the design storm event.

Q_{p25} = 6.77 ft^3/s

- Flow depth = 11 inches (0.88 ft)

- Flow velocity = 0.45 ft/sec **Non-erosive (less than 4 ft/sec) OK**

Add 0.5 feet to the flow depth for freeboard to get overall channel depth equal to 1.38 feet. Set channel to minimum to a depth of 1.5 feet.

Adjust channel dimensions as needed for stable channel design and existing site conditions. If channel dimensions are modified, re-calculate flow depth and velocity values for WQ_v and the design storm event. Repeat until flow depth and velocity meet design requirements for both the WQ_v and the design storm event.

Verify that channel design meets all design requirements as outlined in the open channel design policy as outlined in chapter 6 of the Drainage Design Policy Manual.

The design could end at this step, but if a designer wants to determine the runoff reduction volume credited by the practice, continue to step 8.

8. The water quality volume was already calculated to be 3,195 ft^3.

9. Calculate the runoff reduction volume credited. A grass channel with HSG B soils is credited with 25% runoff reduction.

$$RR_v(credited) = RR_v(RR\%)$$

$$RR_v(credited) = 3{,}195 \times (25\%) = 799\ ft^3$$

Summary

2.6.3 Enhanced Swale

Advantages	Disadvantages
• Water quality benefits • Can be configured to provide stormwater attenuation • Design options suitable for dry or wet conditions • No soil restriction • Moderate maintenance burden	• Potential large land requirement • Limited to small drainage areas • Unsuitable for steep terrains • Moderate capital cost • Potential for odor or mosquitos with wet swale

Description: A vegetated open channel designed and constructed to capture and treat stormwater runoff from the WQ_v rainfall event in dry or wet cells formed by check dams or other means.

Design Considerations:

- Drainage area less than 5 acres
- Longitudinal slope less than 4% with 1% to 2% recommended
- Maximum 18 inches WQ_v ponding depth
- Side slopes of 4:1 or flatter recommended, max 2:1
- Maintain non-erosive velocity for 2-year storm
- Dry swale has multiple underdrain options that provide different runoff reduction credits

Maintenance Considerations:

- Provide adequate access to the BMP and appropriate components
- Maintaining the vegetative cover is essential to the proper operation of the enhanced swale

Applicability for Roadway Projects:

- Space and grade requirements may limit applicability in the linear environment
- Channel shape can be elongated to accommodate roadway applications
- Check dams serve as a design option when existing slopes are too steep

Stormwater Management Suitability:

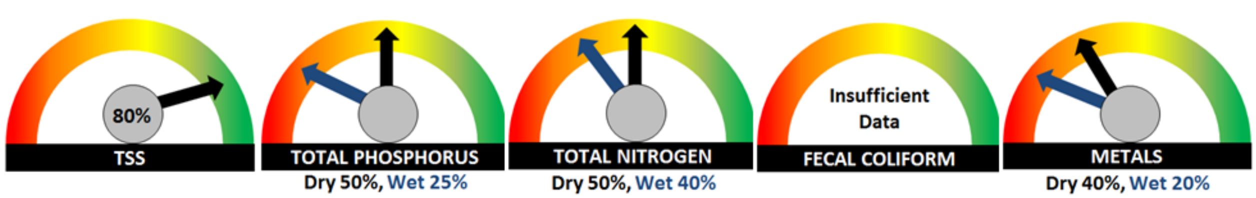
✓ Runoff Reduction
✓ Water Quality
○ Channel Protection
○ Overbank Flood Protection
○ Extreme Flood Protection
○ Temperature Reduction

✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

Enhanced swales are considered a LID/GI design practice. They are capable of blending in with and enhancing the natural landscape.

Treatment Capabilities



2.6.3 Enhanced Swale

Description

Enhanced swales are vegetated open channels designed and constructed to capture and treat stormwater runoff from the water quality rainfall event that collects within a dry or wet cell formed by an outlet control structure or other means. Enhanced swales are a structural BMP and considered an LID/GI practice. The incorporation of specific design features to enhance stormwater pollutant removal effectiveness distinguishes the enhanced swale from a normal drainage ditch or grass channel.

The enhanced swale operates much like a grass channel in that it is a trapezoidal or parabolic-shaped vegetated channel used as a measure for runoff conveyance and attenuation. Enhanced swales work as a type of vegetative filter designed to enhance water quality through the settling of suspended solids through filtration, infiltration, and biofiltration. The enhanced swale additionally incorporates the use of an outlet control structure to retain the water quality volume and promote settling and infiltration.

The two primary enhanced swale designs include:

- **Enhanced Dry Swale** – Includes a filter media of soil and an underdrain system designed to treat the water quality volume through filtration and infiltration. The mostly dry conditions of the dry swale make it the preferred option in areas where standing water may present a safety hazard.
- **Enhanced Wet Swale** – Designed to retain the water quality volume in support of wetland vegetation, wet swales achieve pollutant removal from the water quality volume through sediment accumulation and biological removal. Wet swales are better suited for areas with a high water table or poorly draining soils.

Figure 2.6.3-1 shows examples of both dry and wet swales.

Figure 2.6.3-1 - Enhanced swale examples [(2-17)]

Enhanced Dry Swale   Enhanced Wet Swale

Enhanced swales are designed primarily for stormwater quality and have limited ability in channel protection and conveyance. Enhanced swales are best suited for small drainage areas (less than 5 acres), well suited for incorporation into many applications and land uses, including linear roadway environments, and may also provide aesthetic benefits by accenting the natural landscape.

Stormwater Management Suitability

- Runoff Reduction – A dry swale can provide 50% of the runoff reduction volume, if properly maintained. Enhanced wet swales do not provide runoff reduction volume credits.
- Water Quality – Dry swale systems rely primarily on filtration through an engineered media and/or infiltration into the underlying soils to provide removal of stormwater contaminants. Both the enhanced dry swale and enhanced wet swale provide 80% TSS removal if designed, constructed, and maintained correctly.
- Channel Protection – Generally, only the WQ_v is treated by a dry or wet swale, and another BMP must be used to provide CP_v extended detention. However, for some smaller sites, a swale may be designed to capture and detain the full CP_v.
- Overbank Flood Protection – Enhanced swales must provide flow diversion and/or be designed to safely pass overbank flood flows. Ensure non-erosive velocities for the 25-year event or the 50-year event if the swale is in a sag and armor 1 foot' above this level. Another BMP must be used in conjunction with an enhanced swale system to reduce the post-development peak flow of the 25-year storm (Q_{p25}) to pre-development levels (detention).
- Extreme Flood Protection – Enhanced swales must provide flow diversion and/or be designed to safely pass extreme storm flows. The swale should be sized such that the 100-year storm can pass within the emergency spillway without overtopping the swale in any other location. Another BMP must be used in conjunction with an enhanced swale system to reduce the post-development peak flow of the 100-year storm (Q_f) if necessary.
- Temperature Reduction – Enhanced dry swales can provide for temperature reduction. Enhanced wet swales do not provide temperature reduction.

Pollutant Removal Capabilities

The following average pollutant removal rates may be utilized for enhanced swales: [(2-17)]

- TSS – 80%
- TP – 50% (Dry Swale) / 25% (Wet Swale)
- TN – 50% (Dry Swale) / 40% (Wet Swale)
- Fecal Coliform – Insufficient Data
- Heavy Metals – 40% (Dry Swale) / 20% (Wet Swale)
- Temperature – Temperature reduction is provided (Dry Swale).

Stability is the ultimate goal for all swales located within a highway right-of-way or that impact highway facilities. In addition to water quality design requirements, enhanced swale design is also required to comply with the hydraulic design and freeboard requirements of the Open Channel Design Policy, as outlined in chapter 6 of the Drainage Design Policy Manual.

Application and Site Suitability

Enhanced swales are a moderate cost option appropriate for various transportation applications, including roadways, highways, and non-road areas with a low percentage of impervious cover. The relatively large land requirement limits the incorporation of enhanced swales in high density areas or where a right-of-way may be limited.

Dry swales tend to be more prevalent along rural primary roads and highways. Wet swales generally are not the preferred BMP in high density or urban areas due to the presence of standing water and the potential safety threat, odor, or mosquitos. Wet swales may be used along highways as an element of a landscaped area.

Location and configuration will be largely dependent upon the existing site conditions, the available right-of-way, and the results of a hydraulic analysis. Location and geometry should be determined on a case-by-case basis using sound engineering judgment. The final location should be coordinated with the project environmentalist to ensure compliance with the approved environmental document.

Earth check dams, and/or enhanced swale outlet structures shall not be placed in the median. Additionally, earth check dams and/or enhanced swale outlet structures shall not be placed in the clear zone. Guardrail shall not be placed solely for the purpose of placing any combination of earth check dams and/or enhanced swale outlet structures.

When considering locations for enhanced swales, the following constraints should be considered:

- **Drainage Area** - Contributing drainage area should be less than 5 acres.
- **Space Required** - Enhanced swale design generally requires a surface area equal to approximately 10% to 20% of the contributing impervious area.
- **Depth to Water Table** –
    - The bottom of the underdrain layer should be a minimum of 2 feet above the seasonal high groundwater table for dry swales.
    - A wet swale can be used where the water table is at or near the soil surface, or where there is a sufficient water balance in poorly drained soils to support a wetland plant community. If above an aquifer or treating a hotspot, however, 2 feet is required between the bottom of a wet swale and the elevation of the seasonally high water table. Where wet swales do not intercept the groundwater table, a liner must be installed on HSG A and B soils. A water balance calculation should be performed to ensure an adequate water budget to support the specified wetland species. A water balance analysis may not be necessary if a liner is installed but should be considered regardless if the drainage area is small and/or has a small amount of impervious area. The wet swale size may need to be adjusted to account for lost volume due to seasonal fluctuations in the groundwater table.
- **Trout Stream** – Runoff temperature reduction can be provided by an enhanced dry swale. No runroff temperature reduction is provided by enhanced wet swales.
- **Aquifer Protection** – No exfiltration of hotspot runoff from dry swales is allowed in areas subject to aquifer protection. An impermeable liner should be used, or an infiltration BMP should be avoided in these areas.

- **Airports** – A wet swale should not be located within 5 miles of a public-use airport.

Data for Design

The initial data needed for enhanced swale design may include the following:

- Existing and proposed site, topographic, location maps, and field reviews
- Field measured topography or digital terrain model (DTM)
- Aerial/site photographs
- Drainage basin characteristics
- Preliminary plans including plan view, roadway and drainage profiles, cross sections, utility plans, and soil report
- Environmental constraints
- Design data of nearby structures
- Additional survey information
- Depth to seasonally high groundwater
- Soils data from the Web Soil Survey or other source

General Design Criteria

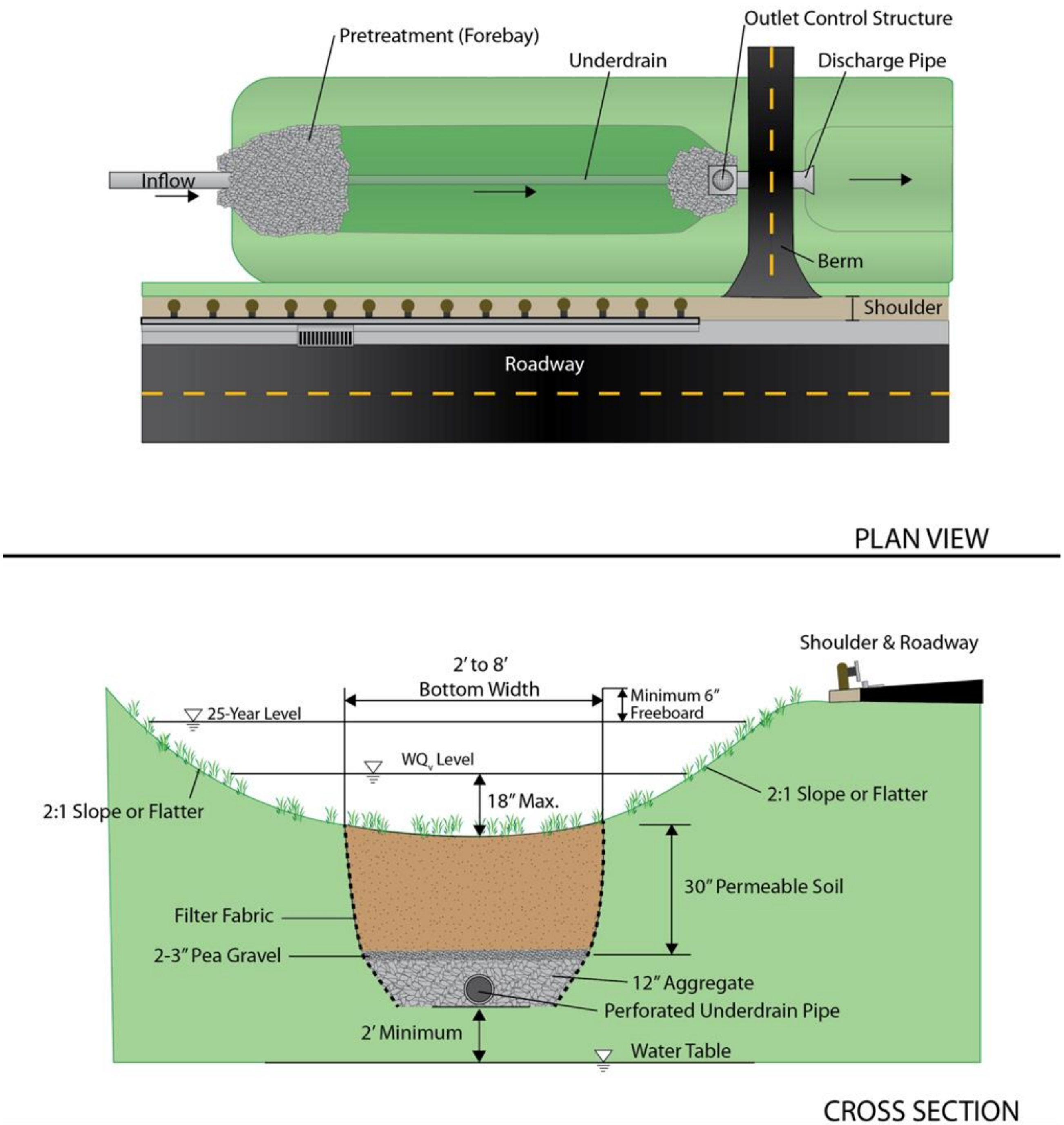
After initial data gathering, the contributing drainage area should be delineated, and the post-project land use should be used to compute the peak flow of the WQ_v (see section 2.4.1.2) draining to the most downstream segment of the enhanced swale.

Next, preliminary values for swale size and slope should be determined. Location and general configuration for the enhanced swale should account for aesthetics and be preliminarily set based on the following design criteria:

- **Slope –** Longitudinal channel slopes between 1% and 2% are recommended. Maximum slope is 4%. Six-inch or 12-inch check dam may be used at minimum 50-foot spacing when needed. Longitudinal slope between 1% and 4% for enhanced dry swales.
- **Base Width –** The minimum base width is 2 feet, and the maximum base width is 8 feet.
- **Side Slope –** Side slopes should be 3:1 or flatter, however can be 2:1 with permission from the Office of Design Policy and Support.
- **Runoff Velocity –** Maintain non-erosive velocity (less than 4 feet per second) within the swale for the 25-year storm event.

The design elements specific to an enhanced dry swale are discussed below and illustrated in Figure 2.6.3-2.

Figure 2.6.3-2 - Enhanced Dry Swale schematic



Pretreatment

Pretreatment of runoff in both a dry and wet swale system is typically provided by a sediment forebay located at the inlet. Vegetated filter strips and gentle side slopes should be provided along the top of channels to provide pretreatment for lateral sheet flows.

Filter Media

The required treatment is achieved as the WQ_v flows through the filter media and potentially infiltrates into the underlying soil. The surface area of the filter media is designed such that the WQ_v has a maximum drawdown time of 48 hours.

- Refer to GDOT Special Provision 169 / Specification 169.
- Minimum soil media infiltration rate (coefficient of permeability) of 2 ft/day.
- Where possible, soil media is recommended to contain a high level of organic material to promote pollutant removal.
- Sod should be obtained from a supplier that grows in nonclay soils where possible. Sod grown in clayey soils can reduce infiltration into the media, causing the basin to retain water longer than desired. Generally, sod should be ‘half cut’ or ‘thin cut’ whereby the soil thickness is approximately half of conventionally available sod to maximize infiltration.[2-26]

Underdrain

An underdrain system is only required for enhanced dry swales.

- Underdrain systems consist of a polyethylene pipe longitudinal underdrain, typically 8 inches in diameter in a 12-inch No. 57 aggregate layer.
- Outlet protection must be used at any discharge point to prevent scour and downstream erosion. Discharge underdrain systems to storm drainage infrastructure or stable outlet.
- Refer to section 2.8.3 of this manual and the GDOT Underdrain Special Construction Detail for additional information regarding underdrain design.

Outlet Control Structure

- There are three potential outlet control structure configurations for the enhanced dry swale: retaining wall outlet, earth berm outlet, and concrete drop inlet. Refer to the Enhanced Dry Swale Outlet Structure Special Construction Detail.
- Retaining wall outlets shall have a minimum 10-feet wide concrete splash pad downstream of the overflow weir. A rip rap apron shall extend from the downstream edge of the concrete splash pad a minimum distance of 5-feet.
- The outlet control structure is designed to both retain the WQ_v in the BMP, as well as safely convey the remaining runoff downstream of the enhanced dry swale.
- The overflow weir is placed at the elevation of the WQ_v, which is a maximum of 18-inches above the bottom of the swale. The overflow weir allows the runoff that does not filter through the media to discharge from the BMP and be conveyed downstream.
- The length of the overflow weir is designed to allow the enhanced swale to safely pass the 25-year, 24-hour storm event with a minimum 6 inches of freeboard.
  - If using the retaining wall outlet structure, the freeboard shall be measured from the 25-year, 24-hour storm event to the top of retaining wall.
  - If using the earth berm outlet structure, the minimum width of the weir shall be 2-feet and the maximum width of the weir shall be 8-feet.

- If using the retaining wall outlet structure, the maximum width of weir shall be 2 feet less than the width of the concrete splash pad.

The following weir equation is used to determine weir length of a broad-crested weir. (2-32)

$$Q = C_d \times L \times H^{\frac{3}{2}}$$

(2.6.3- 2)

Where:

Q = Peak flow (ft^3/s)
C_d = Weir coefficient
L = Length of weir (ft)
H = Depth of water above weir crest (ft)

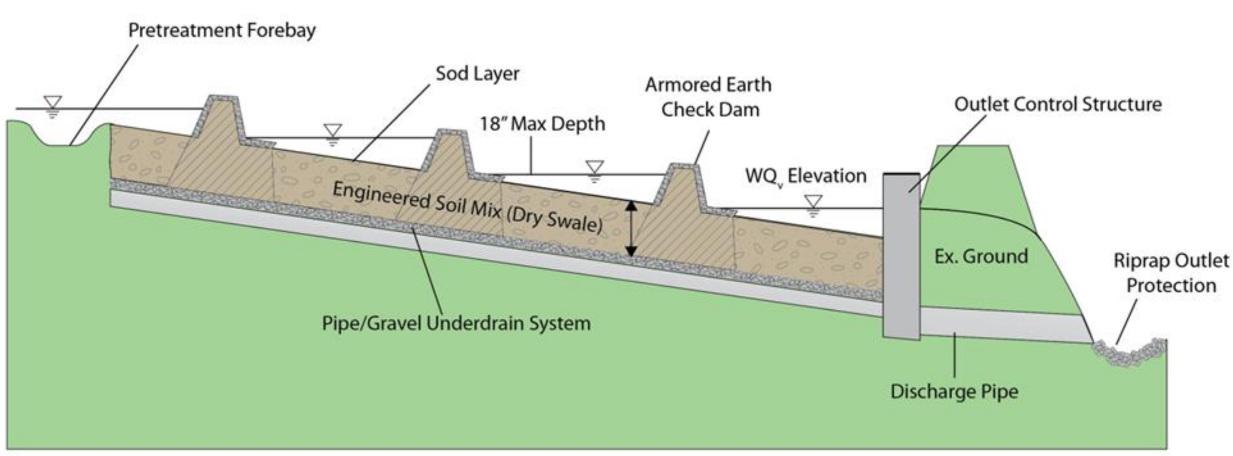
The 100-year storm should pass within the emergency spillway without overtopping the swale in any other location. Ensure non-erosive velocities for the 25-year event or the 50-year event if the swale is in a sag and armor 1 foot above this level. Refer to the guidance given in chapter 6 of the Drainage Design Policy Manual for assistance in sizing the channel and determining an appropriate lining material.

Refer to the Enhanced Dry Swale Outlet Structure Special Construction Detail for required tables to be provided in the special grading plans for an enhanced dry swale outlet structure.

Check Dams

In areas where the surrounding terrain is too steep to maintain 1% to 4% swale slopes, check dams may be incorporated into the design to flatten out sections of the swale. Check dams should be sized/spaced to contain and distribute the design volume across the length of the channel. If utilized, proper outlet and energy dissipation is required to prevent the erosion or failure of the downstream swale adjacent to the check dam. Refer to the Check Dams Special Construction Detail for more information. An example profile of an enhanced dry swale with check dams is shown in Figure 2.6.3-3.

Figure 2.6.3-3 - Example profile utilizing check dams



Access and Driveway Considerations

Adequate access to all elements of the enhanced dry swale must be included in the design to allow for inspection and maintenance. See section 2.10.3 for maintenance access requirements. Driveway crossings can also be located within the limits of the enhanced dry swale, as long as the effective surface area of the filter media is adjusted to account for the driveway.

Signage

The designer shall specify the installation of BMP signs consistent with GDOT's BMP Signs Special Construction Detail.

Enhanced Dry Swale Sizing

1. Determine the goals and primary function of the enhanced dry swale.

   The goals and primary function of the BMP must take into account any restrictions or site-specific constraints. Also take into consideration any special surface water or watershed requirements.

   - Consider if the BMP can be "oversized" to include the channel protection volume.
   - Size flow diversion structure, if needed

2. Determine if the enhanced dry swale will be on-line or off-line. If the enhanced swale will be off-line, a flow regulator (or flow splitter diversion structure) should be supplied to divert the WQ_v, to the swale. The design storm peak flow is needed for sizing an off-line diversion structure. See section 2.8.2 for more information on bypass structures. See section 2.4.1.2 for more information on calculating the water quality volume peak flow.

3A. Calculate the Target Water Quality Volume.

Calculate the water quality volume formula using the following formula:

$$WQ_v = \frac{1.2\ in \times (R_v) \times A \times 43560 \frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

Where:

WQ_v = water quality volume (ft^3)
R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.
A = onsite drainage area of the post-condition basin (acres)

3B. Determine the storage volume of the practice and the pretreatment volume

The actual volume provided in the enhanced swale is calculated using the following formula:

$$VP = PV + VES(N_{ES}) + VA(N_A)$$

Where:

VP = volume provided (temporary storage)
PV = ponding volume
VES = volume of engineered soils
N_{ES} = porosity of engineered soil (For enhanced dry swales, use 0.25)

VA = volume of aggregate
N_A = porosity of aggregate (use 0.4)

Note that if the BMP is being sized for CP_v, the required storage volume for CP_v calculated per section 2.4.2 will replace the WQ_v in the formula above.

If check dams are needed use erosion control sediment storage calculation equation to find the ponding volume within each segment.

Provide pretreatment by using a grass filter strip, as needed (sheet flow), or a forebay (concentrated flow). Where filter strips are used, 100% of the runoff should flow across the filter strip. Pretreatment is also necessary to reduce flow velocities and assist in sediment removal and maintenance. Pretreatment can include a forebay, weir, or check dam. Splash blocks or level spreaders should be considered to dissipate concentrated stormwater runoff at the inlet and prevent scour. Forebays should be sized to contain 0.1 inches per impervious acre of contributing drainage.

3C. Verify total volume provided by the practice is at least equal to the $WQ_{v(target)}$

When the VP ≥ $WQ_{v(target)}$ then the treatment requirements are met for this practice. When the VP < $WQ_{v(target)}$, then the design must be adjusted.

3D. Verify that the enhanced swale will drain in the specified timeframes.

The ponding volume of the enhanced dry swale must drain in less than 48 hours.

$$t_f = \frac{PV(d_f)}{k(h_f + d_f)A_f}$$

Where:

t_f = drain time (days)
PV = ponding volume (ft^3)
d_f = filter media depth (ft)
k = hydraulic conductivity (2-4 ft/day)
h_f = average water depth (ft)
A_f = top surface area of filter media (ft^2)

Note that if the BMP is being sized for CP_v, the required storage volume for CP_v calculated per section 2.4.2 will replace the WQ_v in the formula above.

4. Check 2-year and 25-year velocity erosion potential, if the BMP is online.

Check for erosive velocities and modify design as appropriate. Ensure non-erosive velocities for the 25-year event or the 50-year event if the swale is in a sag and armor 1 foot above this level.

5. Confirm the swale can pass all design requirements with required freeboard.

Refer to chapter 6 of the Drainage Design Policy Manual for freeboard requirements.

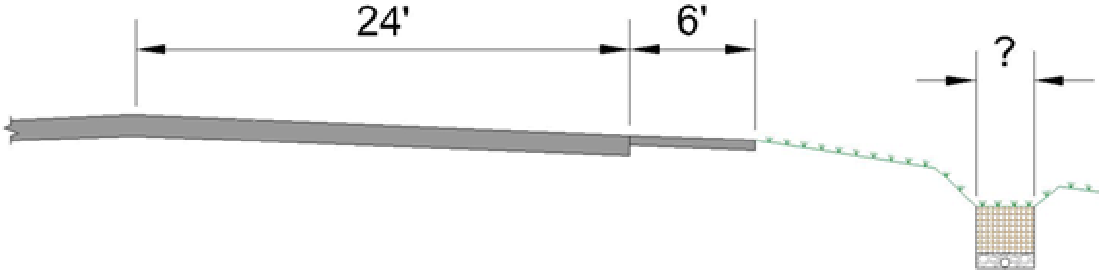
6. Design outlet control structure and emergency overflow.

An overflow must be provided to bypass and/or convey larger flows to the downstream drainage system or stabilized watercourse. The 100-year storm should pass within the emergency spillway without overtopping the swale in any other location. Non-erosive velocities need to be ensured at the outlet point.

Enhanced Dry Swale Example Calculation

GIVEN:

- A roadway widening project located in Savannah, Georgia.
- The proposed project includes 1,500 feet of roadway (in length).
- Approximately 325 feet is available for an enhanced dry swale; good vegetative cover can be established and maintained upgradient of the proposed BMP. Longitudinal slope of 1%.
- 20 feet of available width will be present in the typical section for installation of the enhanced dry swale.
- The site meets all other site constraints such that an enhanced dry swale is appropriate for use.
- The designer has previously calculated the following hydrologic information:
  - WQ_v = 4,312 cuft



FIND:

- The enhanced dry swale size and configuration that meets the site constraints and treats the WQ_v.

SOLUTION:

1. Since the longitudinal slope is 1% and the check dam height is 1.5 ft, check dams will be added at least every 150 ft.
2. The water quality volume was already calculated as 4.312 cuft.
3. The next step is to determine the storage volume of the practice. To complete this step, use the area available as a starting point for the surface area of the enhanced dry swale. In this example, approximately 20 feet by 325 feet are available for the enhanced dry swale. It is recommended that a software program and/or BMP sizing calculator spreadsheet be used at this point. The volume provided by the BMP is calculated using the following formula:

$$VP = PV + VES(N_{ES}) + VA(N_A)$$

Where:

VP = volume provided (temporary storage)
PV = ponding volume

VES = volume of engineered soils
N_{ES} = porosity of engineered soil (For enhanced dry swales, use 0.25)
VA = volume of aggregate
N_A = porosity of aggregate (use 0.4)

Therefore, at least an estimate of the following values is required to calculate the storage volume of the BMP:

- Bottom width of the swale
- Engineered soil mix depth
- Aggregate layer depth

For the purposes of this example, the following values are used as a starting point for sizing the basin. A bottom width of 6.5 ft, 4:1 slopes, a 1% longitudinal slope, and 150 ft long segments:

Volume of ponding per segment:

$(6.5\,ft\ \times 0.01\ \times (150\,ft)^2 \div 2) + \left((0.01)^2 \times \frac{(150\,ft)^3}{6 \times 0.25}\right) + \left((0.01)^2 \times \frac{(150\,ft)^3}{6 \times 0.25}\right)$ = 1181.25 cuft

- Engineered soil mix depth = 30 inches = 2.5 ft
- Aggregate layer depth = 12 inches = 1 ft
- Solve for $VP = PV + VES(N_{ES}) + VA(N_A)$
  - $4{,}312\ cuft \leq (1{,}181.25\ cuft\ \times \#\ segments) + (2.5ft \times 6.5ft \times length \times 0.25) + (6.5ft \times length \times 0.4)$
  - $\#\ segments = length\ \div 150ft$
  - Solution is a swale of 300 ft length, providing 4,361 cuft of WQ_v

As a factor of safety, the void space in the No. 8/No. 89 layer is not part of the storage calculations. This additional volume can serve as a safety buffer for storage in heavy rainfall.

A forebay is the chosen pretreatment method for this enhanced dry swale. Forebays should be sized to contain 0.1 inches per impervious acre of contributing drainage. The required forebay volume is 404 ft^3.

4. Verify the ponded volume will drain in less than 48 hours.

$$t_f = \frac{PV(d_f)}{k(h_f + d_f)A_f}$$

Where:

A_f = top surface area of filter media (1,950 ft^2)
P_V = ponding volume (2,362.5 ft^3)
d_f = filter media depth (2.5 ft)
k = hydraulic conductivity (2 ft/day)
h_f = average water depth (0.75 ft)
t_f = drain time (days)

$$t_f = \frac{2{,}362.5(2.5)}{2(0.75 + 2.5)1{,}950} = 0.347\ days = 11.28\ hours$$

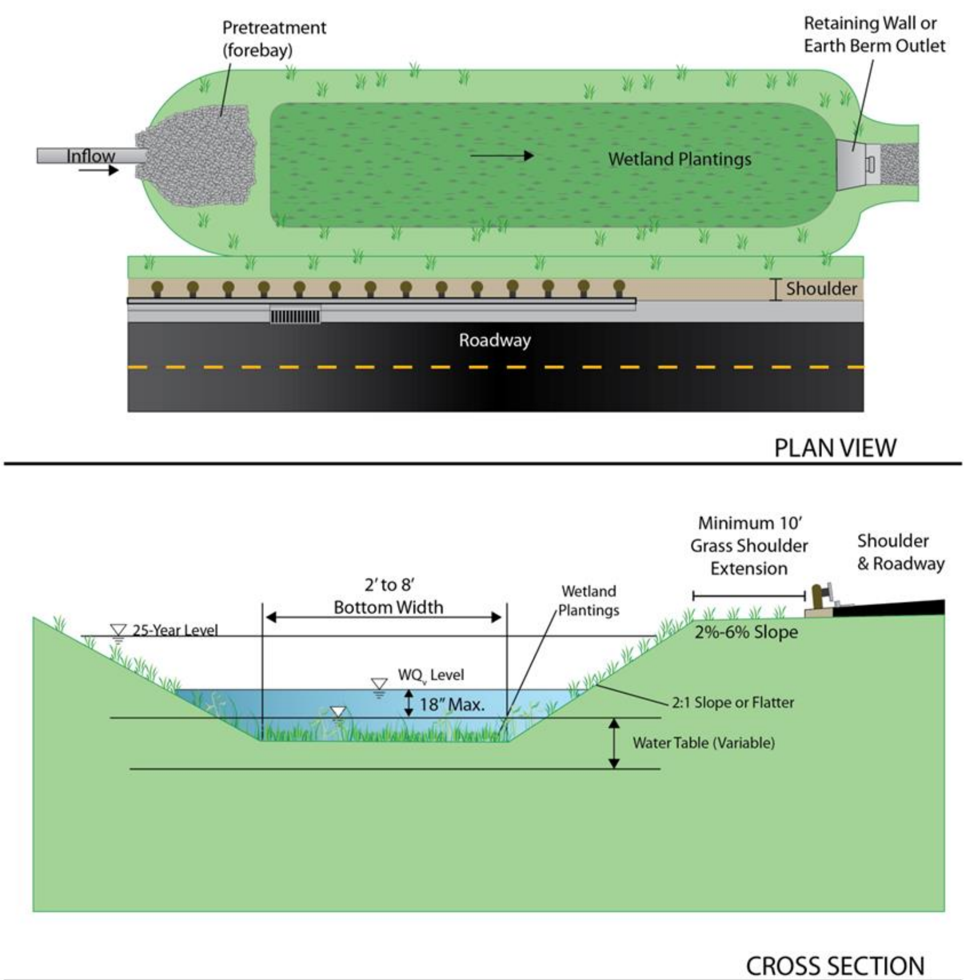
Additional design steps:

1. Check the 2-year and 25-year velocity erosion potential and freeboard.
2. Design outlet control structure and emergency overflow.
3. Size flow diversion structure, if needed.

Design Elements – Enhanced Wet Swale

An enhanced wet swale is designed to retain the WQ_v within the BMP in support of wetland vegetation. Wet swales achieve pollutant removal from the water quality volume through sediment accumulation and biological removal. Wet swales are sized to retain the entire WQ_v with less than 18 inches of ponding above the high water table at the maximum depth point. The outlet control structure in the wet swale includes an orifice that is sized to allow the WQ_v to draw down in a time frame less than 48 hours. Enhanced wet swales do not provide runoff reduction volume credits. The design elements specific to an enhanced wet swale are discussed below and illustrated in Figure 2.6.3-4.

Figure 2.6.3-4 - Enhanced wet swale schematic



One design characteristic, unique to a wet enhanced swale, is the grass shoulder extension as shown in Figure 2.6.3-4. To prevent slope instability along the front slope of the wet swale, a minimum of 10 feet must be provided between the edge of pavement, or paved shoulder, and the slope of the wet enhanced swale.

Outlet Control Structure

A low flow orifice should be incorporated into the outlet structure to allow for the drainage of the water quality volume in less than 48 hours. The orifice elevation shall be a maximum of 18 inches above the high water table. The following orifice equation is used to determine the size of the orifice:

$$Q = C_d \times A \times (2gh)^{0.5}$$

(2.6.3- 3)

Where:

Q = Peak flow (ft^3/s)
C_d = Orifice coefficient = 0.6
A = Area of orifice (ft^2)
g = Gravitational constant (32.2 ft/sec^2)
h = Depth of water to center of orifice (ft)

The low flow orifice invert shall be a maximum of 18 inches below the overflow weir and a maximum of 6 inches above the channel elevation at the outlet structure.

In addition, an overflow weir should be designed to allow the enhanced wet swale to safely pass the 25-year, 24-hour storm event with a minimum 6 inches of freeboard. If using the retaining wall outlet structure, the freeboard shall be measured from the 25-year, 24-hour storm event to the top of retaining wall.

The following weir equation is used to determine weir length of a broad-crested weir. [(2-32)]

$$Q = C_d \times L \times H^{\frac{3}{2}}$$

(2.6.3- 4)

Where:

Q = Peak flow (ft^3/s)
C_d = Weir coefficient = 2.6
L = Length of weir (ft)
H = Depth of water above weir crest (ft)

If using the retaining wall outlet structure, the maximum width of weir shall be 2 feet less than the width of the concrete splash pad.

If using the earth berm outlet structure, the minimum width of the weir shall be 2-feet and the maximum width of the weir shall be 8-feet.Retaining wall outlets shall have a minimum 10-feet wide concrete splash pad downstream of the overflow weir. A rip rap apron shall extend from the downstream edge of the concrete splash pad a minimum distance of 5-feet

Finally, the outlet control structure or outlet conveyance channel must also be designed to adequately carry the extreme flood protection volume (100-year, 24-hour rainfall event). Refer to the Enhanced

Wet Swale Outlet Structure Special Construction Detail for more information and required tables to be provided in the special grading plans for an enhanced wet swale.

Water Balance

Enhanced wet swales must be designed to maintain a permanent pool. Install an impermeable liner if the enhanced wet swale is located on HSG A or B soils and the swale does not intercept the groundwater table. A water balance analysis should be performed for systems on HSG C and D soils. Refer to section 2.2.4 for water balance calculations.

Embankment

The top of the enhanced wet swale shall have an 4 feet wide berm or bench graded all around the basin, both in cut and in fill sections. The top of the berm or bench may be sloped up to 4% towards the inside of the swale.

Landscaping Plan

A landscaping plan should be included as part of the complete design for the enhanced wet swale. The landscaping plan should specify how the enhanced wet swale will be stabilized and vegetation established. It should specify proper grass and wetland plants based on specific site soils and hydric conditions. Refer to GDOT Planting Schedule Special Construction Detail and Special Provision / Specification 169 on Post Construction Stormwater BMP Items for more information.

Access and Driveway Considerations

Adequate access to all elements of the enhanced wet swale should be included in the design to allow for inspection and maintenance. See section 2.10.3 for maintenance access requirements. Driveway crossings can also be located within the limits of the enhanced wet swale, as long as the WQ_v is adjusted to account for the driveway.

Signage

The designer shall specify the installation of BMP signs consistent with GDOT's BMP Signs Special Construction Detail.

Enhanced Wet Swale Sizing

1. Determine the goals and primary function of the enhanced wet swale.

   The goals and primary function of the BMP must take into account any restrictions or site-specific constraints. Also take into consideration any special surface water or watershed requirements.

   - Enhanced wet swales must be designed for the target water quality volume because they do not provide runoff reduction volume credits.
   - Consider if the BMP can be "oversized" to include the channel protection volume.

2. Size flow diversion structure, if needed.

   Determine if the enhanced wet swale will be on-line or off-line. If the enhanced wet swale will be off-line, a flow regulator (or flow splitter diversion structure) should be supplied to divert the WQ_v to the swale. The design storm peak flow is needed for sizing an off-line diversion structure. See section 2.8.2 for more information on bypass structures.

3. Calculate the Target Water Quality Volume

   Calculate the water quality volume formula using the following formula:

$$WQ_v = \frac{1.2\ in \times (R_v) \times A \times 43560\,\frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

Where:

WQ_v = water quality volume (ft^3)
R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.
A = onsite drainage area of the post-condition basin (acres)

4. Determine channel geometry and pretreatment volume required.

   Size bottom width, depth, length, and slope necessary to treat the water quality volume with less than 18 inches of ponding at the downstream end.

5. Check 2-year and 25-year velocity erosion potential, if the BMP is online.

   Check for erosive velocities and modify design as appropriate. Ensure non-erosive velocities for the 25-year event or the 50-year event if the swale is in a sag and armor 1" above this level.

6. Confirm the swale can pass all design requirements with required freeboard.

7. Design the outlet control structure and emergency overflow.

   The orifice should be sized to allow the WQ_v to drain down within 48 hours.

   Determine the flow rate of the WQ_v discharging from the swale within 48 hours.

$$Q = \frac{WQ_v}{(24\ hours)\left(60\ \frac{min}{hr}\right)\left(60\,\frac{s}{min}\right)}$$

   Determine the size of the orifice that allows the WQ_v to drain down within 48 hours.

$$Q = C_d \times A \times (2gh)^{0.5}$$

Where:

Q = Peak flow (ft^3/s)
C_d = Orifice coefficient = 0.6
A = Area of orifice (ft^2)
g = Gravitational constant (32.2 ft/s^2)
h = Depth of water to center of orifice (ft)

   The diameter of the orifice shall be determined as follows:

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

   If the BMP is online, an overflow must be provided to bypass and/or convey larger flows to the downstream drainage system or stabilized watercourse. The 100-year storm should pass within the emergency spillway without overtopping the swale in any other location. Non-erosive velocities need to be ensured at the outlet point. The overflow should be sized to safely pass the peak flows anticipated to reach the practice, up to a 100-year storm event.

The overflow weir should be designed to allow the enhanced wet swale to safely pass the 25-year, 24-hour storm event with a minimum 6 inches of freeboard. The following weir equation is used to determine weir length of a broad-crested weir.

$$Q = C_d \times L \times H^{\frac{3}{2}}$$

Where:

Q = Peak flow (ft^3/s)
C_d = Weir coefficient = 2.6
L = Length of weir (ft)
H = Depth of water above weir crest (ft)

8. <u>If applicable, complete a water balance analysis to verify the enhanced wet swale will maintain its permanent pool.</u>

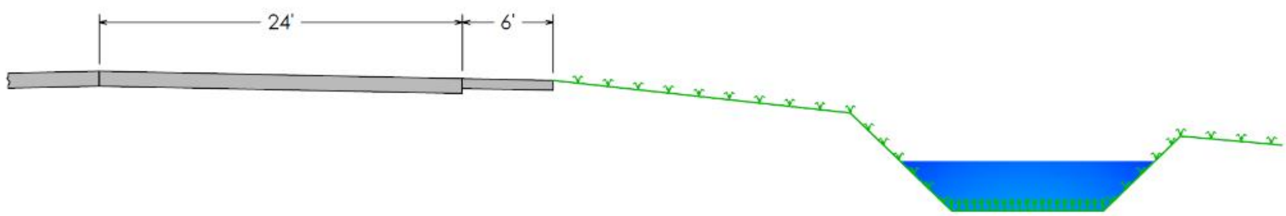
9. <u>Prepare a vegetation and landscaping plan</u>

A landscaping plan for an enhanced wet swale should be prepared to indicate how vegetation will be established. See the Vegetation section above and the GDOT Planting Schedule Special Construction Detail for additional guidance.

Enhanced Wet Swale Example Calculation

GIVEN:

- A new roadway project located in Dallas, Georgia.
- The proposed project includes 1,500 feet of roadway (in length).
- Approximately 300 feet is available for enhanced wet swale; good vegetative cover can be established and maintained upgradient of the proposed BMP.
- The drainage area that discharges to the enhanced wet swale includes the following: two 12-foot lanes, a 6-foot paved shoulder, and a 20-foot wide grassed area, draining via sheet flow.
- 18 feet of available width will be present in the typical section for installation of the enhanced wet swale.
- The maximum permanent pool depth is 6 inches.
- The site meets all other site constraints such that an enhanced wet swale is appropriate for use.
- The designer has previously calculated the following hydrologic information:
    - $WQ_v = 4{,}548\ ft^3$
    - $Q_{wq} = 1.5\ ft^3/s$
    - $Q_{p25} = 10.7\ ft^3/s$



FIND:

- The enhanced wet swale size and configuration that meets the site constraints and provides the required water quality treatment.

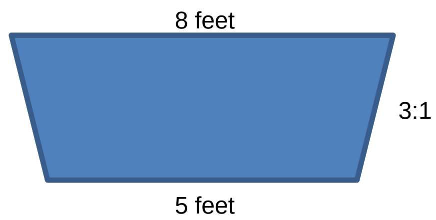
SOLUTION:

1. The enhanced wet swale will be sized for the water quality volume.

2. The water quality volume was already calculated to be 4,548 ft^3.

3. Based on the existing ground geometry, the enhanced wet swale will utilize a length of 300 feet. Size bottom width, depth, and side slopes necessary to treat the water quality volume with less than 18 inches of ponding at the downstream end.

   Assume a base width of 2 feet and side slopes of 3:1. With a permanent pool depth of 0.5 foot (6 inches), the top width of the permanent pool is 8 feet.



The basic volume formula for a trapezoidal prism is:

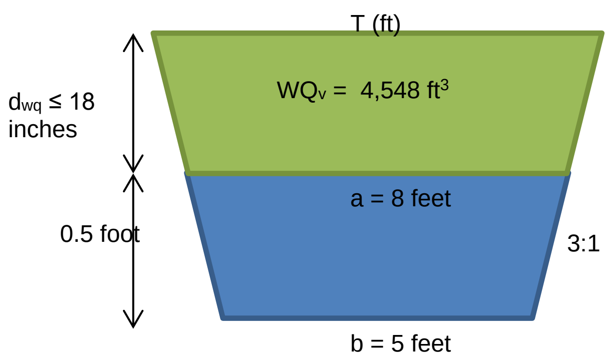
$$V = L \times \left[h \times \frac{(a+b)}{2}\right]$$

Where:

V = Volume of trapezoidal prism
L = Length of prism
h = Height of trapezoid
a = Top length of trapezoid
b = Bottom length of trapezoid

The top width of the channel, T, is a function of the water quality volume depth.

$$T = a + (2 \times 3 \times d_{wq})$$



Solve for the water quality volume depth.

$$4{,}548 = 300 \times \left\{ d_{wq} \times \frac{[8 + (2 \times 3 \times d_{wq})] + 8)}{2} \right\}$$

$$d_{wq} = 1.3\,ft\ (ok\ because\ less\ than\ 18\ inches) \rightarrow round\ up\ to\ 1.5\,ft$$

Check that the top width is less than the 18 feet available.

$$T = 8 + (2 \times 3 \times 1.5) = 17\,feet$$

4. Next, check the 25-year velocity erosion potential and freeboard.
   - Q_{p25} = 10.7 ft^3/s
   - n = 0.1 (assumed for this example)
   - Flow velocity = 1.3 ft/sec **Non-erosive (less than 4 ft/sec) OK**
   - Flow depth = 1.4 ft

   Add 0.5 feet to the flow depth for freeboard and 0.5 foot for the permanent pool to get overall channel depth equal to 2.4 feet.

   Verify that channel design meets all design requirements as outlined in the open channel design policy as outlined in chapter 6 of the Drainage Design Policy Manual.

5. Determine the flow rate of the WQ_v discharging from the swale within 48 hours:

$$Q = \frac{Volume}{Time} = \frac{4{,}548\,ft^3}{(48\,hours)\left(60\,\frac{min}{hr}\right)\left(60\,\frac{s}{min}\right)} = 0.0265\,ft^3/s$$

Next, the following orifice equation is used to determine the size of the orifice:

$$Q = C_d \times A \times (2gh)^{0.5}$$

Where:

Q = Peak flow = 0.0265 ft^3/s
C_d = Orifice coefficient = 0.6
A = Area of orifice (ft^2)
g = Gravitational constant (32.2 ft/s^2)
h = Depth of water to center of orifice (ft) = 1 ft

The area of the orifice is calculated to be 0.0055 ft^2, or 0.79 in^2.

The diameter of the orifice shall be determined as follows:

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

$$d = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4(0.79)}{3.14}} = 1.00\,inches$$

Therefore, the diameter of the orifice is specified to be 1.00 inch.

Additional design steps:

1. Determine the specifics for the forebay (length, width, depth and stone size).
2. Specify the width of the overflow weir and determine the 25-year maximum stage based on the height of flow at the overflow weir.
3. Determine the top and bottom elevations for the outlet control structure.
4. Determine the size of the discharge pipe.
5. Verify that the outlet structure or discharge conveyance channel can safely convey and discharge the 100-year storm event without overtopping the swale in any other location.
6. If applicable, complete a water balance analysis.
7. Prepare a vegetation and landscaping plan.

Maintenance Considerations

Without proper maintenance, BMPs will function at a reduced capacity and may cease to function altogether. A properly designed BMP includes several considerations for maintenance:

- Provide adequate right-of-way.
- Provide access roads and ramps for appropriate equipment to all applicable components (outlet structure, forebay, etc.).
- Provide space to turn around if necessary.
- Check for sufficient area to safely exit and enter the highway, if applicable.
- Provide a valve or other method for dewatering an enhanced wet swale.

Refer to GDOT's *Stormwater System Inspection and Maintenance Manual*, for specific maintenance requirements.

Summary

2.6.4 Infiltration Trench

Advantages	Disadvantages
• Considered a LID/GI control • Provides for groundwater recharge • Good for small sites with porous soils	• Potential for groundwater contamination • Only suitable for smaller drainage areas (5 acres or less) • High clogging potential; should not be used on sites with fine-particled soils (clays or silts) in drainage area • Significant setback requirements • Geotechnical testing required

Description: Infiltration trenches are shallow trenches comprised of an underground reservoir of large crushed stone. The runoff volume slowly exfiltrates (exits the device by infiltrating into the soil) through the bottom and sides of the trench into the subsoil, eventually reaching the water table.

Design Considerations:

- Soil infiltration rate of 0.5 in/hr or greater required
- Excavated trench (2 to 10-foot depth) filled with stone media (1.5 to 2.5-inch diameter); pea gravel and sand filter layers
- Trench is wrapped in non-woven plastic filter fabric (top and sides)
- A forebay, or equivalent upstream pretreatment measure, must be provided.
- Observation well(s) to monitor percolation
- Must not be placed under pavement or concrete

Maintenance Considerations:

- Clogging is a significant concern; locate only in stabilized areas
- Ensure observation well is easily and safely accessible

Applicability for Roadway Projects

- Subsurface drainage direction must be away from the subbase of adjacent roadway or impervious paved area
- Linear nature lends itself well to roadway applications

Direct coordination with the Water Resources Group is required if an Infiltration Trench is determined to be feasible. This coordination needs to occur before submittal of the draft Post Construction Stormwater Report.

Stormwater Management Suitability:

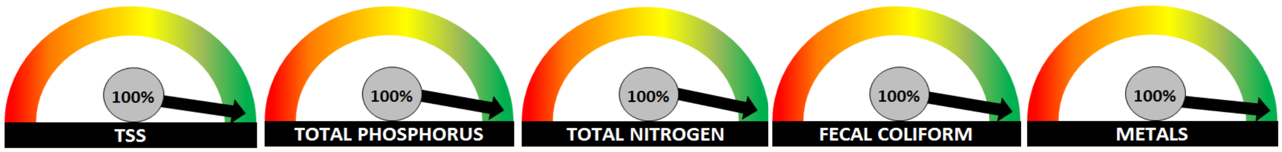
✓ Runoff Reduction
✓ Water Quality
○ Channel Protection
X Overbank Flood Protection
X Extreme Flood Protection
✓ Temperature Reduction

✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

Infiltration trenches are considered a LID/GI control. They have the ability to recharge groundwater, which helps to restore a site's natural hydrology.

Treatment Capabilities



2.6.4 Infiltration Trench

Description

Infiltration trenches are shallow trenches comprised of an underground reservoir of large crushed stone. The runoff volume slowly exfiltrates (exits the device by infiltrating into the soil) through the bottom and sides of the trench into the subsoil over a 2 to 3-day period, eventually reaching the water table. Infiltration trenches must always be designed with pretreatment measures as they can clog easily. Forebays are often utilized as pretreatment. In addition, some other BMPs such as grass channels and filter strips can be used in series with an infiltration trench to protect it from clogging. For runoff from large storm events, an overflow outlet, such as a berm or level spreader, is needed for stormwater that cannot be fully infiltrated by 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 channel protection volume, CP_v. Peak rate control of the 10-year and greater storm events is typically beyond the capacity of an infiltration practice.

By infiltrating runoff into the soil, infiltration trenches serve multiple LID/GI functions including treating the water quality volume, helping to preserve the site's natural water balance, and recharging groundwater. These benefits must be weighed against the tendency for infiltration trenches to become clogged. They should only be incorporated into sites where upstream sediment can be controlled or the upstream drainage area is built out or well stabilized.

Careful attention must be given to avoid siting infiltration trenches where there is potential for groundwater contamination. Also, they cannot be utilized in areas having karst (i.e., limestone) topography as there is potential for sink holes to develop. Figure 2.6.4-1 shows typical infiltration trench components and Figure 2.6.4-2 shows the typical layout of an infiltration trench.

Figure 2.6.4-1 - Typical infiltration trench components

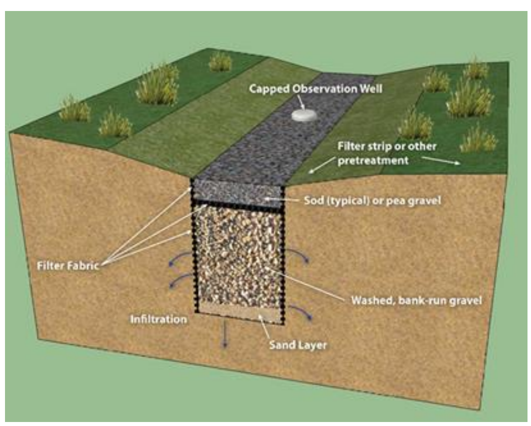
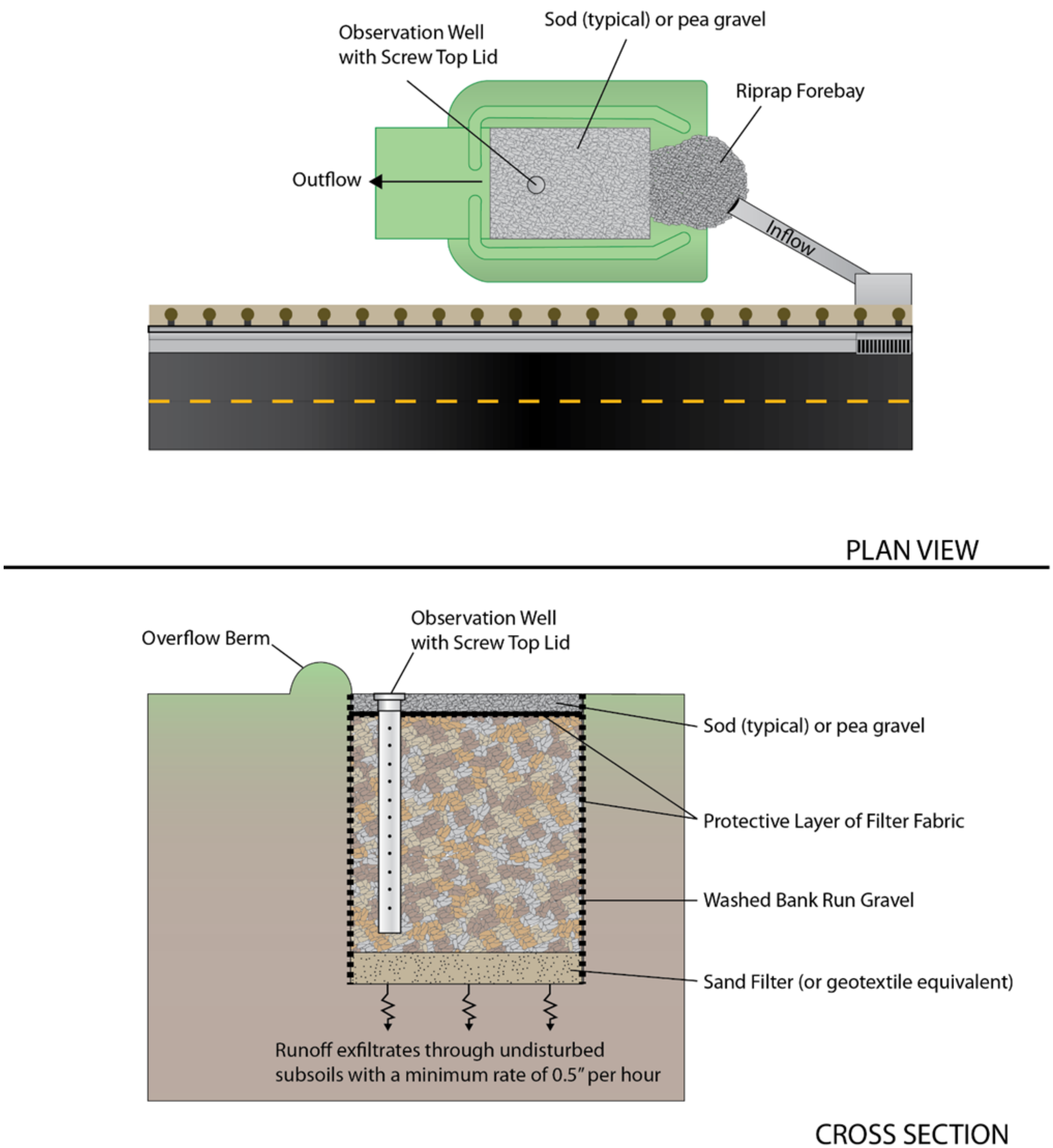


Figure 2.6.4-2 - Typical infiltration trench – plan and profile views



Stormwater Management Suitability

- Runoff Reduction – Infiltration trenches are one of the most effective low impact development (LID) practices that can be used in Georgia to reduce post-construction stormwater runoff and improve stormwater runoff quality. Like other LID practices, they become more effective with higher infiltration rates of native soils. An infiltration trench can be designed to provide 100% of the runoff reduction volume, if properly maintained.

- Water Quality – The infiltration trench is an excellent stormwater treatment practice due to the variety of pollutant removal mechanisms. Each of the components of the infiltration trench is designed to perform a specific function. The grass filter strip (for sheet flow) or grass channel or forebay (for concentrated flow) pre-treatment component reduces incoming runoff velocity and filters particulates from the runoff. The planting soil or rock in the infiltration practice acts as a filtration system, and clay in the soil provides adsorption sites for hydrocarbons, heavy metals, nutrients and other pollutants. An infiltration trench provides 100% TSS removal if designed, constructed, and maintained correctly.
- Channel Protection – For smaller sites, an infiltration trench may be designed to capture the entire channel protection volume (CP_v). Given that an infiltration trench is typically designed to completely drain over 48-72 hours, the requirement of extended detention for the 1-year, 24-hour storm runoff volume will be met. For larger sites, or where only the WQ_v is diverted to the infiltration trench, another control must be used to provide CP_v extended detention.
- Overbank Flood Protection – Another control will likely be required in conjunction with an infiltration trench to reduce the post-development peak flow of the 25-year storm (Q_{p25}) to pre-development levels (detention).
- Extreme Flood Protection – Infiltration trenches must provide flow diversion and/or be designed to safely pass extreme storm flows.
- Temperature Reduction – Infiltration trenches can provide for temperature reduction.

Pollutant Removal Capabilities

The following average pollutant removal rates may be utilized for design purposes:

- TSS – 100%
- TP – 100%
- TN – 100%
- Fecal coliform – 100%
- Heavy metals – 100%
- Temperature – Temperation reduction is provided.

Application and Site Suitability

Direct coordination with the Water Resources Group is required if an Infiltration Trench is determined to be feasible. This coordination needs to occur before submittal of the draft Post Construction Stormwater Report. Infiltration trenches can be utilized in locations where the subsoil is sufficiently permeable to provide a reasonable infiltration rate and a low water table exists to prevent groundwater contamination. Locating infiltration basins on linear projects in urban settings may not be appropriate as these areas tend to have compacted soils. They are applicable primarily for impervious drainage areas where there are not high levels of fine particulates (clay/silt soils) in the runoff and should only be considered for sites where the sediment load is relatively low. [(2-17)]

Infiltration trenches generally have a grassed or gravel surface. Infiltration trenches located adjacent to roadways or impervious paved areas must be designed so the subsurface drainage direction flows to the downhill side (away from the subbase of the pavement) or located lower than the impervious subbase layer. Proper measures should be taken to prevent water from infiltrating into the subbase of impervious pavement. [(2-31)]

Infiltration trenches can either be used to capture sheet flow from a drainage area or function as an off-line device. Due to the relatively narrow shape, infiltration trenches can be adapted to many different types of sites and can be used in retrofit situations. Unlike some other structural stormwater controls, they can fit into the perimeter or other unused areas of developed sites. Median strip infiltration trenches can be combined with a grass filter strip to direct sheet flow to the trench. Multiple trenches can be incorporated on larger sites or in the upland area of large sites to reduce the amount of runoff needing treatment downstream.

Infiltration trenches are frequently used to infiltrate runoff from adjacent impervious surfaces, such as parking lots. In these cases, a filter strip should be installed between the pavement and the device to trap sediment and litter before it is washed into the infiltration trench. Another approach is to construct infiltration devices at the downgradient edges of areas with permeable pavement. In this case, the permeable pavement is the inlet to the device. As water will also infiltrate through the base of the pavement, the size of the infiltration devices can be reduced significantly. [(2-28)]

In areas of high traffic or areas where excessive sediment, litter, and other similar materials may be generated, a pretreatment device (such as a forebay) and/or additional BMPs (such as a filter strip or grassed channel) are needed.

In site development applications, roof drains may be connected to infiltration trenches. Roof runoff generally has lower sediment levels and often is ideally suited for discharge through an infiltration trench. A cleanout with sediment sump should be provided between the building and infiltration trench.

Infiltration trenches are not suitable in areas with karst geology without adequate geotechnical testing by qualified individuals and must be installed in accordance with local requirements.

Siting information and constraints include the following:

- **Drainage Area –** The maximum drainage area to an infiltration trench is 5 acres.
- **Space Required –** Required spacing will vary, dependent upon the depth of the facility.
- **Site Slope** – No more than 6% site slope for preconstruction footprint. Infiltration trenches should be designed with slopes that are as close to flat as possible.
- **Minimum Head** – Elevation difference of 1 foot needed for minimum head at a site from the inflow to the outflow.
- **Depth to Water Table** – Four-foot depth recommended between the bottom of the infiltration trench and the elevation of the seasonally high water table, which may be reduced to 2 feet in coastal areas. The separation distance provided should allow the trench to empty completely within a maximum of 72 hours following a runoff producing event.
- **Infiltration Rate –** Infiltration rate greater than 0.5 inches per hour required (typically HSG A, some HSG B soils).

- Soils exhibiting a clay content of greater than 30% and a silt/clay content greater than 40% are unacceptable in order to prevent clogging and failure.
- Clay lenses, bedrock or other restrictive layers below the bottom of the trench will reduce infiltration rates unless excavated.

- **Setbacks** – See the following setback requirements. Confirm there are no local ordinances or criteria.
  - From a property line – 10 feet
  - From a building foundation – 20 feet downslope and at least 100 feet upslope
  - From a private well – 100 feet
  - From a public water supply well – 1,200 feet
  - From a septic system tank/leach field – 100 feet (notify health official if trench is placed in the vicinity of a septic leach field)
  - From surface waters – 100 feet
  - From surface drinking water sources – 400 feet (100 feet for a tributary)
- **Hotspots** – Do not use for hotspot runoff.
- **Trout Stream** – Runoff temperature reduction is provided.
- **Other Considerations** –
  - Infiltration trenches cannot be placed under pavement or concrete.
  - Infiltration trenches are designed for intermittent flow and must be allowed to drain and allow reaeration of the surrounding soil between rainfall events. They must not be used on sites with a continuous flow from groundwater, or other sources.[(2-17)]
  - Infiltration trenches should not be constructed on or near 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 expected when an infiltration BMP is proposed. [(2-38)]

General Design Criteria

Sizing and specification criteria include:

- Design to fully dewater the entire RR_v within 72 hours after the rainfall event.
- The bottom slope of the trench must be flat length-wise and width-wise to promote uniform infiltration.
- Generally, the trench's total depth ranges from 2 to 10 feet.
- The width of a trench should be less than 25 feet. Trench widths greater than 8 feet require large excavation equipment rather than smaller trenching equipment. Infiltration trenches that are broader and shallower are less likely to clog as they provide a larger area for infiltration.

- The infiltration trench material should be comprised of GDOT No. 3 aggregate. Aggregate contaminated with soil shall not be used. Use a porosity value, n, (void space/total volume) of 0.40 for GDOT No. 3 aggregate in calculations.
- A 6-inch deep layer of clean, washed sand must be installed at the bottom of the trench. This will promote drainage and prevent compaction of the native soil when the stone is added.
- The infiltration trench should be lined on the sides and top by appropriate non-woven plastic filter fabric capable of preventing surrounding soil piping and able to maintain a greater permeability than the surrounding native soil. The top layer of filter fabric should be located 2 inches from the top of the trench and serves to prevent sediment from passing into the stone aggregate. Since this top layer serves as a sediment barrier, it will need to be replaced more frequently and must be readily separated from the side sections.
- The top surface of the infiltration trench above the filter fabric should typically be covered with sod (typical) or pea gravel. The sod/pea gravel layer will improve sediment filtering and maximize the pollutant removal in the top of the trench. In addition, it can easily be removed and replaced should the device begin to clog.
- Refer to Special Provision / Specification 169 for more information.

The required storage volume is equal to the RR_v. For smaller sites, an infiltration trench can be designed with a larger storage volume to include the CP_v. Refer to section 2.4 of this chapter for guidance on calculating these volumes.

Note that it is often the case in roadway systems that length and particularly width are predetermined by constraints such as limited right-of-way and edge of pavement. Depth can often be adjusted to meet sizing requirements unless shallow groundwater or bedrock are present. Note that reduced surface area of the trench increases the likelihood of clogging and tends to yield less stormwater treatment.

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 check that the trench is emptying within the maximum allowable time of 72 hours. Generally, the observation well is constructed of perforated pipe and should extend to the bottom of the trench.

Pretreatment

Pretreatment facilities must always be used in conjunction with an infiltration trench to prevent clogging and failure. 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. Multiple pretreatment measures are recommended such as forebays, or other BMPs including grass channels and filter when implemented in series.

Where sheet flow enters the trench from an adjacent drainage area, the pretreatment system should consist of a vegetated filter strip with a minimum 25-foot length. A vegetated buffer strip around the entire trench is required if the facility is receiving runoff from all directions. If the infiltration rate for the underlying soils is greater than 2 inches per hour, 50% of the RR_v should be pretreated by another method prior to reaching the infiltration trench.

For off-line configurations, pretreatment should consist of a forebay sized to 25% of the WQ_v. Exit velocities from the pretreatment must be nonerosive for the 25-year design storm. See section 2.5.4 for additional information on off-line configurations.

Vegetation

Refer to GDOT Special Provision / Specification 169 – Post-Construction Stormwater BMP Items for guidance on selecting and placing sod.

Emergency Spillway

Because of the small drainage area served by an infiltration trench, an emergency spillway is typically not required; however, a non-erosive overflow channel or storm sewer system must be located at the downstream end of the trench. If an overflow berm surrounding the infiltration trench is incorporated into the design, the emergency spillway can be a depressed portion of the overflow berm, acting as weir, discharging flows in excess of the RR_v to the channel or storm sewer downgradient. Overflow berms are sized to contain the RR_v within the infiltration trench, preventing stormwater from bypassing across or around its surface.

Access and Driveway Considerations

See section 2.10.3 for maintenance access requirements.

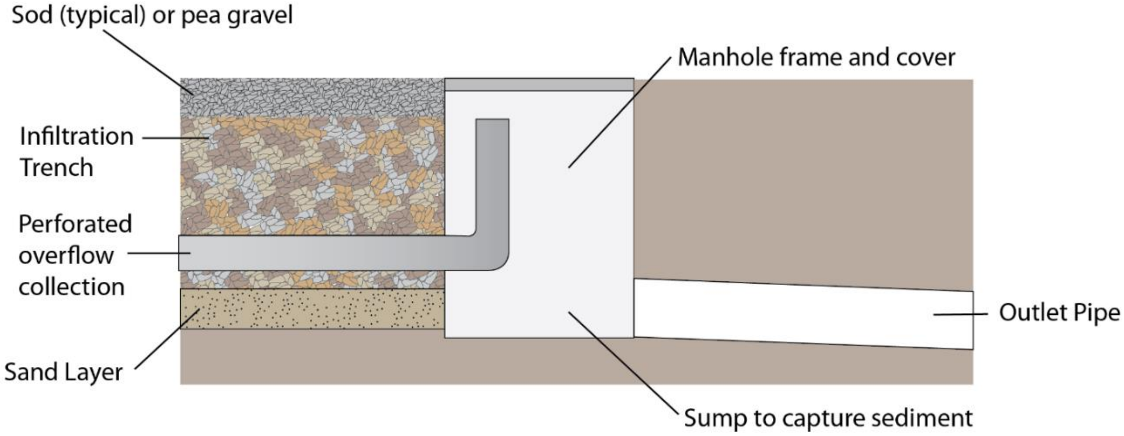
Alternative Design Options

For off-line infiltration trench configurations, the RR_v is diverted to the infiltration trench through the use of a flow bypass structure (see section 2.8.2 of this chapter for guidance on flow bypass structure design). Where stormwater flows are greater than the RR_v, divert the flow to other controls or downstream using a diversion structure or flow splitter.

Provisions for Overflow

Provisions for overflow may be needed for undersized infiltration trenches or for infiltration trenches that treat larger drainage areas. Overflow configurations can include a perforated pipe system with up-turned vertical section, elevated catch basin (similar to a riser), or an emergency spillway channel. The perforated pipe system should be designed similar to an underdrain as presented in section 2.8, Common BMP Components. Pipe material should be polyethylene and consistent with GDOT Specification 573. The pipe and perforations should be sized to convey the desired peak flow (refer to Chapter 5, Table 5.1 of the Drainage Design Policy Manual). See Figure 2.6.4-3 for an example.

Figure 2.6.4-3 - Design example of an infiltration trench overflow system



If an elevated catch basin is used, the rim of the catch basin should be set at the desired design storm elevation and will perform the same function as a riser structure would in a detention pond. Large riser structures are typically not required in infiltration trenches because they typically treat smaller drainage areas. Additional information for both risers and emergency spillway channels can be found in section 2.7, Detention Design.

Infiltration Trench Sizing

1. Determine the goals and primary function of the infiltration trench.

   The goals and primary function of the BMP must take into account any restrictions or site-specific constraints. Also take into consideration any special surface water or watershed requirements.

   - An infiltration trench should be sized to meet the runoff reduction target. *Minimum infiltration rates of the surrounding native soils must be acceptable.*
   - Consider if the BMP can be "oversized" to include the channel protection volume.

2. Calculate the Stormwater Runoff Reduction Target Volume.

$$RR_{v(target)} = \frac{1\,in \times (R_v) \times A \times 43560\,\frac{ft^2}{acre}}{12\,\frac{in}{ft}}$$

   Where:

   $RR_{v(target)}$ = runoff reduction target volume (ft^3)
   A = area draining to this practice (acres)
   R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.

3. Determine if the infiltration trench will be on-line or off-line.

   If the infiltration trench will be off-line, a flow regulator (or flow splitter diversion structure) should be incorporated into the design to divert the RR_v to the infiltration trench. The design storm peak flow is needed for sizing an off-line diversion structure. See section 2.8.2 for more information on bypass structures.

4. Determine the storage volume of the practice and the pretreatment volume

   The actual volume provided in the infiltration trench is calculated using the following formula:

$$VP = PV + VA(N_A)$$

   Where:

   VP = volume provided (temporary storage)
   PV = ponding volume
   VA = volume of aggregate
   N_A = porosity of aggregate (use 0.4)

   Provide pretreatment by using a grass filter strip, as needed (sheet flow), or a grass channel or forebay (concentrated flow). Where filter strips are used, 100% of the runoff should flow across the filter strip. Pretreatment is also necessary to reduce flow velocities and assist in sediment removal and maintenance. Pretreatment can include a forebay, weir, or check dam. Splash blocks or level spreaders should be considered to dissipate concentrated stormwater runoff at the inlet and prevent scour. For off-line configurations, pretreatment should consist of a forebay sized to 25% of the WQ_v. Otherwise, forebays should be sized to contain 0.1 inches per impervious acre of contributing drainage.

5. Verify the total volume provided by the practice is at least equal to the $RR_{v(target)}$

   When the VP ≥ $RR_{v(target)}$ then the runoff reduction requirements are met for this practice. When the VP < $RR_{v(target)}$, then the design must be adjusted or another BMP must be selected and designed for the drainage area.

6. Verify that the infiltration trench will drain in the specified timeframes.

   Verify that the entire volume provided by the BMP will drain within 72 hours.

$$t_f = \frac{VP}{(k_{design})A_a}$$

Where:

t_f = drain time (days)
VP = total volume provided by practice (ft^3)
k_{design}= design infiltration rate of underlying soil (ft/day) The design infiltration rate is equal to the observed, in-situ, infiltration rate divided by the factor of safety (usually 1).
A_a = bottom surface area of aggregate (ft^2)

7. Design outlet control structure and emergency overflow

   An overflow must be provided to bypass and/or convey larger flows to the downstream drainage system or stabilized watercourse. Non-erosive velocities need to be ensured at the outlet point. The overflow should be sized to safely pass the peak flows anticipated to reach the practice, up to a 100-year storm event.

Maintenance Considerations

Without proper maintenance, BMPs will function at a reduced capacity and may cease to function altogether. A properly designed BMP includes the following considerations for maintenance:

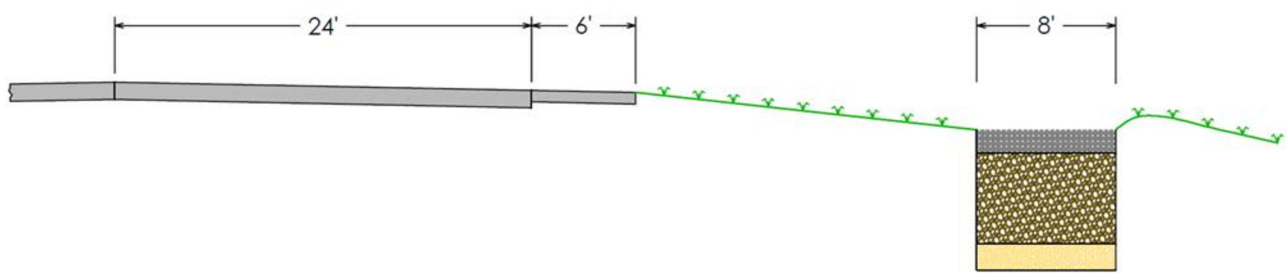
- Provide adequate right-of-way.
- Provide access roads and ramps for appropriate equipment to all applicable components (observation well, forebay, etc.).
- Provide space to turn around if necessary.
- Check for sufficient area to safely exit and enter the highway, if applicable.
- Provide an observation well at an interval of every 50 feet along the entire trench length to provide a means by which dewatering times can be observed.

Refer to GDOT's *Stormwater System Inspection and Maintenance Manual*, for specific maintenance requirements.

Infiltration Trench Example Calculation

GIVEN:

- A new roadway project located in Savannah, Georgia.
- The proposed project includes 1,500 feet of roadway (in length) that discharges into an impaired stream.
- Assume that approximately 300 feet is available for an infiltration trench; good vegetative cover can be established and maintained upgradient of the proposed BMP. Runoff exits the roadway as sheet flow via shoulder sections.
- Assume eight feet of available width will be present in the typical section for installation of the infiltration trench.
- Assume the infiltration rate of the existing soil with a factor of safety is 1.5 inches per hour and the site meets all other site constraints for an infiltration trench to be utilized.
- Assume the CP_v, Q_{p25}, and Q_f requirements are not applicable.
- The designer has previously calculated the following hydrologic information:
    - RR_v = 2,832 ft^3



FIND:

- The infiltration trench depth and configuration that meets the site constraints.

SOLUTION:

1. The infiltration trench will be sized solely for runoff reduction. The CP_v, Q_{p25}, and Q_f requirements are not applicable.
2. The runoff reduction volume was already calculated to be 2,832 ft^3.
3. The actual volume provided in the infiltration trench is calculated using the following formula:

$$VP = PV + VA(N_A)$$

Where:

VP = volume provided (temporary storage)
PV = ponding volume
VA = volume of aggregate
N_A = porosity of aggregate (use 0.4)

Assume 0.5 ft can pond along the length of the infiltration trench. Use the available surface area to find the minimum depth of the infiltration trench.

$$2{,}832\ ft^3 = (0.5\ ft \times 8\ ft \times 300\ ft) + (depth\ \times 8\ ft \times 300\ ft)(0.4)$$

$$depth = 1.7\ feet$$
$$\rightarrow round\ up\ to\ 2.0\ feet\ to\ meet\ minimum\ design\ requirements\ and\ for\ constructibility$$

Therefore, the infiltration trench will be 8 feet wide by 2 feet deep by 300 feet long.

Runoff exits the roadway via sheet flow over a grassed shoulder. The shoulder is presumed to provide adequate pretreatment to prevent clogging of the infiltration trench and no further action is required.

4. The total volume provided (3,312 ft^3) is greater than the target runoff reduction volume (2,832 ft^3).
5. Verify that the entire volume provided by the BMP will drain within 72 hours.

$$t_f = \frac{VP}{(k_{design})A_a}$$

Where:

t_f = drain time (days)
VP = total volume provided by practice (8 ft by 300 ft by 3 ft = 7,200 ft^3)
k_{design}= design infiltration rate of underlying soil (1.5 in/hr)
A_a = bottom surface area of aggregate (8 ft by 300 ft = 2,400 ft^2)

$$t_f = \frac{7{,}200\ ft^3}{\left(\frac{1.5\ in}{hr}\right)\left(\frac{1\ ft}{12\ in}\right)(2{,}400\ ft^2)} = 24\ hours$$

Therefore, the infiltration trench will drain within the specified timeframe.

Summary

2.6.5 Bioslope

Advantages	Disadvantages
• LID/GI design practice • Water quality benefits • Applicable in highly constrained areas • Flexible design options: can provide storage and infiltration	• Sheet flow is required • Unsuitable for steep embankments • Does not typically provide detention

Description: A BMP with engineered media and an underdrain installed on slopes or embankments. Sheet flow from paved areas infiltrates into the highly permeable media where it is filtered before exiting through the underdrain. High flows bypass the bioslope in the form of sheet flow running over the bioslope.

Design Considerations:

- Flow path between edge of pavement and bioslope <30 feet (preferred)
- Bioslope length typically equals the length of paved area treated
- Bioslope width is sized to capture the Q_{wq}
- Pretreatment through filter strip preferred

Maintenance Considerations:

- Provide markers or GPS location as bioslopes are difficult to distinguish from typical roadside embankments
- Provide underdrain cleanouts for inspection and to avert clogging

Applicability for Roadway Projects

- Lateral slope <3:1 (< 4:1 preferred)
- Longitudinal slope ≤5%
- Sheet flow required
- Linear configuration and minimal required space lend itself well to roadway environment

Stormwater Management Suitability:

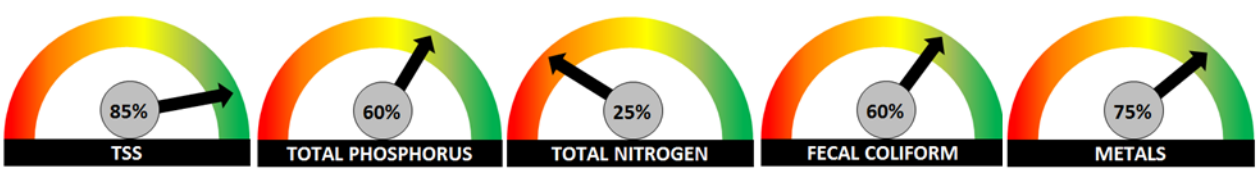
○ Runoff Reduction
✓ Water Quality
○ Channel Protection
X Overbank Flood Protection
X Extreme Flood Protection
✓ Temperature Reduction

✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

Bioslopes exhibit many LID/GI characteristics. Bioslopes treat runoff near the source using natural processes and often promote infiltration.

Treatment Capabilities

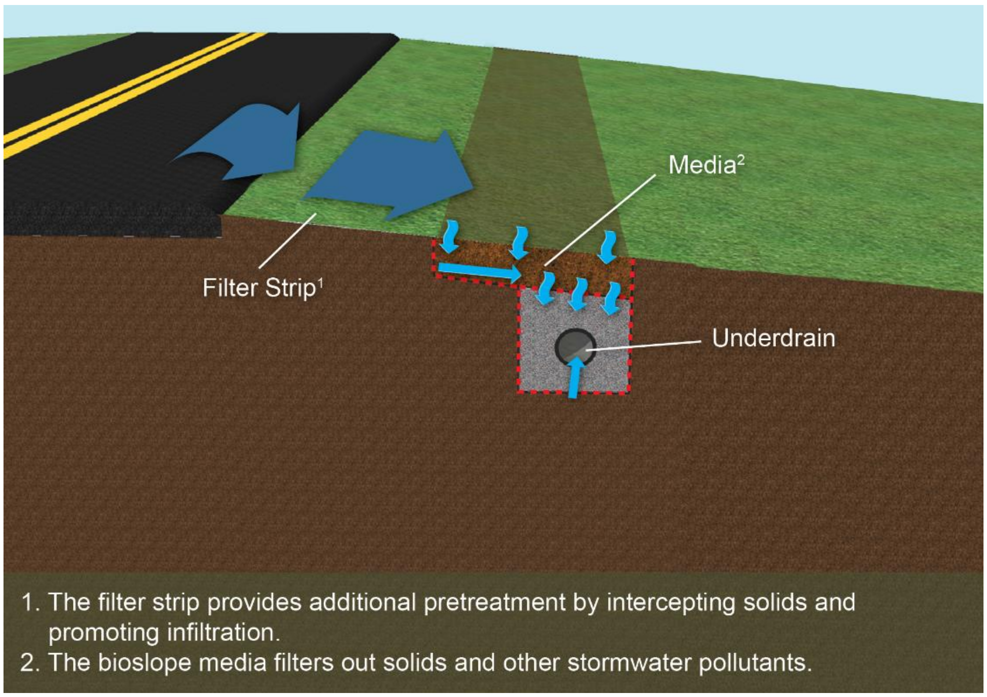


2.6.5 Bioslope

Description

Bioslopes are filtration BMPs that are typically installed in roadway embankments. A special media allows sheet flow from the roadway to rapidly infiltrate and filter through the bioslope where it is then collected and conveyed by an underdrain parallel to the roadway. Runoff in excess of the design flow rate bypasses the bioslope in the form of sheet flow that does not infiltrate. A filter strip is recommended, if space allows, and is typically placed directly upstream of the bioslope for pretreatment where it captures sediment and debris and prevents premature clogging of the bioslope. Bioslopes combine the benefits of filter strips and dry enhanced swales, providing cost effective treatment in areas where it is challenging to implement other BMPs. Figure 2.6.5-1 illustrates the typical bioslope components and treatment processes.

Figure 2.6.5-1 - Typical bioslope components and treatment processes



Stormwater Management Suitability

- Runoff Reduction – Bioslopes can provide 25% of the runoff reduction volume.
- Water Quality – Bioslopes rely primarily on filtration through an engineered media to provide removal of stormwater contaminants. The pretreatment component, commonly a vegetated filter strip, is most effective at sediment/debris removal, whereas the engineered media is capable of removing other pollutants. A bioslope provides 85% TSS removal if designed, constructed, and maintained correctly.
- Channel Protection – Generally, only the WQ_v is treated by a bioslope, so another BMP must be used to provide CP_v extended detention. However, for some smaller sites, a bioslope could provide some benefit towards detaining a portion of the full CP_v.

- Overbank Flood Protection – Bioslopes do not provide stormwater quantity control and should be designed to safely pass overbank flood flows. Another BMP must be used in conjunction with a bioslope to reduce the post-development peak flow of the 25- year storm (Q_{p25}) to pre-development levels (detention).
- Extreme Flood Protection – Bioslopes do not provide stormwater quantity control and should be designed to safely pass overbank flood flows. Another BMP must be used in conjunction with a bioslope to reduce the post-development peak flow of the 100- year storm (Q_f) to pre-development levels (detention).
- Temperature Reduction – Bioslopes can provide for temperature reduction.

Pollutant Removal Capabilities

The following average pollutant removal rates may be utilized for design purposes:

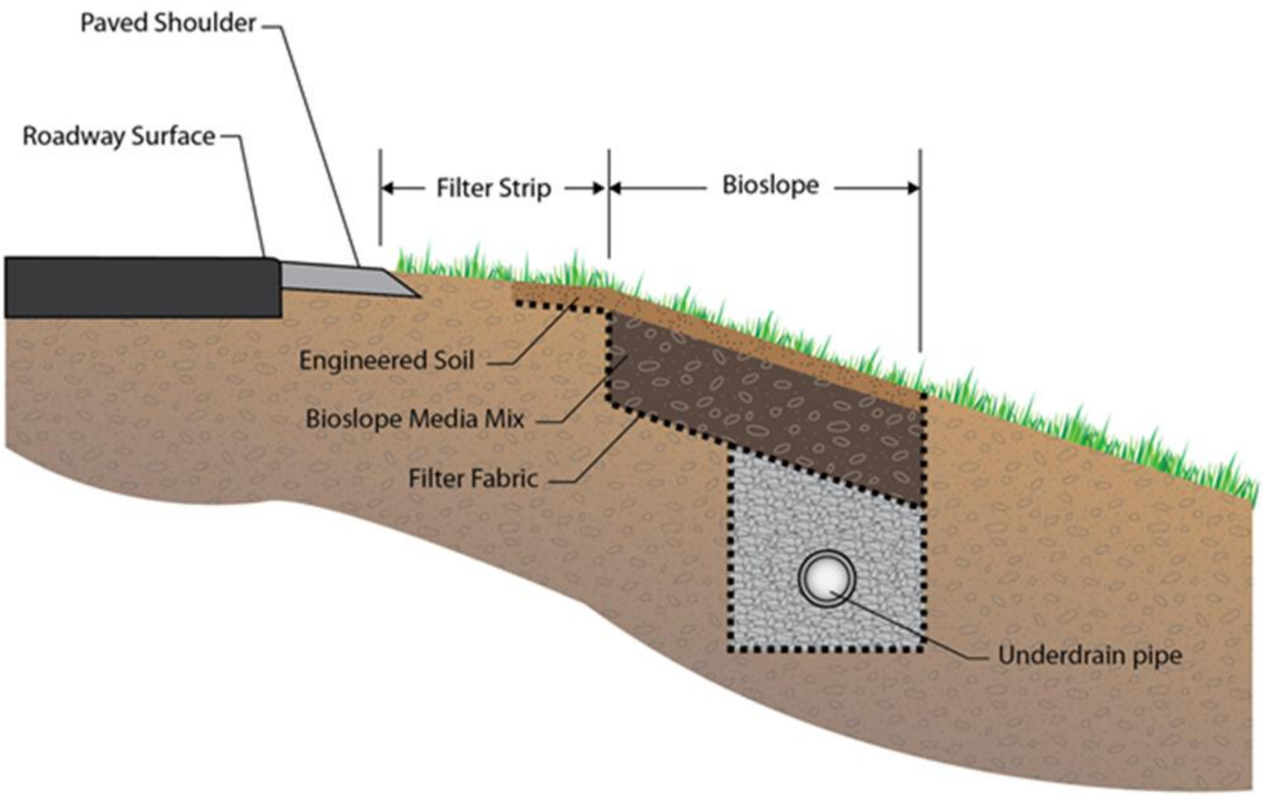
- TSS – 85%
- TP – 60%
- TN – 25%
- Fecal coliform – 60%
- Heavy metals – 75%
- Temperature – Temperature reduction is provided.

Pollutant removals values for TSS, TP, and heavy metals are based on research performed by the Washington State Department of Transportation. [(2-39)] Pollutant removal values for TN and fecal coliform are based on media filter removal rates published in a synthesis performed by the National Cooperative Highway Research Program. [(2-29)]

Application and Site Suitability

Bioslopes are applicable for roadway embankments where runoff exits the pavement as sheet flow. Bioslopes may be most practical in areas where limited right-of-way or other constraints preclude the use of enhanced swales, infiltration trenches, or similar BMPs that would otherwise collect and convey stormwater at the toe of the slope. Under ordinary circumstances, GDOT is not required to implement post-construction BMPs where runoff exits the right-of-way as sheet flow and does not cause instability, erosion, or flooding. Therefore, it may not be feasible to construct bioslopes in many of the areas where they would otherwise be utilized. However, if the project is located within a watershed that has an impaired waters, trout stream protection, or similar permit requirement, bioslopes can provide effective treatment in challenging areas. Figure 2.6.5-2 illustrates a typical bioslope configuration.

Figure 2.6.5-2 - Typical bioslope configuration (adapted from NCHRP, 2006) (2-27)



Sizing and specification criteria include:

- Preferably, the area between the edge of the pavement and the bioslope should be less than 30 feet to prevent flow from reconcentrating and eroding the roadway embankment or bioslope. (2-26)
- **Slopes –** Embankment slopes should be 3:1 or flatter. (2-26) Guardrail shall not be placed for the purpose of installing a bioslope. However, if guardrail is needed regardless of the bioslope, the bioslope may be placed behind guardrail given that the bioslope is no steeper than 3:1. Slopes greater than 4:1 may require additional stabilization such as TRM or plastic turf reinforcement grid products. Longitudinal slopes should be 5% or less. (2-39)
- **Depth to Water Table –** Two feet of separation is required between the bottom of the bioslope and the seasonally high water table.

Data for Design

The initial data needed for bioslope design may include the following:

- Existing and proposed site, topographic and location maps, and field reviews
- Field-measured topography or digital terrain model (DTM)
- Aerial/site photographs
- Drainage basin characteristics

- Preliminary plans including plan view, roadway and drainage profiles, cross sections, utility plans, and soil report
- Environmental constraints
- Design data of nearby hydraulic structures
- Additional survey information
- Groundwater elevations

Pretreatment

Where space allows, filter strips should be installed upstream of bioslopes to prevent the bioslope from clogging. Guidance for filter strips is provided in section 2.6.1 of this manual and should be followed when possible; however, if adequate space is not available, the minimum filter strip width (lateral) is not required when applied upstream of a bioslope. The edge of the filter strip shall be a least 1 ½ feet from the edge of the paved shoulder.

Bioslope Media

The media should have a minimum depth of 12 inches. Bioslope media is a mixture of crushed rock, dolomite, gypsum, and perlite. Crushed rock provides structure to the media; dolomite and gypsum promote the removal of heavy metals from runoff; and perlite enhances moisture retention. The media mixture is described in detail in Special Provision / Specification 169 on Post Construction Stormwater BMP Items. The media mixture is designed for an initial infiltration capacity of 50 inches per hour, with a long-term infiltration capacity of 28 inches per hour. The bioslope is sized using an infiltration rate of 10 inches per hour as a factor of safety. [(2-39)] Provide a minimum of 4-6 feet between the paved shoulder and the bioslope media.

Underdrain

An underdrain collects and conveys the stormwater that has filtered through the media. For bioslope applications, the underdrain trench/aggregate area cross-section should be at least 2-feet wide. [(2-39)] The underdrain pipe should be sized to convey the design flow (typically Q_{wq}) but should be no less than 8 inches in diameter. Filter fabric should completely encase the underdrain coarse aggregate (top, bottom, and sides). Refer to section 2.8.3 of this manual for additional information regarding underdrain design.

Underdrains implemented in bioslopes may be significantly longer than those in other BMPs. For this reason, cleanouts or observation wells should be provided every 100 feet and should connect to the underdrain with a tee fitting such that the water level may be observed, and the underdrain may be flushed. Additionally, no underdrain may extend more than 300 linear feet. In such cases, the underdrain should outlet and then a new underdrain started.

A design data table shall be added to the special grading sheet for each bioslope. See GDOT's Bioslope Special Construction Detail for more information.

Signage

The designer shall specify the installation of BMP signs consistent with GDOT's BMP Signs Special Construction Detail.

Bioslope Sizing

1. Determine the goals and primary function of the bioslope.

   The goals and primary function of the BMP must take into account any restrictions or site-specific constraints. Also take into consideration any special surface water or watershed requirements.

   - A bioslope must be designed for the water quality volume. The bioslope, however, can provide some runoff reduction benefit and reduce the required detention volume downstream. To calculate the RR_v credited for the practice (sized for WQ_v), Steps 2 – 4 have to be met, then proceed to Step 5. Otherwise the design process ends with Step 4.
   - Consider if the BMP can be "oversized" to include the channel protection volume or meet other detention targets.

2. Calculate the Target Water Quality Volume.

   Calculate the water quality volume formula using the following formula:

$$WQ_v = \frac{1.2\ in \times (R_v) \times A \times 43560 \frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

   Where:

   WQ_v = water quality volume (ft^3)

   R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.

   A = onsite drainage area of the post-condition basin (acres)

   Note that if the BMP is being sized for CP_v, the required storage volume for CP_v calculated per section 2.4.2 will replace the WQ_v in the formula above.

3. Calculate the Target Water Quality Volume Peak Flow.

   Calculate the water quality volume peak flow using the guidance in section 2.4.1.2.1.

4. Determine the length and width of the bioslope and the pretreatment volume required.

   The length of the bioslope is typically defined by site constraints and the length of the pavement area desired for treatment. Typically, the length of the bioslope should equal the length of pavement being treated. The width is typically sized such that the rate at which runoff infiltrates into the bioslope is at least as great as the Q_{wq}. Equation 2.6.5-1 should be used to calculate bioslope width. A minimum width of 2 feet is generally used for constructability and to facilitate the overall success and long-term operation of the BMP.

$$W = \frac{CQ_{wq}SF}{kL}$$

(2.6.5- 1)

   Where:

W = bioslope width (perpendicular to roadway) (feet)
C = conversion factor = 43,200 [(in/hr)/(ft/s)]
Q_{wq} = water quality volume peak flow (ft^3/s)
SF = safety factor equal to 1 (unitless, typical throughout Georgia)
k = infiltration, use long-term infiltration rate of 10 (inches/hour)
L = bioslope length (parallel to roadway) (feet)

5. Calculate the runoff reduction volume conveyed to the practice.

$$RR_v = \frac{1\ in \times (R_v) \times A \times 43560\,\frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

Where:

RR_v = runoff reduction volume (ft^3)
A = area draining to this practice (acres)
R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.

6. Calculate the runoff reduction volume credited.

Using Table 2.5-1 - GDOT BMPs and Associated Pollutant Removals, lookup the appropriate runoff reduction percentage (or credit) provided by the practice:

$$RR_v(credited) = RR_v(RR\%)$$

Where:

RR_v (credited) = runoff reduction volume provided by this practice (ft^3)
RR_v = runoff reduction volume conveyed to this practice (ft^3)
RR% = runoff reduction percentage, or credit, assigned to the specific practice

Maintenance Considerations

Without proper maintenance, BMPs will function at a reduced capacity and may cease to function altogether. A properly designed BMP includes several considerations for maintenance:

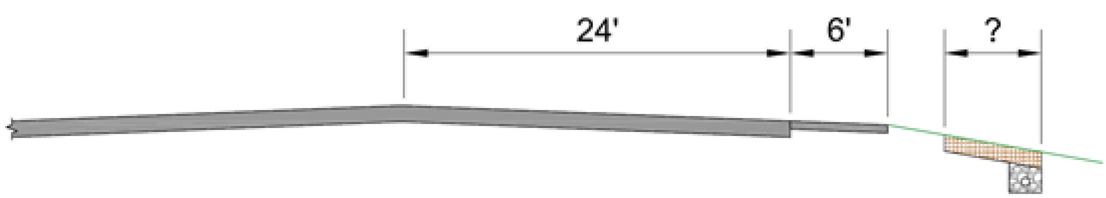
- Provide adequate right-of-way.
- Provide access roads and ramps for appropriate equipment to all applicable components (outlet structure, forebay, etc.).
- Provide space to turn around if necessary.
- Check for sufficient area to safely exit and enter the highway, if applicable.

Refer to GDOT's *Stormwater System Inspection and Maintenance Manual*, for specific maintenance requirements.

Bioslope Example Calculation

GIVEN:

- A new roadway project located in Savannah, Georgia.
- The proposed project includes 200 feet of roadway (in length).
- Heavy sediment loading is not expected.
- The drainage area that discharges to the bioslope includes the following: two 12-foot lanes and a 6-foot paved shoulder draining via sheet flow.
- There is 25 feet available for both a filter strip and the width of the bioslope along the length of the roadway.
- Assume that no stormwater is collected as "off-site" or "bypass" runoff.
- Assume that the existing ground and available right-of-way is sufficient for a bioslope with a longitudinal slope less than 5% and a length of 200 feet (entire length of roadway).
- The designer has previously calculated the following hydrologic information:
  - $WQ_v = 606\ ft^3$
  - $Q_{wq} = 0.16\ ft^3/s$



FIND:

- Determine the required width of the bioslope to treat runoff from the proposed roadway.

SOLUTION:

1. The bioslope must be designed for the water quality volume.
2. The water quality volume was already calculated to be 606 ft^3.
3. The water quality volume peak flow was already calculated to be 0.16 ft^3/s.
4. Calculate the minimum width of the bioslope using the following formula.

$$W = \frac{CQ_{wq}SF}{kL}$$

Where:

W = bioslope width perpendicular to roadway (feet)
C = conversion factor = (43,200 (in/hr)/(ft/s))
Q_{wq} = water quality volume peak flow (0.16 ft^3/s)
SF = safety factor (equal to 1 unless heavy sediment load is expected)
k = infiltration rate (10 inches/hour)

L = bioslope length parallel to roadway (200 feet)

$$W = \frac{(43{,}200)(0.16)(1)}{(10)(200)} = 3.5\ ft$$

Verify that the bioslope meets all design requirements as outlined in this section.

Additional design considerations:

- Complete filter strip design.
- Calculate the runoff reduction credited

Summary

2.6.6 Sand Filter

Advantages	Disadvantages
• LID/GI design practice • Effective TSS, nutrient, and fecal coliform removal • Low land requirement • No soil restriction • Appropriate for small areas with high impervious cover	• High capital cost • High maintenance burden • Limited to drainage areas of 10 acres for surface sand filter and 2 acres for perimeter sand filter

Description: Multi-chamber structures designed to treat stormwater runoff through filtration, using a sediment forebay, a sand bed as the primary filter media, and an underdrain collection system.

Design Considerations:

- Drainage area less than 10 acres for surface sand filter and less than 2 acres for perimeter sand filter
- Detain and treat the WQ_v
- Pretreatment through sediment forebay or chamber
- Maximum drain time of 40 hours for WQ_v
- Minimum elevation head of 5 feet for surface sand filter and 2-3 feet for perimeter sand filter
- Must design outlets for CP_v, Q_{p25}, and Q_f
- Provide minimum 2 feet of separation between bottom of sand filter and seasonal high water table

Maintenance Considerations:

- Provide adequate access to the BMP and appropriate components.

Applicability for Roadway Projects:

- Well suited for small drainage areas with a high percentage of impervious area
- Low land requirement
- Flexibility in basin shape

Stormwater Management Suitability:

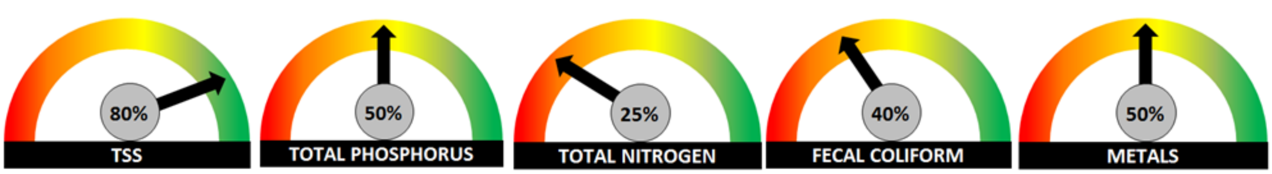
X Runoff Reduction
✓ Water Quality
○ Channel Protection
X Overbank Flood Protection
X Extreme Flood Protection
X Temperature Reduction

✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

Low land requirement and may be incorporated to complement the natural landscape.

Treatment Capabilities



2.6.6 Sand Filter

Description

Sand filters are multi-chamber structures designed to treat stormwater runoff through filtration, using a sediment forebay, a sand bed as the primary filter media, and an underdrain collection system. Sand filters are typically constructed offline for stormwater quality. A sand filter captures and temporarily stores the WQ_v so that it may be filtered through a bed of sand. Filtered runoff may be returned to the conveyance system or allowed to fully or partially exfiltrate into the soil.

A sand filter is typically composed of two chambers: a sediment forebay or sediment chamber, and a filtration chamber. The sediment forebay serves to remove floatables and heavy sediments while the filtration chamber removes additional pollutants by filtration through the sand bed.

There are two primary sand filter system designs, shown in Figure 2.6.6-1:

- **Surface Sand Filter** – A ground-level open air structure typically located off-line. It can be designed as an excavated basin with earthen embankments or as a concrete block structure.
- **Perimeter Sand Filter** – An enclosed filter system consisting of a sedimentation chamber and a sand bed filter, typically constructed in a below grade vault along the edge of an impervious area. The perimeter sand filter is a flexible, easily accessible BMP that provides good phosphorus removal and additional high oil and grease trapping ability. This type of sand filter may be best suited for site development applications and is further discussed in the GSMM.[(2-17)]

Figure 2.6.6-1 - Sand filter examples [(2-17)]

Surface Sand Filter   Perimeter Sand Filter

In sand filter systems, stormwater pollutants are removed through a combination of gravitational settling, filtration, and adsorption. This process effectively removes suspended solids and particulates, biochemical oxygen demand (BOD), fecal coliform bacteria, and other pollutants. Surface sand filters with a grass cover have additional opportunities for bacterial decomposition as well as vegetation uptake of pollutants, particularly nutrients.

While sand filters are well suited for small drainage areas with a high percentage of impervious area and have a low land requirement, which would make the sand filter well suited to a roadway environment, capital costs and maintenance burden are high.

Stormwater Management Suitability

- Runoff Reduction – Another BMP should be used in a treatment train with sand filters to provide runoff reduction as they are not designed to provide RR_v as a stand-alone BMP.
- Water Quality – In sand filter systems, stormwater pollutants are removed through a combination of gravitational settling, filtration, and adsorption. The filtration process effectively removes suspended solids and particulates, biochemical oxygen demand (BOD), fecal coliform bacteria, and other pollutants. Surface sand filters with a grass cover have additional opportunities for bacterial decomposition as well as vegetation uptake of pollutants, particularly nutrients. A sand filter provides 80% TSS removal if designed, constructed, and maintained correctly.
- Channel Protection – For smaller sites, a sand filter may be designed to capture the entire channel protection volume (CP_v) in either an off- or on-line configuration. Given that a sand filter system is typically designed to completely drain over 40 hours, the time requirement of extended detention of the 1-year, 24-hour storm runoff volume will be met. For larger sites or where only the WQ_v is diverted to the sand filter facility, another structural control must be used to provide CP_v extended detention.
- Overbank Flood Protection – Another BMP must be used in conjunction with a sand filter system to reduce the post development peak flow of the 25-year, 24- hour storm (Q_{p25}) to pre-development levels (detention).
- Extreme Flood Protection – Sand filter facilities must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the filter bed and facility.
- Temperature Reduction – Sand filters provide temperature reduction.

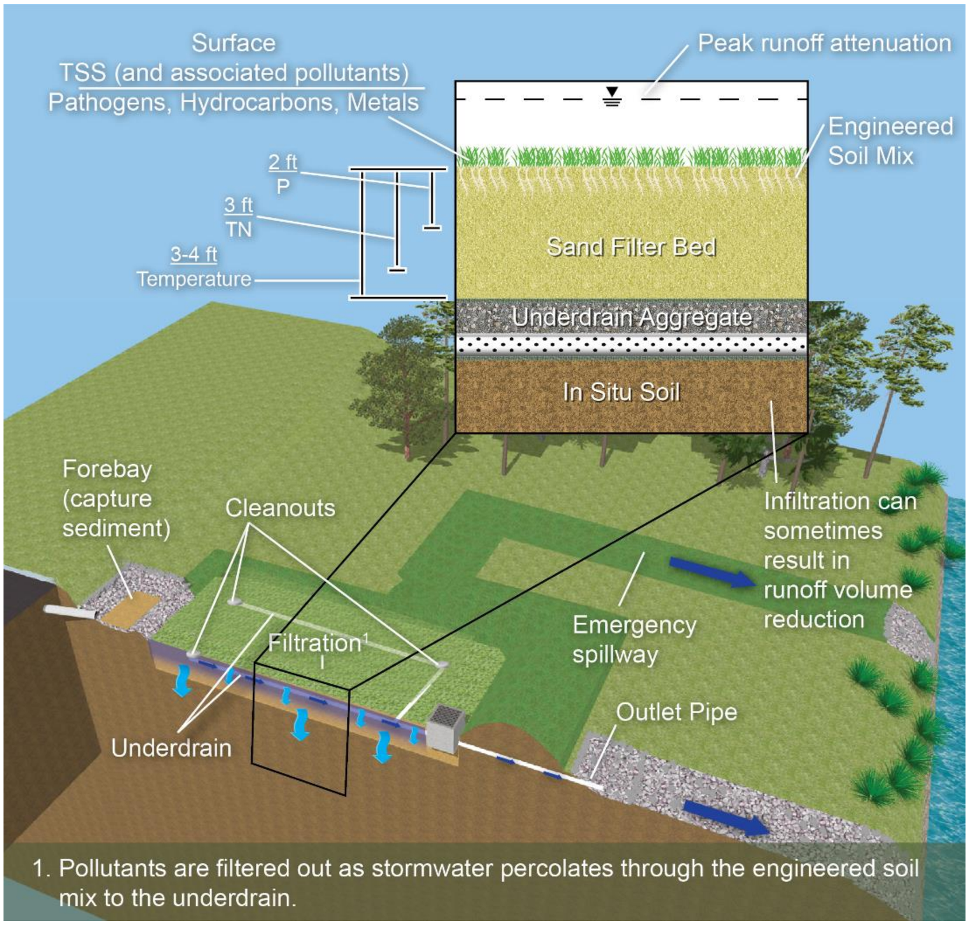
Pollutant Removal Capabilities

The following average pollutant removal rates may be utilized for design purposes: (2-17)

- TSS – 80%
- TP – 50%
- TN – 25%
- Fecal Coliform – 40%
- Heavy Metals – 50%
- Temperature – Temperature reduction is provided.

Figure 2.6.6-2 illustrates the treatment processes and target infiltration depths associated with different pollutants for filtration basins. (2-21)

Figure 2.6.6-2 - Sand filter treatment process and target depths



Application and Site Suitability

Sand filters are a high-cost option appropriate for various transportation applications, including roadways, highways, and non-road areas with a high percentage of impervious cover when pollutant reduction is the primary objective of stormwater treatment. The low land requirement of design and flexibility of the basin shape allows for a sand filter to be utilized in areas where available space or right-of-way may be limited. Sand filters may be incorporated into existing topography and can be shaped in various geometric patterns.

When considering locations for a sand filter, the following constraints should be considered:

- **Drainage Area** – Surface sand filters are best suited for small drainage areas, maximum 10 acres.
- **Drainage Area Characteristics –** Not well suited for locations with high sediment load. Sand filters should be avoided in areas with less than 50% impervious cover or sites with silt/clay soils to avoid rapid clogging and potential failure of the system.
- **Depth to Water Table** – Minimum 2 feet of clearance between the bottom of a surface sand filter and the seasonal high water table
- **Soils** – No soil restrictions, but HSG A soils are generally required for exfiltration.

- **Site Slope** – To promote filtration along and across the entire sand filter surface, maximum 6% slope across the filter location.
- **Minimum Head** – Minimum 5 feet of elevation head required between the inflow and outflow points.
- **Trout Stream** – Runoff temperature reduction may be provided with a sand filter. If discharging to a trout stream where temperature is a concern, evaluate for stream warming.
- **Aquifer Protection** - No exfiltration in areas subject to aquifer protection. Impermeable liner should be used.
- **Other Considerations –**
    - Sand filters should not be located in wetlands or other environmentally sensitive areas such as live streams (only under special circumstances are post-construction BMPs allowed within environmentally sensitive areas, with prior consent from appropriate regulatory agencies)
    - Sand filters should be placed at an appropriate offset (generally defined by the state) from any surface water (i.e., streams, ponds, lakes, or wetlands)
    - Sand filters are designed to completely drain the water quality volume within 40 hours and reaerate between rainfall events. Therefore, sites with continuous interflow from groundwater, sump pumps, or other sources should not be considered.

Data for Design

The initial data needed for sand filter design may include the following:

- Existing and proposed site, topographic and location maps, and field reviews
- Field measured topography or digital terrain model (DTM)
- Drainage basin characteristics
- Preliminary plans including plan view, roadway and drainage profiles, cross sections, utility plans, and soil report
- Environmental constraints
- Location of nearby surface waters and the depth to groundwater
- Design data of nearby hydraulic structures
- Additional survey information

General Design

The surface sand filter is located at ground level and consists of a perforated pipe and gravel underdrain system in addition to the sediment forebay and filtration chamber. A schematic of a surface sand filter is shown in Figure 2.6.6-3.

Stormwater will first enter the sand filter sedimentation chamber, which allows for the settling of debris and larger sediment particles. The stormwater then flows from the sediment forebay/chamber over a riprap dam to the filtration chamber, which contains the sand media filter. The hydraulic loading of the filter bed should be evenly distributed in a non-erosive manner. A perforated pipe and gravel underdrain system then collects the stormwater filtered through the sand bed and discharges stormwater from the filter system. Two typical sand filter sections are shown in Figure 2.6.6-4.

Figure 2.6.6-3 - Surface sand filter

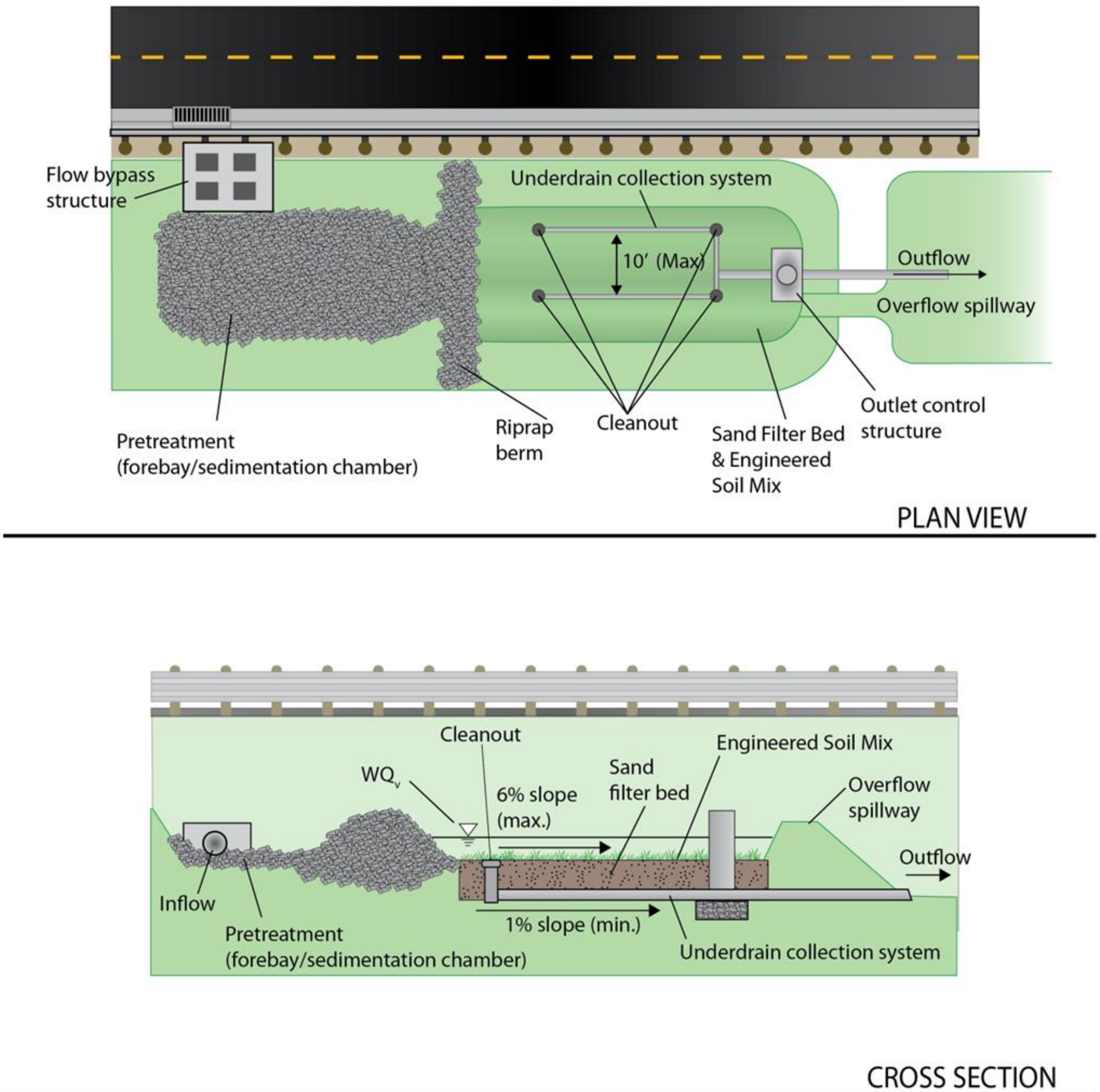
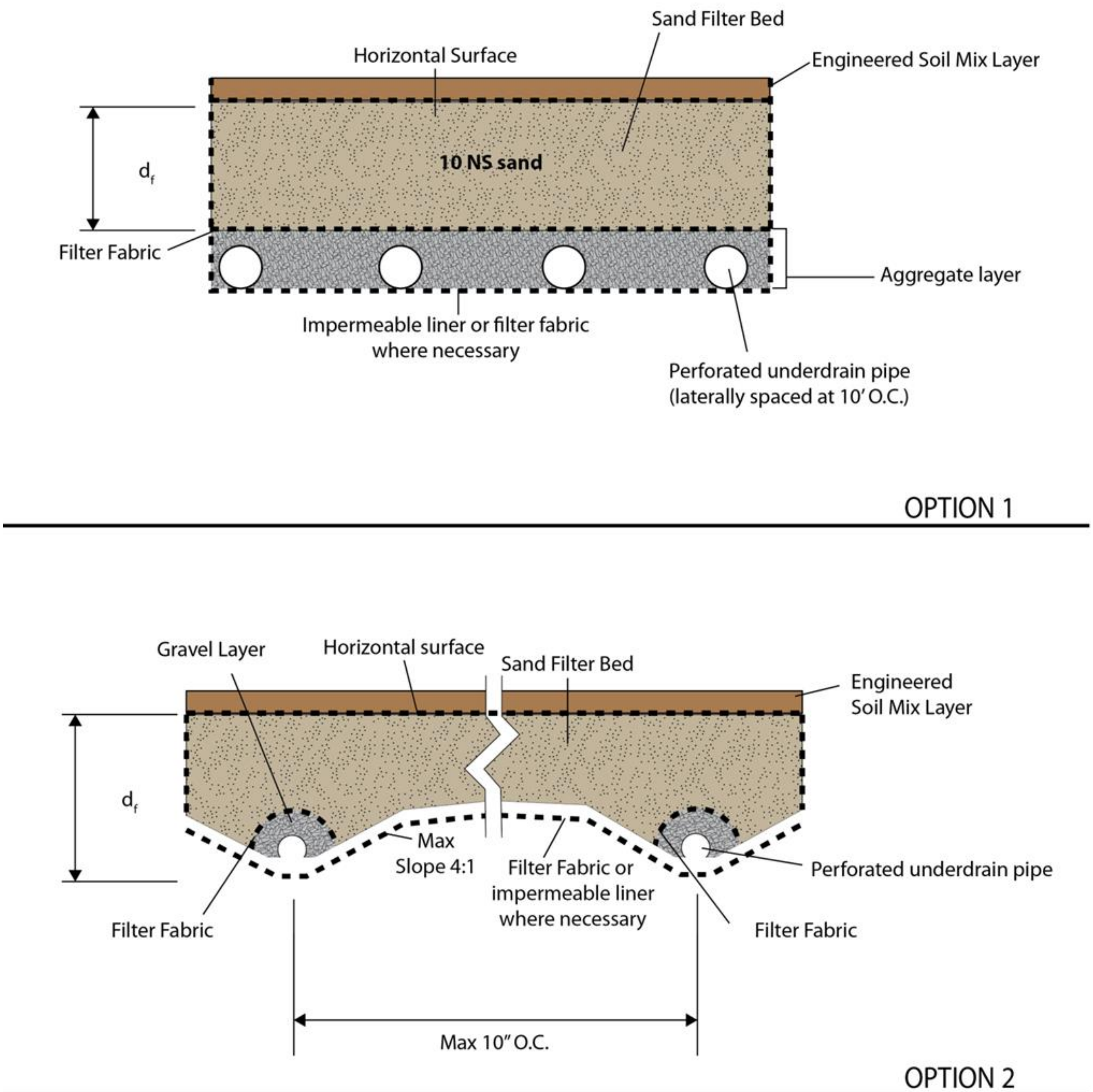


Figure 2.6.6-4 - Typical sand filter sections



The following criteria should be observed in the design of a surface sand filter:

- Sedimentation chamber shall be sized to hold a minimum volume based on 25% of the WQ_v with a minimum length to width ratio of 2:1. The Camp-Hazen equation can be used to calculate the required surface area for the sedimentation chamber:

$$A_s = -\frac{Q_o}{w} \times Ln(1 - E)$$

(2.6.6-1)

Where:

A_s = Sedimentation basin surface area (ft^2)

Q_o = Rate of WQ_v outflow over 24 hours (ft^3/s)

w = Particle settling velocity (ft^3/s)

= 0.0033 ft/s for imperviousness ≥ 75%

= 0.0004 ft/s for imperviousness < 75%

E = Trap efficiency (may use 90% trap efficiency (0.9))

- The filtration chamber can be sized using Equation 2.6.6-2 based on Darcy's Law:

$$A_f = \frac{WQ_v \times d_f}{k(h_f + d_f)t_f}$$

(2.6.6-2)

Where:

A_f = Surface area of filter bed (ft^2)

WQ_v = Water quality volume (ft^3)

d_f = Filter bed depth, sand only (ft)

k = Coefficient of permeability of filter media (ft/day) (3.5 ft/day for sand)

h_f = Average height of water above filter bed (ft)

(1/2 h_{max}, which varies based on design but h_{max} typically ≤ 6 feet)

t_f = Design filter bed drain time (days)

(1.67 days or 40 hours recommended maximum)

System Storage Volume

The entire treatment system (including the sedimentation chamber) must temporarily hold at least 75% of the WQ_v prior to filtration. Figure 2.6.6-5 illustrates the distribution of the volume to be treated (0.75 * WQ_v) among the various components of the surface sand filter.

$$V_{min} = 0.75 \times WQ_v = V_s + V_f + V_{temp}$$

(2.6.6-3)

Where:

WQ_v = Water quality volume (ft^3)

V_f = Filter bed voids volume (ft^3)

= $A_f d_f n$

A_f = Surface area of the filter media (ft^2)

d_f = Depth of filter media (ft)

n = Porosity (0.4 for most applications)

V_{temp} = Temporary volume stored above the filter bed (ft^3)

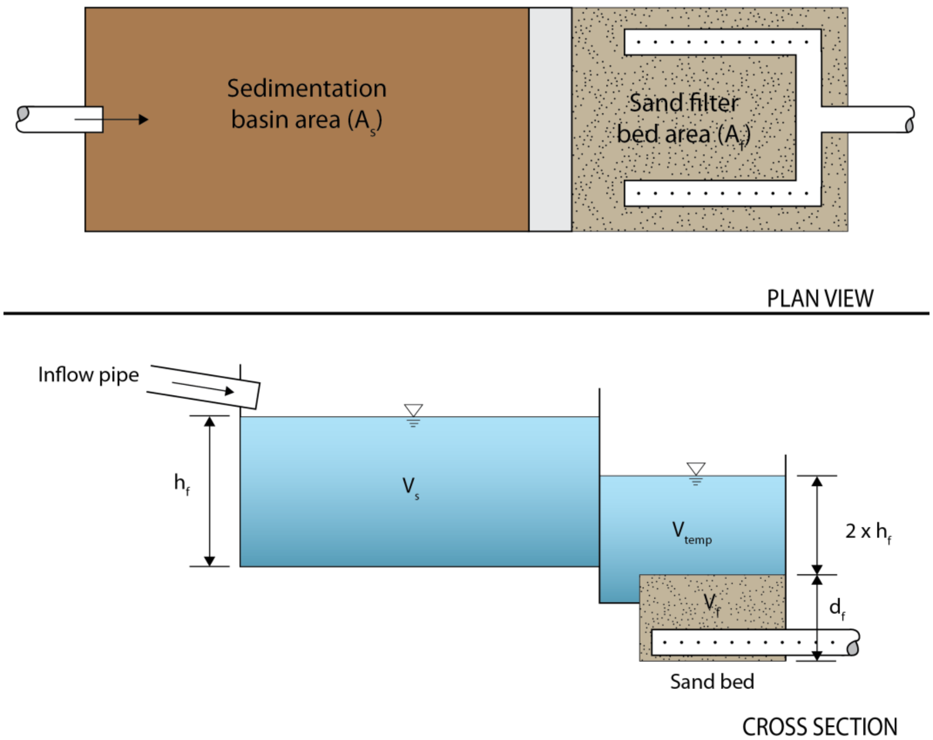
= $2 \times h_f \times A_f$

h_f = Average water depth above filter media (ft)

V_s = Sediment chamber volume (ft^3)

= $V_{min} - V_f - V_{temp}$

Figure 2.6.6-5 - Volume distribution schematic of a surface sand filter



Sand Filter Bed

The filter media consists of an 18-inch minimum 48-in maximum layer of clean washed medium sand (meeting ASTM C-33 concrete sand or GDOT Fine Aggregate Size No. 10) on top of the underdrain system. Design should use a sand soil permeability of 3.5 ft/day with a maximum total drain time of 40 hours.(2-17) Darcy's law can be applied to calculate drain time using the hydraulic conductivity of the filter media.

$$q = \frac{KhA}{12L}$$

(2.6.6-4)

Where:

q = flow rate (ft^3/hr)
K = hydraulic conductivity of the media (in/hr)
h = average head during drawdown period (ft)
A = cross-sectional area of flow (ft^2)
L = length of flow path (ft)

The filter media depth may be increased, depending upon targeted pollutant treatment. Table 2.6.6-1 lists the depths at which treatment has been found to occur for various pollutants. (2-21)

If phosphorus is targeted for removal, the media should be analyzed by a soils laboratory to determine the phosphorus content and corresponding phosphorus index (P-index). Media with high phosphorus levels can export this nutrient into the runoff instead of reducing this potential pollutant. A P-index less than 30 is desirable.

Table 2.6.6-1 Sand filter depth for pollutant treatment	
Targeted Pollutant	**Minimum Sand Filter Depth (ft)**
TP	2
TN	3

Three inches of loose topsoil should be placed over the sand bed. Non-woven plastic filter fabric should be placed both above and below the sand bed to prevent clogging of the sand filter and the underdrain system.

The structure of the surface sand filter may be constructed of impermeable media, such as concrete, or through the use of excavations and earthen embankments. When constructed with earthen walls/embankments, filter fabric should be used to line the bottom and side slopes of the sand filter before installation of the underdrain system and filter media.

Flow Bypass Structure

Sand filters should generally be an off-line BMP where the WQ_v is diverted to the filter through a flow bypass structure. Stormwater flows greater than the WQ_v may be diverted. See section 2.8.2 of this manual for further guidance.

Pretreatment/Inlets

The sedimentation chamber acts as pretreatment. Energy dissipation should be provided at all sand filter inlets. Non-erosive velocities are required for flow from the sedimentation chamber to the filtration chamber. See section 2.8 of this manual for further design guidance.

Underdrain System

Underdrains should be a minimum 8-inch perforated polyethylene pipe used to drain and discharge the treated stormwater from the filter media. Multiple branches of underdrain pipe may be utilized when needed. Spacing between branches should be no greater than 10 feet. Darcy's law can be used to determine the maximum flow rate through the filter media. Manning's equation can then be used to verify adequate underdrain pipe diameter. The orifice equation can then be used to determine an adequate length of underdrain pipe.

Cleanouts should be provided at the end of each underdrain branch and should extend to a height that minimizes inflow in the event that a cap is removed or damaged, burial by sediment, or damage by maintenance equipment.

Refer to Special Provision / Specification 169 on Post Construction Stormwater BMP Items and section 2.8.3 of this manual for further design guidance.

Outlet Structure

Treated stormwater will exit the sand filter system through an outlet pipe from the underdrain system to the discharge point. The discharge point of the outlet pipe should be evaluated to determine if there is need for energy dissipation, but the slow rate of filtration generally makes it unnecessary.

An outlet control structure, emergency, or bypass spillway must also be included in the sand filter system design to safely pass flows above the design storm. This prevents water levels within the filter from overtopping the embankment and causing structural damage. Downstream structures should not be impacted by spillway discharges. Typically, other structural controls must be designed in combination with the sand filter to provide safe passage of the CP_v, Q_{p25}, and Q_f. The peak flow of the proposed conditions peak for Q_{p25} must be limited to existing conditions flow rates.

The outlet structure dimensions shall be based on the following table.

Table 2.6.6-2 Outlet Structure Dimensions

Pipe Diameter	Min Width	Min Length	Max Width / Length	Min Height	Max Height
18 in	4 ft	4 ft	6 ft – 6 in	6 ft	8 ft
24 in	4 ft	4 ft	6 ft – 6 in	6 ft	8 ft
30 in	5 ft	4 ft	6 ft – 6 in	6 ft	13 ft
36 in	5 ft	4 ft	6 ft – 6 in	6 ft	13 ft
42 in	6 ft	4 ft	6 ft – 6 in	7 ft	13 ft
48 in	6 ft	4 ft	6 ft – 6 in	7 ft	13 ft

Dimensions that exceed maximum width or length will require individual structural design. Maximum outlet structure size shall have an inside area of no greater than 49 square feet. Outlet structure shall be constructed at even 6-inch increments. Dimensions of outlet structure shall be shown on special grading plans per special details.

The minimum height and width of an overflow weir shall be 6-inches. The maximum width of a weir shall be the width or length of the outlet structure less 1-foot. For example, if the outlet structure is 5 feet wide, then the maximum weir width on that side of the outlet structure shall be 4 feet. The overflow weir elevation shall be set no less than 3-inches and no more than 72-inches above the surface of the sand filter bed.

See the GDOT Sand Filter Outlet Structure Special Construction Detail for further design guidance.

Emergency Spillway

The emergency spillway is generally an open channel constructed in natural ground (as opposed to the embankment). The emergency overflow elevation shall be established at least one (1) foot below the roadway's normal shoulder break point and within 0.5 ft of the 100-year ponding elevation modeled with an unclogged outlet structure. The spillway shall be capable of conveying the 100-year storm with at least 1-foot of freeboard between the 100-year water surface elevation and the top of dam. The spillway shall be at minimum 8-feet wide. If including an emergency spillway in the design is not possible, size the weir(s) in the outlet structure so that they are capable of conveying the 100-year storm. Refer to the Sand Filter Outlet Structure Special Construction Detail for additional information. Refer to the guidance given in chapter 6 of the Drainage Design Policy Manual for assistance in sizing the channel and determining an appropriate lining material.

Vegetation

Surface filters should be designed with a grass cover to aid in pollutant removal and prevent clogging. The grass should be capable of withstanding frequent periods of wet and dry.

Additional Design Considerations

To prevent access and address safety concerns, fencing around the perimeter of a surface sand filter and gate locks may be incorporated into the design. Fencing should be determined on a case by case basis as warranted and as allowed by GDOT, see section 2.10 for additional information.

Additional design considerations include compliance with regulatory agencies. No exfiltration is allowed in areas subject to aquifer protection by the EPD watershed protection branch. Impermeable liner on earthen structures and watertight structures should be used. Evaluation of stream warming potential on downstream trout waters may warrant a shorter drain time of 24 hours or the incorporation of a micropool extended detention (ED) pond. Refer to the GSMM for further guidance on micropool ED pond design guidance. For more information on the design of a sand filter, see the detailed calculation example located at the end of this section.

Access and Driveway Considerations

See section 2.10.3 for maintenance access requirements.

Signage

The designer shall specify the installation of BMP signs consistent with GDOT's BMP Signs Special Construction Detail.

Sand Filter Sizing

1. Determine the goals and primary function of the sand filter.

   The goals and primary function of the BMP must take into account any restrictions or site-specific constraints. Also take into consideration any special surface water or watershed requirements.

   - Sand filters do not provide runoff reduction volume credits, so the BMP must be sized utilizing the water quality treatment approach.
   - Consider if the BMP can be "oversized" to include the channel protection volume.

2. Calculate the Target Water Quality Volume

   Calculate the water quality volume formula using the following formula:

$$WQ_v = \frac{1.2\ in \times (R_v) \times A \times 43560\,\frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

   Where:

WQ_v = water quality volume (ft^3)
R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.
A = onsite drainage area of the post-condition basin (acres)

3. Size flow diversion structure, if needed.

   A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQ_v to the sand filter facility. The peak rate of discharge for the water quality design storm is needed for sizing of off-line diversion structures. Refer to section 2.4.1.2 for calculation steps. Size low flow orifice, weir, or other device to pass Q_{wq}.

4. Compute the required surface area of the filter bed.

   The filter area is sized using the following equation (based on Darcy's Law):

$$A_f = \frac{WQ_v d_f}{k(h_f + d_f)t_f}$$

   Where:

   A_f = Surface area of filter bed (ft^2)
   d_f = Filter bed depth, sand only (ft)
   k = Coefficient of permeability of filter media (ft/day) (3.5 ft/day for sand)
   h_f = Average height of water above filter bed (ft)
   (1/2 h_{max}, which varies based on design but h_{max} typically ≤ 6 feet)
   t_f = Design filter bed drain time (days)
   (1.67 days or 40 hours recommended maximum)

5. Size sedimentation chamber.

   The sedimentation chamber should be sized to at least 25% of the computed WQ_v and have a length-to-width ratio of 2:1. The Camp-Hazen equation is used to compute the required surface area:

$$A_s = -\frac{Q_o}{w} \times Ln(1 - E)$$

   Where:
   A_s = Sedimentation chamber surface area (ft^2)
   Q_o = Rate of WQ_v outflow over 24 hours (ft^3/s)
   w = Particle settling velocity (ft^3/s)
   = 0.0033 ft/s for imperviousness ≥ 75%
   = 0.0004 ft/s for imperviousness < 75%
   E = Trap efficiency (may use 90% trap efficiency (0.9))

6. Compute V_{min}

$$V_{min} = 0.75 \times WQ_v$$

7. <u>Compute the water volume within the filter bed/gravel/pipe, V_f.</u>

$$V_f = A_f d_f n$$

Where:
V_f = Filter bed voids volume (ft^3)
A_f = Surface area of the filter media (ft^2)
d_f = Depth of filter media (ft)
n = Porosity (0.4 for most applications)

8. <u>Compute the temporary storage volume above the filter bed, V_{temp}.</u>

$$V_{temp} = 2 \times h_f \times A_f$$

Where:
V_{temp} = Temporary volume stored above the filter bed (ft^3)
h_f = Average water depth above filter media (ft)

9. <u>Compute the volume within the sedimentation chamber, V_s.</u>

$$V_s = V_{min} - V_f - V_{temp}$$

10. <u>Compute the sedimentation chamber height, h_s.</u>

$$h_s = \frac{V_s}{A_s}$$

11. <u>Ensure h_s and h_f fit available head and other dimensions still fit. Change as necessary in design iterations until all site dimensions fit.</u>

12. <u>Size distribution chamber and riprap berm to spread flow over filtration media.</u>

13. <u>Design inlets, pretreatment facilities, underdrain system, and outlet structures.</u>

Plan inlet protection for overflow from sedimentation chamber and size overflow weir at elevation h_f in filtration chamber to handle surcharge of flow through filter system from 25-year storm.

Maintenance Considerations

Without proper maintenance, BMPs will function at a reduced capacity and may cease to function altogether. A properly designed BMP includes several considerations for maintenance:

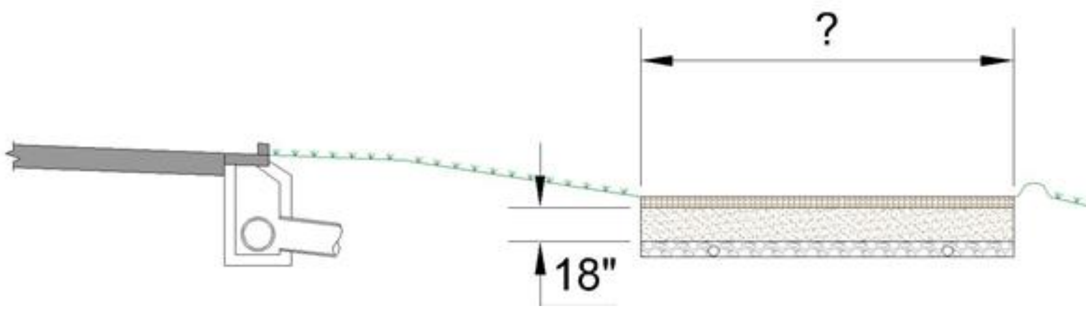
- If the BMP is fenced, provide appropriately sized gates (refer to section 2.10 for additional guidance regarding fencing and other safety considerations).
- Cleanouts should be provided at the end of each underdrain branch and should extend to a height that minimizes inflow in the event that a cap is removed or damaged, burial by sediment, or damage by maintenance equipment.

Refer to GDOT's *Stormwater System Inspection and Maintenance Manual*, for specific maintenance requirements.

Surface Sand Filter Example Calculation

GIVEN:

- A new roadway project located in Savannah, Georgia.
- The proposed project includes 1,500 feet of roadway (in length).
- Assume that an area approximately 50 feet by 50 feet is available for a sand filter.
- Runoff exits the roadway through a storm drain system with an 18" RCP outlet.
- The site meets all other site constraints.
- The designer has previously calculated the following hydrologic information:
  - WQ_v = 3,398 ft^3
  - Basin impervious area percentage = 70%



FIND:

- The surface sand filter size and configuration to meet WQ requirements.

SOLUTION:

1. The target water quality volume was already calculated to be 3,398 ft^3.
2. Using an 18-inch filter media depth, calculate the required surface area of the sand filter.

$$A_f = \frac{WQ_v d_f}{k(h_f + d_f)t_f}$$

Where:

A_f = Surface area of filter bed (ft^2)
d_f = Filter bed depth, sand only (1.5 ft)
k = Coefficient of permeability of filter media (ft/day) (3.5 ft/day for sand)
h_f = Average height of water above filter bed (ft)
(1/2 h_{max}, which varies based on design but h_{max} typically ≤ 6 feet)
t_f = Design filter bed drain time (days)
(1.67 days or 40 hours recommended maximum)

$$A_f = \frac{(3{,}398\ ft^3)(1.5\ ft)}{\left(3.5\frac{ft}{day}\right)(3\ ft + 1.5\ ft)(1.67\ days)} = 194\ ft^2$$

Approximate constructible dimensions required to form the required area. Use 14 feet by 14 feet for the sand filter surface area, now making $A_f = 196\ ft^2$.

3. Use the following equation to calculate the required surface area of the sedimentation chamber.

$$A_s = -\frac{Q_o}{w} \times Ln(1-E)$$

Where:

A_s = Sedimentation chamber surface area (ft^2)
Q_o = Rate of WQ_v outflow over 24 hours (ft^3/s)
w = Particle settling velocity (ft^3/s)
= 0.0004 ft/s for imperviousness < 75%
E = Trap efficiency (may use 90% trap efficiency (0.9))

$$A_s = -\frac{\frac{3{,}398\ ft^3}{24\ hrs} \times \frac{1\ hr}{3{,}600\ s}}{0.0004\ \frac{ft}{s}} \times Ln(1-0.9) = 226\ ft^2$$

4. Compute $V_{min.}$

$$V_{min} = 0.75 \times WQ_v = 0.75 \times 3{,}398\ ft^3 = 2{,}549\ ft^3$$

5. Compute the water volume within the filter bed.

$$V_f = A_f d_f n$$

Where:

V_f = Filter bed voids volume (ft^3)
A_f = Surface area of the filter media (ft^2)
d_f = Depth of filter media (ft)
n = Porosity (0.4 for most applications)

$$V_f = 196\ ft^2(1.5\ ft)(0.4) = 118\ ft^3$$

6. Compute the temporary storage volume above the filter bed, V_{temp}.

$$V_{temp} = 2 \times h_f \times A_f$$

Where:

V_{temp} = Temporary volume stored above the filter bed (ft^3)
h_f = Average water depth above filter media (ft)

$$V_{temp} = 2 \times 1.5\ ft \times 196\ ft^2 = 588\ ft^3$$

7. Compute the volume within the sedimentation chamber, V_s.

$$V_s = V_{min} - V_f - V_{temp}$$

$$V_s = 2{,}549\ ft^3 - 118\ ft^3 - 588\ ft^3 = 1{,}843\ ft^3$$

The sedimentation chamber (or forebay) should hold a minimum of 25% of the WQ_v but may be larger.

$$25\%(3{,}398\ ft^3) = 850\ ft^3 \leq 1{,}843\ ft^3 \therefore OK$$

8. Compute the sedimentation chamber height, h_s.

$$h_s = \frac{V_s}{A_s} = \frac{1{,}843\ ft^3}{226\ ft^2} = 8.2\ ft$$

A riprap forebay will be used as the sedimentation chamber and its height is limited to 5.5 feet.

Therefore, recalculate the area of the sedimentation chamber using a maximum height of 5.5 feet (new h_s).

$$A_s = \frac{V_s}{h_s} = \frac{1{,}843\ ft^3}{5.5\ ft} = 335\ ft^2$$

The sedimentation chamber should have a length-to-width ratio of 2:1. For constructability, use minimum 13 feet by 26 feet sedimentation chamber.

A sedimentation chamber that is 13 feet by 26 feet and a sand filter area of 14 feet by 14 feet fit into the available 50 foot by 50 foot area.

Summary

2.6.7 Bioretention Basin

Advantages	Disadvantages
• LID/GI design practice • Effective pollutant removals • Low land requirement • No native soil restriction • Appropriate for small areas with high impervious cover • Pleasing aesthetics	• High capital cost • High maintenance burden • Generally limited to drainage areas of 5 acres or less • Not intended for discharge attenuation

Description: Filtration BMP with mulch, diverse vegetation, engineered soil media, and an underdrain.

Design Considerations:

- Drainage area less than 5 acres
- Multiple underdrain options that provide different runoff reduction credits
- Detain the RR_v or treat the WQ_v
- Provide pretreatment to prevent clogging of media
- Ponding depth: 12 inches or less, 9 inches preferred
- Maximum ponding volume drain time of 24 hours
- Engineered soil media is composed of sand, fines, and organic matter
- A landscaping plan is required and vegetation should be carefully selected; trees should not be used

Maintenance Considerations:

- Provide adequate access to the BMP and appropriate components
- Provide mulch that resists floating to avoid erosion and clogging of the outlet structure

Applicability for Roadway Projects:

- Well suited for small drainage areas with a high percentage of impervious area
- Low land requirement
- Flexibility in basin shape
- Can be tailored to fit constrained sites

Stormwater Management Suitability

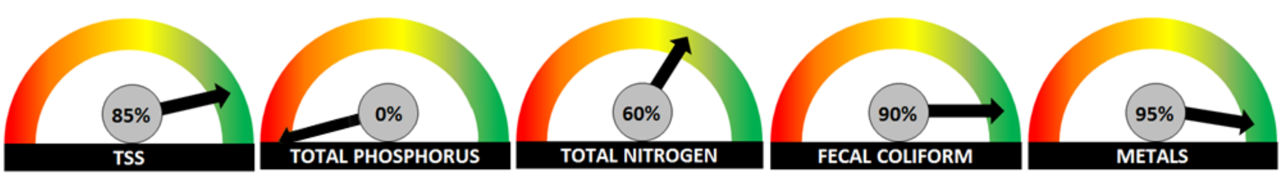
✓ Runoff Reduction
✓ Water Quality
○ Channel Protection
X Overbank Flood Protection
X Extreme Flood Protection
✓ Temperature Reduction

✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

Low land requirement, adaptable to many situations, and often a small BMP used to treat runoff close to the source.

Treatment Capabilities



2.6.7 Bioretention Basin

Description

Bioretention basins are structural BMPs that serve to reduce stormwater pollution through infiltration, filtration, biological uptake, and microbial activity using landscape vegetation, engineered soil mix, and an underdrain.

Bioretention basins are effective in reducing TSS, nutrients, heavy metals, pathogens and temperature. After pretreatment, runoff is temporarily detained in the bioretention basin to allow it to percolate through an engineered soil mix. Vegetation is purposefully selected and planted to enhance pollutant removal and aesthetics. If the native soils allow for infiltration, bioretention basins can provide runoff quantity control, particularly for smaller runoff volumes.

The design process of a bioretention basin varies depending on the intended goals and primary function of the basin. If native soils have low infiltration rates, the bioretention basin will be designed to treat the water quality volume. Runoff filters through the engineered soil mix, is collected by the underdrain system, routed to an outlet structure and then discharged through the outlet pipe.

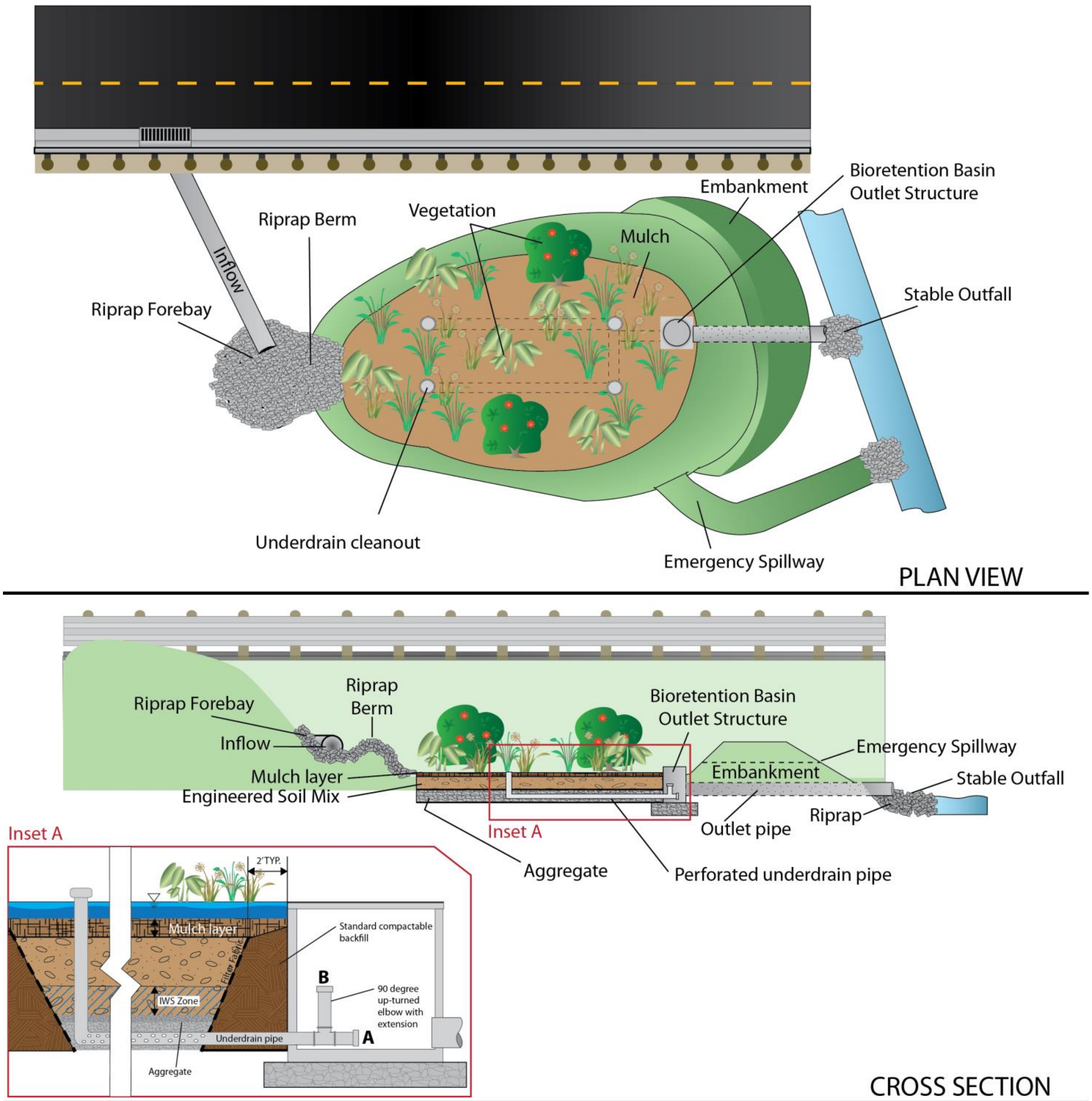
A bioretention basin may be designed with an upturned underdrain within the outlet control structure to create an internal water storage (IWS) zone. An upturned underdrain increases the runoff reduction credit of the BMP and is also a beneficial configuration for nitrogen removal. The IWS maintains a saturated zone where anaerobic conditions develop and increase nitrogen removal. [(2-30)] The IWS media depth should be at least 12 inches. [(2-26)]

If native soils allow for infiltration, the bioretention basin can be designed for runoff reduction. When designed for runoff reduction, stormwater runoff filters through the engineered soil mix and then infiltrates into the underlying soil.

The underdrain configuration provided in Figure 2.6.7-1 is a single design that can be used for all bioretention basin designs. Removable screw caps may be included at the underdrain discharge point in the outlet control structure at point A as well as the top of the upturned underdrain at point B, depending on the goals and primary function of the bioretention basin. For a bioretention basin sized for water quality, point B will be capped. For a BMP sized with an IWS zone, point A will be capped, but point B will be open. For a BMP sized for runoff reduction, both points A and B will be capped so that the BMP functions as though no underdrain is present. The underdrain is included in the design in this scenario only as a safety measure to provide a method to drain standing water for maintenance and in the event the BMP does not function as designed.

Bioretention terminology is often confusing and inconsistent. Bioretention BMPs are described as cells, basins, facilities, etc. The term rain garden is sometimes used to describe small, residential bioretention BMPs. Depending on the agency or jurisdiction, an underdrain may be required, allowed, or restricted (filtration versus infiltration).

Figure 2.6.7-1 - Typical bioretention basin configuration



Stormwater Management Suitability

- Runoff Reduction – Bioretention basins are one of the most effective low impact development (LID) practices that can be used in Georgia to reduce post-construction stormwater runoff and improve stormwater runoff quality. Like other LID practices, they become even more effective when constructed in native soils with high infiltration rates. A bioretention basin with a capped underdrain can provide 100% of the runoff reduction volume, if properly maintained. In order to design a bioretention basin with a capped underdrain, the footprint must be in HSG A or B and Worksheet B-1 must find infiltration potentially suitable. Infiltration testing will be performed during construction if the design is for a capped underdrain. A bioretention basin with an upturned underdrain can provide 75% of the runoff reduction volume if the IWS zone is at least equal to the target runoff reduction volume. An upturned underdrain should not be used in soils that have significant clay/rock content due to the clogging potential created during construction. Finally, a bioretention basin with a typical underdrain configuration can provide 50% of the runoff reduction volume, if properly maintained.

- Water Quality – A bioretention basin is an excellent stormwater treatment practice due to its variety of pollutant removal mechanisms. The pre-treatment component reduces incoming runoff velocity and filters particulates from the runoff. The ponding area provides for temporary storage of stormwater runoff prior to its evaporation, infiltration, or uptake and provides additional settling capacity. The organic or mulch layer provides filtration as well as an environment conducive to the growth of microorganisms that degrade hydrocarbons and organic material. The engineered soil mix in the bioretention basin acts as a filtration system and clay in the soil provides adsorption sites for hydrocarbons, heavy metals, nutrients, and other pollutants. Plants in the ponding area provide vegetative uptake of runoff and pollutants and also serve to stabilize surrounding soils. A bioretention basin with an open or upturned underdrain provides 85% TSS removal if designed, constructed, and maintained correctly. A bioretention basin with a capped underdrain provides 100% TSS removal if designed, constructed, and maintained correctly.
- Channel Protection – For smaller sites, a bioretention basin may be designed to capture the entire channel protection volume (CP_v). Given that a bioretention basin is typically designed to completely drain over 72 hours, the requirement of extended detention for the 1-year, 24-hour storm runoff volume will be met. For larger sites, or where only the WQ_v is diverted to the bioretention basin, another control must be used to provide CP_v extended detention.
- Overbank Flood Protection – Another control will be required in conjunction with a bioretention basin to reduce the post-development peak flow of the 25-year storm (Q_{p25}) to pre-development levels (detention).
- Extreme Flood Protection – Bioretention basins must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the ponding area, mulch layer and vegetation.
- Temperature Reduction – Bioretention basins can provide for temperature reduction.

Pollutant Removal Capabilities

Bioretention basins designed for runoff reduction with a capped underdrain system are credited with a 100% pollutant removal capability. The following average pollutant removal rates may be utilized for bioretention basins with an open or upturned underdrain:

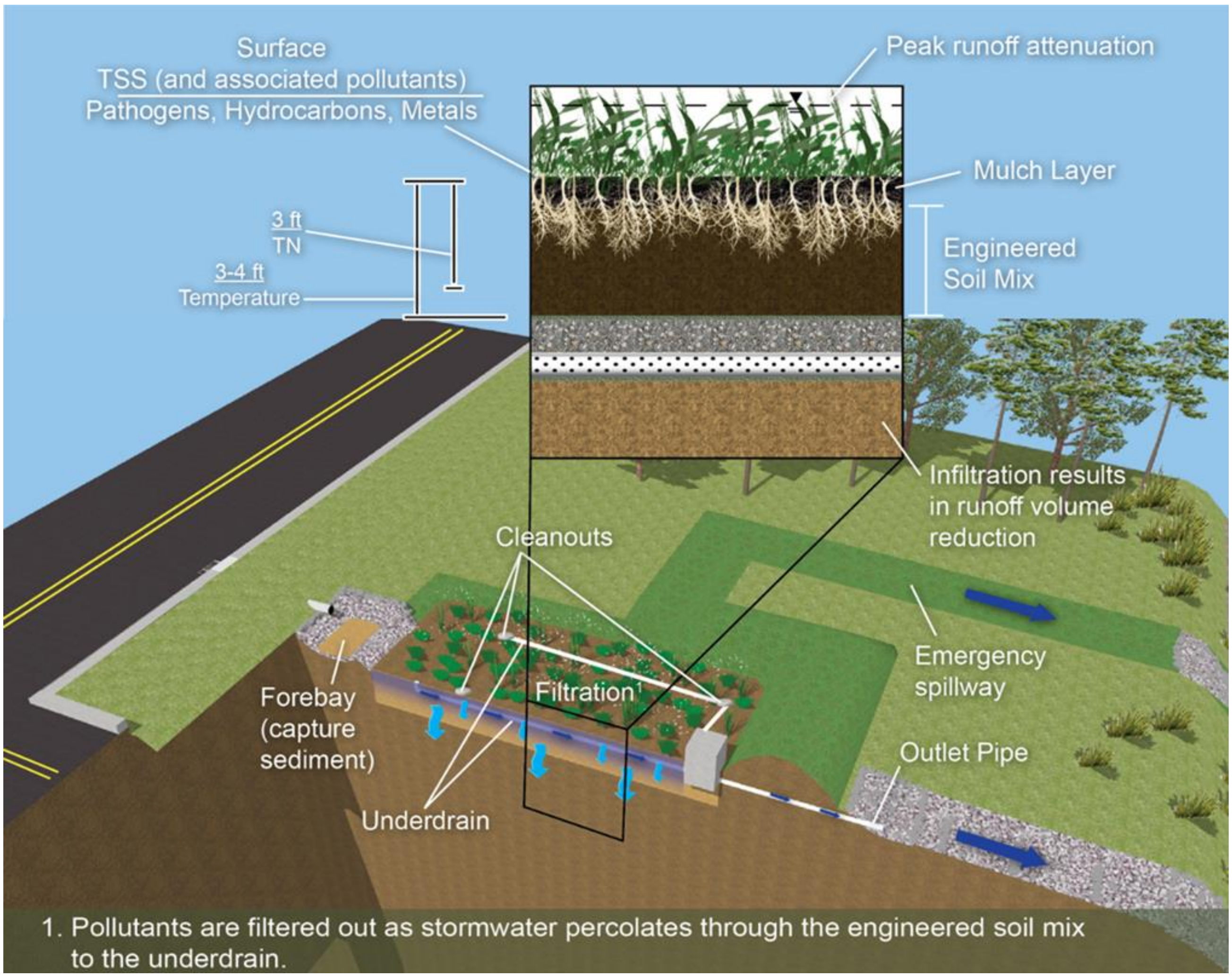
- TSS – 85% [(2-28)]
- TP – 0%
- TN – 60% [(2-28)]
- Fecal coliform – 90% [(2-22)]

- Heavy metals – 95% (2-22)
- Temperature – Temperature reduction is provided.

*GDOT does not take credit for phosphorus removal in bioretention basins due to conflicting scientific reports.

Bioretention basins that meet the minimum design criteria outlined in this section are expected to perform well and significantly reduce stormwater pollutants. However, where practicable, bioretention basin design should be optimized and tailored to the specific pollutants of concern for the given drainage area and receiving water. Pollutant removal for individual constituents is largely dependent on the media depth provided. For example, pathogens and hydrocarbons are removed at the surface, while temperature reduction typically occurs at 3 to 4 feet of depth. Figure 2.6.7-2 should be used to determine the optimum filtration depth for various pollutants.

Figure 2.6.7-2 - Typical bioretention basin components and treatment processes, and pollutant removal zones (2-26)



Application and Site Suitability

Bioretention basin designs have been adapted to fit many challenging urban applications. Size, shape, and configuration are flexible and can be adjusted to fit many transportation-related sites. However, due to the added aesthetics and maintenance associated with the landscape vegetation, GDOT bioretention basins may be best suited for highly visible locations such as rest areas, roadway median strips, or municipal interchange quadrants receiving a higher level of maintenance.

When considering locations for a bioretention basin, the following constraints should be considered:

- **Drainage Area** – Due to the limited ponding depths and inlet velocities, bioretention basins usually serve smaller drainage areas (5 acres or less). If the drainage area is greater than 5 acres, consider multiple bioretention basins or providing additional pretreatment and/or inlet protection to reduce the velocity and energy of stormwater entering the practice. Inlet protection may include splash blocks, a stone diaphragm, a level spreader, or another similar device.
- **Space Required** – For general planning purposes, the amount of space that is often needed by the basin is approximately 3 to 6% of the contributing drainage area. The value can vary significantly, however, depending on the design (configuration and components) of the bioretention basin, the percent imperviousness of the drainage area, and the volume of runoff captured.
- **Site Slope** – Bioretention basins are not intended to serve steep contributing slopes. Contributing slopes should be a maximum of 20%, although slopes of 5% or less are ideal.
- **Depth to Water Table** – Two feet of vertical separation from the bottom of the media to the seasonally high water table should be provided to avoid groundwater from ponding inside the filter bed, which could lead to groundwater contamination.
- **Soils** – Determine the HSG from the Web Soil Survey. If the HSG is A or B, and Worksheet B-1 determines that infiltration is potentially suitable, infiltration testing will be performed during construction. Engineered soil mix, as specified in Specification 169, is needed.
- **Hotspots** – Do not use for hotspot runoff.
- **Damage to existing structures and facilities** – Consideration should be given to the impact of water exfiltrating the bioretention basin on nearby road bases. To avoid the risk of seepage, bioretention basins should not be hydraulically connected to pavement or structure foundations.[(2-37)] In addition, the maximum water surface elevation or top of the engineered soil media should not be placed above the subgrade of the adjacent roadway.
- **Setbacks** – Although there are no specific setback requirements, careful consideration should be given to the potential negative impacts for locating bioretention basins in close proximity to water supply wells, septic systems, utilities, and private property. Recommended setbacks are listed below:
    - 10 feet from building foundations
    - 100 feet from private water supply wells
    - 200 feet from public water supply reservoirs (measured from edge of water)

  - 1,200 feet from public water supply wells

- **Trout Stream –** Runoff temperature reduction is provided when a bioretention basin is designed for infiltration. If discharging to a trout stream where temperature is a concern, evaluate for stream warming when an open underdrain system is used.

Data for Design

The initial data needed for bioretention basin design may include the following:

- Existing and proposed site, topographic and location maps, and field reviews
- Aerial photographs of the drainage basin to estimate land use areas (grassed, paved, etc.)
- Preliminary plans including plan view, roadway and drainage profiles, cross sections, and utility plans
- Location of nearby surface waters and the depth to seasonally high groundwater
- Soils data from the Web Soil Survey or other source
- Design data from nearby hydraulic structures

Flow Bypass Structure

Due to the presence of a mulch layer and engineered soil mix, it may be beneficial to implement the bioretention in an offline configuration using a flow bypass structure. Flows from large storm events can wash out and displace the mulch and media. Refer to section 2.8.2 and the GDOT Bypass Structure Special Construction Detail for additional information.

Pretreatment

Pretreatment is vital to the successful operation of filtration BMPs as the media can quickly become clogged from high sediment loads if otherwise left without pretreatment. Where possible, forebays should be provided. Refer to section 2.8.1 and the GDOT Riprap Forebay Special Construction Detail for additional information guidance on forebays. Filter strips and grass channels can be used for pretreatment in a treatment train application. The location of bioretention basins on unique sites often constrains the use of pretreatment options by application type or available space. Flow exiting the pretreatment device and entering the bioretention basin should be nonerosive to avoid eroding the mulch and engineered soil media.

Filter Media

Bioretention basins have engineered soil mix designed to sustain landscape vegetation and filter pollutants. If the rate of infiltration is too slow, allowing for extended periods of water ponding at the surface, the growth of vegetation will be impeded causing bioretention to be ineffective. Careful consideration is given to the composition of this layer; it is generally engineered and imported from offsite sources. Special Provision / Specification 169 prescribes the engineered soil mix to use in bioretention basins. The engineered soil mix should be covered by hardwood mulch that is resistant to floating per Special Provision / Specification 169. Mulch provides multiple benefits, such as removing metals and retaining soil moisture.

It is especially important to protect the integrity of the media during construction to prevent clogging or compaction that would reduce its treatment capabilities. Refer to Special Provision / Specification 169 for additional construction considerations.

As shown in Figure 2.6.7-2, the depth of the engineered soil mix also plays a role in pollutant removal. The minimum engineered soil depth is 24 inches. Additional depth may be added for nitrogen and temperature reduction. The maximum engineered soil depth is 48 inches.

Vegetation

Landscape vegetation is an important design component of bioretention basins. Roots enhance soil qualities and help create a suitable environment for beneficial microbial activity. The vegetation helps to uptake nutrients that have been filtered out of stormwater within the media.

A landscaping plan is required for bioretention basin design. The landscaping plan must include a list of the proposed plant species, source of where the plants are obtained, the planting sequences, and post-nursery maintenance requirements. Vegetation should be selected based on the zone of hydric tolerance. A bioretention basin has essentially three zones. The lowest elevation requires plants that can withstand standing and fluctuating water levels. Plants located in the middle elevation also need to withstand fluctuating water levels but are generally tolerant to dryer conditions. The highest elevation supports plants adapted to dryer conditions. Although trees typically provide added water quality benefits, they can obstruct maintenance operations and roots can damage underdrains. Therefore, tree species are not recommended for use within bioretention basins.

A professional landscape architect may be consulted. Native species are preferred. However, non-native, ornamental species may be used as long as they are not invasive. Vegetation should cover at least 90% of the surface area in the bioretention cell within 2 years. Refer to the GDOT Planting Schedule Special Construction Detail for additional guidance.

Underdrain System

Underdrains collect and convey the stormwater that has filtered through the soil mix. Underdrain systems consist of small diameter perforated pipe surrounded by coarse aggregate. Multiple branches are typically required, and at least two branches are recommended in case one becomes clogged. Refer to section 2.8.3 of this manual and the GDOT Underdrain Special Construction Detail for additional information regarding underdrain design.

Provisions for Overflow

Provisions for overflow should be provided for most bioretention configurations. Exceptions may include small bioretention basins with flow bypass structures. Overflow configurations can include riser boxes and/or emergency spillway channels.

If an elevated catch basin is used, the edge of the inlet should be set at the WQ_v elevation and will perform the same function as would a riser structure in a detention pond. Large riser structures are typically not required in bioretention basins because they typically treat smaller drainage areas.

The outlet structure dimensions shall be based on the following table.

Table 2.6.7-1 Outlet Structure Dimensions					
Pipe Diameter	Min Width	Min Length	Max Width / Length	Min Height	Max Height
18 in	4 ft	4 ft	7 ft – 6 in	5 ft – 2 in	8 ft – 0 in
24 in	4 ft	4 ft	7 ft – 6 in	5 ft – 2 in	8 ft – 0 in
30 in	5 ft	4 ft	7 ft – 6 in	6 ft – 0 in	8 ft – 3 in
36 in	5 ft	4 ft	7 ft – 6 in	6 ft – 0 in	8 ft – 3 in
42 in	6 ft	4 ft	7 ft – 6 in	7 ft – 0 in	8 ft – 3 in
48 in	6 ft	4 ft	7 ft – 6 in	7 ft – 0 in	8 ft – 3 in

Dimensions that exceed maximum width or length will require individual structural design. Maximum outlet structure size shall have an inside area of no greater than 49 square feet. Outlet structure shall be constructed at even one-foot (1 ft) increments. Dimensions of outlet structure shall be shown on special grading plans per special details.

The minimum height and width of an overflow weir shall be 6-inches. The maximum width of a weir shall be the width or length of the outlet structure less 1-foot. For example, if the outlet structure is 5 feet wide, then the maximum weir width on that side of the outlet structure shall be 4 feet. The overflow weir elevation shall be set no less than 3-inches and no more than 12-inches above the surface of the mulch.

Refer to the GDOT Bioretention Basin Outlet Structure Special Construction Detail for additional information.

If an emergency spillway is utilized in the design the overflow elevation shall be established at least one (1) foot below the roadway's normal shoulder break point and within 0.5 ft of the 100-year ponding elevation. The spillway shall be capable of conveying the 100-year storm. The spillway shall be at minimum 8-feet wide. If including an emergency spillway in the design is not possible, size the weir(s) in the outlet structure so that they are capable of conveying the 100-year storm. Refer to the guidance given in chapter 6 of the Drainage Design Policy Manual for assistance in sizing the channel and determining an appropriate lining material.

Access and Driveway Considerations

See section 2.10.3 for maintenance access requirements.

Signage

The designer shall specify the installation of BMP signs consistent with GDOT's BMP Signs Special Construction Detail..

Bioretention Basin Sizing

1. <u>Determine the goals and primary function of the bioretention basin.</u>

The goals and primary function of the BMP must take into account any restrictions or site-specific constraints. Also take into consideration any special surface water or watershed requirements.

- Determine whether the bioretention basin is intended to meet the runoff reduction target or channel protection volume. Please note that bioretention basins have approximately the same size when sized for the runoff reduction volume compared to when they are sized for the water quality volume, so the default calculations are for runoff reduction, regardless of the HSG. However, be sure to indicate in the plans whether construction should perform infiltration testing.

2. Determine if the bioretention basin will be on-line or off-line. If the bioretention basin will be off-line, a flow regulator (or flow splitter diversion structure) should be supplied to divert the RR_v or CP_v to the bioretention basin. The design storm peak flow is needed for sizing an off-line diversion structure. See section 2.8.2 for more information on bypass structures. See section 2.4.1.2 for more information on calculating the water quality volume peak flow.

3A. <u>Calculate the Stormwater Runoff Reduction Target Volume.</u>

$$RR_v = \frac{1\ in \times (R_v) \times A \times 43560 \frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

Where:

$RR_{v(target)}$ = runoff reduction target volume (ft^3)
A = area draining to this practice (acres)
R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.

3B. <u>Determine the storage volume of the practice and the pretreatment volume</u>

The actual volume provided in the bioretention basin is calculated using the following formula:

$$VP = PV + VES(N_{ES}) + VA(N_A)$$

Where:

VP = volume provided (temporary storage)
PV = ponding volume
VES = volume of engineered soils
N_{ES} = porosity of engineered soil (For bioretention basins, use 0.25)
VA = volume of aggregate
N_A = porosity of aggregate (use 0.4)

Provide pretreatment by using a grass filter strip or as needed (sheet flow), or a grass channel or forebay (concentrated flow). Where filter strips are used, 100% of the runoff should flow across the filter strip. Pretreatment is also necessary to reduce flow velocities and assist in sediment removal and maintenance. Pretreatment can include a forebay, weir, or check dam. Splash blocks or level spreaders should be considered to dissipate concentrated stormwater runoff at the inlet and prevent scour. Forebays should be sized to contain 0.1 inches per impervious acre of contributing drainage.

3C. <u>Verify total volume provided by the practice is at least equal to the $RR_{v(target)}$</u>

When the VP ≥ $RR_{v(target)}$ then the runoff reduction requirements are met for this practice. When the VP < $RR_{v(target)}$, then the design must be adjusted, the BMP must be sized according to the WQ_v treatment method (see Step 4), or another BMP must be considered and designed.

3D. Verify that the bioretention basin will drain in the specified timeframes.

The ponding area of the bioretention basin must drain within 24 hours (1 day) and the entire bioretention basin must drain within 72 hours (3 days).

$$t_f = \frac{PV(d_f)}{k(h_f + d_f)A_f}$$

Where:

A_f = top surface area of filter media (ft^2)
PV = ponding volume (ft^3)
d_f = filter media depth (ft)
k = hydraulic conductivity (2 ft/day)
h_f = average water depth (ft)
t_f = drain time (days)

If the HSG is A or B and Worksheet B-1 indicates that infiltration is potentially suitable, verify that the entire volume provided by the BMP will drain within 72 hours.

$$t_f = \frac{VP}{(k_{design})A_a}$$

Where:

VP = total volume provided by practice (ft^3)
k_{design}= design infiltration rate of underlying soil (2 ft/day).
A_a = bottom surface area of aggregate (ft^2)

4. Design outlet control structure and emergency overflow

An overflow must be provided to bypass and/or convey larger flows to the downstream drainage system or stabilized watercourse. Non-erosive velocities need to be ensured at the outlet point. The overflow should be sized to safely pass the peak flows anticipated to reach the practice, up to a 100-year storm event.

5. Prepare a vegetation and landscaping plan

A landscaping plan for the bioretention basin should be prepared to indicate how vegetation will be established. See the Vegetation section above and the GDOT Planting Schedule Special Construction Detail for additional guidance.

Maintenance Considerations

Without proper maintenance, BMPs will function at a reduced capacity and may cease to function altogether. A properly designed bioretention basin includes the following considerations to facilitate maintenance:

- Access:
    - Provide adequate right-of-way.

- Provide access roads and ramps for appropriate equipment to all applicable components (outlet structure, forebay, etc.).
- Provide space to turn around if necessary.
- Check for sufficient area to safely exit and enter the highway, if applicable.
- If the BMP is fenced, provide appropriately sized gates (refer to section 2.10 for additional guidance regarding fencing and other safety considerations).

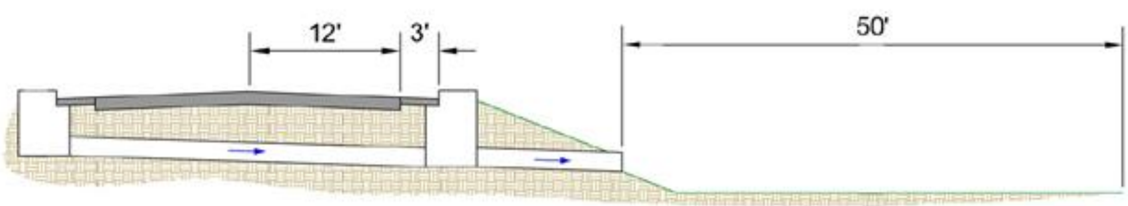
- Avoid outlet structure configurations that are prone to clogging.
- Hardwood mulch resistant to floating should be used to minimize loss of mulch that results in clogging of the outlet structure.

Refer to GDOT's *Stormwater System Inspection and Maintenance Manual*, for specific maintenance requirements.

Bioretention Example Calculation

GIVEN:

- A new roadway project located in Savannah, Georgia.
- The proposed project includes 1,100 feet of roadway (in length).
- The drainage area that discharges to the bioretention basin includes the following: two 12-foot lanes and two 3-foot shoulders that will be conveyed via curb and gutter.
- An area, approximately 50 feet by 50 feet, is available for the bioretention basin taking into account access for maintenance and required clear zones.
- Runoff exits the roadway through storm drain system with an 18" RCP outlet.
- The site satisfies all other site constraints.
- Assume CP_v, Q_{p25} and Q_f requirements do not apply.
- HSG A
- The designer has previously calculated the following hydrologic information (See section 2.4 for additional guidance):
  - $RR_v = 2,832\ ft^3$
  - $WQ_v = 3,398\ ft^3$



FIND:

- The bioretention size and configuration to retain the RR_v.

SOLUTION:

1. Determine whether the bioretention basin is intended to meet the runoff reduction target or water quality target. Initially, review the infiltration rate of the native soils using the Web Soil Survey and use Worksheet B-1 to determine if it should be indicated on the plans that infiltration testing is needed during construction.
2. The runoff reduction volume was already calculated as 2,832 ft^3.
3. The next step is to determine the storage volume of the practice. To complete this step, use the area available as a starting point for the surface area of the bioretention basin. In this example, approximately 50 feet by 50 feet is available for the bioretention basin. It is recommended that a software program and/or BMP sizing calculator spreadsheet be used at this point. The volume provided by the BMP is calculated using the following formula:

$$VP = PV + VES(N_{ES}) + VA(N_A)$$

Where:

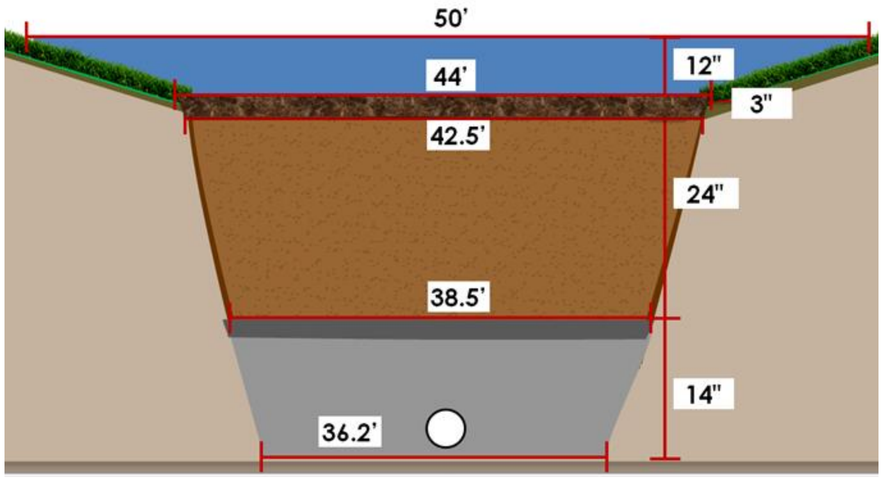
VP = volume provided (temporary storage)
PV = ponding volume
VES = volume of engineered soils
N_{ES} = porosity of engineered soil (For bioretention basins, use 0.25)
VA = volume of aggregate
N_A = porosity of aggregate (use 0.4)

Therefore, at least an estimate of the following values is required to calculate the storage volume of the BMP:

- Top surface area of ponding volume
- Bottom surface area of pond volume/top surface area of engineered soil mix
- Maximum ponding height
- Bottom surface area of the engineered soil mix/top surface area of the aggregate layer
- Engineered soil mix depth
- Bottom surface area of the aggregate layer
- Aggregate layer depth

For the purposes of this example, the following values are used as a starting point for sizing the basin.

- Top surface area of ponding volume = 50 ft x 50 ft = 2,500 ft^2
- Top surface area of engineered soil mix = 42.5 ft x 42.5 ft = 1,806 ft^2
- Maximum ponding height = 12 inches = 1 ft
- Bottom surface area of the engineered soil mix/top surface area of the aggregate layer = 38.5 ft x 38.5 ft = 1,482 ft^2
- Engineered soil mix depth = 24 inches = 2 ft
- Bottom surface area of the aggregate layer = 36.2 ft x 36.2 ft = 1,310 ft^2
- Aggregate layer depth = 14 inches = 1.167 ft



The volume of each layer is approximately the following:

- Ponding volume = 2,218 ft^3
- Engineered soils = 3,288 ft^3
- Aggregate = 1,629 ft^3

$$VP = PV + VES(N_{ES}) + VA(N_A)$$

$$VP = 2218 + 3{,}288(0.25) + 1{,}629(0.4) = 3{,}692\ ft^3$$

A forebay is the chosen pretreatment method for this bioretention basin. Forebays should be sized to contain 0.1 inches per impervious acre of contributing drainage. The required forebay volume is 275 ft^3.

4. The volume provided (3,692 ft^3) is greater than the minimum volume of the practice (2,832 ft^3) to meet the runoff reduction requirement. It is now an iterative process to design the bioretention basin so that the volume provided more closely matches the minimum required volume to maximize the efficiency of the design.

5. Verify the ponded volume will drain within 24 hours and the entire bioretention basin will drain within 72 hours. For the purposes of this example, assume the values provided in Step 4 are used for the design.

$$t_f = \frac{PV(d_f)}{k(h_f + d_f)A_f}$$

Where:

A_f = top surface area of filter media (1,806 ft^2)
PV = ponding volume (2,218 ft^3)
d_f = filter media depth (2 ft)
k = hydraulic conductivity (2 ft/day)
h_f = average water depth (0.5 ft)
t_f = drain time (days)

$$t_f = \frac{2{,}218(2)}{2(0.5 + 2)1{,}806} = 0.49\ days = 11.76\ hours$$

Therefore, the ponded volume will drain within 24 hours.

Now, verify that the entire volume provided by the BMP will drain within 72 hours.

$$t_f = \frac{VP}{(k_{design})A_a}$$

$$t_f = \frac{3{,}692}{(2)1{,}310} = 1.409\ days = 33.8\ hours$$

Therefore, the total volume provided by the BMP will drain within 72 hours.

6. Therefore, the 50 feet by 50 feet (2,500 ft^2) available area is adequate for the bioretention basin assuming slopes and other site constraints are not limiting.

   The shape of the bioretention basin should conform to the available area and site topography.

Additional design considerations:

- Design the flow diversion structure, if needed.
- Design the outlet structure in accordance with the GDOT Bioretention Basin Outlet Structure Special Construction Detail.
- Develop the landscaping plan.

Summary

2.6.8 Dry Detention Basin

Advantages	Disadvantages
• May be less costly than other detention BMPs • Space may be utilized for other purposes during dry conditions • Can be used for large and small drainage areas	• Standing water can create a safety concern and may require fencing or guardrail. See section 2.10.2 for information on public safety

Description: A basin designed to attenuate peak flows and completely drains between storm events.

Design Considerations:

- Can be used to comply with CP_v, Q_{p25}, and Q_f requirements; other requirements may apply
- Outflow hydrograph should mimic the predevelopment hydrograph
- Maximum drainage area of 75 acres
- Maximum basin depth should be 10 feet
- Side slopes should be 3:1 or flatter if possible
- Basin bottom should be a minimum of 2 feet above the seasonal high water table

Maintenance Considerations:

- Provide adequate access to the BMP and appropriate components
- Design outlet structure to resist clogging

Applicability for Roadway Projects

- Space and grade requirements may limit applicability in the linear environment
- Basin shape can be elongated to accommodate roadway applications
- May be best suited for interchange areas

Stormwater Management Suitability:

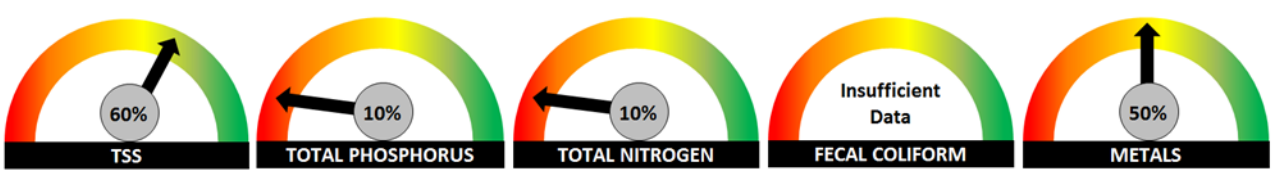
X Runoff Reduction
○ Water Quality
✓ Channel Protection
✓ Overbank Flood Protection
✓ Extreme Flood Protection
X Temperature Reduction

✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

Dry detention is generally not considered LID/GI. However, dry detention basins do provide some infiltration and evapotranspiration. Further, they can be used for small drainage areas close to the source and help to restore predevelopment hydrology.

Treatment Capabilities



2.6.8 Dry Detention Basin

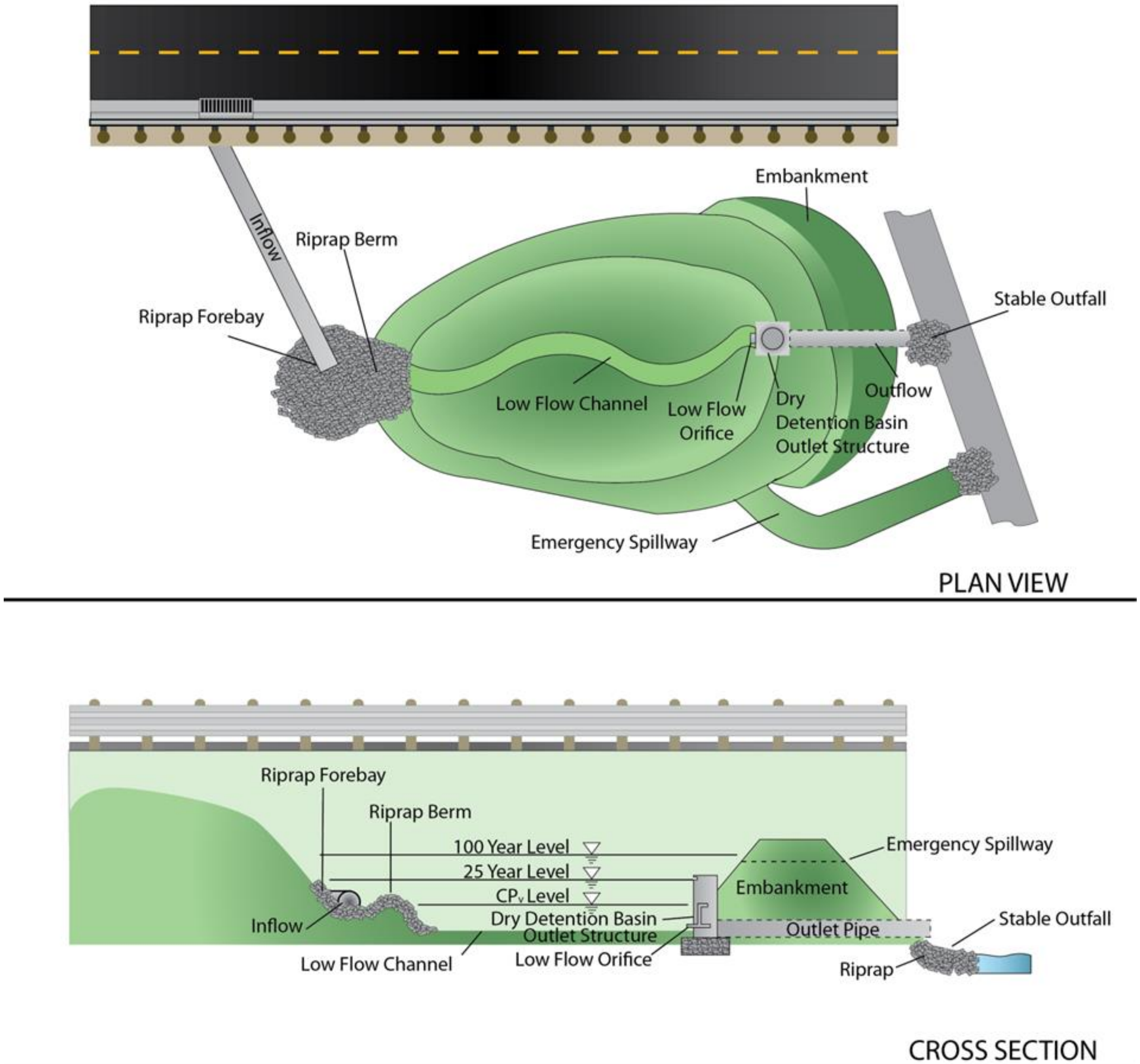
Description

Dry detention basins are earthen impoundments designed to temporarily store stormwater runoff and drain completely following storm events. Their primary purpose is to reduce the proposed condition rate of discharge (the rate of runoff after final project completion) to the existing condition rate of discharge (the rate of runoff before roadway construction activities begin). Detention may reduce the potential to overload existing downstream drainage systems, reduce the potential for soil erosion, and minimize the adverse effects of sedimentation. Detention basins can be used to help meet WQ_v, CP_v, Q_{p25}, and Q_f requirements. A riser with a small orifice at the bottom allows the basin to temporarily detain the design storm and slowly release it over a period of time (24 hours). Runoff in excess of the design storm is released through additional weirs/orifices higher on the riser, the top of the riser, and/or an emergency spillway channel. Figure 2.6.8-1 illustrates a typical dry detention configuration.

Alternative detention structures include underground detention and multipurpose detention. Underground detention is discouraged for use at GDOT facilities due to the high cost and maintenance burden. However, these facilities may be considered in areas where constraints restrict the use of other BMPs and where flooding may impact life or property. Prior approval is required directly from the Office of Design Policy and Support before designing underground detention facilities. Follow the procedure outlined in section 2.5.3 when submitting for approval to design underground detention or multipurpose detention facilities.

Multipurpose detention areas are facilities that are used primarily for purposes other than detention. Detention can be incorporated into parking lots, rooftops, athletic fields, and other open spaces. Areas of temporarily ponded water are typically shallow, relatively isolated, and graded to drain. Multipurpose detention is generally used for the Q_{p25} and Q_f. Extended detention is precluded because the areas need to be made available for their primary purpose shortly after the rainfall event. Underground and multipurpose detention facilities are covered in greater detail in the GSMM.

Figure 2.6.8-1 - Typical dry detention basin configuration



Stormwater Management Suitability

- Runoff Reduction – Another BMP should be used in a treatment train with dry detention basins to provide runoff reduction as they are not designed to provide RR_V as a stand-alone BMP.
- Water Quality – If installed to include the water quality volume and water quality volume orifice per the recommended design criteria and properly maintained, 60% total suspended solids removal will be applied to the water quality volume (WQ_v) flowing to the dry detention basin. Another BMP should be used in a treatment train with dry detention basins.

- Channel Protection – Dry detention basins can be sized to store the channel protection volume (CP_v) and to completely drain over 24-72 hours, meeting the requirement of extended detention of the 1-year, 24-hour stormwater runoff volume.
- Overbank Flood Protection – Dry detention basins are intended to provide overbank flood protection (peak flow reduction of the 25-year, 24-hour storm, Q_{p25}).
- Extreme Flood Protection – Dry detention basins can be designed to control the extreme flood (100-year, 24-hour storm, Q_f) rainfall event.

Pollutant Removal Capabilities

Dry detention basins provide water quality benefits when properly maintained. The following average pollutant removal rates may be utilized for design purposes: [(2-28) (2-29)]

- TSS – 60%
- TP – 10%
- TN – 30%
- Fecal Coliform – Insufficient Data
- Heavy Metals – 50%
- Temperature – Temperature reduction is not provided.

Application and Site Suitability

Dry detention basins should be considered in areas where flooding is a concern. Pre-existing drainage deficiencies such as inadequate downstream channel capacity and flooding conditions should be considered in the overall project design. The construction of dry detention basins within floodplains is strongly discouraged. When the situation is deemed unavoidable, the following must be thoroughly evaluated and shown in the MS4 Post-Construction Stormwater Report:

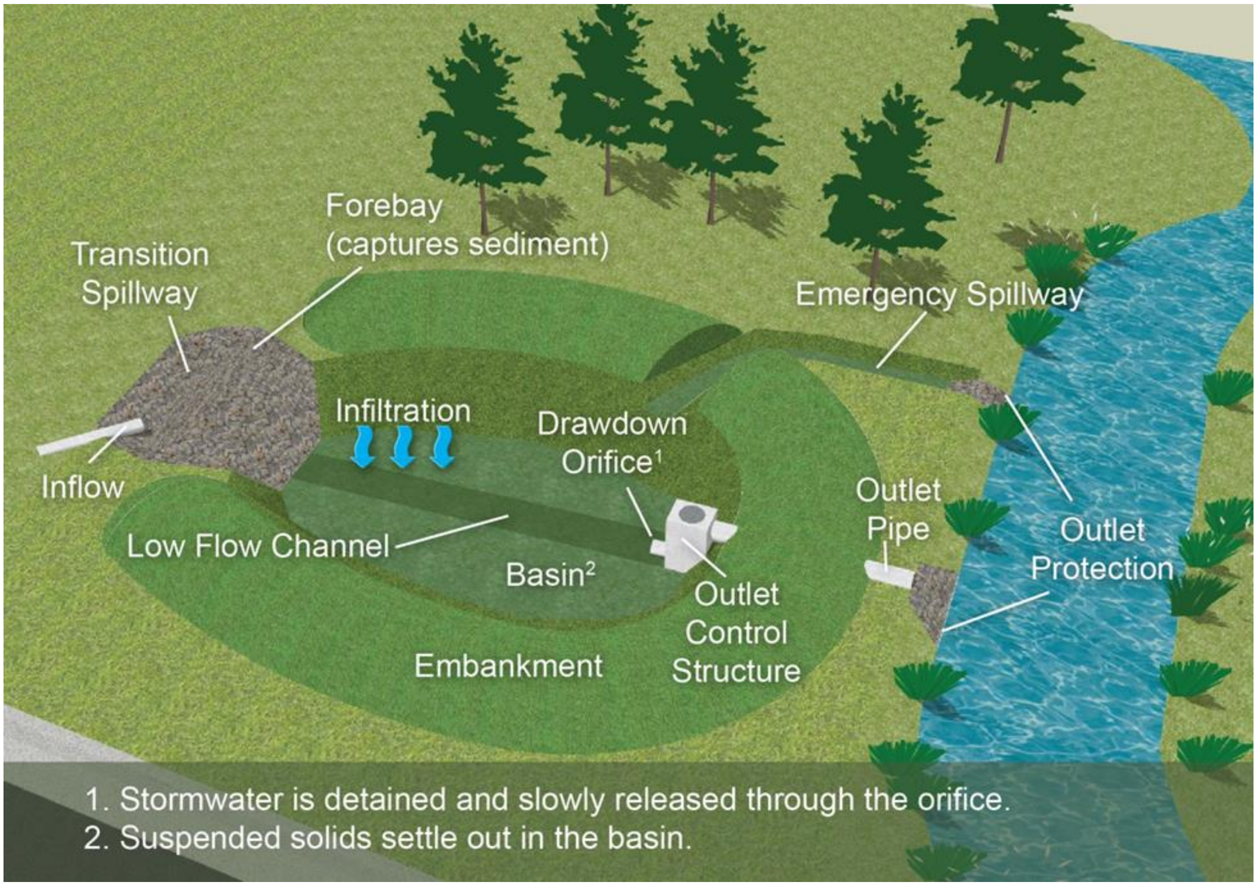
- The proposed basin functions effectively during the 10-year flood event.
- The proposed basin is structurally sound and safe under the 100-year flood conditions.
- The impacts to the characteristics of the 100-year floodplain due to the basin.

When basin construction is proposed within a floodplain, construction and permitting must comply with all applicable regulations under FEMA’s National Flood Insurance Program.

Outlet structures shall not be placed in the clear zone. Guardrail may only be placed for the purpose of protecting a dry detention basin if the BMP is warranted based on one or more of the section 2.2.3 warranting criteria.

In addition to attenuating stormwater runoff, another primary goal of detention design for roadway construction projects is to remove pollutants from the roadway construction activities. Dry detention basins may also reduce the required capacity, and therefore cost, of downstream drainage structures. Figure 2.6.8-2 illustrates typical dry detention basin components and treatment processes.

Figure 2.6.8-2 - Typical dry detention basin components and treatment processes



The location of the dry detention basin should be determined by considering a number of factors including: topography, cost, surrounding land use and development, and access. The location should be determined on a case-by-case basis using sound engineering judgment. As a general rule, detention basins should not be located in wetlands or other environmentally sensitive areas such as live streams. Under special circumstances, post-construction BMPs may be allowed within environmentally sensitive areas with prior consent from appropriate regulatory agencies. Siting information and constraints include:

- **Drainage Area** – Limit the contributing drainage area to 75 acres.
- **Site Slope** – Can be used on site with slopes up to about 15%.
- **Bedrock** – Avoid areas with shallow bedrock.
- **Depth to Water Table** – The bottom of the pond should have a minimum of 2 feet of separation from the seasonally high water table if over a water supply aquifer.
- **Hot Spots** – Can accept runoff from stormwater hotspots but need significant separation from groundwater when used for this purpose.
- **Trout Stream** – Should not be used were receiving water temperature is a concern. In addition, careful consideration should be given to the potential for perched or raised groundwater levels.

Challenges associated with roadway configurations include limited right-of-way and clear recovery zone requirements. Basins may be elongated to better fit the linear environment, if necessary. In addition, maintenance must be considered during the design and can often be challenging and hazardous for roadway BMPs. It is recommended that basins be designed with at least a 2:1 length to width ration.

Data for Design

The initial data needed for dry detention basin design includes the following:

- Existing and proposed site, topographic and location maps, and field reviews
- Aerial photographs of the drainage basin to estimate land use areas (grassed, paved, etc.)
- Preliminary plans including plan view, roadway and drainage profiles, cross sections, utility plans, and soil report
- Calculations and details from existing nearby detention facilities (if they have a hydrologic effect on the dry detention basin being designed)

The size and configuration of the dry detention basin will depend on stormwater management goals. Typically, detention basins are designed to capture and slowly release the CP_v over 24 hours, maintain the Q_{p25} at existing condition rates, and to adequately control the Q_f. However, one or more of these goals may be waived as described in section 2.4.

After initial data gathering and determining stormwater management requirements, the designer should proceed with an initial basin volume estimate using one of the following four methods as detailed in section 2.7, Detention Design:

- Hydrograph method
- Triangular hydrograph method
- NRCS procedure
- Regression equation

Next, a location and general configuration for the basin should be determined using the following criteria:

- Maximum depth of 10 feet
- Embankment side slopes should be 3:1 or flatter, however can be 2:1 with permission from the Office of Design Policy and Support

After a rough location and configuration are determined, follow the remaining steps outlined in section 2.7, Detention Design, for sizing and hydrograph routing. Then, integrate the remaining BMP components into the design. Remember that the cumulative flow from multiple detention basins within the same watershed can negatively impact receiving waters if hydrograph timing is not considered. Perform a hydrologic analysis for the project's zone of influence as described in section 2.2.3 of this chapter. For more information on the design of a dry detention basin, see the detailed calculation example located at the end of this section.

Pretreatment

Forebays should be provided at basin inlet areas to capture solids before the runoff enters the main basin. This will reduce clogging of drawdown orifices, extend the life of the BMP, and facilitate maintenance. Forebays should be sized for 0.1 inches of runoff per impervious acre. A small weir or transition spillway exiting the forebay may need to be included to direct low flows into the low flow channel.

Refer to section 2.8, Common BMP Components, for further guidance.

Low Flow Channel

A low flow channel constructed of riprap, or preferably a turf reinforcement mat to promote infiltration and interception of suspended sediments, should be provided to reduce the potential of nuisance conditions such as odors, insects, and weeds. Maximize the flow length of the channel by using a sinuous path to promote infiltration. Consider the drainage area size and groundwater levels when sizing the low flow channel. Refer to chapter 6 of the Drainage Design Policy Manual for channel design guidance and chapter 1 of this manual for additional guidance on rolled erosion control products.

Vegetation

Vegetation within the basin, on the side slopes, on the embankment, and the area immediately surrounding the basin should generally consist of a hearty turfgrass to prevent erosion. Alternatively, shrub species and other herbaceous species may be considered for highly visible areas where aesthetics are a greater concern. Do not plant trees in dry detention basins unless approved by the Office of Design Policy and Support.

Outlet Structure

The configuration of the outlet structure can vary greatly and will depend on stormwater requirements (i.e., WQ_v, CP_v, Q_{p25}, and Q_f). A typical configuration uses a riser/barrel configuration and emergency spillway to meet all requirements. The riser is typically a concrete structure with a low flow orifice at the elevation of the basin bottom for WQ_v treatment. The low flow orifice is used to detain the WQ_v and slowly release it over a 24-hour period. Alternatively, a metal cage with wire mesh and gravel can be used in lieu of a trash screen.

An additional low flow orifice used to detain the CP_v is located at the top of the water quality volume. This orifice should be properly sized and designed to release the difference between the CP_v and WQ_v over a 24-hour period.

According to GDOT’s published special construction details, a dry detention basin orifice is a hole drilled into the end of a PVC cap threaded onto a PVC pipe. The PVC pipe size shall be selected using the largest orifice designed for the dry detention basin. If the largest orifice designed for the basin is 2.0”-2.9” then the PVC pipe size will be 6”. If the largest orifice designed for the basin is 3.0”-5.0” then the PVC pipe size will be 8”. If an orifice less than 2” or greater than 5” is needed for a dry detention basin, contact the Office of Design Policy and Support before incorporating that orifice into the stormwater report or plans.

Weirs located near the top of the riser or the open throat of the riser are typically used to accommodate the Q_{p25}. The minimum height and width of the weir shall be 6-inches. The maximum width of a weir shall be the width or length of the outlet structure less 1-foot. For example, if the outlet structure is 5 feet wide, then the maximum weir width on that side of the outlet structure shall be 4 feet. Outlet protection should also be provided downstream of the outlet structure to protect against erosion (refer to chapter 7 of Drainage Design Policy Manual). Maximum release rates from the outlet structure should be equal or less than existing condition rates, for the storm events that are required to be studied. Refer to the GDOT Dry Detention Basin Outlet Structure Special Construction Detail for more information.

The hydrograph routing procedures and weir and orifice equations outlined in section 2.7 of this chapter are used to size the components of the outlet structure.

The buoyancy of the outlet structure should be determined and offset with proper anchoring and/or concrete. Refer to the American Concrete Pipe Association's (ACPA) document entitled, *Design Data 41 Manhole Flotation (*2008) [(2-3)] for additional information.

The outlet structure dimensions shall be based on the following table.

Table 2.6.8-1 Outlet Structure Dimensions

Pipe Diameter	Min Width	Min Length	Max Width / Length	Min Height	Max Height
18 in	4 ft	4 ft	8 ft	5 ft – 6 in	11 ft – 6 in
24 in	4 ft	4 ft	8 ft	5 ft – 6 in	11 ft – 6 in
30 in	5 ft	4 ft	8 ft	6 ft – 6 in	11 ft – 6 in
36 in	5 ft	4 ft	8 ft	6 ft – 6 in	11 ft – 6 in
42 in	6 ft	4 ft	8 ft	7 ft – 6 in	11 ft – 6 in
48 in	6 ft	4 ft	8 ft	7 ft – 6 in	11 ft – 6 in

Dimensions that exceed maximum width or length will require individual structural design. Maximum outlet structure size shall have an inside area of no greater than 49 square feet. Outlet structure shall be constructed at even one-foot (1 ft) increments. Dimensions of outlet structure shall be shown on special grading plans per special details. Refer to the Dry Detention Basin Outlet Structure Special Construction Detail for additional information.

Emergency Spillway

The emergency spillway is generally an open channel constructed in natural ground (as opposed to the embankment). The emergency overflow elevation shall be established at least one (1) foot below the roadway's normal shoulder break point and within 0.5 ft of the 100-year ponding elevation modeled with an unclogged outlet structure. The spillway shall be capable of conveying the 100-year storm modeled with a clogged outlet structure. The spillway shall be at minimum 8-feet wide. If including an emergency spillway in the design is not possible, size the weir(s) in the outlet structure so that they are capable of conveying the 100-year storm. Refer to the Dry Detention Basin Outlet Structure Special Construction Detail for additional information. Refer to the guidance given in chapter 6 of the Drainage Design Policy Manual for assistance in sizing the channel and determining an appropriate lining material.

Embankment

The embankment is a small earthen dam or fill section used to create the downslope side of the basin. Embankments must be designed to be less than 25 feet in height and detain less than 100 acre-feet in volume. Any pond volume equal to or greater than 10 acre-ft shall be coordinated directly with ODPS. Embankment height is measured from the elevation of the downstream toe to the maximum water storage elevation. Embankments that exceed these limits should be avoided and are subject to the Georgia Safe Dams Act of 1978. [(2-13)]

Side slopes should be 3:1 or flatter, however can be 2:1 with permission from the Office of Design Policy and Support. Overland flow should be minimized down embankment side slopes. A slope stability analysis is recommended for embankments higher than 10 feet and is required for slopes steeper than 2:1. Appropriate seepage control should be provided according to the size of the embankment and characteristics of the soils and basin configuration. Refer to the NRCS's Agriculture Handbook 590 [(2-33)] for additional guidance. Since shallow bedrock beneath the embankment may act as a conduit for seepage through the embankment, additional seepage prevention measures may be needed in these areas. Finally, the embankment should have 1 foot of freeboard above the 100-year flood elevation with additional consideration for embankment settlement.

The top of the dry detention basin should have an 8 feet wide berm or bench graded all around the basin, both in cut and in fill sections. The top of the berm or bench may be sloped up to 4% towards the inside of the basin.

Refer to GDOT Special Provision / Specification 169 on Post-Construction Stormwater BMP Items for additional design guidance and construction considerations.

Access and Driveway Considerations

See section 2.10.3 for maintenance access requirements.

Dry Detention Basin Sizing

1. Determine the goals and primary function of the dry detention basin.

   The goals and primary function of the BMP must take into account any restrictions or site-specific constraints. Also take into consideration any special surface water or watershed requirements. Consider whether the dry detention basin is intended to:

   - Meet a water quality (treatment) target in addition to providing detention.
   - Provide a possible solution to a drainage problem

2. Calculate the Target Water Quality Volume

   Calculate the water quality volume formula using the following formula:

$$WQ_v = \frac{1.2\ in \times (R_v) \times A \times 43560\,\frac{ft^2}{acre}}{12\,\frac{in}{ft}}$$

Where:

WQ_v = water quality volume (ft^3)

R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.
A = onsite drainage area of the post-condition basin (acres)

3. Calculate the CP_v, Q_{p25}, and Q_f flow rates and volumes.
4. Determine the pretreatment volume.

   A sediment forebay is provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the basin. The forebay should be sized to contain 0.1 inch per impervious acre of contributing drainage.
5. Design the outlet control structure.
6. Design embankment(s) and spillway(s).

   Size the emergency spillway, calculate the 100-year water surface elevation, set the top of the embankment elevation, and analyze safe passage of the Q_f.
7. Investigate potential basin hazard classification.

   The design and construction of the dry detention basin may be required to meet the Georgia Dam Safety standards.

Maintenance Considerations

Without proper maintenance, BMPs will function at a reduced capacity and may cease to function altogether. A properly designed BMP includes the following considerations for maintenance:

- Access:
  - Provide adequate right-of-way.
  - Provide access roads and ramps for appropriate equipment to all applicable components (outlet structure, forebay, etc.).
  - Provide space to turn around if necessary.
  - Check for sufficient area to safely exit and enter the highway, if applicable.
  - If the BMP is fenced, provide appropriately sized gates (refer to section 2.10 for additional guidance regarding fencing and other safety considerations).
- Provide a valve or other method for dewatering the basin if deemed appropriate.

Refer to GDOT's *Stormwater System Inspection and Maintenance Manual,* for specific maintenance requirements.

Dry Detention Basin Example Calculation

GIVEN:

- A new roadway project located in Savannah, GA.
- The proposed project includes 1,300 feet of roadway (in length).
- The drainage area that discharges to the dry detention basin includes the following: two 12-foot lanes, a 6-foot paved shoulder, and a 20-foot wide grassed area, draining via sheet flow.
- Assume no stormwater is collected as “off-site” or “bypass” runoff.
- Assume the basin depth will be 3 feet for the purposes of this example.
- Note that a separate hydrograph routing example calculation is given to illustrate the calculations associated with the Q_{p25} and Q_f.
- Assume that water quality treatment will be provided upstream (prior to) the detention basin.

FIND:

- Size the dry detention basin and drawdown orifice to capture and release the CP_v over a 24-hour period.

SOLUTION:

1. Since the dry detention basin will not be sized for the water quality volume, the first step is to size the basin by determining CP_v using guidelines and information from section 2.4.2:

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

Where:

Q = accumulated direct runoff (in) (Q = CP_v in this case)
P = accumulated rainfall (in)
S = potential maximum soil retention (in)

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

Where:

CN = SCS curve number (most drainage areas will require a composite CN)

A comprehensive list of curve numbers is provided in TR-55. A composite curve number should be calculated for multiple land uses. For example:

$$CN_{composite} = \frac{CN_1A_1 + CN_2A_2 + CN_3A_3}{A_1 + A_2 + A_3}$$

Where:

A = surface area

$$CN_{composite} = \frac{(98)(30\ ft \times 1{,}300\ ft) + (69)(20\ ft\ \times 1{,}300\ ft)}{(50\ ft \times 1{,}300\ ft)} = 86.4$$

$$S = \frac{1000}{86.4} - 10 = 1.574$$

From NOAA Precipitation Frequency Data Server (Atlas 14):

P (1-yr, 24 –hr Savannah) = 3.86 in

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

$$CP_v = \frac{[3.86 - 0.2(1.574)]^2}{[3.86 + 0.8(1.574)]} = 2.46\ inches\ of\ runoff$$

$$CP_v = (2.46\ in)\left(\frac{1\ ft}{12\ in}\right)(1{,}300\ ft\ \times 50\ ft) = 13{,}325\ ft^3\ (required\ volume)$$

2. The next step is to determine the pretreatment volume. The forebay should be sized to contain 0.1 inch per impervious acre of contributing drainage.

$$V_{pretreat} = 0.1\ inches\ \times \frac{1\ ft}{12\ inches} \times (30\ ft\ \times 1{,}300\ ft) = 325\ ft^3$$

3. Size the orifice to release CPv in 24 hours:

$$Q = C_D A\sqrt{2g\Delta H}$$

Where:

Q = Discharge, ft^3/s
C_D = Coefficient of discharge, 0.6 for a sharp-edged orifice
A = Area of the orifice, ft^2
g = Acceleration of gravity = 32.2 ft/s^2
ΔH = Difference in head across the orifice, ft

The average required flow rate (Q) from the orifice can be determined by dividing the CP_v by the 24-hr detention time.

$$Q = \left(\frac{13{,}325\ ft^3}{24\ hr}\right)\left(\frac{1\ hr}{60\ min}\right)\left(\frac{1\ min}{60\ sec}\right) = 0.154\ ft^3/s$$

The orifice equation can be rearranged to solve for area:

$$A = \frac{Q}{C_D\sqrt{2g\Delta H}}$$

As an approximation, we can use the basin depth of 3 feet to assume an average ΔH of 1.5 ft for the entire 24-hr detention period.

$$A = \frac{0.154}{0.6\sqrt{2(32.2)(1.5)}} = 0.0261\ ft^2$$

Finally, assuming a round orifice, the orifice diameter can be determined.

$$D = \left(\frac{4A}{\pi}\right)^{1/2} = 0.182\ ft = 2.19\ in$$

The orifice diameter should be no larger than 2.19 inches and should be rounded down to the nearest constructible value. Because the orifice is less than 3 inches in diameter, internal orifice protection should be provided.

Note that detention of the CP_v is not required for discharges less than 2.0 ft^3/s under normal circumstances. Using the TR-55 method, peak flow from the 1-yr, 24-hr storm for this site was estimated at 3.4 ft^3/s.

Summary

2.6.9 Wet Detention Pond

Advantages	Disadvantages
• Provides aesthetic value • Cost-effective BMP that provides good treatment • Provides wildlife habitat	• Requires a large footprint • Difficulties in maintaining the permanent pool may arise • Standing water can create a safety concern. See section 2.10.2 for information on public safety

Description: An earthen pond with permanent pool and temporary storage for attenuating peak flows.

Design Considerations:

- Size to store the WQ_v (part or all of which can be in the permanent pool) plus CP_v and release over 24 hours; other requirements may apply
- Outflow hydrograph should mimic the predevelopment hydrograph
- Drainage area should be between 10 and 75 acres
- Maximum permanent pool depth of 8 feet, 6 feet preferred
- Minimum permanent pool depth of 3 feet
- Maximum total depth of 18 feet
- Maximum side slopes of 3:1

Maintenance Considerations:

- Provide a means of draining the basin for maintenance activities
- Design outlet structure to resist clogging

Applicability for Roadway Projects:

- Space requirements and flooding concerns may limit applicability in the linear environment
- GDOT does not typically own sufficient right-of-way and property is usually expensive in locations where wet ponds are most desired (urban communities)

Stormwater Management Suitability:

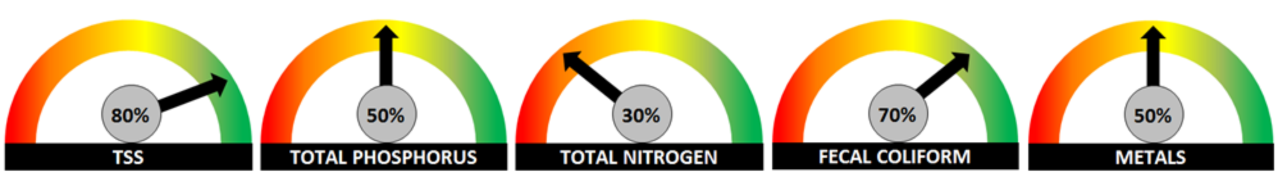
X Runoff Reduction
✓ Water Quality
✓ Channel Protection
✓ Overbank Flood Protection
✓ Extreme Flood Protection
X Temperature Reduction

✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

It is generally not practical or cost-effective to design small ponds close to the source of runoff as LID dictates. However, wet ponds employ multiple LID/GI characteristics such as providing infiltration and evapotranspiration. In addition, wet ponds create the opportunity for water harvesting if there is a demand for irrigation on adjacent properties.

Treatment Capabilities



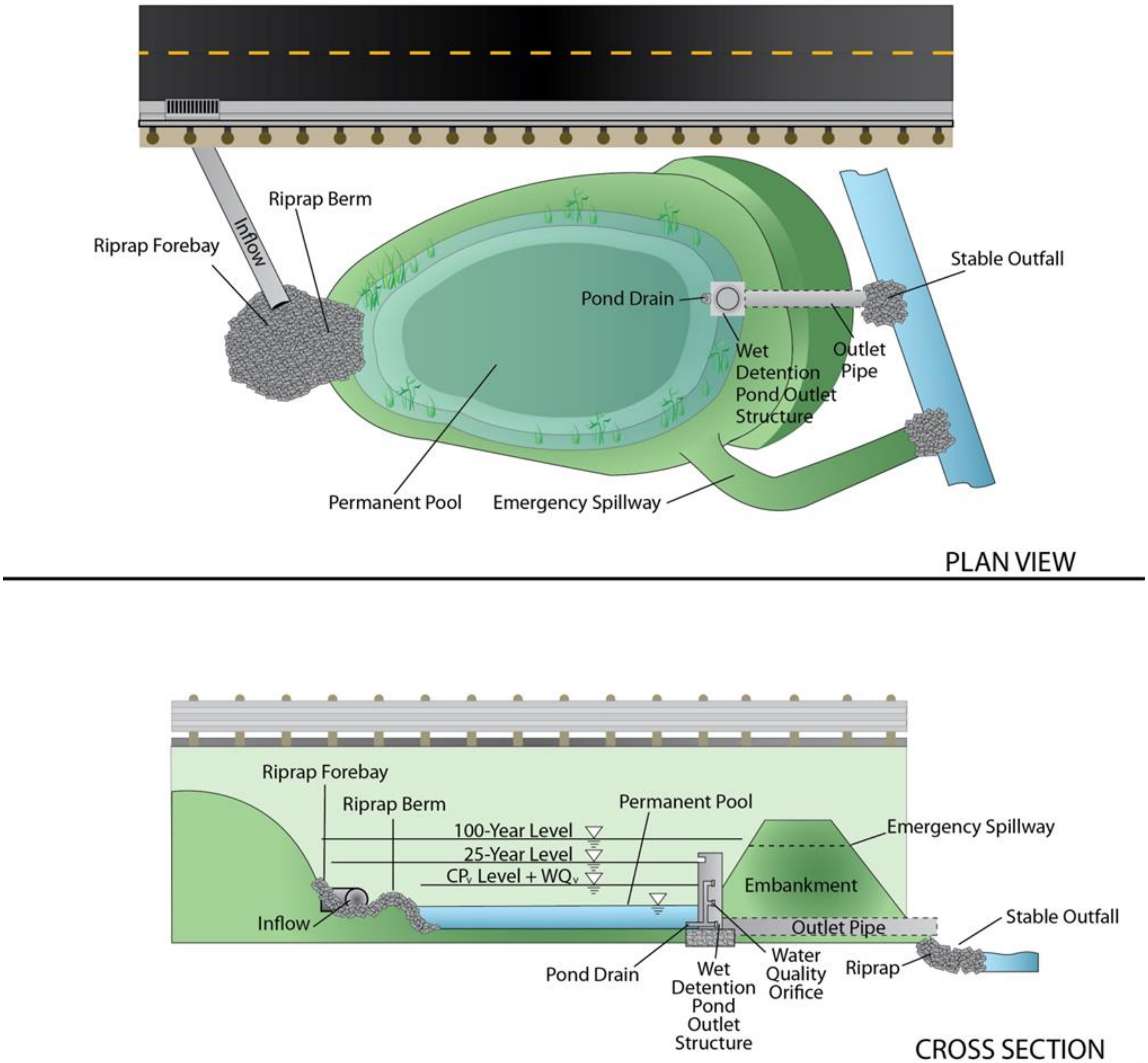
2.6.9 Wet Detention Pond

Description

A wet detention pond is an earthen impoundment that maintains a permanent pool of water and has additional storage for detaining runoff and attenuating peak flows. As such, wet detention ponds provide benefits similar to dry detention basins (i.e., reducing peak flows to existing condition rates and preventing stream channel erosion). Wet detention ponds also provide runoff water quality treatment. The permanent pool provides an area for sediment storage, reducing TSS and the associated pollutants adhered to these particles. Contact with the permanent pool and surrounding vegetation results in chemical and biological processes that reduce nutrients, metals, and pathogens.

Wet detention ponds can be used to meet WQ_v, CP_v, Q_{p25}, and Q_f requirements. A riser with a small orifice that is elevated a few feet off of the basin bottom creates the permanent pool and allows the pond to store additional runoff for a short period of time (24 hours for CP_v). The dimensions of the permanent pool can vary depending on the space available. To address the different stormwater requirements previously listed, the GSMM [(2-17)] presents multiple types of wet ponds (e.g., wet extended detention pond, micropool extended detention pond). Runoff in excess of the CP_v is released through additional weirs/orifices higher on the riser, the top of the riser, and/or an emergency spillway channel. Figure 2.6.9-1 illustrates a typical wet detention pond configuration.

Figure 2.6.9-1 - Typical wet detention pond configuration



Stormwater Management Suitability

- Runoff Reduction – Wet detention ponds provide negligible stormwater volume runoff reduction. Another BMP should be used in a treatment train with stormwater ponds to provide runoff reduction.
- Water Quality – Wet detention ponds treat incoming stormwater runoff by physical, biological, and chemical processes. The primary removal mechanism is gravitational settling of particulates, organic matter, metals, bacteria, and organics as stormwater runoff resides in the pond. Pollutant removal is also provided through uptake by algae and wetland plants in the permanent pool—particularly of nutrients. Volatilization and chemical activity also work to

break down and eliminate a number of other stormwater contaminants, such as hydrocarbons. A wet detention pond provides 80% TSS removal if designed, constructed, and maintained correctly.

- Channel Protection – A portion of the storage volume above the permanent pool in a wet detention pond can be used to provide control of the channel protection volume (CP_v). This is accomplished by releasing the 1-year, 24-hour storm runoff volume over 24 hours (extended detention).
- Overbank Flood Protection – A stormwater pond can also provide storage above the permanent pool to reduce the post-development peak flow of the 25-year, 24-hour storm (Q_{p25}) to pre-development levels (detention).
- Extreme Flood Protection – In situations where it is required, stormwater ponds can also be used to provide detention to control the 100-year, 24-hour storm peak flow (Q_f). Where this is not required, the pond structure is designed to safely pass extreme storm flows.

Pollutant Removal Capabilities

Wet detention ponds provide good treatment and detention and can be cost-effective BMPs in certain applications. [(2-17)] The following average pollutant removal rates may be utilized for design purposes: [(2-17)]

- TSS – 80%
- TP – 50%
- TN – 30%
- Fecal coliform – 70%
- Heavy metals – 50%
- Temperature – Temperature reduction is not provided.

Application and Site Suitability

Although they provide many water quality benefits, wet detention ponds are sometimes difficult to implement in roadway settings due to space requirements and safety concerns associated with the permanent pool. Further, an adequate supply of runoff is necessary to maintain the permanent pool. Figure 2.6.9-2 illustrates typical wet detention pond components and treatment processes.

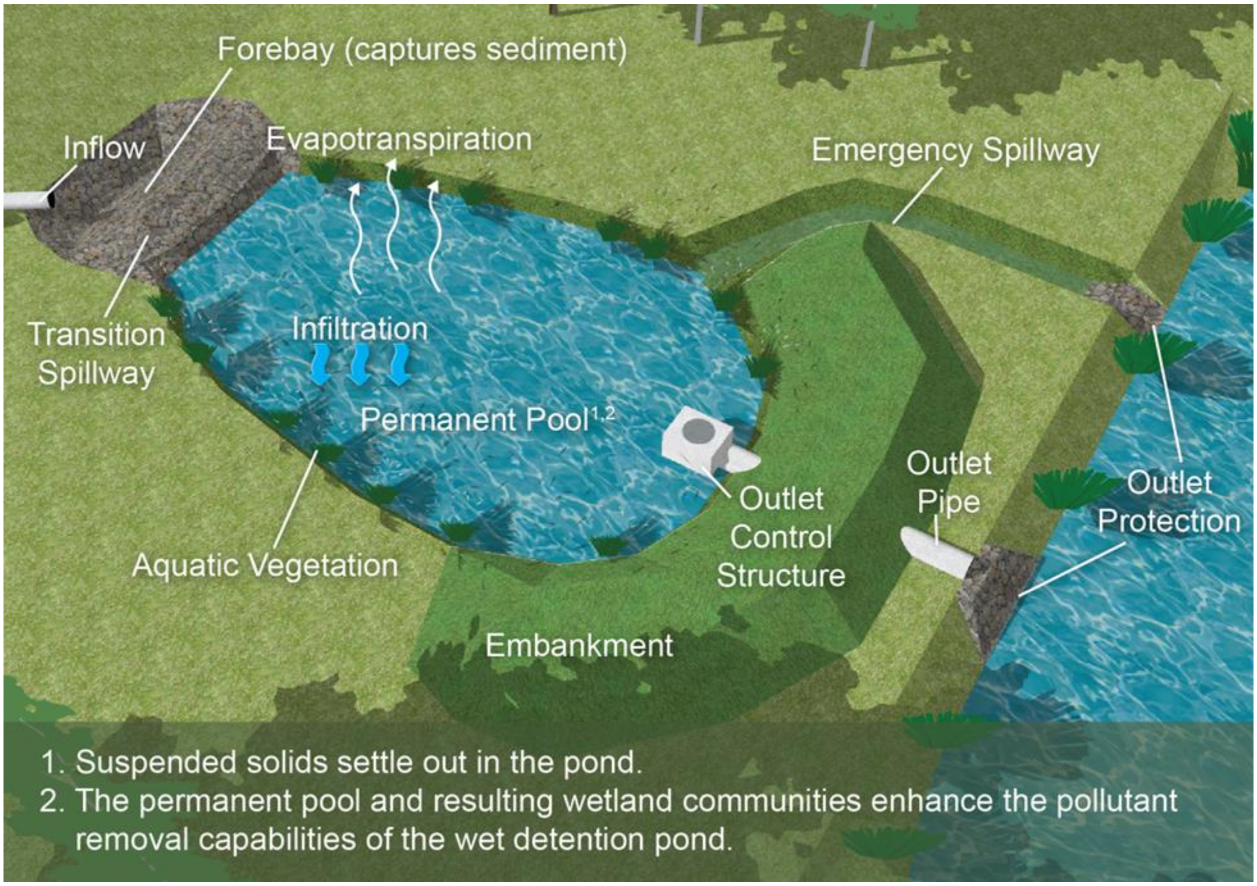
The construction of wet detention ponds within floodplains is strongly discouraged. When the situation is deemed unavoidable, the following must be thoroughly evaluated and shown in the MS4 Post-Construction Stormwater Report:

- The proposed pond functions effectively during the 10-year flood event.
- The proposed pond is structurally sound and safe under the 100-year flood conditions.
- The impacts to the characteristics of the 100-year floodplain due to the pond.

When pond construction is proposed within a floodplain, construction and permitting must comply with all applicable regulations under FEMA’s National Flood Insurance Program.

Outlet structures shall not be placed in the clear zone. Guardrail may only be placed for the purpose of protecting a wet detention basin if the BMP is warranted based on one or more of the section 2.2.3 warranting criteria.

Figure 2.6.9-2 - Typical wet detention pond components and treatment processes



The location of the wet detention pond will be determined by considering a number of factors including topography, cost, surrounding land use and development, and access. The location should be determined on a case-by-case basis using sound engineering judgment. As a general rule, detention ponds should not be located in wetlands or other environmentally-sensitive areas such as live streams. Under special circumstances, post-construction BMPs may be allowed within environmentally sensitive areas with prior consent from appropriate regulatory agencies. Wet pond depths should be varied to meet different objectives. Impacts to adjacent properties resulting from wetland systems requiring shallow depths (e.g., odors, insects) must be evaluated. Alternatively, deep water may be desired to provide a cool water release and/or fish habitat. Siting information and constraints include:

- **Drainage Area –** The contributing drainage area should be limited to 75 acres. Minimum drainage area of 10 acres is required to maintain the permanent pool (unless groundwater is present).
- **Space Required –** The pond usually occupies 2 to 3% of the total drainage area.

- **Depth to Water Table** – A wet detention pond can be used where the water table is at or near the soil surface, or where there is a sufficient water balance in poorly drained soils to support a wetland plant community. If above an aquifer or treating a hotspot, however, 2 feet is required between the bottom of the pond and the elevation of the seasonally high water table. Where wet detention ponds do not intercept the groundwater table, a liner must be installed on HSG A and B soils. A water balance calculation should be performed to ensure an adequate water budget to support the specified wetland species. A water balance analysis may not be necessary if a liner is installed but should be considered regardless if the drainage area is small and/or has a small amount of impervious area. The wet detention pond size may need to be adjusted to account for lost volume due to seasonal fluctuations in the groundwater table.
- **Site Slope –** There should not be more than 15% slope across the drainage area to the pond.
- **Minimum Head** – 6-8 feet of elevation difference needed onsite from the inflow to the outflow.
- **Setbacks –**
    - Property lines – 10 feet (site development projects only)
    - Private wells – 100 feet
    - Septic systems – 50 feet
    - Public-use airports – 5 miles
- **Trout Stream** – Consideration should be given to the thermal influence of stormwater pond outflows on downstream trout waters. Wet detention ponds can be designed off-line and under shade to minimize their thermal impact.

Challenges associated with roadway configurations include limited right-of-way and clear recovery zone requirements. Basins may be elongated to better fit the linear environment, if necessary. In addition, because it can often be challenging and hazardous to maintain roadway BMPs, maintenance access is an important consideration during BMP design.

Data for Design

The initial data needed for wet detention pond design include the following:

- Existing and proposed site, topographic and location maps, and field reviews
- Aerial photographs of the drainage basin to estimate land use areas (grassed, paved, etc.)
- Preliminary plans including plan view, roadway and drainage profiles, cross sections, utility plans, and soil report
- Calculations and details from existing nearby detention facilities
- Water table information

The size and configuration of the wet detention pond will depend on stormwater management requirements. Typically, wet detention ponds are designed to provide treatment for water quality, capture and slowly release the CP_v over 24 hours, maintain the Q_{p25} at existing condition rates, and to adequately control the Q_f. However, one or more of these goals may be waived as described in section 2.4 of this manual.

After initial data gathering and determination of stormwater management requirements, the designer should proceed with an initial basin volume estimate using one of the following four methods as detailed in section 2.7, Detention Design, in this manual:

- Hydrograph method
- Triangular hydrograph method
- NRCS procedure
- Regression equation

Next, a location and general configuration for the basin should be determined using the following criteria:

- Permanent pool depths greater than 8 feet are not recommended. If a permanent pool depth greater than 8 feet is proposed, additional approval is required from the Office of Design Policy and Support.
- Minimum permanent pool depth of 3 feet
- Maximum total depth of 18 feet
- Side slopes should be 3:1 or flatter
- Embankment side slopes should be 3:1 or flatter, however can be 2:1 with permission from the Office of Design Policy and Support
- Minimum length to width ratio of greater than 2:1 is preferred

After a rough location and configuration are determined, follow the remaining steps outlined in section 2.7, Detention Design, for sizing and hydrograph routing. Then, integrate the remaining BMP components into the design. Remember that the cumulative flow from multiple detention facilities within the same watershed can negatively impact receiving waters if hydrograph timing is not considered. Perform a hydrologic analysis for the project's zone of influence as described in section 2.2.3 of this chapter. For more information on the design of a wet detention pond, see the detailed calculation example located at the end of this section.

Pretreatment

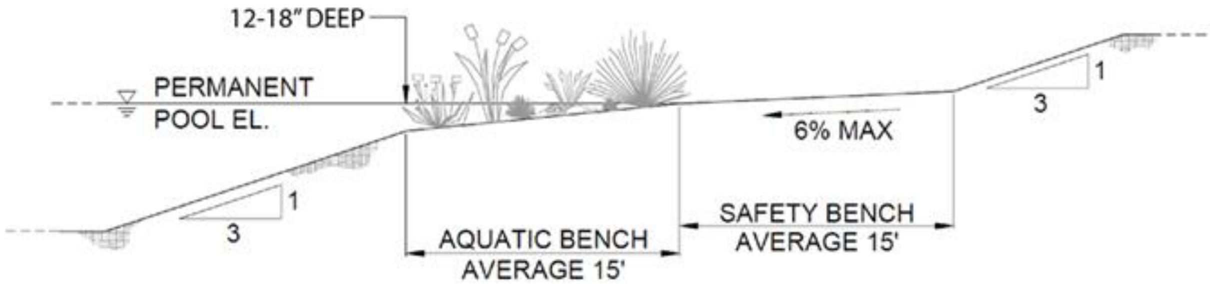
Forebays should be provided at basin inlet areas to capture solids before the runoff enters the main basin. This will reduce clogging of drawdown orifices, extend the life of the BMP, and facilitate maintenance. Forebays should be sized for 0.1 inches of runoff per impervious acre.

Refer to section 2.8, Common BMP Components, for further guidance.

Aquatic and Safety Benches

A safety bench should be provided to help prevent maintenance personnel and the public from slipping into the pond. The safety bench should start at the edge of the permanent pool and extend outward approximately 15 feet (may be less for smaller ponds). The maximum slope of the safety bench should be 6%. The safety bench may be omitted for ponds with side slopes of 4:1 or less. In addition, an aquatic bench should be provided for emergent wetland vegetation. Shallow areas with wetland vegetation provide additional treatment. The aquatic bench should also be approximately 15 feet for average and large ponds. The aquatic bench begins at the edge of the permanent pool and extends inwards to a depth of 12 to 18 inches. Figure 2.6.9-3 provides an illustration of a typical aquatic and safety bench configuration.

Figure 2.6.9-3 - Typical aquatic and safety bench configuration (adapted from GSMM Vol. 2) (2-17)



Vegetation

A landscaping plan is required for wet detention pond design. The landscaping plan will include a list of the proposed plant species, source of where the plants are obtained, the planting sequences, and post-nursery maintenance requirements. A professional landscape architect may be consulted.

Vegetation surrounding the normal pool and along the safety bench should be water tolerant wetland species. Native, non-invasive species are preferred. Aquatic vegetation helps remove pollutants and provides wildlife habitat and aesthetic benefits. The remaining areas should generally consist of a hearty turfgrass to prevent erosion. Refer to the GDOT Planting Schedule Special Construction Detail for more information.

Although trees typically provide added water quality benefits, they can obstruct maintenance operations and roots can damage underdrains. Only if conditions allow, taller vegetation and trees may be planted around the wet detention pond to discourage waterfowl from taking residence in the pond as they can add to nutrient and bacteria loads. Woody vegetation (e.g., trees and shrubs) should not be planted on the embankment.

Outlet Structure

The configuration of the outlet structure can vary and will depend on stormwater requirements (i.e., WQ_v, CP_v, Q_{p25}, and Q_f). A typical configuration uses a riser/barrel configuration and emergency spillway to meet all requirements. The riser is typically a concrete structure with a small orifice that is elevated a few feet off of the basin bottom to set the normal pool elevation. The normal pool dimensions can be adjusted so that the BMP will fit within the allowable area. The minimum normal pool volume should be equal to 0.1 inches per impervious acre. For larger areas, the normal pool should be equal to the WQ_v, since this exceeds the 0.1 inches per impervious acre.

The outlet structure should be designed to allow the water level in the pond to rise above the permanent pool elevation as runoff (usually the CP_v) is detained, and then slowly draw it down over 24 hours. This 24-hour period may be reduced to 12 hours where runoff temperature is a concern, near trout streams for example. In addition, the orifice can be positioned lower to draw off cooler water.

According to GDOT's published special construction details, a wet detention basin orifice is a hole drilled into the end of a PVC cap threaded onto a PVC pipe. The PVC pipe size shall be selected using the largest orifice designed for the wet detention basin. If the largest orifice designed for the basin is 2.0"-2.9" then the PVC pipe size will be 6". If the largest orifice designed for the basin is 3.0"-5.0" then the PVC pipe size will be 8". If an orifice less than 2" or greater than 5" is needed for a wet detention basin, contact the Office of Design Policy and Support before incorporating that orifice into the stormwater report or plans.

Weirs created towards the top of the riser, or the open throat of the riser, are typically used to accommodate the Q_{p25} and should be located at an elevation that allows for the storage of the WQ_v and the CP_v. The minimum height and width of the weir shall be 6-inches. The maximum width of a weir shall be the width or length of the outlet structure less 1-foot. For example, if the outlet structure is 5 feet wide, then the maximum weir width on that side of the outlet structure shall be 4 feet. Outlet protection should be provided downstream of the outlet structure to protect against erosion (refer to chapter 9 of this manual). Maximum release rates from the outlet structure should be targeted towards pre-project rates. The outlet structure contains a small pipe with a threaded end cap at the lowest elevation of the pond in the event that the pond needs to be drained completely. Accessibility to the cap may be difficult at times, depending on the design depth and configuration of the pond, so it is best that the location of the outlet control structure itself be as close to the embankment as possible to accommodate access.

The outlet structure dimensions shall be based on the following table.

Table 2.6.9-1 Outlet Structure Dimensions

Pipe Diameter	Min Width	Min Length	Max Width / Length	Min Height	Max Height
18 in	4 ft	4 ft	6 ft – 6 in	5 ft – 6 in	19 ft – 6 in
24 in	4 ft	4 ft	6 ft – 6 in	5 ft – 6 in	19 ft – 6 in
30 in	5 ft	4 ft	6 ft – 6 in	6 ft – 6 in	19 ft – 6 in
36 in	5 ft	4 ft	6 ft – 6 in	6 ft – 6 in	19 ft – 6 in
42 in	6 ft	4 ft	6 ft – 6 in	7 ft – 6 in	19 ft – 6 in
48 in	6 ft	4 ft	6 ft – 6 in	7 ft – 6 in	19 ft – 6 in

Dimensions that exceed maximum width or length will require individual structural design. Maximum outlet structure size shall have an inside area of no greater than 30.5 square feet. Outlet structure shall be constructed at even one-foot (1 ft) increments. 4 foot by 4 foot outlet structures shall have a maximum height of 8 feet. Dimensions of outlet structure shall be shown on special grading plans per special details. Refer to the GDOT Wet Detention Pond Outlet Structure Special Construction Detail for additional guidance.

The hydrograph routing procedures and weir and orifice equations outlined in section 2.7 of this chapter are used to size the components of the outlet structure.

The buoyancy of the outlet structure should be determined and offset with proper anchoring and/or concrete. Refer to the ACPA document entitled, ***Design Data 41 Manhole Flotation*** (2008) (2-3) for additional information.

Water Balance

Install an impermeable liner if the wet detention pond is located on HSG A or B soils and the pond does not intercept the groundwater table. A water balance analysis should be performed for systems on HSG C and D soils. Refer to section 2.2.4 for water balance calculations. Infiltration testing will be completed during construction to determine if a liner is needed if the HSG is C or D. Specify if infiltration testing is needed on the plans.

Emergency Spillway

The emergency spillway is generally an open channel constructed in natural ground (as opposed to the embankment). The emergency overflow elevation shall be established at least one (1) foot below the roadway's normal shoulder break point and within 0.5 ft of the 100-year ponding elevation modeled with an unclogged outlet structure. The spillway shall be capable of conveying the 100-year storm modeled with a clogged outlet structure. If including an emergency spillway in the design is not possible, size the weir(s) in the outlet structure so that they are capable of conveying the 100-year storm. Refer to the Wet Detention Basin Outlet Structure Special Construction Detail for additional information. Refer to the guidance given in chapter 6 of the Drainage Design Policy Manual for assistance in sizing the channel and determining an appropriate lining material.

Embankment

The embankment is a small earthen dam or fill section used to create the downslope side of the basin. Embankments must be designed to be less than 25 feet in height and should detain less than 100 acre-feet in volume. The roadway embankment shall not be used as a dam for impounding water except when the wet detention basin has a volume of less than 5 acre-ft. When this is the case, a 10 ft berm shall be separate the roadway embankment from the top of the basin.

Embankment height is measured from the elevation of the downstream toe to the maximum water storage elevation. Embankments that exceed these limits should be avoided and are subject to the Georgia Safe Dams Act of 1978 (OCGA 12-5-370) (2-13) unless the basin has been excavated and fill was not used to create the dam.

Embankment side slopes should be 3:1 or flatter, however can be 2:1 with permission from the Office of Design Policy and Support. Overland flow should be minimized down embankment side slopes. A slope stability analysis is recommended for embankments higher than 10 feet and is required for slopes steeper than 2:1. Appropriate seepage control should be provided according to the size of the embankment and characteristics of the soils and basin configuration. Refer to the NRCS's Agriculture Handbook 590 (2-33) for additional guidance. Since shallow bedrock beneath the embankment may act as a conduit for seepage through the embankment, additional seepage prevention measures may be needed in these areas. Finally, the embankment should have 1 foot of freeboard above the 100-year flood elevation with additional consideration for embankment settlement.

The top of the wet detention basin shall have an 8 feet wide berm or bench graded all around the basin, both in cut and in fill sections. The top of the berm or bench may sloped up to 4% towards the inside of the basin.

Refer to GDOT Special Provision / Specification 169 on Post-Construction Stormwater BMP Items for additional design guidance and construction considerations.

Access and Driveway Considerations

See section 2.10.3 for maintenance access requirements.

Wet Detention Pond Sizing

1. Determine the goals and primary function of the wet detention pond.

   The goals and primary function of the BMP must take into account any restrictions or site-specific constraints. Also take into consideration any special surface water or watershed requirements.

2. Calculate the Target Water Quality Volume

   Calculate the water quality volume formula using the following formula:

$$WQ_v = \frac{1.2\ in \times (R_v) \times A \times 43560 \frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

   Where:

   WQ_v = water quality volume (ft^3)
   R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.
   A = onsite drainage area of the post-condition basin (acres)

3. Determine the permanent pool volume.

   - Wet Pond: Size permanent pool volume to 1.0 WQ_v
   - Wet ED Pond: Size permanent pool volume to 0.5 WQ_v and extended detention volume to 0.5 WQ_v
   - Micropool ED Pond: Size permanent pool volume to 25-30% of WQ_v and extended detention volume to remainder of WQ_v

4. Determine the pretreatment volume.

   A sediment forebay is provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the basin. The forebay should be sized to contain 0.1 inch per impervious acre of contributing drainage.

5. Determine the pond location and preliminary geometry. Conduct pond grading and determine storage volume available for the permanent pool (and water quality extended detention volume as appropriate).

   This step involves initially grading the pond (establishing contours) and determining the elevation-storage relationship for the pond.

   - Include safety and aquatic benches

- Set WQ_v permanent pool elevation (and WQ_v-ED elevation for wet ED and micropool ED ponds)

6. If applicable, complete a water balance analysis to verify the wet detention pond will maintain its permanent pool.

- For the infiltration component of the water balance, the vertical projection of both the side slopes and pond bottom to the pond surface should be used for the permanent pool area to account for infiltration through the side slopes.

7. Compute extended detention orifice release rate(s) and size(s), and establish CP_v elevation.

- Wet Pond: The CP_v elevation is determined from the stage-storage relationship and the orifice is then sized to release the difference between the water quality volume and channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water stream basins).
- Wet ED Pond and Micropool ED Pond: Based on the elevations established in Step 5 for the extended detention portion of the water quality volume, the water quality orifice is sized to release this extended detention volume in 24 hours. The CP_v elevation is then determined from the stage-storage relationship. The invert of the channel protection orifice is located at the water quality extended detention elevation, and the orifice is sized to release the difference between the water quality volume and channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams).

8. Calculate the Q_{p25} release rate and water surface elevation.

   Set up a stage-storage-discharge relationship for the control structure for the extended detention orifice(s) and the 25-year, 24-hour rainfall event.

9. Design embankment(s) and spillway(s).

   To size the emergency spillway, calculate the 100-year, 24-hour storm water surface elevation. The emergency overflow elevation should be set at the ponding elevation for the 100-year storm event and should be at least 1 foot below the roadway's normal shoulder break point and analyze safe passage of the Extreme Flood Volume (Qf). At final design, provide safe passage for the 100-year, 24- hour rainfall event.

10. Verify pond embankment design does not trigger Georgia Safe Dams hazard classification.
    Embankments must be designed to be less than 25 feet in height and should detain less than 100 acre-feet in volume. Embankment height is measured from the elevation of the downstream toe to the maximum water storage elevation. Embankments that exceed these limits should be avoided and are subject to the Georgia Safe Dams Act of 1978 (OCGA 12-5-370) [(2-13)] unless the basin has been excavated and fill was not used to create the dam.

11. Prepare a site vegetation and landscaping plan.
    A landscaping plan for a stormwater pond and its buffer should be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation. See the GDOT Planting Schedule Special Construction Detail for more information.

Maintenance Considerations

Without proper maintenance, BMPs will function at a reduced capacity and may cease to function altogether. A properly designed BMP includes several considerations for maintenance:

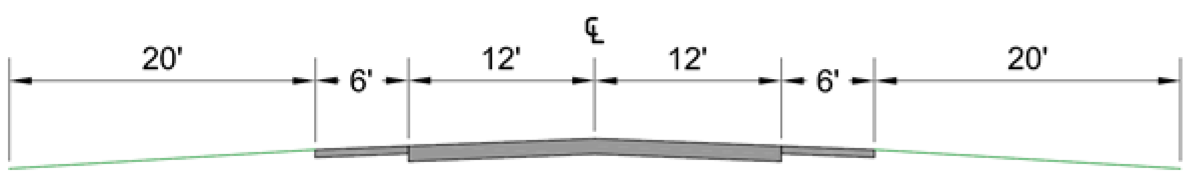
- Access:
    - Provide adequate right-of-way.
    - Provide access roads and ramps for appropriate equipment to all applicable components (outlet structure, forebay, etc.).
    - Provide space to turn around if necessary.
    - Check for sufficient area to safely exit and enter the highway, if applicable.
    - If the BMP is fenced, provide appropriately sized gates (refer to section 2.10 for additional guidance regarding fencing and other safety considerations).
    - Adequate access for a small boat may be needed for sediment depth measurements.

Refer to GDOT's *Stormwater System Inspection and Maintenance Manual*, for specific maintenance requirements.

Wet Detention Basin Example Calculation

GIVEN:

- A new roadway project located in Savannah, GA.
- The proposed project includes 3,000 feet of roadway (in length).
- The drainage area that discharges to the wet detention pond includes the following: two 12-foot lanes, two 6-foot paved shoulders, and two 20-foot wide grassed areas (on either side of the road) draining via sheet flow.
- Soil underlying the wet detention basin is sandy clay loam.
- Offsite stormwater also provides supplemental runoff to maintain permanent pool (assume 5 acres (217,800 ft^2) of undeveloped land for the purposes of this example).
- Pond dimensions were simplified and assumed for the purposes of this example.
- The designer has previously calculated the following hydrologic information:
    - Min permanent pool = 0.1 inches × Impervious Acreage = 0.021 ac-ft (915 ft^3)
    - Upper end permanent pool = WQ_v = 0.251 ac-ft (10,934 ft^3) (See section 2.4.1.2 for additional guidance)
    - CP_v = 13,325 ft^3 (See section 2.4.2 for additional guidance)



FIND:

- Size the wet detention pond permanent pool, temporary storage, and drawdown orifices to capture and release the WQ_v and CP_v over 24 hours.
- Perform a water balance calculation to verify that the permanent pool will be maintained to an acceptable degree.
- Note that a separate hydrograph routing example calculation is given in section 2.7 to illustrate the calculations associated with the Q_{p25} and Q_f.

SOLUTION:

1. The target water quality volume was already calculated to be 10,934 ft^3.
2. The permanent pool can vary anywhere from 915 ft^3 to 10,934 ft^3. The approximate 10-acre drainage area for this site is relatively small for a wet detention pond and may not support the permanent pool unless groundwater contributes additional baseflow. Therefore, the micropool

(915 ft^3) option will be evaluated. Any portion of the WQ_v not accounted for in the permanent pool should be provided for through extended detention.

3. The pretreatment (forebay) volume is calculated as:

$$Forebay\ volume = 0.1\ inches\ \times Impervious\ Acreage$$

$$Forebay\ volume = 0.1\ in \times \frac{36\ ft\ x\ 3{,}000\ ft}{43{,}560\frac{ft^2}{ac} \times 12\frac{in}{ft}} = 0.021\ ac - ft = 915\ ft^3$$

4. The minimum depth of the permanent pool should be 3 feet with a length to width ratio of at least 2:1. Therefore, the permanent pool dimensions can be approximated at 3 ft deep × 13 ft wide × 26 ft long. The area of the permanent pool is approximately 338 ft^2 or 0.0078 acres.
5. A **water balance calculation** should be performed to verify that the permanent pool has adequate depth. This example assumes no baseflow. Refer to Table 2.6.9-2:
   a. Determine the average monthly precipitation for your site.
   b. Obtain monthly evaporation distribution values from Table 2.2-2
   c. Calculate the volume of runoff from the contributing drainage area minus the pond (Ro) for each month (*Example calculations below are for January*)

$$I = \frac{36\ ft\ x\ 3{,}000\ ft}{76\ ft\ x\ 3{,}000\ ft\ +\ 217{,}800\ ft^2} = 24.2\%$$

$$R_v = 0.05 + 0.009(I) = 0.05 + 0.009(24.2) = 0.27$$

$$Q = 0.9PR_v = 0.9(3.69)(0.27) = 0.897 inches$$

$$R_o = \frac{QA_{site-pond}}{12} = \frac{0.897(10.23ac - \ 0.0078ac)}{12} = 0.76\ acre - ft$$

   d. Calculate the volume of precipitation that falls on the pond (P_{pond}).

$$P_{pond} = \frac{P(A_{pond})}{12} = \frac{3.69(0.0078)}{12} = 0.002\ ac - ft$$

   e. Obtain the free water surface evaporation value from Figure 2.2-4. For Savannah, this value is approximately 46 inches.

   f. Calculate the volume of evaporation that occurs over the open water surface of the pond (E).

$$E = \frac{Evap.Dist. \times Free\ Water\ Surface\ Evaporation \times A_{pond}}{12} = \frac{3.2\% \times 46 \times 0.0078}{12} = 0.001\ ac - ft$$

   g. Determine the saturated hydraulic conductivity (k_h) of the soil using Table 2.2-1. For sandy clay loam, k_h = 0.34 ft/day.

h. Calculate infiltration (I). For this example, assume G_h = 1.

$$I = Ak_hG_h = 0.0078ac \times \frac{0.34ft}{day} \times 1 \times 31days = 0.082ac - ft$$

i. Calculate the difference between the inflows and outflows.

$$Balance = \left(Ro + P_{pond}\right) - (E + I) = (0.76 + 0.002) - (0.001 + 0.082) = 0.679\ ac - ft$$

j. Calculate the accumulated total. Assume that all volume above the 3-foot depth (0.023 acre-feet) overflows and is lost in the trial design.

Table 2.6.9-2 shows that there are higher inflows than outflows for every month, and the pond can maintain a permanent pool of at least 3 feet.

Table 2.6.9-2. Summary Water Balance Calculations												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Days/Mo	31	28	31	30	31	30	31	31	30	31	30	31
[1]Precip. (in)	3.69	2.79	3.73	3.07	2.98	5.95	5.6	6.56	4.58	3.69	2.37	2.95
Evap. Dist.	3.2%	4.4%	7.4%	10.3%	12.3%	12.9%	13.4%	11.8%	9.3%	7.0%	4.7%	3.2%
Ro (ac-ft)	0.76	0.58	0.77	0.64	0.62	1.23	1.16	1.36	0.95	0.76	0.49	0.61
P_{pond} (ac-ft)	0.002	0.002	0.002	0.002	0.002	0.004	0.004	0.004	0.003	0.002	0.002	0.002
E (ac-ft)	0.001	0.001	0.002	0.003	0.004	0.004	0.004	0.004	0.003	0.002	0.001	0.001
I (ac-ft)	0.082	0.074	0.082	0.079	0.082	0.079	0.082	0.082	0.079	0.082	0.079	0.082
Balance (ac-ft)	0.683	0.504	0.691	0.555	0.533	1.153	1.077	1.277	0.869	0.682	0.412	0.530
Running Balance (ac-ft)	0.023	0.023	0.023	0.023	0.023	0.023	0.023	0.023	0.023	0.023	0.023	0.023

[1]https://www.ncdc.noaa.gov/cdo-web/datatools/normals

6. Extended Detention (for the remaining WQ_v):

   Note that the pretreatment and permanent pool volume can be subtracted from the WQ_v to determine the remaining WQ_v that will be treated through extended detention:

$$WQ_v = 10{,}934\ ft^3 - 915\ ft^3 - 915\ ft^3 = 9{,}104\ ft^3$$

   With the addition of the 15-ft wide aquatic bench, the pond dimensions become 28 ft × 41 ft resulting in a depth of approximately 8 ft. This is likely deeper than desired (without considering the CP_v) due to added embankment design challenges and potential safety concerns. Increasing the dimensions to 50 ft × 100 ft results in a depth of 4.5 ft (when looking at the combined WQ_v and CP_v) which is more manageable.

   The water quality drawdown device should be positioned on the outlet control structure such that it maintains the 3-foot deep permanent pool. Its orifice should be sized to draw down the WQ_v within 24 hours. See section 2.6.8 for an example that illustrates the orifice sizing process.

7. Channel Protection:

   Based on the approximate geometry of the wet detention pond, the portion of the WQ_v treated through extended detention requires a depth of approximately 1.8 feet [9,104 / (50×100)]. Therefore, the CP_v drawdown device should be located approximately 1.8 feet above the WQ_v drawdown device and to draw down the CP_v over a 24-hour period.

   The pond must be positioned within the available footprint and designed to fit the site's topography. A stage-storage relationship should be established to more accurately represent storage volumes associated with various water surface elevations. The stage-storage relationship will more accurately reflect the pond's side slopes and any irregular topography. The riser and emergency spillway should be designed to control the Q_{p25} and the Q_f. Verify that all other design requirements and constraints have been met.

Summary

2.6.10 Stormwater Wetland

Advantages	Disadvantages
• Offers good treatment and provides wildlife habitat • Maintenance requirements are typically minimal	• Requires a large footprint • More costly than some BMPs • Difficulties in maintaining the permanent pool may arise

Description: A shallow impoundment with a permanent pool designed to mimic natural wetlands.

Design Considerations:

- Two design variations (level 1 and level 2) achieve different pollutant removals
- Outflow hydrograph should mimic the existing conditions hydrograph, where applicable
- Minimum preferred drainage area of 5 acres
- Various wetland zones (e.g., deep pools, high marsh) create diverse wetland communities
- The design of stormwater wetlands should include a water balance analysis and landscaping plan

Maintenance Considerations:

- Provide adequate access to the BMP and appropriate components
- Design outlet structure to resist clogging

Applicability for Roadway Projects:

- Space requirements and flooding concerns may limit applicability in the linear environment; however, linear-shaped wetlands can offer many of the same benefits as traditional stormwater wetlands
- May be best suited for low lying, flat areas

Stormwater Management Suitability:

X Runoff Reduction
✓ Water Quality
✓ Channel Protection
✓ Overbank Flood Protection
✓ Extreme Flood Protection
X Temperature Reduction

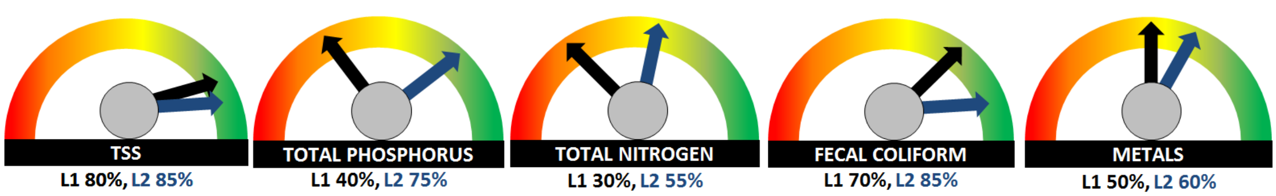
✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

It is generally not practical or cost-effective to design small wetlands close to the source of runoff as LID dictates. However, stormwater wetlands employ several LID/GI characteristics such as mimicking natural systems and providing infiltration and evapotranspiration.

Treatment Capabilities

■ Level 1 Stormwater Wetlands (L1) ■ Level 2 Stormwater Wetlands (L2)



2.6.10 Stormwater Wetland

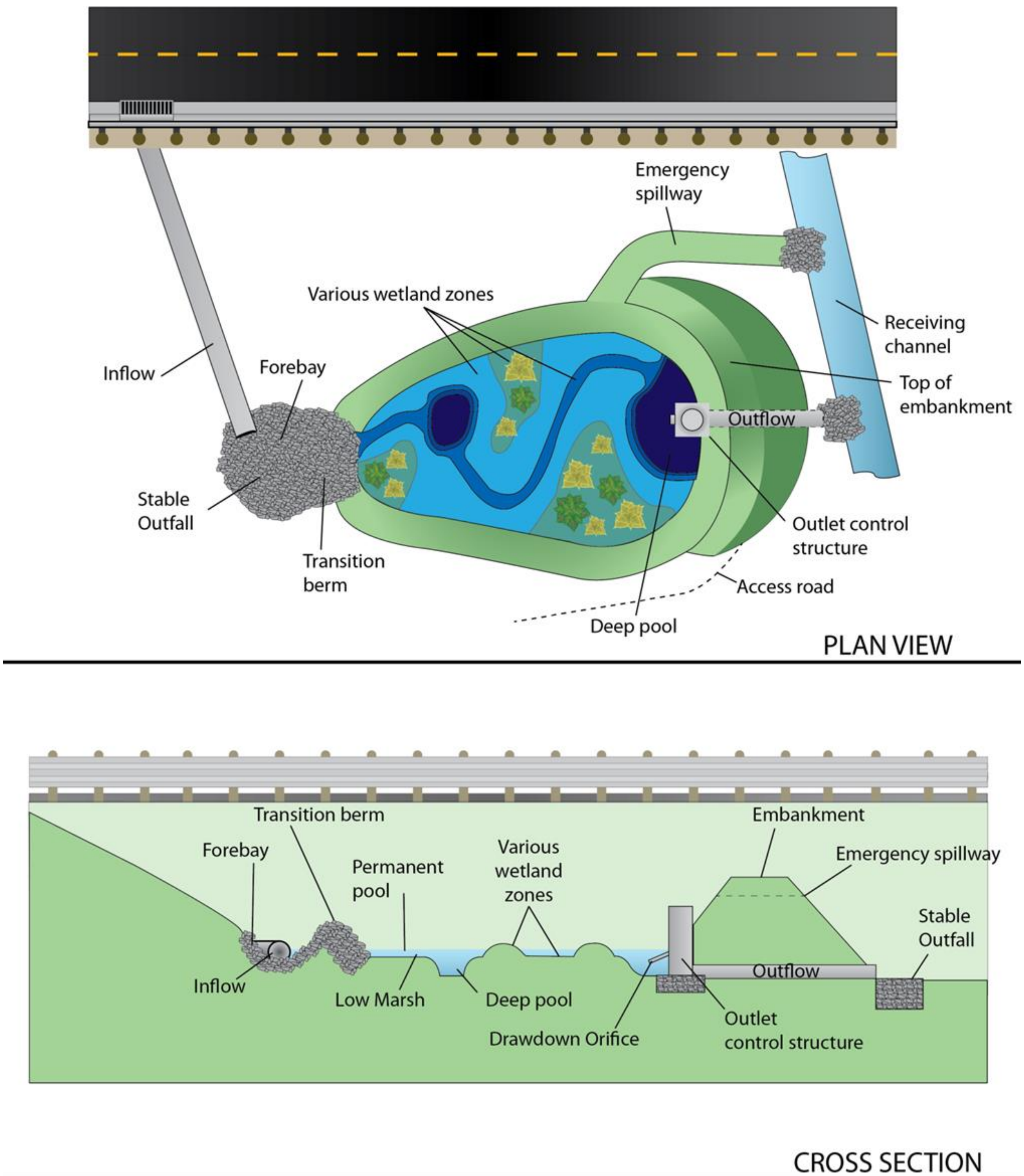
Description

Stormwater wetlands function similar to wet detention ponds. Stormwater wetlands are earthen impoundments that maintain a permanent pool of water and may have additional storage for detaining runoff and attenuating peak flows. However, stormwater wetlands are shallower than wet detention ponds and have greater areas of wetland vegetation. Varying shallow water depths (wetland zones) increase aquatic plant diversity. Stormwater wetlands can provide detention benefits such as reduced peak flows and preventing stream channel erosion. Stormwater wetlands also provide runoff water quality treatment. The permanent pool provides an area for sediment storage, reducing TSS and the associated pollutants adhered to these particles. Contact with the permanent pool and wetland vegetation results in chemical and biological processes that reduce nutrients, metals, and pathogens.

Recent research and lessons learned during the past 20 years of stormwater wetland implementation have led to additional design recommendations that can enhance the pollutant removal ability and wildlife benefits of stormwater wetlands. This section presents two types of stormwater wetlands. Level 1 wetland designs are based on the stormwater wetland approach presented in the GSMM with some modifications and suggestions based on lessons learned. Level 2 wetland designs are based on guidance from the Center for Watershed Protection. [(2-8)]

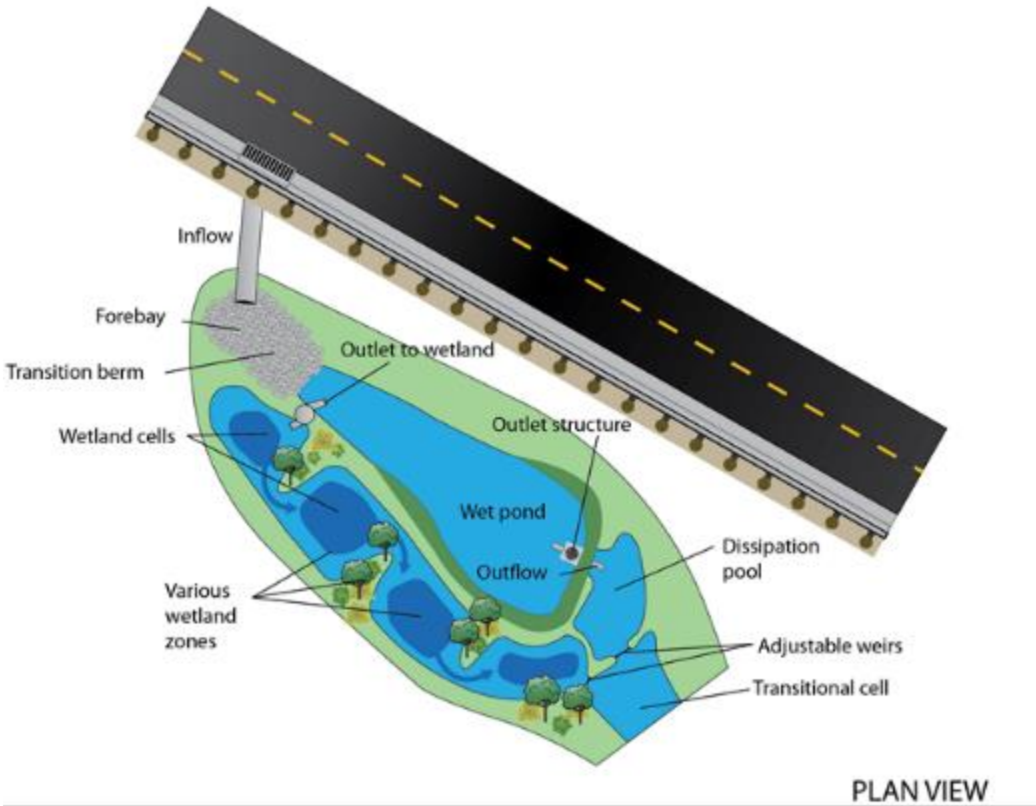
Level 1 stormwater wetlands can be used to meet WQ_v, CP_v, Q_{p25}, and Q_f requirements. A riser with a small orifice that is elevated above the bottom of the wetland creates a shallow permanent pool and allows the wetland to store additional runoff for a short period of time (24 hours for CP_v). Runoff in excess of the design volume is released through the top of the riser and/or an emergency spillway channel. Figure 2.6.10-1 illustrates a typical level 1 stormwater wetland configuration.

Figure 2.6.10–1 - Typical Level 1 stormwater wetland configuration



Level 2 stormwater wetlands are intended to meet water quality requirements only; they cannot be used for extended detention. Therefore, the outlet structure design can be simplified. Level 2 wetlands can be installed parallel to wet detention ponds to meet detention requirements and to help maintain the wetland permanent pool level. Figure 2.6.10-2 illustrates this option.

Figure 2.6.10–2 - Level 2 wetland with wet pond (adapted from CWP, 2008) (2-8)



Level 1 wetlands provide sufficient water quality treatment for most sites and have the added flexibility of providing detention. For these reasons, level 1 wetlands will likely be the desired choice for most sites. However, level 2 wetlands may be more applicable where additional water quality treatment is needed due to receiving water impairments or similar issues. Further, level 2 wetlands should be considered where wildlife habitat is of particular concern and in cases where its application will not be considerably more costly than level 1 wetlands.

Stormwater Management Suitability

- Runoff Reduction – Stormwater wetlands do not provide runoff reduction credits. Although stormwater wetlands provide moderate to high removal of many of the pollutants of concern typically contained in post-construction stormwater runoff, recent research shows that they provide little, if any, reduction of post-construction stormwater runoff volumes. (2-20)
- Water Quality – Pollutants are removed from stormwater runoff in a wetland through uptake by vegetation and algae, filtering, and gravitational settling in the slow-moving marsh flow. Other pollutant removal mechanisms are also at work in a stormwater wetland, including chemical and biological decomposition, and volatilization. A level 1 wetlands provides 80% TSS removal if designed, constructed, and maintained correctly. A level 2 wetlands provides 85% TSS removal if designed, constructed, and maintained correctly.

- Channel Protection – The storage volume above the permanent pool/water surface level in a stormwater wetland is used to provide control of the channel protection volume (CP_v) by releasing the 1-year, 24-hour storm runoff volume over 24 hours (extended detention). It is best to do this with minimum vertical water level fluctuation, as extreme fluctuation may stress vegetation.
- Overbank Flood Protection – A stormwater wetland can also provide storage above the permanent pool/water surface level to reduce the post-development peak flow of the 25-year storm (Q_{p25}) to pre-development levels (detention). If a wetland facility is not used for overbank flood protection, it should be designed as an off-line system to pass higher flows around rather than through the wetland system.
- Extreme Flood Protection – In situations where it is required, stormwater wetlands can also be used to provide detention to control the 100-year, 24-hour storm peak flow (Q_f). Where Q_f peak control is not required, a stormwater wetland must be designed to safely pass extreme storm flows.

Pollutant Removal Capabilities

Level 1 stormwater wetlands provide good treatment and detention but are less cost-effective than wet detention ponds because they require a greater land area. The following average pollutant removal rates for level 1 wetlands may be utilized for design purposes: (2-17)

- TSS – 80%
- TP – 40%
- TN – 30%
- Fecal coliform – 70%
- Heavy metals – 50%
- Temperature – Temperature reduction is not provided.

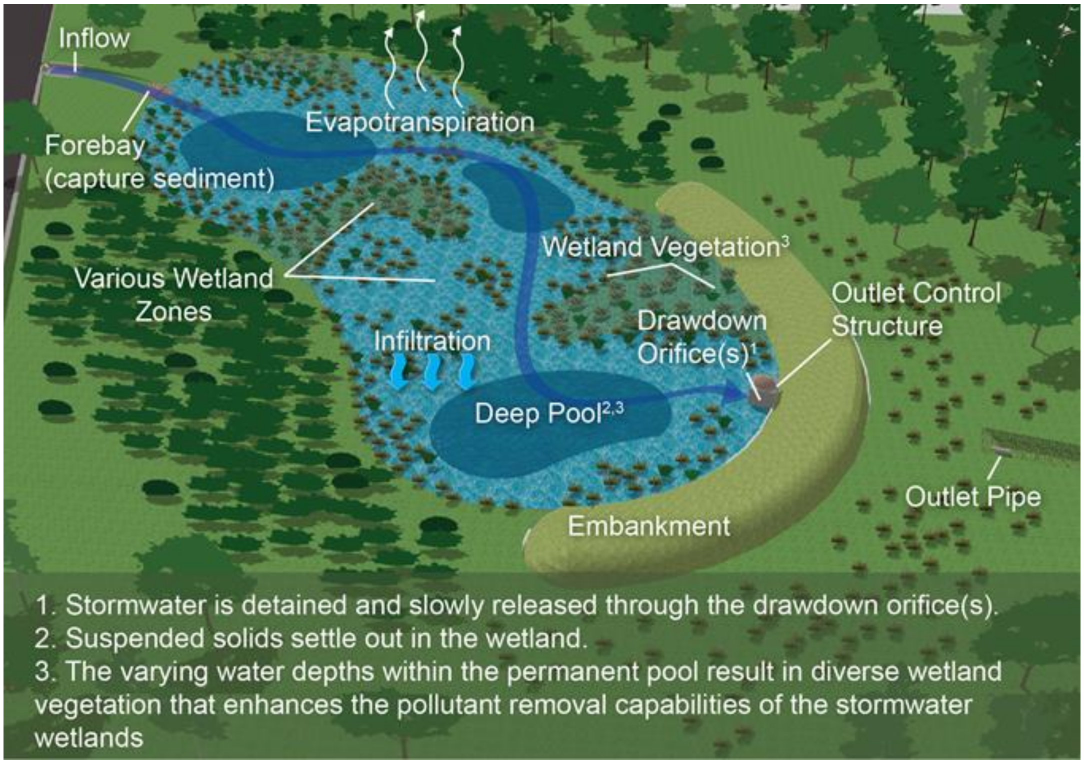
Research shows that level 2 wetland designs achieve the following pollutant removals:

- TSS – 85%
- TP – 75%
- TN – 55%
- Fecal coliform – 85%
- Heavy metals – 60%
- Temperature – Temperature reduction is not provided.

Application and Site Suitability

Stormwater wetlands are most applicable in low lying, flat sites with plenty of space, which can limit their application to roadway settings. Further, an adequate supply of runoff or groundwater is necessary to maintain the permanent pool. Figure 2.6.10-3 illustrates typical stormwater wetland components and treatment processes.

Figure 2.6.10–3 - Typical stormwater wetland components and treatment processes



The location of the stormwater wetlands should be determined on a case-by-case basis using sound engineering judgment with consideration for topography, cost, surrounding land use and development, and access. As a general rule, stormwater wetlands should not be located in natural wetland areas or other environmentally-sensitive areas such as live streams. Under special circumstances, post-construction BMPs may be allowed within environmentally sensitive areas with prior consent from appropriate regulatory agencies. For example, if a naturally occurring wetland or other environmentally-sensitive area is impacted, whether it is within an MS4 area or not, post-construction stormwater BMPs may be warranted to protect the impacted area. Siting information and constraints include:

- **Drainage Area –** Minimum drainage area of 5 acres is required to maintain the permanent pool. In some cases, the 5-acre minimum drainage area can be waived.
- **Depth to Water Table** – Stormwater wetlands can be used where the water table is at or near the soil surface, or where there is a sufficient water balance in poorly drained soils to support a wetland plant community. If located above an aquifer or being used to treat a hotspot, however, 2 feet is required between the bottom of a stormwater wetland and the elevation of the seasonally high water table. It is recommended, especially for Level 2 wetlands that the bottom elevation of the wetland intercept the groundwater table. Where stormwater wetlands do not intercept the groundwater table, a liner must be installed on HSG A and B soils. A water balance calculation should be performed to ensure an adequate water budget to support the specified wetland species. A water balance analysis may not be necessary if a liner is installed but should be considered regardless if the drainage area is small and/or has a small amount of impervious area. The stormwater wetland size may need to be adjusted to account for lost volume due to seasonal fluctuations in the groundwater table.

- **Space Required** – The wetland usually occupies approximately 3-5% of the total drainage area.
- **Minimum Head** – The required elevation difference from the inflow to outflow is typically 2-3 feet.
- **Setbacks** –
  - Property lines – 10 feet (site development projects only)
  - Private wells – 100 feet
  - Septic systems – 50 feet
  - Public-use airports – 5 miles
- **Trout Stream** – Consideration should be given to the thermal influence of stormwater wetland outflows on downstream trout waters.

Challenges associated with roadway configurations include limited right-of-way and clear recovery zone requirements. Stormwater wetlands may be elongated to better fit the linear environment, if necessary. In addition, maintenance must be considered during the design and can often be challenging and hazardous for roadway BMPs.

Data for Design

The initial data needed for stormwater wetland design includes the following:

- Existing and proposed site, topographic and location maps, and field reviews
- Aerial photographs of the drainage basin to estimate land use areas (grassed, paved, etc.)
- Preliminary plans including plan view, roadway and drainage profiles, cross sections, utility plans, and soil report
- Calculations and details from existing nearby detention facilities
- Water table information

The size and configuration of the stormwater wetland will depend on stormwater management requirements. Level 1 stormwater wetlands are often designed to capture and slowly release the CP_v over 24 hours, maintain the Q_{p25} at existing condition rates, and to adequately control the Q_f. However, one or more of these goals may be waived as described in section 2.4.

After initial data gathering and determination of stormwater management requirements, the designer should proceed with an initial wetland volume estimate. Methods outlined in section 2.7, Detention Design can be used for level 1 designs. The WQ_v method should be used for level 2 designs and for level 1 wetlands that are not designed to meet detention requirements.

If possible, at least two alternating planting peninsulas (or other forms of micro-topography) should extend into the wetland perpendicular to flow. The peninsulas should extend at least 80% of the way across the wetland. This creates a shallow meandering channel that extends the dry weather flow path. It also provides varying permanent pool depths for a diverse wetland ecosystem. Table 2.6.10-1 gives approximate wetland zone criteria that can be used to configure the wetland.

Table 2.6.10-1 Approximate Level 1 and 2 Dimensional Information for Various Wetland Zones			
Wetland Zone	**Criteria**	**Level 1 Design**	**Level 2 Design**
Deep Pools	Depth	-18” to -72”	-18” to -48”
	% of Total Volume	20 %	25%
Low Marsh	Depth	-6” to -18”	N/A
	% of Total Volume	20%	N/A
High Marsh	Depth	-6” to 0”	-6” to +6”
	% of Total Volume	10%	70%
Low Land	Depth	0”+	N/A
	% of Total Volume	50%	N/A

Table 2.6.10-2 Level 1 and 2 Wetland Design Criteria		
Criteria	**Level 1**	**Level 2**
WQ_v	As presented in section 2.4.1.2	As presented in section 2.4.1.2
Deep pools	2 (forebay and outlet)	3 (forebay, middle, outlet)
Wetland side slopes (max)	3:1	5:1
Slope profile	8% across the site	Should generally be flat; use multiple cells if needed; max drop of 1’ between cells
Normal flow path (distance from inlet to outlet)	1:1	1.5:1
Dry weather flow path	Not required	5:1
Vegetation	Can use solely herbaceous	Include woody vegetation (trees and shrubs)
Average wetland depth	Can be >1	Should be ≤1
Extended detention	Limit to 1’ vertically	Not allowed

Micro-topography is an important aspect of level 2 wetland designs. The previously discussed planting peninsulas are often the preferred method. The following methods can be used to enhance micro-topography:

- Snags
- Inverted root wads
- Tree peninsulas

- Coir fiber islands
- Internal pools
- Cobble sand weirs

Consult a stream restoration specialist for additional guidance on these items.

After a rough location and configuration are determined, follow the remaining steps outlined in section 2.7, Detention Design, for sizing and hydrograph routing. Then, integrate the remaining BMP components into the design. Remember that the cumulative flow from multiple detention facilities within the same watershed can negatively impact receiving waters if hydrograph timing is not considered. Perform a hydrologic analysis for the project's zone of influence as described in section 2.2.3 of this chapter. For more information on the design of a stormwater wetland, see the detailed calculation example located at the end of this section.

Pretreatment

Forebays should be provided at wetland inlet areas to capture solids before the stormwater runoff enters the wetland. This will reduce clogging of drawdown orifices, extend the life of the BMP, and facilitate maintenance. Forebays should be sized for 0.1 inches of stormwater runoff per impervious acre and should be 4 to 6 feet deep.

Refer to section 2.8, Common BMP Components, for further guidance.

Vegetation & Landscaping Plan

A vegetation & landscaping plan is an important component of the design of stormwater wetlands. A variety of species should be selected for the various zones of the wetland. Native, non-invasive species are preferred. Aquatic vegetation helps remove pollutants and provides wildlife habitat and aesthetic benefits. Consult a landscaping professional for plant selection. If conditions allow, taller vegetation and trees may be planted around the stormwater wetlands to discourage waterfowl from taking residence in the wetland as they can add to nutrient and bacteria loads. Taller vegetation also provides shade for better thermal control. Woody vegetation, which enhances pollutant removal, infiltration, and evapotranspiration, should be included in level 2 wetlands design. Woody vegetation should not be planted within 15 feet of the embankment or maintenance access areas. [(2-8)] Refer to GDOT Planting Schedule Special Construction Detail for more information.

Outlet Structure

The configuration of the outlet structure can vary and will depend on stormwater requirements (i.e., WQ_v, CP_v, Q_{p25}, and Q_f). A typical level 1 configuration uses a riser/barrel configuration and emergency spillway to meet all requirements. The level 1 normal pool size can be adjusted so that the BMP will fit with in the allowable area. The minimum level 1 normal pool size should be 50% of the WQ_v. For larger areas, the normal pool should be equal to the WQ_v.

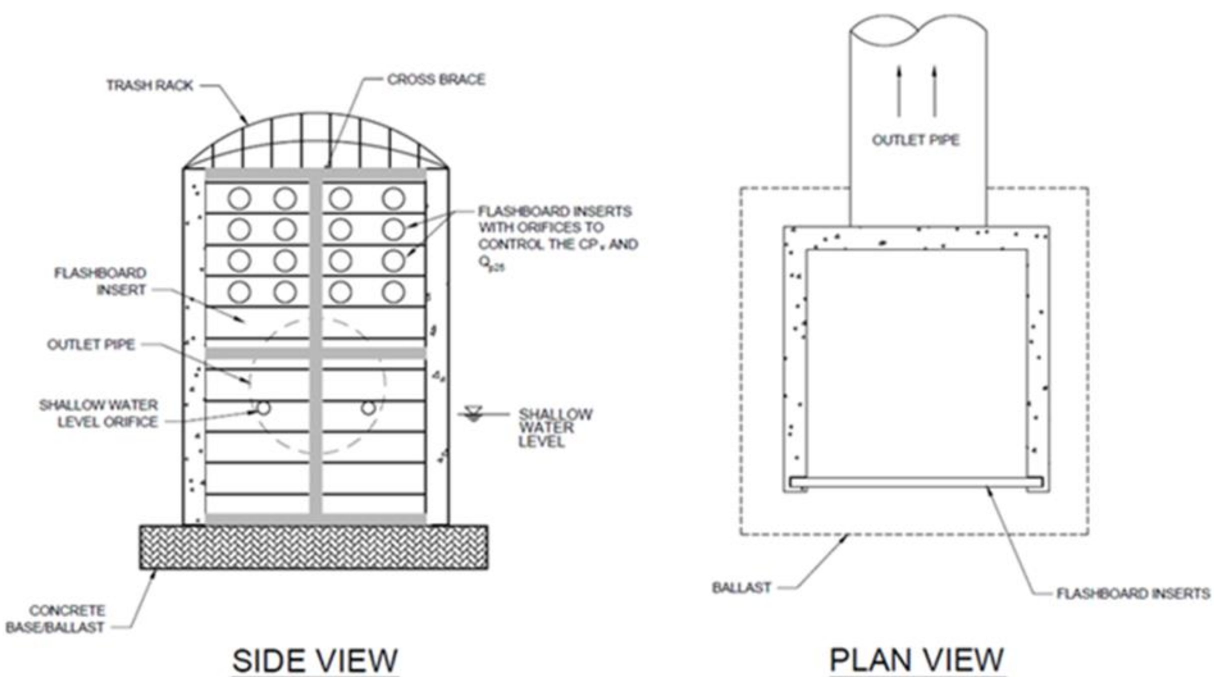
A deep pool is required at the outlet to prevent clogging and resuspension of sediment. The outlet structure should be designed to allow the water level in the wetland to rise above the permanent pool elevation as runoff (usually the CP_v) is detained, and then slowly draw it down over 24 hours. This 24-hour period may be reduced to 12 hours where runoff temperature is a concern. Additionally, the orifice can be positioned lower to draw off cooler water.

Weirs created towards the top of the riser or the open throat of the riser are typically used to accommodate the Q_{p25} and should be located at an elevation that allows for the storage of the WQ_v plus the CP_v. Outlet protection should be provided downstream of the outlet structure to protect against erosion (refer to chapter 9 of this manual). Maximum release rates from the outlet structure should be targeted towards existing condition rates.

The outlet structure contains a small pipe with a threaded end cap at the lowest elevation of the wetland in the event that the wetland needs to be drained completely. Accessibility to the cap may be difficult at times, depending on the design depth and configuration of the wetland, so it is best that the location of the outlet control structure itself be as close to the embankment as possible to accommodate access. Refer to the GDOT Wet Detention Pond Outlet Structure Special Construction Detail for additional guidance.

Alternatively, a flashboard riser design may be used. Drawdown occurs through orifice holes in the boards located on the front face of the flashboard riser, as shown in Figure 2.6.10-4. These boards can easily be modified or replaced to adjust the water level as needed for maintenance or the health of the wetland vegetation. A baffle can be used to prevent clogging of orifices by floating debris.

Figure 2.6.10–4 - Typical flashboard riser configuration



For level 2 designs, the normal pool should encompass the entire WQ_v. The outlet structure may be simplified since detention requirements are not permitted for level 2 designs. For this reason, assume that the water level fluctuation associated with the WQ_v design storm should be limited to 6-8 inches. Similarly, the water level fluctuation associated with the CP_v storm should be limited to 12 inches. This can be accomplished by using a long weir structure capable of conveying large flow rates with little hydraulic head or bypassing larger storm events altogether by using an upstream diversion structure.

The hydrograph routing procedures and weir and orifice equations outlined in section 2.7 of this chapter are used to size the components of the outlet structure. The outlet structure should be designed such that the outflow hydrograph resembles the existing condition hydrograph to the maximum extent practicable.

The buoyancy of the outlet structure should be determined and offset with proper anchoring and/or concrete. Refer to the ACPA document entitled, *Design Data 41 Manhole Flotation* (2008) [(2-3)] for additional information.

Water Balance

Install an impermeable liner if the stormwater wetland is located on HSG A or B soils and the wetland does not intercept the groundwater table. A water balance analysis should be performed for systems on HSG C and D soils. Refer to section 2.2.4 for water balance calculations. In-situ infiltration testing may be completed if determined to be necessary based on engineering judgement to ensure that the permanent pool will not be completely drawn down.

Emergency Spillway

The emergency spillway is generally an open channel constructed in natural ground (as opposed to the embankment). The emergency overflow elevation shall be established at least one (1) foot below the roadway's normal shoulder break point and within 0.5 ft of the 100-year ponding elevation modeled with an unclogged outlet structure. The spillway shall be capable of conveying the 100-year storm modeled with a clogged outlet structure. If including an emergency spillway in the design is not possible, size the weir(s) in the outlet structure so that they are capable of conveying the 100-year storm. Refer to the guidance given in chapter 6 of the Drainage Design Policy Manual for assistance in sizing the channel and determining an appropriate lining material.

Embankment

The embankment is a small earthen dam or fill section used to create the downslope side of the wetland. Embankments shall be designed to be less than 25 feet in height and detain less than 100 acre-feet in volume. Embankment height is measured from the elevation of the downstream toe to the maximum water storage elevation. Embankments that exceed these limits should be avoided and are subject to the Georgia Safe Dams Act of 1978. [(2-13)]

Embankment side slopes should be 3:1 or flatter, however can be 2:1 with permission from the Office of Design Policy and Support. Overland flow should be minimized down embankment side slopes. A slope stability analysis is recommended for embankments higher than 10 feet and is required for slopes steeper than 2:1. Appropriate seepage control should be provided according to the size of the embankment and characteristics of the soils and wetland configuration. Refer to the NRCS's Agriculture Handbook 590 [(2-33)] and geotechnical reports prepared for the project site for additional guidance. Since shallow bedrock beneath the embankment may act as a conduit for seepage through the embankment, additional seepage prevention measures may be needed in these areas. Finally, the embankment should have 1 foot of freeboard above the 100-year flood elevation with additional consideration for embankment settlement.

The top of the stormwater wetland shall have an 8 feet wide berm or bench graded all around the basin, both in cut and in fill sections. The top of the berm or bench may be sloped up to 4% towards the inside of the basin.

Access and Driveway Considerations

See section 2.10.3 for maintenance access requirements.

Signage

The designer shall specify the installation of BMP signs consistent with GDOT's BMP Signs Special Construction Detail.

Stormwater Wetlands Sizing

1. Determine the goals and primary function of the stormwater wetlands.

   The goals and primary function of the BMP must take into account any restrictions or site-specific constraints. Also take into consideration any special surface water or watershed requirements. Determine whether a level 1 or level 2 design is more appropriate.

2. Calculate the Target Water Quality Volume

   Calculate the water quality volume formula using the following formula:

$$WQ_v = \frac{1.2\ in \times (R_v) \times A \times 43560\,\frac{ft^2}{acre}}{12\,\frac{in}{ft}}$$

   Where:

   WQ_v = water quality volume (ft^3)

   R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.

   A = onsite drainage area of the post-condition basin (acres)

3. Determine the pretreatment volume.

   A sediment forebay is provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the basin. The forebay should be sized to contain 0.1 inch per impervious acre of contributing drainage.

4. Determine the wetlands location and preliminary geometry. Conduct stormwater wetlands grading and determine storage volume available for the permanent pool.

   This step involves initially grading the wetlands (establishing contours) and determining the elevation-storage relationship for the wetlands.

5. If applicable, complete a water balance analysis to verify the stormwater wetlands will maintain its permanent pool.

6. Compute extended detention orifice release rate(s) and size(s) and establish CP_v elevation.

   The CP_v elevation is determined from the stage-storage relationship and the orifice is then sized to release the difference between the water quality volume and channel protection storage volume over a 24- hour period (12-hour extended detention may be warranted in some cold water stream basins).

7. Calculate the Q_{p25} release rate and water surface elevation.

Set up a stage-storage-discharge relationship for the control structure for the extended detention orifice(s) and the 25-year, 24-hour rainfall event.

8. Design embankment(s) and spillway(s).

   To size the emergency spillway, calculate the 100-year, 24-hour storm water surface elevation. Set the top of the embankment elevation at least one foot higher and analyze safe passage of the Extreme Flood Volume (Q_f).

9. Investigate potential basin hazard classification.

   The design and construction of stormwater management ponds are required to follow the latest version of the State of Georgia dam safety rules.

10. Prepare a site vegetation and landscaping plan.

    A landscaping plan for stormwater wetlands and its buffer should be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation. See the GDOT Planting Schedule Special Construction Detail for more information.

Construction Considerations

The following items should be considered during the design and, if warranted, included as notes on the design drawings, in the details or special provisions:

- Place the embankment in shallow lifts under controlled compaction conditions.
- Provide an adequate water-tight seal between the outlet structure and pipes or other appurtenances to avoid leaks and possible failure of the structure.
- Remove sediment from construction activities and establish vegetation before the stormwater wetland is brought online.
- Make holes dug for planting larger to allow for root growth in order to counteract compaction within the wetland, which may limit the growth of newly planted vegetation.

Maintenance Considerations

Without proper maintenance, BMPs will function at a reduced capacity and may cease to function altogether. A properly designed BMP includes the following considerations for maintenance:

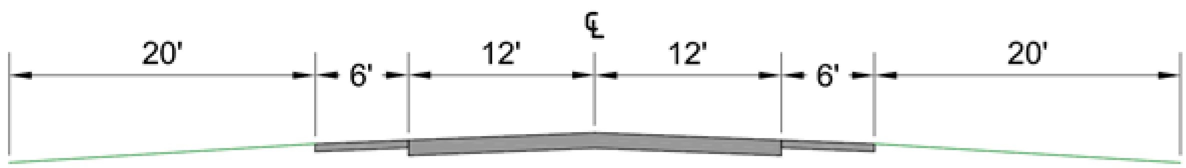
- Access:
  - Provide adequate right-of-way.
  - Provide access roads and ramps for appropriate equipment to all applicable components (outlet structure, forebay, etc.).
  - Provide space to turn around if necessary.
  - Check for sufficient area to safely exit and enter the highway, if applicable.
  - If the BMP is fenced, provide appropriately sized gates (refer to section 2.10 for additional guidance regarding fencing and other safety considerations).
  - Adequate access for a small boat may be needed for sediment depth measurements.
- Provide a method for dewatering the wetland.

Refer to GDOT's *Stormwater System Inspection and Maintenance Manual*, for specific maintenance requirements.

Stormwater Wetland Example Calculation

GIVEN:

- A new roadway project located in Savannah, GA.
- The proposed project includes 3,000 feet of roadway (in length).
- The drainage area that discharges to the stormwater wetland includes the following: two 12-foot lanes, two 6-foot paved shoulders, and two 20-foot wide grassed areas (on either side of the road), draining via sheet flow.
- Offsite stormwater also provides supplemental runoff to maintain the permanent pool (assume 2 acres for the purposes of this example).
- A level 2 stormwater wetland is desired to provide additional water quality benefits and wildlife habitat.
- Wetland dimensions were simplified and assumed for the purposes of this example.
- The designer has previously calculated the following hydrologic information:
    - Permanent pool = WQ_v = 10,936 ft^3



FIND:

- Size the stormwater wetland permanent pool, individual wetland zones, and outlet structure to treat the WQ_v.
- Perform a water balance calculation to verify that the permanent pool will be maintained to an acceptable degree.
- Note that a separate hydrograph routing example calculation is given in section 2.7 to illustrate the calculations associated with the Q_{p25} and Q_f.
- Note that extended detention should not be used in level 2 stormwater wetlands.

SOLUTION:

1. The target water quality volume was already calculated to be 10,936 ft^3 which will be the volume of the permanent pool.
2. For level 2 wetlands, the following wetland zones should be provided:

Wetland Zone	Depth (relative to permanent pool)	% of Total Volume
Deep Pools	-18” to -48”	25%
High Marsh	-6” to +6”	70%

Note that these are approximate guidelines. The 5% that is unaccounted for is the result of short transition zones from high marsh to deep pools.

At least three deep pools should be provided. One of which is the forebay.

$$Total\ deep\ pool\ volume = 25\% \times 10{,}936\ ft^3 = 2{,}734\ ft^3$$

$$Forebay\ volume = 0.1\ inches\ \times Impervious\ Acreage$$

$$Forebay\ volume = 0.1\ in \times \frac{36\ ft\ x\ 3{,}000\ ft}{43{,}560\frac{ft^2}{ac} \times 12\frac{in}{ft}} = 0.021\ ac - ft = 915\ ft^3$$

Remaining pools (assume two):

$$\frac{2{,}734 - 915}{2} = 910\ ft^3$$

$$Total\ deep\ pool\ volume = 75\% \times 10{,}936\ ft^3 = 8{,}202\ ft^3$$

3. A water balance calculation should be performed to verify that there is adequate runoff supply to maintain the permanent pool. Refer to the wet detention pond example calculation in section 2.6.9.
4. A simple weir outlet structure will be used for this example. Extended detention is not permitted for level 2 wetlands. For this reason, assume the water level rise associated with the WQ_v design storm should be limited to 8 inches and the CP_v storm should be limited to 12 inches. Use this information and the guidance in section 2.7 to design the outlet structure.

Additional Considerations:

The wetland must be positioned within the available footprint and designed to fit the site’s topography. Incorporate the various zones and configure the planting peninsulas into the site plan. A qualified professional should develop a planting plan that utilizes various woody species.

A stage-storage relationship that reflects the wetland’s side slopes and any irregular topography should be established to more accurately represent storage volumes associated with various water surface elevations. Note that the wetland should be designed to convey the 100-yr storm for the total drainage area (including offsite runoff) without failure unless it is designed to be offline.

Summary

2.6.11 Open-Graded Friction Course

Photo Description: OGFC on Left, Portland Cement Concrete on Right

Photo credit: Dr. Michael E. Barrett

Advantages	Disadvantages
• Offers increased safety benefits on wet roadway conditions • Can be applied to more area per ton than conventional asphalt pavement • Can be cost effective • Removes TSS	• Drainage can be impeded by sediment deposition • Improper installation leads to rapid deformation

Description: Open-graded friction course (OGFC) is a thin, permeable layer of asphalt that encompasses a support structure consisting of a uniform, coarse aggregate size with minimal fines, and serves as an overlay to conventional asphalt pavements. OGFC has a high void content which keeps water from sitting on the pavement surface which reduces spray, and therefore reduces washing of vehicle undercarriage.

Design Considerations:

- Leveling of existing asphalt overlay required prior to installation of OGFC overlay
- Porous nature requires installation during optimal temperatures as specified in standard specifications
- Requires coordination with other offices and project team members

Maintenance Considerations:

- Drainage and lateral flow should not be impeded by compaction

Applicability for Roadway Projects:

- Highly suitable for roadway pavement and resurfacing projects with higher annual average daily traffic volumes and may be used in conjunction with additional stormwater BMPs if adequate right-of-way is available

Stormwater Management Suitability:

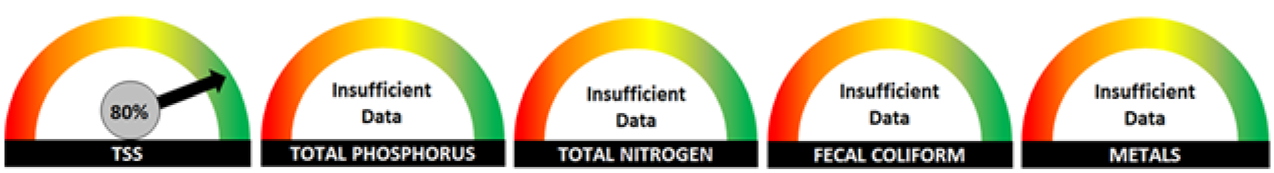
X Runoff Reduction
○ Water Quality
X Channel Protection
X Overbank Flood Protection
X Extreme Flood Protection
X Temperature Reduction

✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

Since OGFC offers water quality treatment through stormwater filtration, it can be considered an LID/GI control when used for this purpose.

Treatment Capabilities

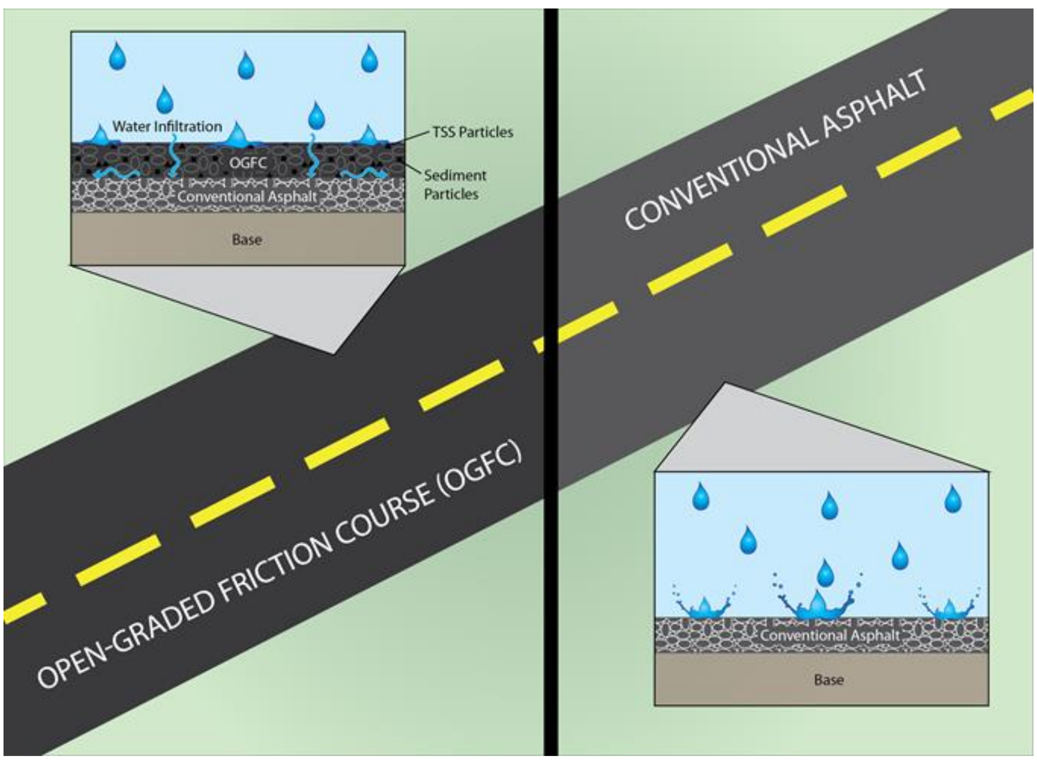


2.6.11 Open-Graded Friction Course

Description

Open-graded friction course (OGFC) is a thin, permeable layer of asphalt that encompasses a support structure consisting of a uniform, coarse aggregate size with minimal fines, and serves as an overlay to conventional asphalt pavements. OGFC has traditionally been used to reduce vehicle spray, absorb noise from vehicle traffic, and also has an increased resistance to surface friction. The permeability of OGFC allows for water to enter and flow through the aggregate matrix, and not directly off the pavement surface. As a result, OGFC not only increases the safety of motorists by decreasing splash and spray, reduces the potential for hydroplaning, and improves the visibility of pavement markings, but it also serves as a benefit to the environment by providing a reduction in TSS. The large number of void spaces within the structure of OGFC provides a stormwater detaining effect, a proven reduction of TSS within stormwater runoff, and a minimization of sediment impacts. This applies to all GDOT types of OGFC including conventional, modified, and porous European mix (PEM). Figure 2.6.11-1 illustrates the typical design structure of OGFC.

Figure 2.6.11-1 - OGFC (left) and conventional asphalt (right) cross sections



Stormwater Management Suitability

- Runoff Reduction – OGFC does not provide runoff reduction credits.
- Water Quality – The large number of void spaces within the structure of OGFC provides a stormwater detaining effect, a proven reduction of TSS within stormwater runoff, and a minimization of sediment impacts. When sized correctly, OGFC provides a 80% TSS removal efficiency.
- Channel Protection – Another practice must be used to provide CP_v extended detention.
- Overbank Flood Protection – Another control will be required in conjunction with OGFC to reduce the post-development peak flow of the 25-year storm (Q_{p25}) to pre-development levels (detention).
- Extreme Flood Protection – Another practice must be used to provide extreme flood protection.

Pollutant Removal Capabilities

Research has shown that the use of OGFC results in a delayed runoff rate, minimization of sediment impacts due to the reduction of wash off from the undercarriage of vehicles, and a removal of TSS concentrations. If properly maintained, water quality benefits can last throughout the design life of the pavement.

Similar transportation related research sponsored by the Federal Highway Administration and Texas Department of Transportation can also be referenced. These studies consistently reported TSS removal rates of 90-91%.[2-4, 2-5] Thus, a conservative TSS pollutant removal rate of 80% may be utilized for water quality design purposes.

Application and Site Suitability

In relation to post-construction stormwater benefits, OGFC is most applicable for roadway segments that span areas adjacent to and across sensitive water bodies. Research has shown that resurfacing of these roadway segments with an OGFC overlay can provide the same functionality as other structural BMPs, such as TSS removal. Roadside filter strips combined with OGFC offer additional water quality benefits on highways without curb and gutter systems. Refer to section 2.6.1 for additional information regarding filter strips.

OGFC is commonly applied to roadway projects and resurfacing routes with a high volume of annual average daily traffic (AADT). Therefore, OGFC can be a cost-effective BMP, particularly for projects requiring resurfacing (i.e., widening and bridge replacement projects). OGFC may also prove to be feasible along rural and low traffic roadways. An alternative to OGFC may be necessary when considering the design for areas with severe turning movements such as parking lots. Collaboration may be required between the design engineer, structural engineer, and material divisions within GDOT to coordinate the practicalities of OGFC in its desired location. This may be the case for potential use on bridge approaches and decks, as an example.

Design

The OGFC mix and specifications are typically determined by the Office of Materials and Research. Designers shall coordinate with the Office of Materials and Research to verify acceptable locations for OGFC and to make all members of the project team aware that OGFC is part of the post-construction stormwater management plan. OGFC shall only be used when recommended by the Office of Materials and Testing based on the Criteria for Use of Asphaltic Concrete and Mix Types. A uniform cross-section must be maintained to ensure lateral drainage toward the road shoulder. Changes to the pavement design may result in the need for additional BMPs.

Additional Design Considerations

OGFC has some limitations when compared to conventional pavements. These include an increased potential for stripping, raveling, and shoving which result in decreased structural value of the pavement. Special snow and ice control methods and rehabilitation techniques that allow for proper drainage through the OGFC overlay are also required.

Construction Considerations

It is important to adhere to Section 400 within the *Georgia Department of Transportation's Standard Specifications Construction of Transportation Systems, 2013 Edition* during construction as there are many practices to consider while installing the OGFC. The OGFC layer should be installed during optimal temperatures. Cold temperatures tend to inhibit the bond between the OGFC and existing pavement, and installation during windy conditions may cool the mixture too rapidly. Special care should be taken not to impede the drainage of the OGFC. Improper practices during construction activities that allow mud and sand to enter the pavement area can make the porous nature of the OGFC overlay vulnerable to clogging. Clogging of the voids within the OGFC reduces its drainage capacity and should be avoided. It is important that erosion and sediment control devices associated with construction projects remain in place until all areas are permanently stabilized with vegetation.

Maintenance Considerations

Note that for areas where OGFC is used to meet post-construction stormwater management requirements, it is likely not acceptable to resurface with conventional asphalt without implementing additional BMPs.

Maintenance of the OGFC is largely dependent upon the AADT. As with any stormwater BMP, OGFC will not function properly if it lacks appropriate maintenance. Shear failures, cracking, raveling, delamination, and the clogging of voids within its porous structure are conditions that warrant maintenance. The sealing of cracks and installation of patches can create areas that retain water over time, which may eventually contribute to additional problems. If the OGFC overlay requires patching, it should be repaired with OGFC. Periodic maintenance may be required to remove sediment buildup on the roadway shoulders caused by low traffic volumes in these areas. The lateral flow of water through the OGFC overlay must be maintained to sustain its functionality.

Refer to GDOT's *Stormwater System Inspection and Maintenance Manual*, for specific maintenance requirements.

Summary

2.6.12 Regenerative Stormwater Conveyance

Source: Anne Arundel County – Design Guidelines for Step Pool Stormwater Conveyance (SPSC) Systems, May 2022

Advantages	Disadvantages
• RSC systems provide high total suspended solids and soluble pollutant removal rates. • Effective at restoring eroded outfalls, ditches, and channels.	• High capital cost • Medium maintenance burden • Limited to drainage areas of 50 acres

Description: Regenerative stormwater conveyance (RSC) provides treatment, and conveyance through the combination of sand, wood chips, native vegetation, riffles (with either cobble rocks or boulders), and shallow pools. RSCs are designed to convey water while minimizing the effects of erosion. A regenerative stormwater conveyance (RSC) is an emerging linearized BMP that uses wetland concepts to treat stormwater. (2-8) RSCs may be used in special situations with prior coordination and approval from the appropriate GDOT personnel and regulatory agencies.

Design Considerations:

- Drainage area less than 50 acres (typically 10-30 acres)
- Detain and treat the WQ_v
- May provide CP_v
- Channel slopes less than 10%
- Pools should drain within 72 hours for WQ_v

Maintenance Considerations:

- Provide adequate access to the BMP and appropriate components.
- Monitor for head cutting around weirs and riffle structures

Applicability for Roadway Projects:

- Well suited for small drainage areas with a high percentage of impervious area
- Moderate land requirement
- Can handle moderately steep slopes

Stormwater Management Suitability:

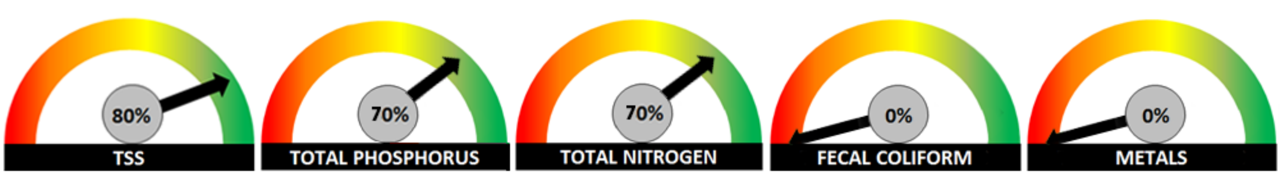
X Runoff Reduction
✓ Water Quality
X Channel Protection
X Overbank Flood Protection
X Extreme Flood Protection
✓ Temperature Reduction

✓ Suitable for this practice ○ May provide partial benefits X Not suitable

LID/GI Considerations

Moderate land requirement and may be incorporated to complement the natural landscape.

Treatment Capabilities



2.6.12 Regenerative Stormwater Conveyance

Description

Regenerative stormwater conveyance (RSC) systems are BMPs that are designed to restore incised and eroded channels, ditches, and intermittent (ephemeral) streams. They are constructed with a series of shallow pools, riffles, cascades, weirs, and outfalls that dissipate stormwater runoff energy and allow for temporary ponding, internal storage, and infiltration. The temporary ponding area of a RSC provides settling time for total suspended solids. The wood chip/sand layer provides filtration as well as an environment conducive to the growth of microorganisms that degrade hydrocarbons and organic material. See Figure 2.6.12-1 for a schematic of a regenerative stormwater conveyance system.

Note: Designers must receive approval from the Office of Design Policy and Support prior to including on projects if the BMP is to be used for MS4 Permit compliance reasons.

Stormwater Management Suitability

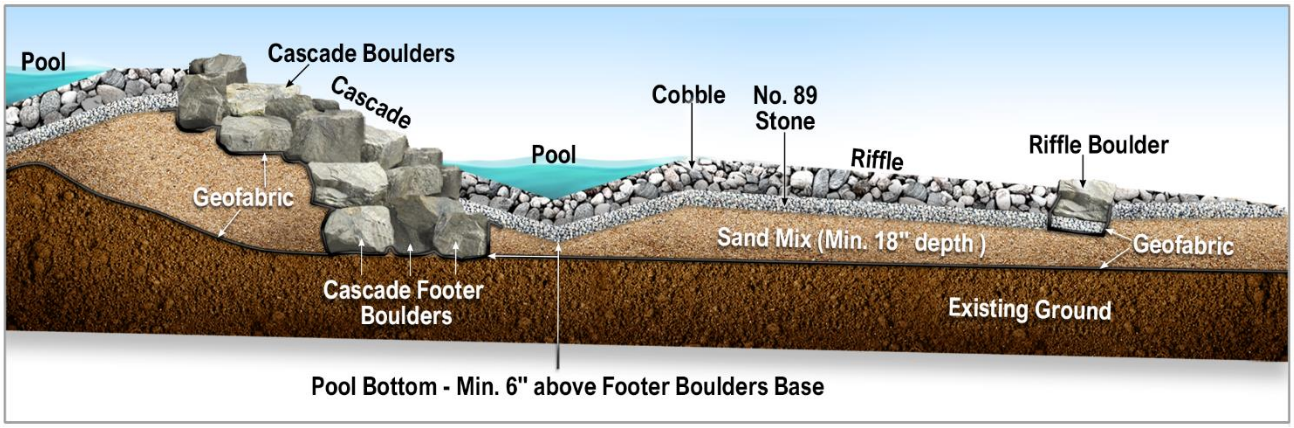
- Runoff Reduction – RSC systems are not designed to provide runoff reduction.
- Water Quality – If installed as per the recommended design criteria and properly maintained, 80% total suspended solids removal can be applied to the water quality volume (WQv) flowing to the RSC system
- Channel Protection – RSC systems do not provide channel protection. Another BMP should be used in a treatment train with RSC systems to provide channel protection or runoff reduction.
- Overbank Flood Protection – RSC systems do not provide overbank flood protection. Another BMP should be used in a treatment train with RSC systems to provide overbank flood protection or runoff reduction.
- Extreme Flood Protection – RSC systems do not provide extreme flood protection. Another BMP should be used in a treatment train with RSC systems to provide extreme flood protection or runoff reduction.
- Temperature Reduction – RSC systems provide temperature reduction.

Pollutant Removal Capabilities

The following average pollutant removal rates may be utilized for design purposes: [(2-17)]

- TSS – 80%
- TP – 70%
- TN – 70%
- Fecal Coliform – 0%
- Heavy Metals – 0%

Figure 2.6.12-1 – Typical RSC Profile



Application and Site Suitability

Regenerative stormwater conveyance systems are usually used to retrofit or repair an existing channel. They can be designed to receive stormwater runoff from up to 50 acres, usually highly impervious. They can also be designed for new construction projects and roadway designs when site conditions allow. Although an RSC system can receive relatively high volume and rates of runoff, they are not considered for control of the CPv, Qp25, and Qf.

When considering locations for a RSC, the following constraints should be considered:

- **Drainage Area –** 50 acres or less, but typically 10 to 30 acres.
- **Drainage Area Characteristics –** No restrictions for roadway drainage.
- **Depth to Water Table** – Shallow ponding areas should include storage volume above the seasonally high groundwater table to allow for temporary ponding in a majority of the pools and storage of the water quality volume.
- **Soils** – No soil restrictions.
- **Site Slope** – Drainage channel slopes should be 10% or less for water quality treatment.
- **Trout Stream** – The ponding and settling functions provided by RSC systems allow for a reduction of the thermal impacts and pollutant loads of runoff from highly urbanized areas.
- **Other Considerations –**
    - Hot spots – RSC systems should not be used for hot spot runoff.
    - Damage to existing structures and facilities – Ensure that runoff through the RSC system is conveyed in a safe, non-erosive manner to minimize damage to existing structures and facilities.
    - Proximity – The following is a list of specific setback requirements for the location of a regenerative stormwater conveyance system:
        - 10 feet from building foundations
        - 10 feet from property lines
        - 100 feet from private water supply wells

- 200 feet from public water supply reservoirs (measured from edge of water)
- 1,200 feet from public water supply wells

Data for Design

The initial data needed for RSC design may include the following:

- Existing and proposed site, topographic and location maps, and field reviews
- Field measured topography or digital terrain model (DTM)
- Drainage basin characteristics
- Preliminary plans including plan view, roadway and drainage profiles, cross sections, utility plans, and soil report
- Environmental constraints
- Location of nearby surface waters and the depth to groundwater
- Design data of nearby hydraulic structures
- Additional survey information

General Design

RSC systems are best used to restore ecological functions to an existing eroded ditch, outfall, channel, or ephemeral stream. RSC systems are designed for intermittent flow and must be allowed to drain and reaerate between rainfall events.

A RSC consists of the following:

- A sequence of pools, riffles, and cascades to assist in treating, detaining, and conveying storm flow.
- Organic/mulch layer to protect planting media.
- A grade control structure and settling pool should be used if the slope of the channel is greater than 5%.
- Pretreatment maybe required to keep sediment and large debris out of the practice.

Physical Specifications / Geometry

- Recommended total length of each grade control structure and each pool is 10 feet or more.
- The invert of the upstream elevation of the grade control structure should be 1 foot higher than the elevation of the downstream grade control structure.
- The width of the grade control structure should be equal or greater than the width of the 100-year storm flowing across the grade control structure.

The Georgia Stormwater Management Manual recommends using the following equation to determine the length of the grade control structure for RSC:

$$L_{GCS} = L_{pool} = \frac{L_{RSC\ Path}}{\frac{\Delta E}{2}}$$

(2.6.12-1)

Where:

L_{GCS} = Length of grade control structure

L_{pool} = Length of pool

$L_{RSC\ Path}$ = Length of the RSC flow path

ΔE = Change in Elevation

- Four inches should be the maximum depth of flow going over the grade control structure.
- Cascades should have a maximum slope of 2H:1V, a maximum vertical drop of 5 feet, and followed by three pools instead of the usual one.
- Pool widths should be greater than the width of the grade control structure.
- Sand layer should be a mixture of sand and wood chips with a ratio of 4:1. This layer should run along the length of the RSC system.
- Maximum width of the sand bed is 14 feet.
- The velocity of the water going through the pool should be less than 4 ft/s.
- Footer boulders should be inserted 6 inches lower than the invert of the pool.
- Flow velocity going through the RSC should be less than the maximum allowable velocity for the cobble size that was selected, use Table 2.6.12-1 to size the cobble stones based on the velocity of flow during the 100-year design storm.

Table 2.6.12-1 Cobble Diameter Based on Flow Velocity

Cobble Diameter, inches	Allowable Velocity (ft/s)
Type 3 Rip-Rap	7.9
Type 1 Rip-Rap	10.4

Source: Georgia Stormwater Management Manual Volume 2

Grade Control Structure Boulders

Boulders shall be type C or type D.

The designer shall note on the design plans that the edges of the boulders should be placed as tightly against one another as possible, creating a continuous structure. All voids between boulders shall be chinked with cobble or boulder fragments from behind the structure to fill voids and promote surface flow over the boulders.

Pretreatment

Pretreatment is vital to the successful operation of filtration BMPs as the media can quickly become clogged from high sediment loads if otherwise left without pretreatment. Where possible, forebays should be provided. Refer to section 2.8.1 and the GDOT Riprap Forebay Special Construction Detail for additional information guidance on forebays. Filter strips and grass channels can be used for pretreatment in a treatment train application. Flow exiting the pretreatment device and entering the RSC should be nonerosive.

Outlet Structure

The outlet of the RSC should end with an outlet pool with a grade control structure just upstream of the outlet pool. The outlet pool elevation should match the existing grade.

Access and Driveway Considerations

Adequate access to all elements of the RSC must be included in the design to allow for inspection and maintenance. See section 2.10.3 for maintenance access requirements.

Regenerative Stormwater Conveyance Sizing

1. Determine the goals and primary function of the RSC.

   The goals and primary function of the BMP must take into account any restrictions or site-specific constraints. Also take into consideration any special surface water or watershed requirements.

2. Calculate the Target Water Quality Volume

   Calculate the water quality volume formula using the following formula:

$$WQ_v = \frac{1.2\ in \times (R_v) \times A \times 43560\,\frac{ft^2}{acre}}{12\ \frac{in}{ft}}$$

   Where:

   WQ_v = water quality volume (ft^3)
   R_v = volumetric runoff coefficient. See section 2.4 for volumetric runoff coefficient calculations.
   A = onsite drainage area of the post-condition basin (acres

3. Determine the storage volume of the practice and the pretreatment volume

   The actual volume provided in the RSC is calculated using the following formula:

$$VP_{Total} = VP_{Sand} + \sum VP_{Pools}$$

   Where:

   VP_{Total} = Total volume provided
   VP_{Sand} = Volume provided in the sand layer
   VP_{Pools} = Volume provided in the pools throughout the RSC system

To determine the volume provided for the sand layer in the RSC, use the following equation:

$$VP_{SAND} = (VSB)*\ (N)$$

Where:
VP_{SAND} = Volume provided in sand layer
VSB = Volume of Sand Bed
N = Porosity (0.4)

To determine the volume provided for the shallow pools in the RSC, use the following equation:

$$VP_{POOLS} = (V_{POOL\_1}) + (V_{POOL\_2}) + (V_{POOL\_3}) + \ldots$$

Where:
VP_{POOLS} = Volume provided in the pools throughout the RSC system
V_{POOL} = Volume of a single storage pool

4. Verify total volume provided by the practice is at least equal to the WQv(target)
When the VP ≥ $WQ_{v(target)}$ then the water quality treatment requirements are met for this practice. When the VP < $WQ_{v(target)}$, then the design must be adjusted, or another BMP must be considered and designed.

5. Design grade control structure, pools, and cascades based on 100-year storm event.
Check velocities and use appropriate stone to prevent erosion and head cutting.

2.7 Detention Design

Overview

Detention sizing and design require employing the following steps:

- Estimating storage volume
- Estimating peak flow reduction
- Defining the stage-storage relationship
- Defining the stage-discharge relationship (performance curve) including:
    - Outlet design
    - Emergency spillway design
    - Conducting hydrograph routing

Estimating Storage Volume

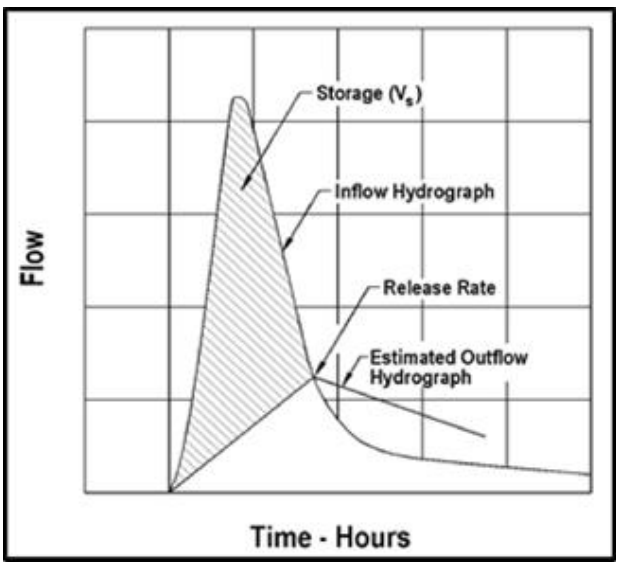
In order to establish the size of the storage basin, a preliminary estimate of the needed storage capacity and the shape of the storage facility are required. This is an iterative process, requiring multiple trials to ensure the necessary peak reduction and desired outflow hydrograph are achieved. The number of trials necessary can be reduced by ensuring accurate computations in the early stages of this process.

The following sections present four methods for determining an initial estimate of the storage required to provide a specific reduction in peak discharge, including the hydrograph method, triangular hydrograph method, NRCS procedure, and regression equation. Once the initial estimate is established, routing is required to finalize the design based on storage volume and outlet structure configuration.

Hydrograph Method

For storage calculations using the hydrograph method, the inflow hydrograph and desired release rate must be determined. The inflow hydrograph represents the runoff from the watershed flowing into the detention basin. The outflow hydrograph is unknown and will be established based on flow attenuation provided by the storage facility. However, for the initial estimation of the needed storage, the outflow hydrograph must be estimated by approximating by straight lines or sketching an assumed outflow curve as shown on Figure 2.7-1. The peak of this estimated outflow hydrograph must not exceed the desired peak outflow from the detention basin. With these values established, the detention basin discharge curve can be estimated and sketched. The shaded area between the curves represents the estimated required storage. To determine the necessary storage, the shaded area can be planimetered or computed mathematically by using a reasonable time period.

Figure 2.7-1 - Hydrograph method for estimating required storage [(2-7)]



Triangular Hydrograph Method

In the triangular hydrograph method, a preliminary estimate of the storage volume (V_s) required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with standard triangular shapes. This method should not be applied if the hydrographs cannot be approximated by a triangular shape; doing so would introduce additional errors to the preliminary estimate of the required storage. This procedure is illustrated by Figure 2.7-2. The required Vs may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph as defined by Equation 2.7-1. [(2-7)]

$$V_s = 0.5 t_b (Q_i - Q_o)$$

(2.7-1)

Where:

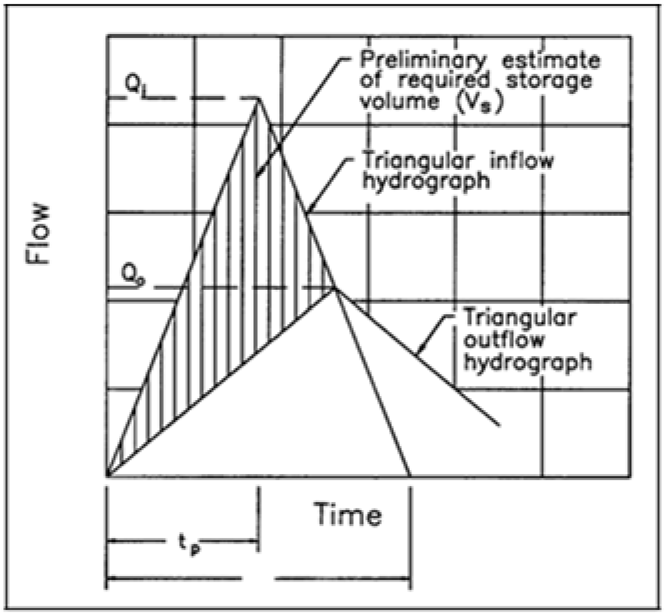
V_s = Storage volume estimate (ft^3)

Q_i = Peak proposed inflow rate into the basin (ft^3/s)

Q_o = Peak existing condition outflow rate out of the basin (ft^3/s)

t_b = Duration of basin inflow(s)

Figure 2.7-2 - SCS detention basin routing curves [(2-7)]



The duration of basin inflow should be derived from the estimated inflow hydrograph. The triangular hydrograph procedure was found to compare favorably with more complete design procedures involving reservoir routing. Refer to the FHWA's HEC No. 22 [(2-7)] for additional information regarding the triangular hydrograph method.

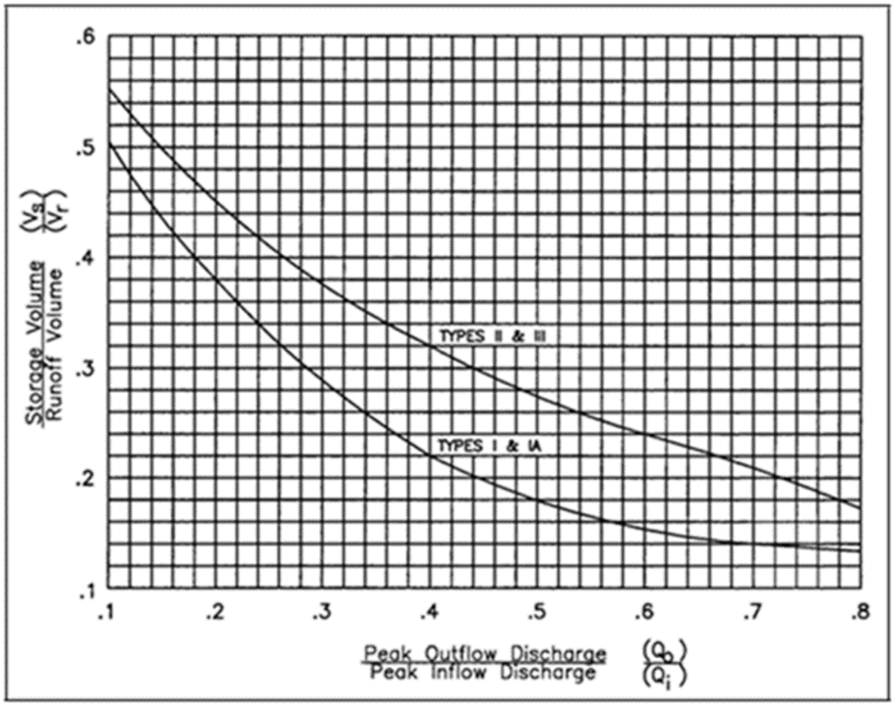
NRCS TR55 Method

The NRCS, in its TR-55 Second Edition Report [(2-34)], describes a manual method for estimating required storage volumes based on peak inflow and outflow rates (www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-tr55.html). The TR55 method is based on average storage and routing effects observed for a large number of structures. A dimensionless figure relating the ratio of V_s to the inflow runoff volume (V_r) with the ratio of Q_o to Q_i was developed, as illustrated in Figure 2.7-3. This procedure for estimating V_s may have errors up to 25 percent of the actual volume and, therefore, should only be used for preliminary estimates.

The procedure for estimating the detention storage required is described as follows:

1. Determine Q_i and Q_o (using the NRCS TR-55 method)
2. Compute the ratio Q_o / Q_i
3. Compute V_r, for the design storm

Figure 2.7-3 - SCS detention basin routing curves (2-7)



$$V_r = K_r Q_D A_m$$

(2.7-2)

Where:

V_r = Inflow volume of runoff (ac-ft)

K_r = 53.33 (dimensionless)

Q_D = Depth of runoff (in)

A_m = Area of watershed (mi^2)

4. Using Figure 2.7-3, determine the ratio V_s/V_r.
5. Determine V_s as

$$V_s = V_r \left(\frac{V_s}{V_r}\right)$$

(2.7-3)

Regression Equation Method

An estimate of V_s required for a specified peak flow reduction can be obtained by using the following regression equation method first presented by Wycoff & Singh. (2-42)

1. Determine the V_r in the inflow hydrograph, Q_o, t_b, and the time to peak of the inflow hydrograph (t_p).
2. Calculate a preliminary estimate of the ratio V_s/V_r using the input data from step 1 and the following equation:

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

(2.7-4)

3. Multiply V_r times the volume ratio computed from Equation 2.7-4 to obtain an estimate of the required V_s.

Estimating Peak Flow Reduction

Similarly, if V_s is known and the designer wants to estimate the peak discharge, two methods can be used. First, the TR-55 method as demonstrated in Figure 2.7-3 can be solved backwards for the ratio of Q_o/Q_i. Secondly, a preliminary estimate of the potential peak flow reduction can be obtained by rewriting the regression Equation 2.7-4 in terms of discharges. This use of the regression equations is demonstrated below.

1. Determine V_r, Q_i, t_b, t_p, and V_s.
2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation.

$$\left(\frac{Q_o}{Q_i}\right) = 1 - 0.712\left(\frac{V_s}{V_r}\right)^{1.328}\left(\frac{t_b}{t_p}\right)^{0.546}$$

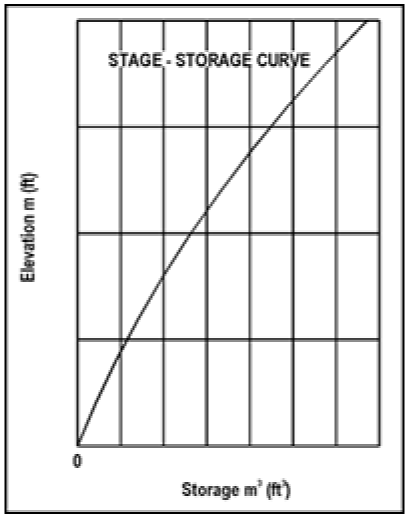
(2.7-5)

3. Multiply Q_i times the potential peak flow reduction ratio calculated from step 2 to obtain Q_o for the selected V_s.

Stage-Storage Relationship

A stage-storage relationship defines the relationship between the depth of water and V_s in the storage facility. The volume of storage can be calculated by using simple geometric formulas expressed as a function of storage depth. This relationship between V_s and depth defines the stage-storage curve. A typical stage-storage curve is illustrated in Figure 2.7-4.

Figure 2.7-4 - Example stage-storage curve (2-7)



After the required storage has been estimated, the configuration of the storage basin must be determined so that the stage-storage curve can be developed. Detention facilities can take various shapes including:

- Rectangular
- Trapezoidal
- Pipes and conduits
- Natural basins

Stage-storage calculations vary depending on the shape of the facility. Refer to HEC 22 (2-7) for additional information. Additionally, popular software packages, such as HydroCAD and Bentley PondPack and InRoads, are capable of generating stage-storage data.

Stage-Discharge Relationship (Performance Curve)

A stage-discharge (performance) curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility will have both a principal and an emergency outlet. The principal outlet is typically designed to convey the design storms below the 100-year recurrence interval. The 100-year, 24-hour storm can be designed to safely pass via the emergency spillway. The principal outlet structure typically consists of a pipe culvert, weir, orifice, or other appropriate hydraulic control device. Multiple outlet control devices are often used to provide discharge controls for multiple frequency storms (i.e., CP_v, Q_{p25}, and Q_f).

Development of a composite stage-discharge curve requires consideration of the discharge rating relationships for each component of the outlet structure. The following sections present design relationships for typical outlet controls.

Orifices and Weirs

Orifices can be used to control flow rates. Values for C_D typically range from 0.6 for square-edged, uniform orifice entrance conditions to 0.4 for ragged edged orifices (e.g., holes cut in corrugated pipe using a torch).

Outlet pipes smaller than 1 foot in diameter may be analyzed as a submerged orifice as long as the change in head (H) divided the diameter of the orifice (D) is greater than 1.5. Pipes greater than 1 foot in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account.

Flow through multiple orifices can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

Wiers can also be used to control flow rates. Values for C_D range from 3.33 for sharp-crested weirs, to 2.34–3.32 for broad-crested weirs. Weir calculations are commonly needed for discharge locations through the sides of risers, through the tops of risers, and over emergency spillways.

Additional guidance for weir and orifice flow can be found in HEC 22. [(2-7)]

Discharge Pipes

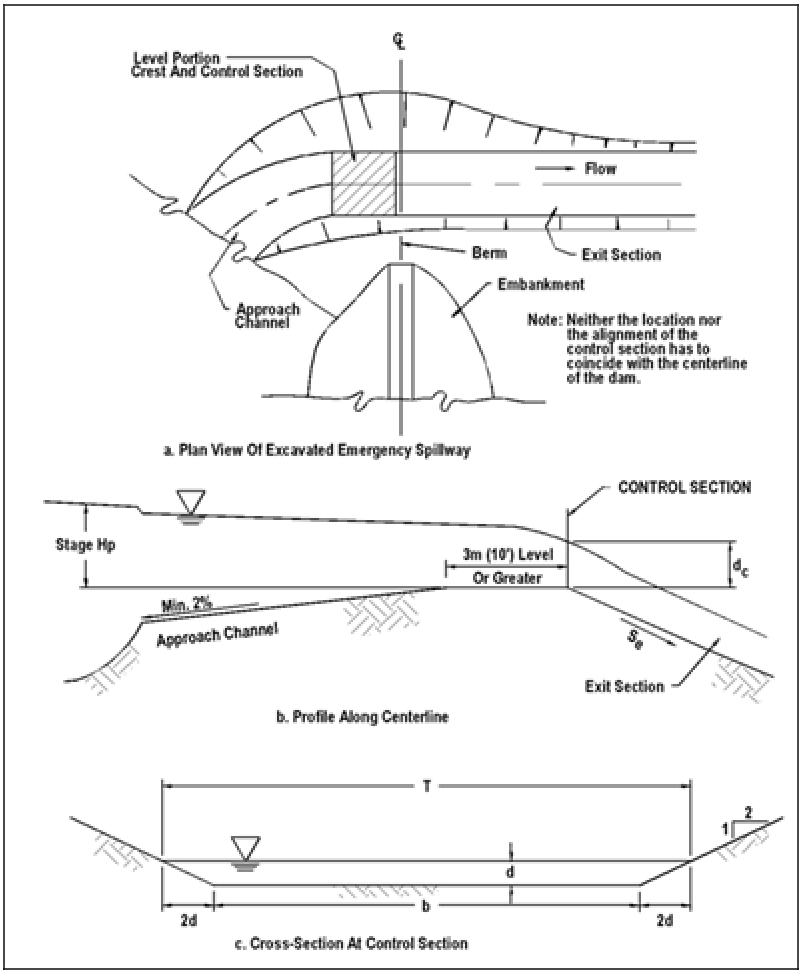
Discharge pipes are often used as outlet structures for detention facilities and can be designed for single- or multi-stage discharges. A single-stage discharge system consists of a single culvert entrance designed to carry emergency flows according to design procedures outlined in HDS-5. [(2-32)] A multi-stage inlet includes a control structure at the inlet end of the pipe (referred to as the outlet structure) that is designed so that design flows discharge through a weir or orifice in the lower levels of the structure and emergency flows pass over the top of the structure. The outlet pipe should have capacity to carry the full range of flows from a drainage area including the emergency flows.

Design of multi-stage structures begins with determination of peak discharges that must be passed through the facility. Second, the designer should select a pipe with the capacity to pass the peak flow within the allowable headwater and develop a performance curve for the pipe. Third, the designer should develop a stage-discharge curve for the outlet control structure incorporating the discharge pipe headwater as the tailwater condition for the outlet structure. Lastly, the designer should perform basin routing using the stage-discharge curve.

Emergency Spillway

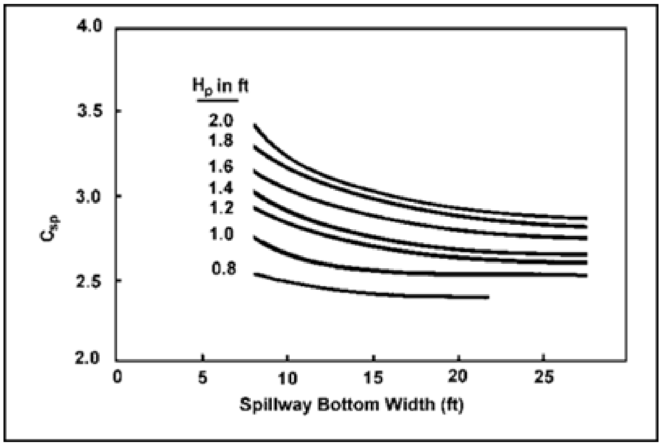
The emergency spillway is generally an open channel constructed in natural ground (as opposed to the embankment). The purpose of an emergency spillway is to provide a controlled overflow relief for storm flows in excess of the design discharge for the storage facility. Detention storage facilities for highway applications often use a broad-crested overflow weir cut through the original ground next to the embankment for overflow passage, as illustrated in Figure 2.7-5. The transverse cross-section of the weir cut is typically trapezoidal in shape for ease of construction. The emergency overflow elevation shall be established at least one (1) foot below the roadway's normal shoulder break point and within 0.5 ft of the 100-year ponding elevation modeled with an unclogged outlet structure. The spillway shall be capable of conveying the 100-year storm modeled with a clogged outlet structure. If including an emergency spillway in the design is not possible, size the weir(s) in the outlet structure so that they are capable of conveying the 100-year storm. Refer to the Dry Detention Basin Outlet Structure Special Construction Detail for additional information. Refer to the guidance given in chapter 6 of the Drainage Design Policy Manual for assistance in sizing the channel and determining an appropriate lining material.

Figure 2.7-5 - Discharge coefficients for emergency spillways (2-7)



The weir equation presented in chapter 10 of HEC-22 may be used to calculate flow through the emergency spillway at various stages. C_D varies based on spillway bottom width (L) and head (H). Figure 2.7-6 can be used to determine C_D for emergency spillway flow for grassed channels with a Manning's n of 0.040. Equations presented in HEC 22 (2-7) can be used for different configurations. The top of the spillway section and channel extending down the slope should be protected with a temporary Type 1 Turf Reinforcement Matting (TRM1).

Figure 2.7-6 - Discharge coefficients for emergency spillways [(2-7)]

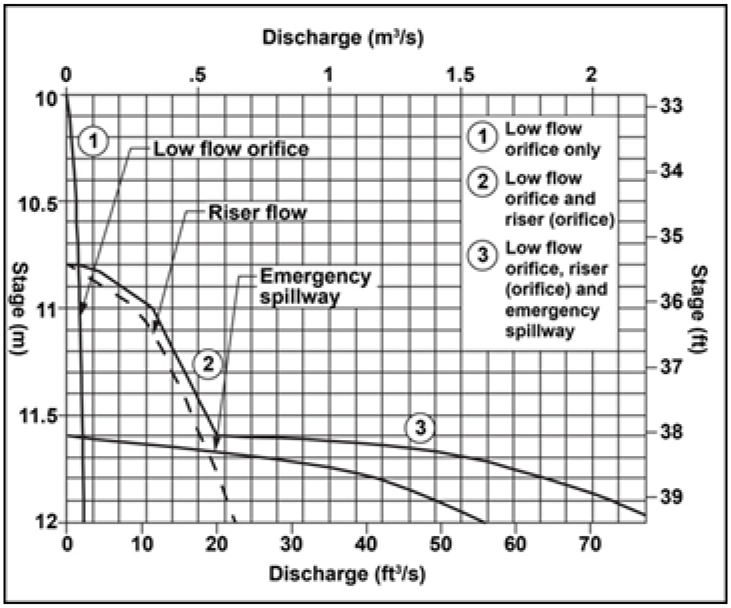


Composite Stage Discharge Curves

As indicated by the discussions in this section, development of a stage-discharge curve for a particular outlet control structure depends on the interaction between each component of the control structure. Figure 2.7-7 illustrates the construction of a stage-discharge curve for an outlet control device consisting of a low flow orifice and a riser pipe connected to an outflow pipe. The structure also includes an emergency spillway.

The impact of each element in the control structure can be seen in Figure 2.7-7. Initially, the low flow orifice controls the discharge. At an elevation of 35.4 feet, the water in the storage facility reaches the top of the riser pipe and begins to flow into the riser. The flow at this point is a combination of the flows through the orifice and the riser. Orifice flow through the riser controls the riser discharge above a stage of 36.1 feet. At an elevation of 38.0 feet, flow begins to pass over the emergency spillway. Beyond this point, the total discharge from the facility is a summation of the flows through the low flow orifice, the riser pipe, and the emergency spillway. Additionally, the designer needs to verify that the outlet pipe from the detention basin is large enough to carry the total flows from the low flow orifice and the riser section to prevent the outlet pipe from controlling the flow from the basin.

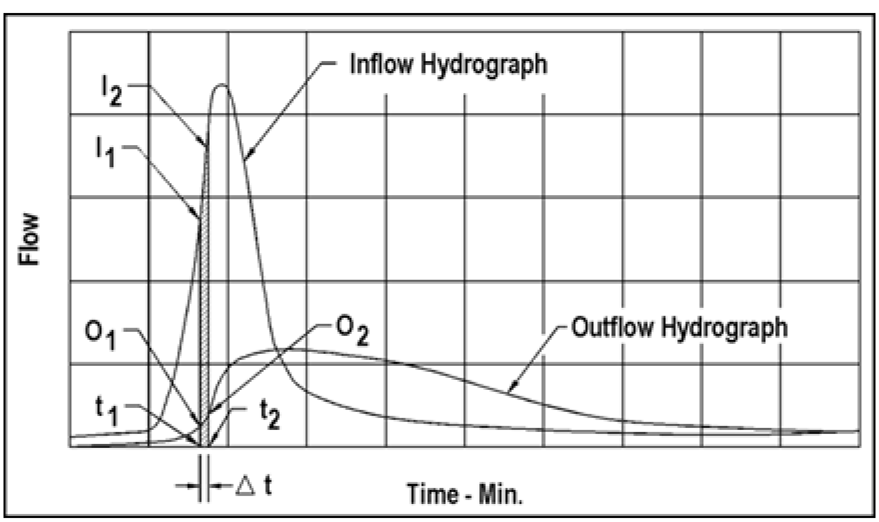
Figure 2.7-7 - Typical combined stage-discharge relationship (2-7)



Generalized Routing Procedure

Various software packages can be used to assist in completing the detention basin design steps and routing. The example calculation provided at the end of this section describes the general approach for using software to assist in detention design. The manual routing procedure will be briefly described to give designers a basic understanding of the underlying principles. Additional guidance and example calculations can be found in HEC 22 (2-7).The most commonly used method for routing an inflow hydrograph through a detention pond is the Storage Indication or Modified Puls method. This method begins with the continuity equation, which states that the inflow minus the outflow equals the change in storage (I - O = DS). By taking the average of two closely spaced inflows and two closely spaced outflows, the method is expressed by Equation 2.7-6. This relationship is illustrated graphically in Figure 2.7-8.

Figure 2.7-8 - Schematic of routing hydrograph (2-7)



$$\frac{\Delta S}{\Delta t} = \frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2}$$

(2.7-6)

Where:

ΔS = Change in storage, ft^3

t = Time interval, min

I = Inflow, ft^3

O = Outflow, ft^3

Equation 2.7-6 can be rearranged as shown in Equation 2.7-7. The following procedure can be used to perform routing through a reservoir or storage facility using Equation 2.7-7.

$$\frac{I_1 + I_2}{2} + \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right)$$

(2.7-7)

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

Step 2. Select a routing time period, Dt, to provide a minimum of five points on the rising limb of the inflow hydrograph.

Step 3. Use the stage-storage and stage-discharge data from step 1 to develop a storage indicator numbers table that provides storage indicator values, S/Dt + O/2, versus stage. A typical storage indicator numbers table contains the following column headings:

(1)	(2)	(3)	(4)	(5)	(6)
Stage	Discharge (O)	Storage (S)	$O_2/2$	S_2/Dt	S_2/Dt+ $O_2/2$
ft	ft^3/s	ft^3	ft^3/s	ft^3/s	

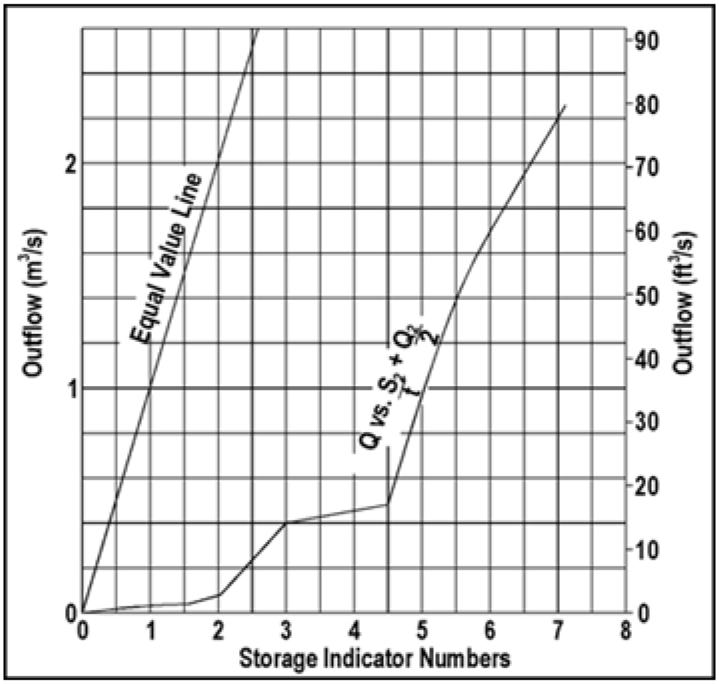
a. The O and S are obtained from the stage-discharge and stage-storage curves, respectively.

b. The subscript 2 is arbitrarily assigned at this time.

c. The time interval (Dt) must be the same as the Dt used in the tabulated inflow hydrograph.

Step 4. Develop a storage indicator numbers curve by plotting the outflow (column 2) vertically against the storage indicator numbers in column (6). An equal value line plotted as O_2 = S_2/Dt + O_2/2 should also be plotted. If the storage indicator curve crosses the equal value line, a smaller time increment (D_t) is needed (refer to Figure 2.7-9).

Figure 2.7-9 - Storage indicator curve (2-7)



Step 5. **A supplementary curve of storage (column 3) vs. $S_2/Dt + O_2/2$ (column 4) can also be constructed. This curve does not enter into the mainstream of the routing; however, it is useful for identifying storage for any given value of $S_2/Dt + O_2/2$. A plot of storage vs. time can be developed from this curve.**

Step 6. **The routing can now be performed by developing a routing table for the solution of Equation 2.7-7 as follows:**

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Time	Inflow	$(I_1+I_2)/2$	$S_1/t+O_1/2$	O_1	$S_2/t+O_2/2$	O_2
(hr)	(ft^3/s)	(ft^3/s)	(ft^3/s)	(ft^3/s)	(ft^3/s)	(ft^3/s)

a. Columns (1) and (2) are obtained from the inflow hydrograph.

b. Column (3) is the average inflow over the time interval.

c. The initial values for columns (4) and (5) are generally assumed to be zero since there is no storage or discharge at the beginning of the hydrograph when there is no inflow into the basin.

d. The left side of Equation 2.7-7 is determined algebraically as columns (3) + (4) - (5). This value equals the right side of Equation 2.7-7 or $S_2/Dt + O_2/2$ and is placed in column (6).

e. Enter the storage indicator curve with $S_2/Dt + O_2/2$ (column 6) to obtain O_2 (column 7).

f. Column (6) ($S_2/Dt + O_2/2$) and column (7) (O_2) are transported to the next line and become $S_1/Dt + O_1/2$ and O_l in columns (4) and (5), respectively. Because ($S_2/Dt + O_2/2$) and O_2 are the ending values for the first-time step, they can also be said to be the beginning values for the second time step.

g. Columns (3), (4), and (5) are again combined and the process is continued until the storm is routed.
h. Peak storage depth and discharge (O_2 in column (7)) will occur when column (6) reaches a maximum. The storage indicator numbers table developed in Step 3 is entered with the maximum value of $S_2/Dt + O_2/2$ to obtain the maximum amount of storage required. This table can also be used to determine the corresponding elevation of the depth of stored water.
i. The designer needs to make sure that the peak value in column (7) does not exceed the allowable discharge as prescribed by the stormwater management criteria.

Step 7. Plot O_2 (column (7)) versus time (column (1)) to obtain the outflow hydrograph.

Detention Design Example Calculation

Manual Calculation:

Refer to HEC 22 for a step-by-step guide to detention design including example calculations.

Software-Assisted Design:

Practitioners use various software packages to assist in the design of detention facilities. The following guidance describes the general process required for most software products.

Inputs:

1. Enter hydrologic information.
   - Users can usually enter tabular inflow hydrograph data directly, if it is available (i.e., time vs inflow). Alternatively, most software packages will assist in calculating the inflow hydrograph using precipitation and drainage area input.
   - Precipitation data consists of:
     - Rainfall intensity-duration-frequency (IDF) curves
     - The desired design storms
   - Define drainage area characteristics (sometimes referred to as catchments)
     - Area
     - Land cover (CN)
     - Times of concentration
2. Define drainage system configuration.
   - Often multiple drainage areas or subcatchments drain to one detention facility.
   - The various components of the system are often defined by nodes and reaches to calculate the aggregate runoff and time of concentration.
3. Enter assumed basin geometry.
   - Estimate the required storage volume using one of the methods described in this section.
   - The stage-storage relationship can then be defined using one of a few options depending on site constraints and information available.
     - Tabular stage-storage data can be entered, if available.
     - Or, the basin geometry can be defined.
       - A trapezoidal basin can be defined by length, width, depth, and slope.
       - If the basin has already been laid out in the site plan, the contours can be used to define the basin volume by entering the elevation and area of each of the contours.
4. Define basin outlet controls.
   - Outlet sizes such as weir lengths and orifice openings should be estimated by setting the flow equal to the pre-development peak flow.
   - Common outlet components often consist of:

- Risers
  - Orifices
  - Weirs
  - Pipes

Outputs:

Most software packages will calculate and generate plots and tables for stage vs time, storage vs time, and the outflow hydrograph. Use this data to determine if the basin design meets the flow requirements and adjust the outlet controls and basin size if it does not. Review the BMP's design guidance in section 2.6 and determine if all criteria (such as freeboard) have been met.

2.8 Common BMP Components

This section provides design guidance for components commonly found in many BMPs.

If the project will have a special design concrete structure such as a pond with retaining walls or a modified outlet structure design, designers will be required to submit special design details to the GDOT Office of Bridge and Structural Design.

2.8.1 Forebay

Adequate pretreatment is an essential component of many BMPs. Pretreatment facilities extend the life of BMPs and reduce maintenance frequency and effort. Pretreatment is required for some BMPs and is optional for others. For example, swales do not typically require pretreatment. However, BMPs that use filtration, infiltration, or small orifices should have pretreatment to reduce sediment and debris loading.

Forebays remove coarser suspended solids and debris, dissipate energy, and prevent erosion at the BMP inlet(s). Forebays should typically be provided at any inlet that contributes concentrated flow that is over 10% of the total flow to the BMP. A forebay is not necessary in cases where inflow to the BMP is non-erosive and enters as filtered sheet flow from a device, such as a vegetated filter strip. Filter strips used as pretreatment should meet the requirements of section 2.6.1.

The forebay should be designed as a separate cell and lined or armored to prevent erosion. The bottom and sides of the forebay are typically lined with woven plastic filter fabric and riprap. The overflow spillway, which is the downslope section where runoff exits the forebay and enters the BMP, is often constructed using gabion baskets or concrete in order to form a better defined spillway. Spillways must be designed to safely convey the 10-year storm event. Figure 2.8-1 shows a typical forebay configuration.

Figure 2.8-1 - Typical forebay configuration (in a bioretention basin) (photo courtesy of NCDOT)

Forebays should be sized for 0.1 inches of runoff per impervious acre of contributing drainage area.

For example, assume a 2.5-acre impervious area drains to a BMP through a single inlet that requires a forebay. The forebay size should be determined as follows:

$$Forebay\ Volume = 0.1 \frac{inches}{impervious\ acre}$$

$$Forebay\ Volume = 0.1\ in \times 2.5\ ac \times \frac{1\ ft}{12\ in} \times \frac{43{,}560\ ft^2}{1\ ac}$$

$$Forebay\ Volume = 907.5\ ft^3$$

Embankment side slopes should be 3:1 or flatter, however can be 2:1 with permission from the Office of Design Policy and Support. Forebays should be 3 to 4 feet deep for large scale BMPs (where the drainage area is greater than 5 acres). Forebay criteria may be reduced for smaller BMPs. Although not desirable, forebays may be eliminated for small BMPs where very minimal sediment and debris are expected and inlets are designed or determined to be non-erosive.

The forebay's pretreatment volume may be located within the main basin of the BMP and included in the calculation of the total treatment volume, if needed.

A fixed vertical sediment depth marker should be installed in the forebay to measure sediment deposition.

Riprap forebay dams shall have a maximum height of 4 feet as measured from the downstream toe of slope to the spillway elevation and maximum height of 5 ½ feet as measured to the top of dam.

The spillway width shall be a minimum of 8 feet for dry detention ponds or wet detention ponds. The spillway width shall match the media width for enhanced dry swales or the channel bottom width for enhanced wet swales.

Bioretention basins shall have a riprap forebay for each inlet receiving more than 1 acre of drainage. Riprap forebays shall also be provided at any inlet that contributes concentrated flow that is over 10% of the total flow to the stormwater BMP.

Refer to the GDOT Riprap Forebay Special Construction Detail for more information.

2.8.2 Flow Bypass Structure

Flow bypass structures, sometimes called flow diverters or flow splitters, are often used for small-scale BMPs to prevent erosion of BMP surfaces or other modes of failure by diverting the WQ_v to the BMP and bypassing excess volume to another location. In order to use a bypass structure, prior approval from the Office of Design Policy and Support shall be required before incorporating a bypass structure into the design. BMPs that typically require flow bypass structures include infiltration trenches and sand filters, but they may also be used with other offline BMPs. Flow bypass structures can also be used upstream of the BMP to help reduce the size of BMP outlet control system or eliminate the need for it altogether.

Flow bypass structures can be designed for a desired volume or flow rate. A weir bypass structure, as shown in Figures 2.8-2 and 2.8-3, is designed to divert a given volume. All stormwater runoff is directed into the BMP, with overflow of the weir occurring when the WQ_v is exceeded, releasing the additional volume to the stormwater drainage system. Similarly, the bypass structure can be designed such that water backs up into the bypass structure as the water level in the BMP rises. A second outlet pipe with its invert at a higher elevation releases runoff as the WQ_v is exceeded. Computer modeling is recommended for the sizing and design of these structures and backwater conditions should be evaluated.

Flow bypass structures using flow rate as the controlling factor include a small diameter pipe, orifice, or similar hydraulic control device sized for the maximum water quality peak discharge that, when exceeded, directs any additional flow to the stormwater drainage system. Figure 2.8-2 illustrates an example of this configuration assuming the outlet pipe to the BMP is sized to restrict flow. For these types of bypass structures, BMP outlet control systems must be provided as the BMP's capacity (volume) can be exceeded during low intensity storms. Refer to chapter 5 of the Drainage Design Policy Manual if the hydraulic control is a small diameter pipe. Refer to chapter 8 of HEC-22 if other hydraulic control devices such as weirs and orifices are used.

Flow bypass structures are often prone to clogging which can result in roadway flooding. For this reason, flow bypass structures should be readily accessible for maintenance.

Figure 2.8-2 - Commonly used flow bypass structure (adapted from GSMM Vol. 2) (2-17)

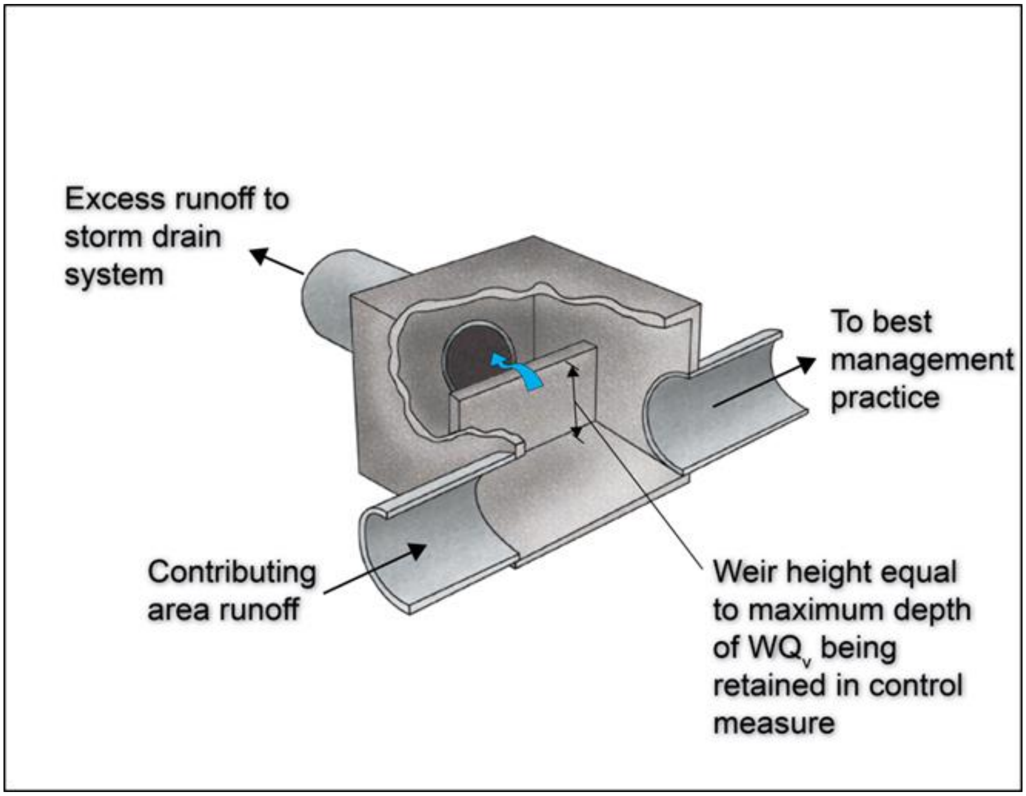


Figure 2.8-3 - A commonly used flow bypass structure configuration that bypasses flow when the capacity of the outlet pipe supplying the BMP is exceeded (photo courtesy of NCSU-BAE)

2.8.3 Underdrains

Underdrains are perforated piping used to drain and discharge the treated stormwater from filtration BMPs. Underdrains, however, may also be included in infiltration-type BMPs as a safety measure to allow the BMP to drain in the event the BMP gets clogged or is not functioning as designed. If underdrains are provided for infiltration-type BMPs, the end at the outlet control structure shall be capped. Multiple branches of underdrain pipe may be utilized when needed. Spacing between branches should be no greater than 10 feet. The branches of the underdrain should come together within the BMP such that only one pipe enters the outlet structure or penetrates the embankment. Underdrains should generally be composed of 8-inch polyethylene pipe, unless being utilized as pipe storage. Perforations are typically set at 3/8-inch diameter and spaced 6 inches on center with 4 rows running longitudinally while the pipe is placed at a minimum slope of 0.5%. These criteria are typically sufficient to provide proper drainage for most BMPs; however, it is prudent to perform calculations to verify the underdrain is adequately designed. Darcy's law can be used to determine the maximum flow rate through the BMP's media. Manning's equation can then be used to verify adequate underdrain pipe diameter. Using the size, spacing, and configuration of the perforations, the orifice equation can be used to determine if the length of the underdrain pipe is sufficient.

Refer to GDOT Specification 573, Underdrains, Special Provision / Specification 169 on Post-Construction Stormwater BMP Items, and the GDOT Underdrain Special Construction Detail for additional information.

Cleanouts should be provided at the end of each underdrain branch. Cleanouts should extend to an elevation that is appropriate for site conditions based on best professional judgment. Consideration should be given for possible inflow of stormwater should a cap become damaged or removed. Consideration should also be given to potential burial by sediment and damage by maintenance equipment. Cleanouts should be placed along underdrains at a maximum spacing of 100 linear feet.

See Figure 2.8-4 for an example of a typical underdrain installation.

Figure 2.8-4 - Typical underdrain installation (photo courtesy of NCDOT)

2.8.4 Level Spreaders

Level Spreaders for Concentrated Flow

Level spreaders with concrete troughs are typically needed to convert concentrated flow to sheet flow. Level spreaders are typically only used in conjunction with filter strips and riparian buffers but may be used for other BMPs as needed. Figure 2.8-6 provides an illustration of a typical level spreader configuration. Level spreaders should be designed to minimize the potential for erosion in downgradient areas and flow bypass systems are often needed to partially divert higher intensity storms. The erosivity of downgradient areas is a function of ground cover, slope, and soils. Flow rate is influenced by the hydrology of the contributing drainage area and the design of the flow bypass structure. The length of the level spreader can be adjusted to distribute the flow over an appropriate area.

Figure 2.8-6 - Typical level spreader configuration (photo courtesy of NCDOT)

The length of the level spreader can be determined using the same methodology for determining filter strip width. Refer to the variation of Manning's equation presented in section 2.6.1 to determine an allowable q. This method assumes an allowable depth of flow (1 or 2 inches) that will not cause erosion in the filter strip.

The design storm for the peak discharge should then be determined. Typically, the flow rate associated with the water quality volume (Q_{wq}) is used for the design of level spreaders but may vary depending on the downgradient BMP and stormwater quality goals. Refer to section 2.4.1.2.1 for guidance on calculating Q_{wq}. The length is then determined by taking Q_{wq} / q. Note that a detention BMP may be used upgradient of the level spreader to control the peak flow to the level spreader, reducing the required length. Detention BMPs may also be used as flow bypass systems. Generally, level spreaders should be limited to 100 feet. [(2-24)] It is difficult to maintain a precisely level lip for lengths in excess of 100 feet, which can cause flow to concentrate in one area of the level spreader.

Level spreaders should be a minimum of 1.5-feet deep and 2-feet wide to provide stilling of flow and allow for some sediment storage. Additional width may be desired, as shown in Figure 2.8-8, to allow maintenance equipment to enter the trough to remove sediment. Widths of 5.5 feet are generally sufficient for small equipment. The lip of the level spreader should be vertical, but the other sides can be sloped for safety (2:1) and to allow for entry by maintenance equipment (4:1). The lip of the level spreader should extend 4 inches above the downgradient ground surface to prevent vegetation from growing over the lip and causing flow to concentrate. Permanent erosion protection liners such as turf reinforcement matting (TRM), may be needed directly downgradient of the lip to stabilize the soil.

Finally, drawdown systems may be included in the design of the trough where standing water is a concern. Figures 2.8-7, 2.8-8, and 2.8-9 show a level spreader configuration used by the NCDOT.

Figure 2.8-7 - Plan view: typical level spreader layout with buffer (adapted from NCDOT)

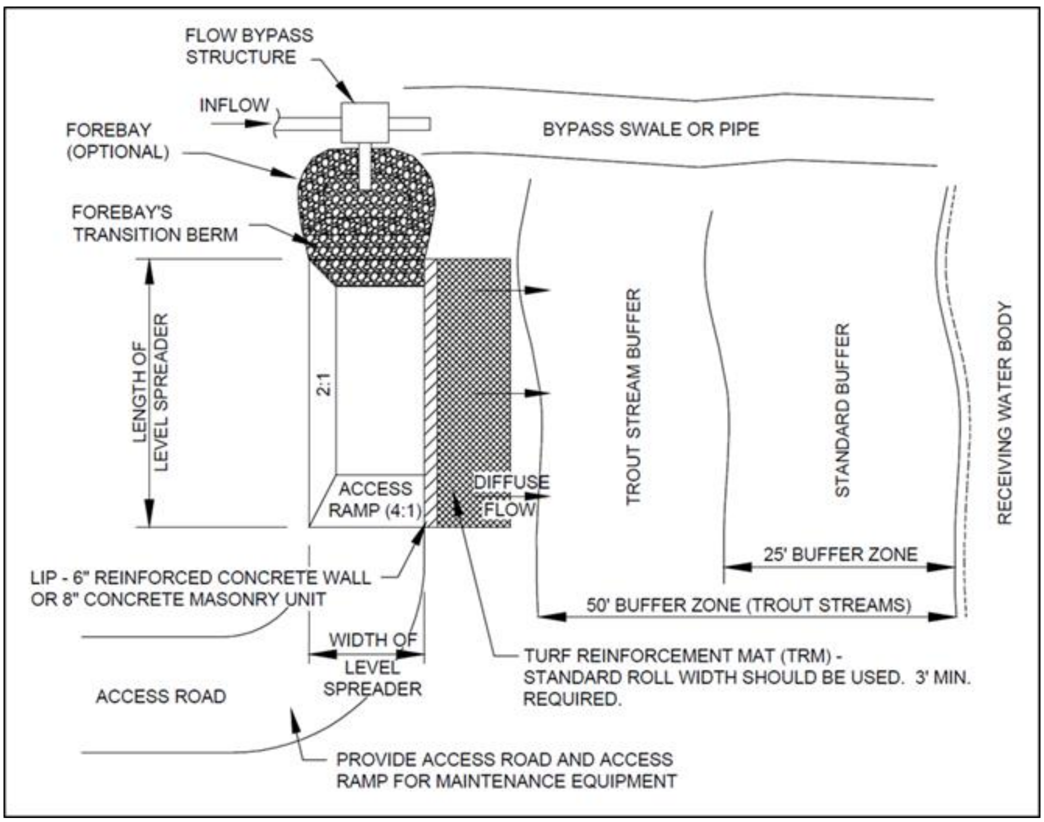


Figure 2.8-8 - Profile view: typical level spreader details and components (adapted from NCDOT)

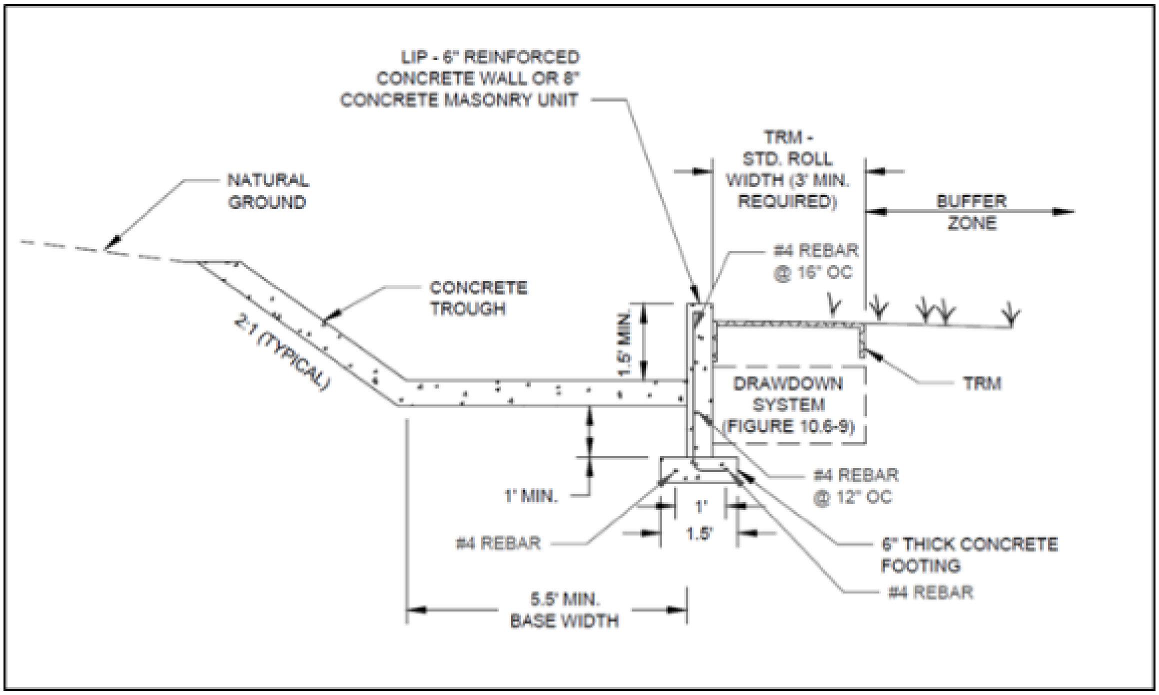
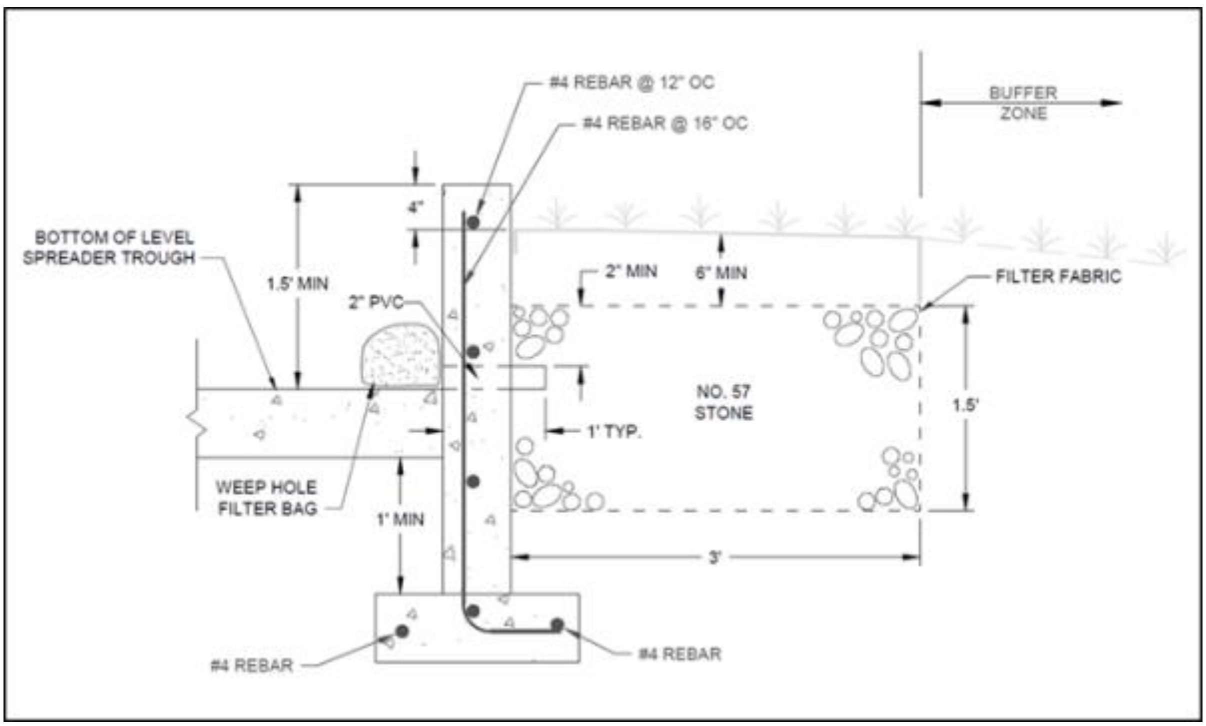


Figure 2.8-9 - Profile view: weep hole dry cell detail for level spreader (adapted from NCDOT)



2.9 Bridge Stormwater Quality Considerations

Drainage design and stormwater management are typically more challenging for bridge runoff because of additional safety hazards and environmental concerns. Increased scrutiny is often placed on stormwater runoff from bridges because these locations can create a direct link between the roadway system and surface waters or other environmentally sensitive areas (ESA). For this reason, deviation from the standard bridge deck drainage system (as described in chapter 9 of the Drainage Design Policy Manual) is typically required in MS4 areas to eliminate direct discharge of stormwater. However, bridge surface area and subsequent runoff volume are often small relative to the body of water that is spanned. These features should be considered when designing the bridge drainage system.

2.9.1 Bridge Stormwater Challenges

The following challenges are often associated with bridge drainage design and stormwater management:

- Structural constraints
- Limited space available for conveyance and treatment
- Limited grade available to achieve positive flow

Special drainage structures and post-construction BMPs designed for use in high-density urban areas may be needed to overcome these challenges.

Note that runoff discharging from bridge deck drains that are at elevations significantly higher than the discharge point area will be dispersed as it falls and lessens the likelihood of erosion in any buffer, wetland, or other vegetated ESA. As mentioned in section 2.4, the requirements associated with stream channel / aquatic resource protection, overbank flood protection, and extreme flood protection are waived for discharge points draining directly to channels that have drainage areas larger than five square miles. Stream channel / aquatic resource protection requirements are also waived if the peak flow is less than 2.0 ft^3/s. Bridges over large bodies of water, such as the Intracoastal Waterway, produce relatively little stormwater runoff when compared to the water body itself, and collecting and conveying this runoff for treatment is often not practicable or feasible. In such instances, stormwater treatment is not required.

2.9.2 Minimizing Direct Discharge

Closed deck drainage systems are often necessary where direct discharge is prohibited. A closed deck drainage system is a network of pipes below the deck drains that captures and conveys runoff to the bridge ends where it is treated by post construction stormwater controls before discharging to the water body. Roadway, bridge, and hydraulics designers should coordinate closely to create an integrated stormwater system that meets drainage and water quality requirements.

Closed deck drainage systems are costly to construct and present a maintenance burden (including costs and safety issues). Design guidance for closed deck drainage systems is provided in chapter 9 of the Drainage Design Policy Manual. Alternatives to closed deck drainage systems include:

1. Deck drains can sometimes be shifted such that discharge over the water body or ESA is avoided. Follow the guidance presented in chapter 9 of the Drainage Design Policy Manual and HEC-21 [(2-44)] to confirm that the safety of the motorist is not compromised.
2. Widening the bridge to increase the shoulder width can sometimes allow runoff to be conveyed safely via the gutter to the bridge abutments, eliminating deck drains altogether.
3. Similarly, shoulders can be shifted on superelevated bridges such that the shoulder on the low side is wider than the shoulder on the high side, providing more space to convey runoff via the gutter.
4. Designing the bridge to crest in the center essentially halves the conveyance capacity that would be required by a bridge with all runoff draining to one side. This may or may not assist in eliminating the need for a closed deck drainage system. Regardless, designing the bridge to crest in the center usually provides twice as much space to manage half the runoff.

2.9.3 Bridge Best Management Practices

There are several practices that should be considered for bridge drainage designs:

Roadway Drainage System Integration

The roadway drainage system must be integrated with the bridge drainage system to effectively convey runoff to the water body. Roadway runoff should be transported down the embankment through an appropriately designed channel (chapter 6 of the Drainage Design Policy Manual) or a drainage structure (chapter 5 of the Drainage Design Policy Manual). Appropriate energy dissipation should be provided at the discharge location (chapter 7 of the Drainage Design Policy Manual).

Slope Stabilization and Ground Cover

Embankments and surrounding areas should have adequate ground cover and stabilization. Careful consideration should be given to materials selected and as to whether or not conditions (e.g., stream stability, shade beneath the bridge) will support vegetative growth. Guidance for riprap aprons at bridges can be referenced in chapter 8, section 8.3 of the Drainage Design Policy Manual.

Energy Dissipation

Energy dissipation is typically needed at the discharge of all conveyances and may be required for areas below deck drains to mitigate the impact of falling runoff and the channelization of flow from multiple deck drains. Refer to chapter 7 of the Drainage Design Policy Manual for additional guidance on energy dissipation design.

Post-Construction Best Management Practices

The post-construction BMPs presented in section 2.6 should be used to meet stormwater management requirements. Refer to sections 2.2 and 2.6 of this manual for further guidance on MS4 permit requirements and post-construction BMPs.

2.10 Safety Considerations for Stormwater BMPs

Stormwater best management practices can present unique safety concerns to motorists, the public, and GDOT maintenance personnel. Guidance for designing an adequate drainage system for the roadway is covered in chapters 4 and 5 of Drainage Design Policy Manual. Dam safety requirements were previously discussed in this chapter and are further defined in the Georgia Safe Dams Act of 1978 (OCGA 12-5-370). Downstream flooding is another concern and is addressed in section 2.2. This section addresses safety concerns associated with stormwater BMPs.

2.10.1 Motorist Safety

Motorist safety is a primary concern for drainage design. Some BMPs or their components can present road hazards and must be placed outside of the clear recovery zone. Refer to the AASHTO Roadside Design Guide (2011) [(2-2)] for additional information relating to clear recovery zone requirements.

2.10.2 Public Safety

While some BMPs are associated with interstate highway systems, others are located in areas that are frequented by pedestrians and the general public. Many BMPs create a temporary (during storm events) or permanent pool of standing water and can present a drowning hazard. The type of BMP, its configuration, and the surrounding areas (e.g., location of nearby schools or playgrounds) should be taken into consideration when determining possible safety measures. Steep embankments and drop-offs should be avoided and safety benches should be provided where possible to minimize the potential for slips and falls. Railings may be an alternative in some areas. Trash racks should be provided over risers to discourage entry by people and animals. Fencing is often added around BMPs to prevent the public from entering the area. Most facilities that meet one or both of the following criteria will require perimeter fencing:

- Stormwater facilities that are located in areas that are subject to frequent visits by the public and/or located adjacent to schools, playgrounds, recreation areas, or urban areas
- Stormwater facilities such as natural ponds, detention ponds, and water quality ponds that contain water over 24-inches deep for an extended period of time (greater than 48 hours)

Perimeter fencing should meet the following guidelines:

- 6-feet height chain link wire fence, in accordance with GDOT standard specification 643.
- Self-closing and self-latching gates
- Adequate space to be provided for routine maintenance

Although fencing may be a good option, in some configurations, it can inhibit maintenance and diminish the aesthetic appeal of the BMP.

BMPs are designed to collect pollutants that are washed off the roadway. For this reason, swimming and fishing is typically discouraged in BMPs with permanent pools. Consider posting signage warning the public of these dangers.

2.10.3 Maintenance Personnel Safety

The safety of maintenance personnel should also be considered during the design process. Safety benches should be provided where applicable to facilitate mowing and other activities. A minimum of 10 feet shall be provided between fences and BMPs to allow for mowing and maintenance activities.

The designer shall coordinate operation and maintenance access with the appropriate GDOT District.

With the exception of bioslopes, OGFC, grass channels, and filter strips, the designer shall provide a maintenance access driveway to all post-construction BMPs.

Where the driveway is provided from a roadway with a minimum 10 foot wide paved shoulder for a minimum of 200 feet in length past the driveway, the driveway shall meet the following requirements:

1. Minimum width of 12 feet
2. Grade 3:1 or flatter
3. Provide access from the roadway to the BMP berm
4. Shall be either grassed or paved as a commercial driveway

Where the driveway is provided from a roadway without a minimum 10 foot wide paved shoulder for a minimum of 200 feet in length past the driveway, the driveway shall meet the following requirements:

1. Minimum width of 16 feet
2. Maximum grade of 15 percent
3. Driveway shall allow for a 32-foot service vehicle with a 30 foot trailer to exit the roadway and re-enter the roadway without backing into the travel lanes
4. Driveway shall be paved as either an asphalt or concrete commercial driveway .

2.11 Chapter 2 References

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Appendix A. MS4 Areas

Phase I MS4s

Acworth
Alpharetta
Atlanta
Augusta-Richmond County
Austell
Avondale Estates
Berkeley Lake
Bloomingdale
Buford
Chamblee
Chatham County
Clarkston
Clayton County
Cobb County
College Park
Columbus
Dacula
Decatur
DeKalb County
Doraville
Duluth
East Point
Fairburn
Forest Park
Forsyth County
Fulton County
Garden City
Grayson
Gwinnett County
Hapeville
Jonesboro
Kennesaw
Lake City
Lawrenceville
Lilburn
Lithonia
Lovejoy
Macon-Bibb County
Marietta
Morrow
Palmetto
Pine Lake
Pooler
Port Wentworth
Powder Springs
Riverdale
Roswell
Savannah
Smyrna
Snellville
Stone Mountain
Sugar Hill
Suwanee
Thunderbolt
Tybee Island
Union City

Phase II MS4s

Counties:

Athens-Clarke
Barrow
Bartow
Carroll (2017 permit)
Catoosa
Cherokee
Columbia
Coweta
Dawson (2017 permit)
Dougherty
Douglas
Effingham (2017 permit)
Fayette (2017 permit)
Floyd
Glynn
Hall
Henry
Houston
Jackson (2017 permit)
Jones
Lee
Liberty
Long
Lowndes
Madison (2017 permit)
Murray (2017 permit)
Newton
Oconee
Paulding
Peach
Rockdale
Spalding
Walker
Walton
Whitfield

Cities:

Albany (Dougherty Co.)
Allenhurst (Liberty Co.)
Auburn (Barrow Co.)
Bogart (Oconee Co.)
Braselton (Jackson Co.) (2017 permit)
Brunswick (Glynn Co.)
Byron (Peach Co.)
Canton (Cherokee Co.)
Cartersville (Bartow Co.) (2017 permit)
Centerville (Houston Co.)
Chatsworth (Murray Co.) (2017 permit)
Chickamauga (Walker Co.)
Conyers (Rockdale Co.)
Cordele (Crisp Co.)
Covington (Newton Co.)
Cumming (Forsyth Co.)
Dallas (Paulding Co.)
Dalton (Whitfield Co.)
Douglasville (Douglas Co.)
Dunwoody (DeKalb Co.)
Emerson (Bartow Co.)
Eton (Murray Co.) (2017 permit)
Euharlee (Bartow Co.) (2017 permit)
Fayetteville (Fayette Co.)
Flemington (Liberty Co.)
Flowery Branch (Hall Co.)
Fort Oglethorpe (Catoosa Co.)
Gainesville (Hall Co.)
Griffin (Spalding Co.)
Grovetown (Columbia Co.)
Hahira (Lowndes Co.) (2017 permit)
Hampton (Henry Co.)
Hephzibah (Richmond Co.)
Hinesville (Liberty Co.)
Hiram (Paulding Co.)
Holly Springs (Cherokee Co.)
Hoschton (Jackson Co.) (2017 permit)
Johns Creek (Fulton Co.)
Leesburg (Lee Co.)
Locust Grove (Henry Co.) (2017 permit)
Loganville (Walton Co.)
Lookout Mountain (Walker Co.)
McDonough (Henry Co.)
Milton (Fulton Co.)
Mountain Park (Fulton Co.)
Newnan (Coweta Co.)
Oakwood (Hall Co.)
Oxford (Newton Co.)
Peachtree City (Fayette Co.)
Perry (Houston Co.) (2017 permit)
Porterdale (Newton Co.)
Remerton (Lowndes Co.)
Richmond Hill (Bryan Co.) (2017 permit)
Ringgold (Catoosa Co.)
Rome (Floyd Co.)
Rossville (Walker Co.)
Sandy Springs (Fulton Co.)
Senoia (Coweta Co.) (2017 permit)
Stockbridge (Henry Co.)
Temple (Carroll Co.) (2017 permit)
Tunnel Hill (Whitfield Co.)
Tyrone (Fayette Co.)
Valdosta (Lowndes Co.)
Varnell (Whitfield Co.)
Villa Rica (Carroll Co.) (2017 permit)
Walnut Grove (Walton Co.) (2017 permit)
Walthourville (Liberty Co.)
Warner Robins (Houston Co.)
Watkinsville (Oconee Co.)
Winterville (Clarke Co.)
Woodstock (Cherokee Co.)

Appendix B. Geo-Hydrologic Assessment

1 Introduction

1.1 Objective

The objective of this appendix to the GDOT Drainage Design for Highways Manual is to provide a guide to assess geotechnical and groundwater conditions as these factors affect the feasibility of infiltration-type stormwater Best Management Practices (stormwater infiltration BMPs).

Stormwater infiltration BMPs are those BMPs that are designed such that water leaves the BMP solely through infiltration into the underlying soil rather than discharging through an underdrain and outlet control structure. Infiltration testing and groundwater characterization is required to verify the infiltration rate of the underlying soil to ensure the BMP will drawdown in the specified design timeframe. Stormwater infiltration BMPs include infiltration trenches and bioretention basins.

Users of this guidance will be designers, consultants, or other individuals or companies that engage in design of roadways and other facilities for GDOT for which stormwater infiltration BMPs are required.

1.2 Safety

Field work and related soil and groundwater testing will be required at many sites. Attention to applicable Occupational Safety and Health Administration (OSHA) regulations and local guidelines related to earthwork and excavation is required. Digging and excavation should never be conducted without adequate notification through the Georgia One Call system (www.georgia811.com or 1-800-282-7411). Excavations should never be left unsecured or unmarked, and all applicable authorities, including GDOT, should be notified prior to any work.

The Design Team is responsible for ensuring the field evaluations discussed in this manual are conducted in compliance with OSHA 29CFR 1926. Field work must also adhere to local (City, County, etc.) and industry safety guidelines. Where OSHA and local guidelines are in conflict, the more stringent guideline shall apply. The Design Team is also responsible for ensuring traffic control is provided, if necessary, according to GDOT requirements.

1.3 Definitions

The following terminology and definitions are adopted for the purposes of this guidance.

1.3.1 Infiltration Rate

Infiltration rate is the rate at which water penetrates the ground surface and enters soil (distance/time). Infiltration rate is typically determined by the thickness of ponded water that flows downward into the soil over given period of time. Infiltration rate typically decreases with time from the beginning of infiltration, and eventually reaches a steady state as the soil becomes saturated. Infiltration rate is a function of soil layering, initial moisture deficit, soil suction, and the hydraulic conductivity of each layer.

1.3.2 Percolation Rate

Percolation rate is the rate at which water flows through a soil mass (distance/time) at hydraulic gradients on order of 1.0 or less. No distinction is made between the vertical and horizontal components of the total percolation rate, thereby limiting the interpretation of percolation test results.

The steady-state infiltration rate may be similar to percolation rate for a uniform soil mass but can vary significantly when soils near the ground surface differ from underlying soils at depth.

1.3.3 Permeability

Permeability is the term for the rate (distance/time) at which fluid flows through a soil mass when subjected to a given hydraulic gradient. Permeability values may be different in the horizontal, vertical, or an intermediate direction based on soil layering.

1.3.4 Hydraulic Conductivity

Hydraulic conductivity is the specific term for the rate (distance/time) at which liquid water flows through a soil mass when subjected to a given hydraulic gradient. As used in the applications addressed by this document, hydraulic conductivity is the same as permeability.

Hydraulic conductivity is typically reported in terms of its horizontal component (Kh) or vertical component (Kv) in most civil engineering projects, which can vary significantly depending on soil layering.

2 PFPR Desktop Feasibility Screening

2.1 Description

Planning and consideration for stormwater infiltration BMPs should be implemented as early as practical in the project design process, ensuring that such planning is incorporated throughout the project. To this end, two stages of evaluation are recommended to determine site-specific suitability for stormwater infiltration BMPs:

- PFPR Desktop Feasibility Screening - A preliminary screening and planning phase during which the feasibility is assessed in consideration of a global set of site physical conditions and constraints. To streamline the coordination process, it is recommended to contact the Water Resources Group before beginning B-1. Direct coordination with the Water Resources is required if an infiltration trench is determined to be feasible prior to submittal of the draft Post Construction Stormwater Report.
- PFPR Field Study (see Section 3) - If an infiltration trench is proposed, a more rigorous analyses – including site specific testing and data gathering – is used to develop a design for the trench. PFPR Field Study is not needed for capped bioretention basins as construction will perform testing if PFPR Desktop Feasibility Screening indicates that infiltration is potentially suitable.

2.1.1 Objective

The objectives of the feasibility screening phase are twofold, namely:

- To identify the potential impact of site physical conditions and constraints on the potential to implement infiltration BMPs; and
- To determine whether infiltration BMPs should be given further consideration. If infiltration BMPs are found to be unsuitable, do not proceed with PFPR Field Study.
- Outcome and Reporting

  Worksheet B-1 *PFPR Screening for Stormwater Infiltration*, is provided as a resource to the Design Team to help assess the feasibility of stormwater infiltration BMPs. This worksheet is required to be submitted for BMPs that pass concept level infiltration feasibility screening.

2.2 Assessment of Site Suitability

2.2.1 Regional Geographic Factors

Georgia has five distinct physiographic provinces, each of which present different challenges to the investigation and evaluation of subsurface conditions for design of stormwater structures.

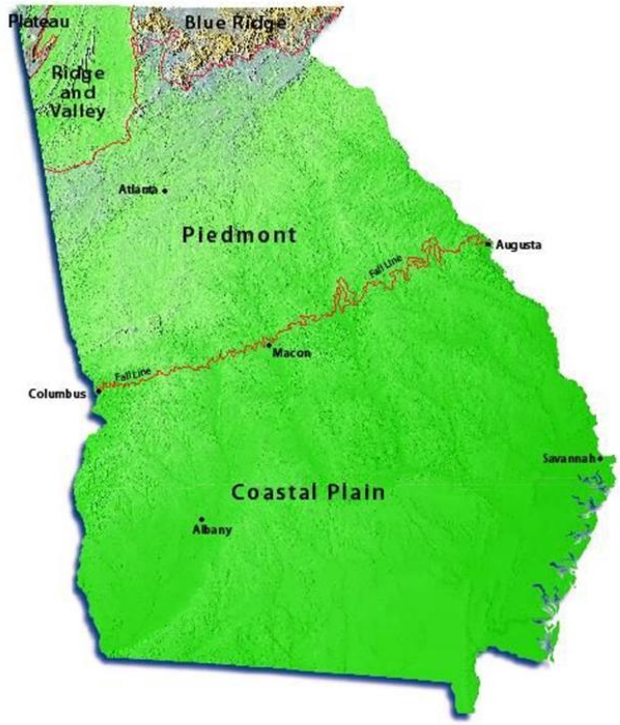

Figure 3-1. Georgia's Physiographic Provinces

- Coastal Plain - The Coastal Plain is characterized by relatively flat, low topographic relief and relatively higher groundwater levels. Development of stormwater infiltration BMPs in this physiographic region may face challenges related to high groundwater level and relatively heterogeneous subsurface conditions.
- Piedmont - Development of stormwater infiltration BMPs in this physiographic region will face challenges posed by the heterogeneous residual soils that lie above the relatively impervious bedrock. The occurrence of sound rock can be difficult to predict.
- Blue Ridge - Development of stormwater infiltration BMPs in this physiographic region may face challenges posed by near surface, relatively impervious bedrock, as well as concerns regarding embankment stability and landslides. Groundwater flow can be complex, occurring in the soil-rock interface and/or in fractures within rock masses.
- Ridge and Valley - Like the Blue Ridge, the region is characterized by high ground surface elevations and steep slopes. By virtue of its geology and topography, the area is well known for historic problems with landslides and ground collapse due to karst-related sinkholes.
- Appalachian Plateau - Development of stormwater infiltration BMPs in this physiographic region will face challenges similar to those posed by the Ridge and Valley. Infiltration may be limited by relatively impervious bedrock. Uncontrolled, infiltration can create hazards to embankment stability, or contribute to landslides. Groundwater flow in the uplands portion of this region can be complex, occurring in the soil-rock interface and in fractures within rock masses.

2.2.2 Site Layout and Geometric Constraints

Planning for stormwater infiltration BMPs must include careful consideration of site-specific constraints. The constraints listed below may make certain structural BMPs infeasible for all or a portion of a roadway project.

1. Site Layout. Available ROW, steep slopes, embankment stability, high water table, proximity to protected waters, construction and/or maintenance access.

2. Geometric Constraints. Prospective BMP locations that are near retaining walls or foundations must ensure that these structures are designed can withstand the purpose-built forces they resist plus any additional load imposed water infiltration (for example, potential increases in lateral pressures and potential reductions in soil strength).

2.2.3 Soils and Hydrogeologic Factors

The PFPR desktop assessment includes a review of publicly available sources to identify potential infiltration issues related to the site physical setting.

- Karst Topography **(Figure 3-2**)
- Acid Producing Rock (**Figure 3-3**)
- Landslide Prone Areas (**Figure 3-4**)
- Potentially expansive soils (**Figure 3-5**)
- Groundwater Recharge area (**Figure 3-6**)

2.2.4 Environmental Factors

PFPR desktop assessment (and future field assessments, if needed) must consider potential environmental impacts related to stormwater infiltration. These effects can include, but are not limited to, the following:

- Areas of contaminated soil or groundwater;
- Hazardous sites;
- Nearby areas of active environmental remediation;
- Groundwater recharge areas;
- Public and private well fields;
- Actively operating underground storage tanks (USTs); and
- Brownfield sites.

2.2.5 Preliminary Site Classifications

2.2.5.1 Unsuitable

A site is considered unsuitable for infiltration BMPs when any of the following conditions are present in the areas where infiltration BMPs are planned:

Table 3-1. BMP Suitability According to Physical Setting

Physical Setting	Unsuitability Conditions
Geologic	Located in an area of Karst Topography, see **Figure 3-2**
	Located in an area of Acid Producing Rock, see **Figure 3-3**
	Located in Landslide Prone Areas see **Figure 3-4**
Soils	Hydrologic Soil groups C or D
	Located in an area of potentially expansive soils, see **Figure 3-5**.
Groundwater	Located in Identified Groundwater Recharge area, see, **Figure 3-6**.
Environmental	Areas with contaminated soil or groundwater.
	Near brownfield sites or active remediation sites.
	Near hazardous sites.
	Near existing underground storage tank (UST) sites.
Structural	Within 50 feet of structure foundation (e.g., bridge, retaining wall, building, etc.).
	Within 20 feet of buried utilities.
Topographic	Preconstruction slopes outside allowable limits in Chapter 10.6 of this manual.
	BMP footprint within 25 feet of the existing crest or toe of a slope steeper than 4:1.
	BMP footprint within a distance 1.5 times the height of the nearest fill slope steeper than 4:1.
	Less than one-foot elevation difference between inflow and outflow locations.
	Constructed within on or near fill sections.

2.2.5.2 Potentially Suitable

This classification indicates the site is potentially well-suited for infiltration BMPs. In general, this classification is designated for sites found to be absent of the concerns discussed in previous site suitability classifications, but additional field study is needed before a full infiltration BMP (infiltration trench) can be recommended.

2.2.6 USGS and GA EPD Geologic Maps

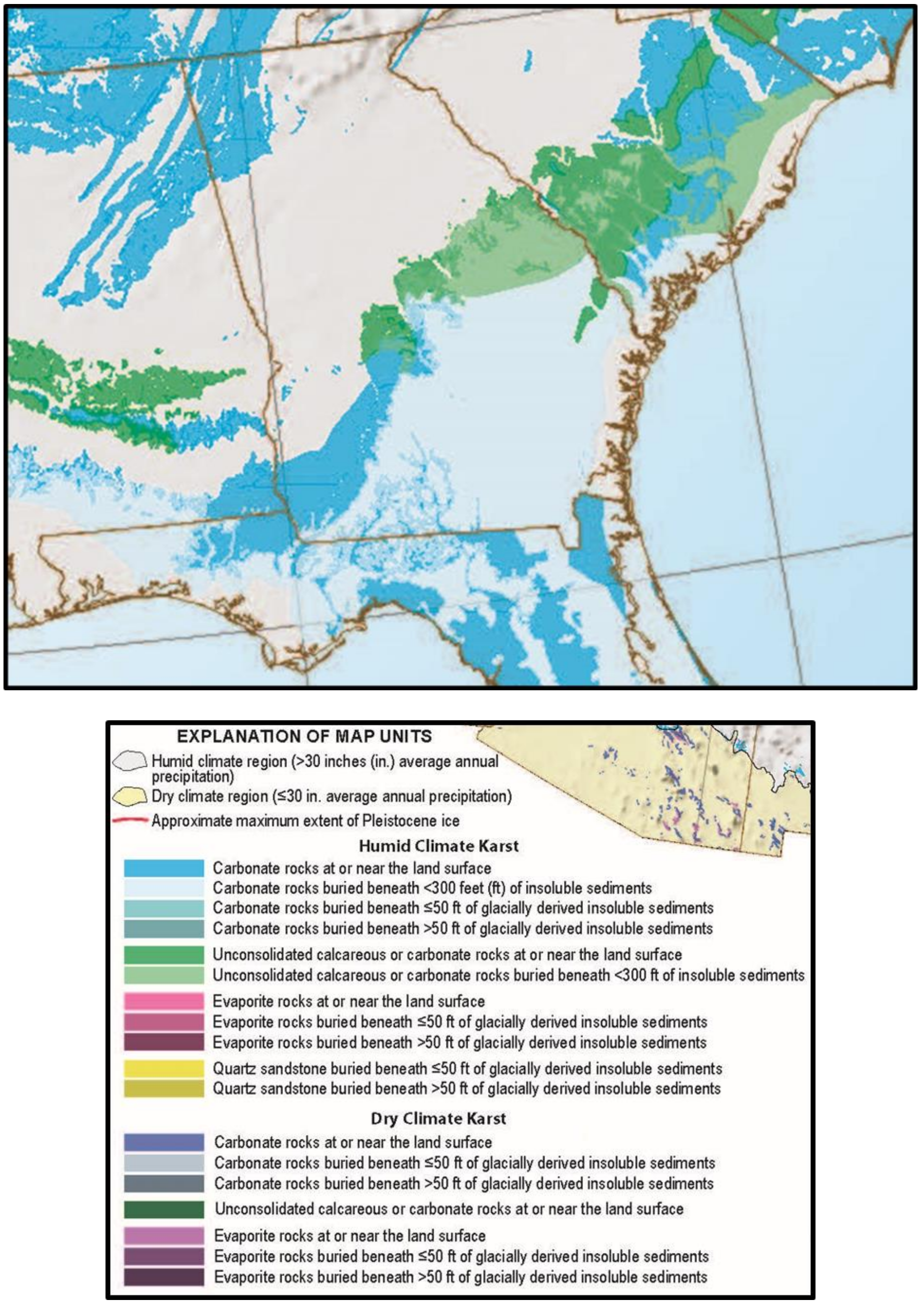

Figure 3-2. Approximate Distribution of Karst and Potential Karst in the Southeastern U.S.
(source: USGS 2014)

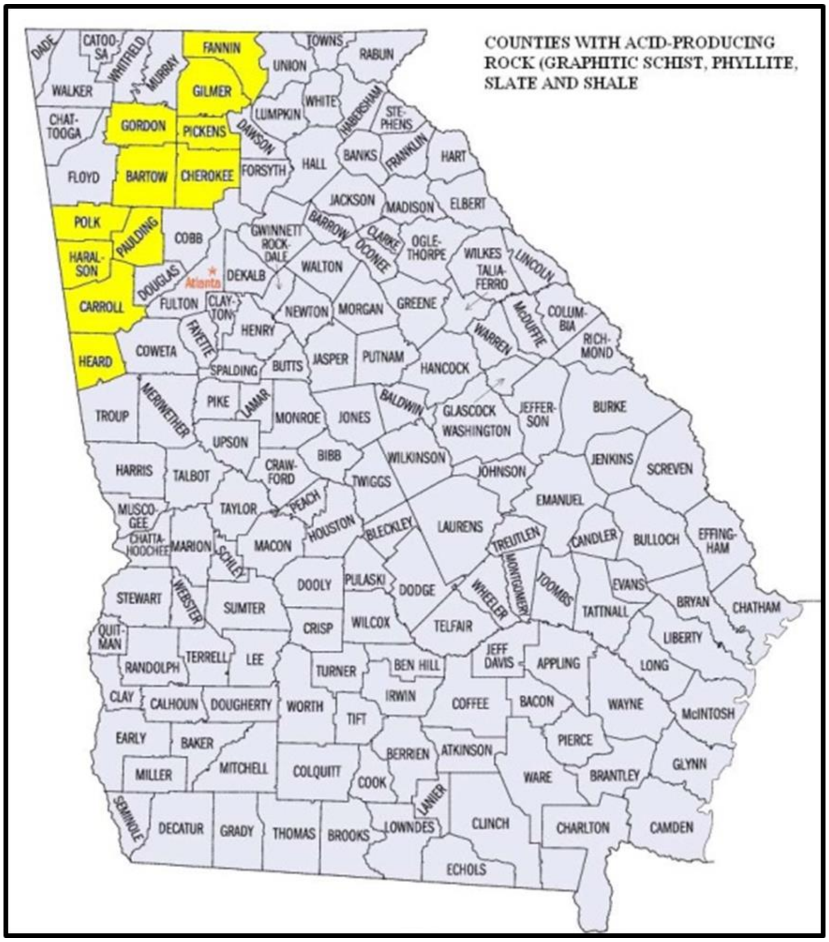

Figure 3-3. Distribution of Acid-Producing Rock in Georgia
(source: GDOT 2016).

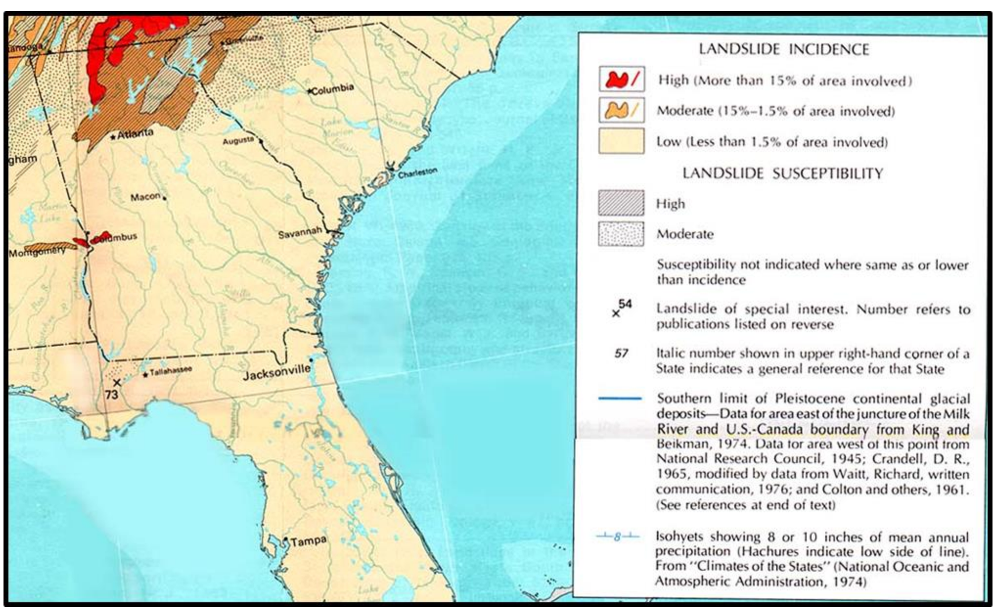

Figure 3-4. Overview of Landslide Risk in Georgia
(source: USDOI 1982)

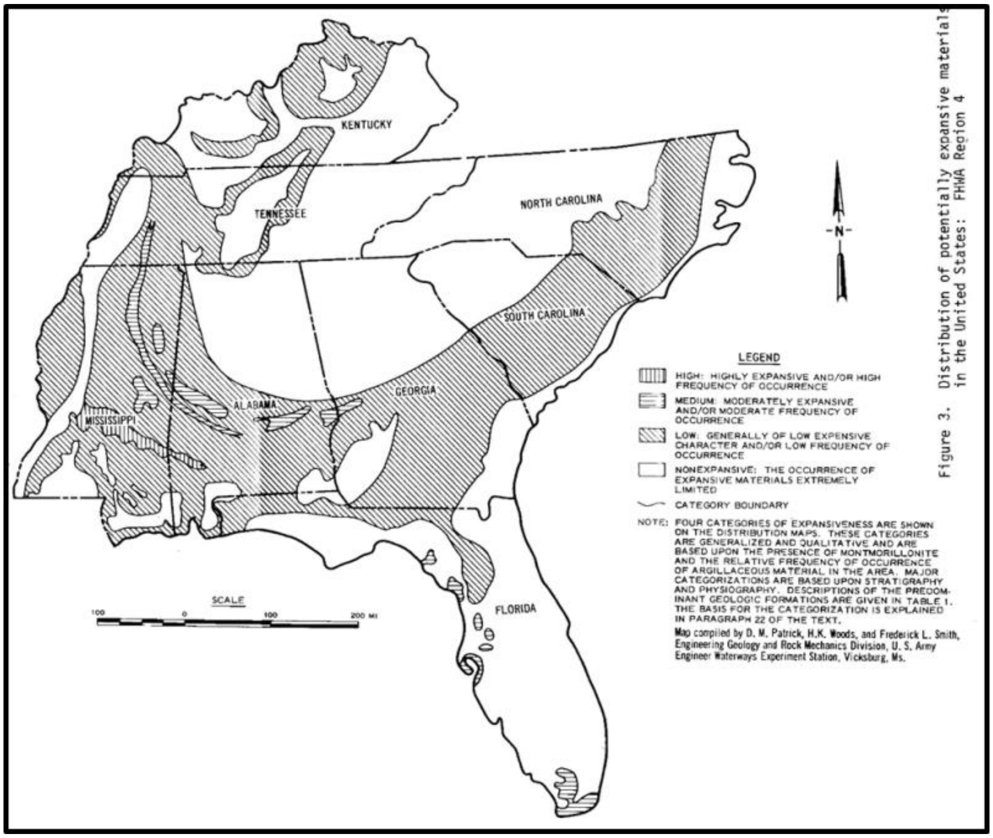

Figure 3-5. Approximate Distribution of Expansive Soils in the Southeastern U.S.
(source: FHWA 1975)

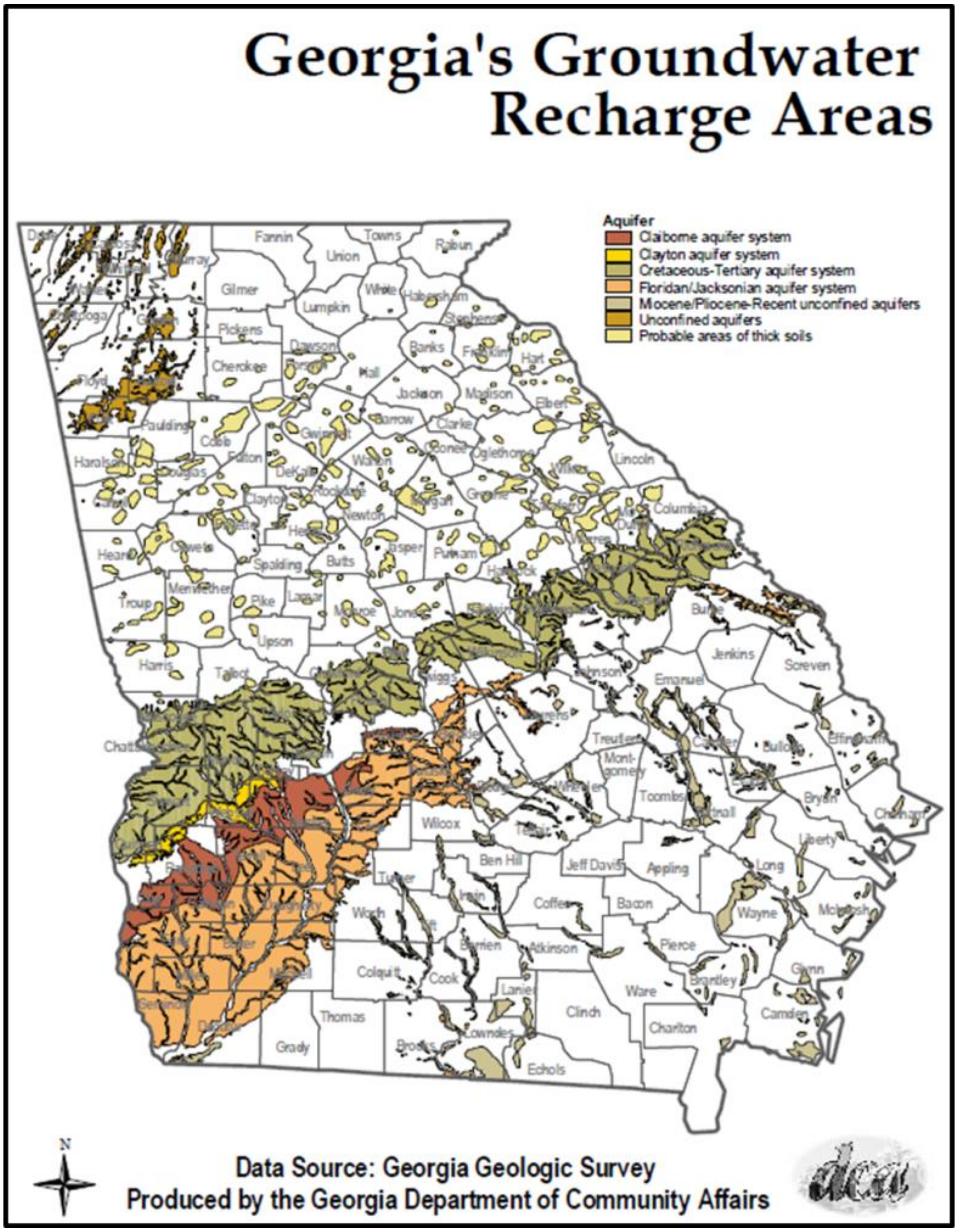

Figure 3-6. Georgia's Groundwater Recharge Areas
(source: Georgia Geologic Survey)

http://www.georgiaplanning.com/documents/atlas/gwrecharge.pdf

3 PFPR Field Study

3.1 Description

3.1.1 Objective

PFPR field study applies only to BMP locations where infiltration is potentially feasible as determined from the Desktop Feasibility Screening. The objective of the PFPR Field Study obtain information that is adequately site-specific and reliable to support design (e.g., depth to groundwater, design infiltration rate; analysis of geotechnical risks and mitigation approaches). The PFPR Field Study is only required for sites with proposed infiltration trenches. Other infiltration BMPs have backup drainage systems, and thus, do not require field study.

3.1.2 Characteristics

This phase is generally performed as part of Preliminary Design in GDOT's PDP framework. The PFPR Field Study consists of two elements, often performed contemporaneously:

- A site-specific geotechnical exploration, hereafter termed "Field Exploration" and
- In-situ Testing.

If infiltration is determined to be feasible for infiltration trenches in PFPR Desktop Screening, In-Situ Testing will be conducted at the same time as PFPR Field Exploration. It is the responsibility of the Design Team to acquire the services of a geo-professional to perform the PFPR Field Study. These components are described in further detail in the following sections.

3.1.3 Outcome and Reporting

The outcome of this phase should be a more rigorous assessment of feasibility, with selection and layout of infiltration trenches, as supported by location specific testing, integrated with project design.

Worksheet B-2 provides guidance for preparation of this PFPR Infiltration Trench Suitability Field Study Report to determine if an infiltration trench is feasible. The report will be submitted to ODPS for review. At a minimum, this report should include the scope of documentation described below.

1. Part 1. Introduction and Summary. Describe the objective and scope of the PFPR Field Study. The report should address requirements for stormwater infiltration as understood at this level of design. The findings of the PFPR Field Study should be summarized .
2. Part 2. Site-Specific Evaluation. The findings of the site-specific assessments of subsurface conditions and the infiltration/percolation rates and capacities should address the site-specific considerations listed below.
    i. Geology of the site area, with a focus on its potential influence on the project requirements for infiltration.
    ii. Surface and subsurface soil and geologic conditions as they may affect infiltration and migration of water.
    iii. The depth to groundwater, groundwater quality, and likely variations in the high seasonal groundwater elevations.
    iv. Results of subsurface exploration and laboratory testing should be tabulated in the body of the report. Records of the testing, including raw data, should be appended.

v. Results of infiltration/percolation testing should be tabulated in the body of the report. Records of the testing, including raw data, should be appended.

vi. To the extent the work considers stormwater at various BMP locations, provide discussion regarding infiltration rates or capacities in each sub-basin.

vii. Provide a concluding opinion regarding whether or not the proposed onsite stormwater infiltration trench can be implemented without damage to GDOT or adjacent properties.

viii. Provide a judgment regarding site suitability for infiltration trenches.

The PFPR Infiltration Trench Suitability Field Study Report should be supplemented, as appropriate by plans, graphics, photographs, etc. that will enable users of the report to clearly understand the text. The PFPR Infiltration Trench Suitability Field Study Report shall be submitted to ODPS for review. Include the approved PFPR Infiltration Trench Suitability Field Study Report as an appendix in the MS4 Post-Construction Stormwater Report for infiltration trenches which are considered potentially suitable by the PFPR Desktop Feasibility Screening.

3.2 Field Exploration for Infiltration Trenches

The Field Exploration should develop site-specific stratigraphy and soil properties in the areas of prospective infiltration trenches. The Field Exploration should include soil borings and/or test pits ("exploration points") extended to at least 10 feet beyond the expected depths of the infiltration trenches.

3.2.1 Subsurface Investigation Methods

Either test pits and/or soil borings must be undertaken for characterization of the subsurface at the location of prospective infiltration trenches. These tools each allow visual observation of the soil horizons and overall soil conditions at an infiltration location. A sufficient number of carefully logged and sampled borings or test pits should be conducted such that the soil conditions are understood both horizontally and vertically in the portion of the site under consideration for infiltration trenches. Laboratory testing of representative samples may be used to supplement descriptions of the subsurface.

In general, the use of test pits is much preferred over soil borings as a field investigation tool. Test pits allow clear visual observation of the subsurface, while such clarity is narrowly limited in a soil boring. The soil boring does not allow observation of the soil horizons in situ, requiring qualitative judgment (e.g., the indications of the drilling rate, soil return from augers, etc.) to assess the subsurface. Certain circumstances (for example, the depth to the base of the planned BMP) early in the design process may drive the use of the soil borings.

Laboratory testing of representative samples may be used to supplement descriptions of the subsurface. However, such testing should be limited only to that necessary to support classification of the subsurface. The use of laboratory testing to establish infiltration rates of the BMP location is not acceptable.

Table 4-2 summarizes the utility of test pits, soil borings and laboratory testing as tools for determining subsurface conditions and infiltration rates at BMP locations.

Table 4-2. Utility of Test Pits, Soil Borings and Laboratory Testing for Assessment of the Subsurface.

Tool	General Applicability	Capabilities	Limitations
Test Pits	Bulk sampling, *in situ* testing, visual inspection.	Fast, economical, able to access more difficult sites. Typically extendable to ±8 feet depth.	Bench for unbraced personnel access if D > 4 feet, stability affected by groundwater. Limited undisturbed sampling.
Soil Borings	General determination of the soil profile to depths in excess of >10 feet with *in situ* testing and undisturbed sampling	Allows *in situ* testing, undisturbed and disturbed soil sampling	Limited access. Casing obscures visual inspection. Penetration can be limited by hard soils, cobbles or boulders.
Laboratory Testing	Quantitative supplement to the logging of test pits and borings.	ASTM classification of soil strata.	May be used as a supplement only. Laboratory testing to establish infiltration rates is not acceptable.

3.2.1.1 Test Pits

Where applicable, test pits are the preferred survey method due to improved visual representation of subsurface soil types, layering, and groundwater. A test pit excavation allows visual observation of the soil horizons and overall soil conditions both horizontally and vertically in that portion of the site.

It is important that the test pit provide information related to conditions at the bottom of the proposed infiltration trench. The designer is cautioned regarding the proposal of infiltration trenches that are significantly lower than the existing topography. The suitability for infiltration may decrease, and risk factors are likely to increase.

The designer and contractors should minimize grading and earthwork to the extent practical to reduce site disturbance and compaction so that a greater opportunity exists for testing and stormwater management in subsequent phases.

3.2.1.2 Soil Borings

Soil borings provide limited sampling of the subsurface relative to test pits and are generally discouraged as a primary investigation options for infiltration purposes. Additionally, production rates for soil borings are typically less than that for test pits. However, in cases where test pits cannot be performed due to site constraints or cannot be sampled to the required depth of investigation, soil borings are acceptable as a primary exploration method. For example, soil borings should be used where proposed finished grade is significantly below existing grade and test pits are unable to sample the soil zone of interest.

3.2.1.3 Allowance for Alternative Testing Procedures

Some laboratory testing methods can be used to assess a soil's suitability for infiltration for early screening. In certain instances, laboratory testing may be used for verification.

For instance, if the infiltration trenches are not located precisely over the test locations, alternate testing or investigations can be used to verify that the soils are the same as the soils that yielded the earlier test results. However, designers should document these verification test results or investigations.

Decisions to utilize laboratory testing should be made by the geo-professional.

3.2.1.4 Index Testing

Laboratory index testing on select samples collected during the field investigation may be performed to confirm field classifications and to aid in the characterization of subsurface stratigraphy. Laboratory index testing should be performed on each different soil type identified in the field logging. Determination of the frequency of testing is the responsibility of the Design Team and may vary significantly depending on geologic formation and expected variability. Index tests should include the following:

- Moisture Content (ASTM D2216);
- Atterberg Limits (ASTM D4318);
- Particle-size distribution (ASTM D422); and,
- Soil classification after ASTM D2488.

3.2.1.5 Density Testing

Undisturbed sampling of soil (for example, thinwall tube sampling after ASTM D1587) may be undertaken in certain instances.

3.2.1.6 Laboratory Hydraulic Conductivity

Laboratory hydraulic conductivity testing on relatively undisturbed thin-wall ('Shelby tube') samples can be performed as part of the screening analysis. Such tests may be preferred in instances where limited site access or other factors that exist limit the feasibility of field infiltration tests.

Laboratory testing methods for preliminary design purposes may include ASTM D2434 or ASTM D5084. However, it is recommended that in-situ field tests be performed whenever practical.

3.3 In-Situ Testing for Infiltration Trenches

3.3.1 Preferred Field Test Methods

In-situ infiltration/percolation testing will provide quantitative data regarding in-situ hydraulic conductivity of soils. These data can be used to confirm and/or calibrate estimates developed from published correlations with grain-size, plasticity, and/or geologic formation provided in the Field Exploration. Selection of a method of in-situ testing is within the Designer's and Geo-professional's discretion. Among other factors, the choice of in-situ testing method will depend on:

- The confidence level that site soils are suitable for infiltration trenches; and
- Certainty of the proposed infiltration trench footprint and depth.

Field testing methodologies preferred by GDOT include:

- Double-Ring Infiltrometer tests (ASTM D3385 or 5093);
- Single-Ring Infiltrometer (modified from ASTM D5126);
- Borehole Infiltration Test (ASTM D6391); and,
- Percolation tests (such as for on-site wastewater systems).

See Table 6-1 for a summary of the methods.

3.3.2 Sampling and Testing Frequency

The PFPR Field Study should include at least one (1) exploration point per proposed infiltration trench. For larger infiltration areas (i.e. more than 10,000 square-feet [SF] in plan or more than 150 linear-feet [LF] in length), multiple exploration points should be evenly distributed within the BMP area at the rate of one (1) additional test per 10,000 SF of infiltration trench area or every 100 LF of infiltration trench length, whichever is more frequent. Exploration points should be located within the footprint of proposed infiltration trench if practicable, but no greater than 50 feet beyond if preconstruction site constraints are present. Table 4-1 summarizes the recommended minimum testing frequencies.

Table 4-1. Recommended Minimum Testing Frequency.

Primary Method for Estimating Infiltration Rate	Minimum Number of Tests / Data Points per Infiltration Trench	Minimum Number of Borings / Test Pits per Infiltration Trench
Single-Ring Infiltrometer (where applicable)	2	1
Double-Ring Infiltrometer	2	1
Borehole Infiltration Test	2	1
Percolation Test	4	1
Grain-size Correlations (Site-Specific Lab Data)	4	1

4 References

4.1 Test Standards

ASTM D 3385 – 03. Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer. West Conshohocken, PA. 2003.

ASTM D 5126 – 16. Standard Guide for Comparison of Field Methods for Determining Hydraulic Conductivity in Vadose Zone. West Conshohocken, PA. 2016.

ASTM D 5093 – 02. Standard Test Method for Field Measurement of Infiltration Rate Using a Double-Ring Infiltrometer with a Sealed-Inner Ring. West Conshohocken, PA. 2002.

ASTM D 6391 – 06. Standard Test Method for Field Measurement of Hydraulic Conductivity Limits of Porous Materials Using Two Stages of Infiltration from a Borehole. West Conshohocken, PA. 2006.

U.S. Department of the Interior. A Field Method for Measurement of Infiltration. Geological Survey Water-Supply Paper 1544-F. 1991.

U.S. Department of the Interior, Bureau of Reclamation, "Procedure for Performing Field Permeability Testing by the Well Permeameter Method (USBR 7300-89)," in Earth Manual, Part 2, Materials Engineering Branch Research and Laboratory Services Division, 3rd edition, Denver, Colorado, 1990.

4.2 State and Federal Manuals

Georgia Department of Transportation (“GDOT 2016”). Geotechnical Manual. Updated June 24, 2016.

Federal Highway Administration (“FHWA 1975”). A Review of Engineering Experiences with Expansive Soils in Highway Subgrades. BY D.R. Snethen and others. June 1975.

4.3 Miscellaneous Technical References

Chapuis RP. 2004. Predicting the Saturated Hydraulic Conductivity of Sand and Gravel Using Effective Diameter and Void Ratio. Can Geotech J 41: 787-795.

Georgia Geologic Survey. Georgia’s Groundwater Recharge Areas. http://www.georgiaplanning.com/documents/atlas/gwrecharge.pdf

Terzaghi, K. and Peck, R.B., (1967), “Soil Mechanics in Engineering Practice”, John Wiley and Sons, 729p.

US Department of the Interior (“USDOI 1982”). Landslide Overview Map of the Conterminous United States. Geological Survey Professional Paper 1183. 1982.

USGS (“USGS 2014”). Karst in the United States: A Digital Map Compilation and Database. By David J. Weary and Daniel H. Doctor. Open-File Report 2014-1156. 2014.

Attachment A: Worksheets

PFPR Screening for Stormwater Infiltration						Worksheet B-1
Outfall Basin Name:						
Category	Parameter	Yes	No	Not Sure	Data Source / Reference	Comments / Justification
Geologic	Located in area of Karst Topography				Figure 3-2	
	Located in area of Acid Producing rock				Figure 3-3	
	Located in Landslide prone area				Figure 3-4	
Soils	Hydrologic Soil groups C or D				NRCS Soil Survey	
	Located in an area of potentially expansive soils?				Figure 3-5	
Groundwater	Located in Identified Groundwater Recharge area				Figure 3-6	
Environmental	Areas of contaminated soil or groundwater					
	Near a hazardous site?				GA EPD Hazardous Site Inventory	
	Near brownfield sites or active remediation sites				GA EPD Brown Fields	
	Near an existing underground storage tank (UST) site				GA EPD USTs	
Structural	Within 50 feet of structure foundation (e.g., bridge, retaining wall, building, etc.)?					
	Within 20 feet of buried utilities					
Topographic	Preconstruction slopes outside allowable limits in Chapter 10.6 of this manual					
	BMP footprint within 25 feet of the existing crest or toe of a slope steeper than 4H:1V					
	BMP footprint within a distance 1.5 times the height of the nearest fill slope steeper than 4:1					
	Less than one-foot elevation difference between inflow and outflow locations					
	Constructed within on or near fill sections					

Desktop Site Classification	Mark one (X)
Unsuitable	
Potentially Suitable	

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PFPR Infiltration Trench Suitability Field Report	Worksheet B-2
Section	**Content**
1	**Introduction**

A. Project Description. Provide a description of the subject project, with reference to the potential need for infiltration trenches. Establish the design phase addressed by the report.

B. Objective of This Study. Provide a succinct statement of the objective of the work reported.

C. Abstract of Current Phase Assessment. Provide a summary of the PFPR Field Study findings and recommendations.

Field Study Site Classification	Mark one
Unsuitable	
Suitable	

2	**Site Description**

A. Regional Geology. Provide a description of geologic setting of the site, with focus on influence of the near surface geology on the project requirements for infiltration. This review may rely on the findings of previous studies. Graphics should be used to support discussion.

B. Site Conditions.

a. Surface Conditions. Utilizing available survey and preliminary project documentation, provide description of the site. A description of the site surface topography should be provided in detail, providing maps to support this discussion. Utilize graphics/maps/ photos, as appropriate, to discuss other relevant descriptions of the site.

b. Subsurface. Provide a description of the near surface soil and rock units, taking care to distinguish between naturally occurring deposits and areas of artificial fill. If fill is planned for the site and may affect stormwater infiltration trenches, such fill should be noted. Support descriptions of the subsurface by the indications of soil borings, test pits, etc. If relevant, utilize the indications of laboratory testing to support soil descriptions.

c. Groundwater. Describe groundwater elevation across the site, addressing any apparent groundwater gradient. Address historical high groundwater levels.

d. Surface Water. Describe surface water to the degree it may affect the site or has historically affected the site. Documentation from flood mapping should be cited.

Worksheet B-2 Page 2 of 3

Section	Content
3	**Subsurface Exploration Or Laboratory Testing**

A. Subsurface Exploration. Provide a description of the scope of the field subsurface exploration. Summarize the types of testing conducted, with references to appendices that provide details (boring logs, logs of test pits, etc.). This discussion must be supported by at least one figure that shows the location of all field exploration points. Field exploration points must be described in terms of GPS locations and elevation.

B. Laboratory Testing. Provide a description of the scope of laboratory testing. Summarize the types of testing conducted, including ASTM references. Tabulate the findings of laboratory testing in summary form in the body of the report. Details regarding laboratory testing should be appended.

Section	Content
4	**Infiltration / Percolation Testing**

A. Summary of Testing. Provide a description of the scope of infiltration and/or percolation testing undertaken for this study.

Utilize tables and graphics to depict the locations of the various types of testing conducted. Discussion should also be provided regarding the reasons for selection of testing methodologies.

Discussion regarding the testing should reference appendices that provide details of all work, including test methodologies, etc. This discussion must be supported by at least one figure that shows the location of all field exploration points. Field exploration points must be described in terms of GPS locations and elevation.

B. Discussion of Results. Provide discussion regarding the indications of the testing. Utilize tables for presentation of specific recommended design parameters for specific stormwater infiltration trenches.

As appropriate, distinguish recommended design values for different subsurface soil units.

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Section	**Content**
5	**Discussion and Recommendations**
A. Discussion. Utilizing the information developed from this assessment review in summary the data developed in Sections 1-4. B. Recommendations. Provide recommendations from a geologic and geotechnical perspective for implementation of infiltration trenches as addressed by the subject report. These recommendations should address, at a minimum, the site considerations listed below. 1. Design Basis Infiltration Rates. Provide design basis infiltration rates for specific soil units for specific infiltration trenches. If the infiltration rate is less than 0.5/hr, infiltration trenches are unsuitable.	
6	**References**
Provide a listing of references used in preparation of the report.	
Appendices	**Project Documentation**
Attach records of borings, test pits, laboratory testing, field testing (if applicable), etc. as separate appendices to Worksheet B-2.	