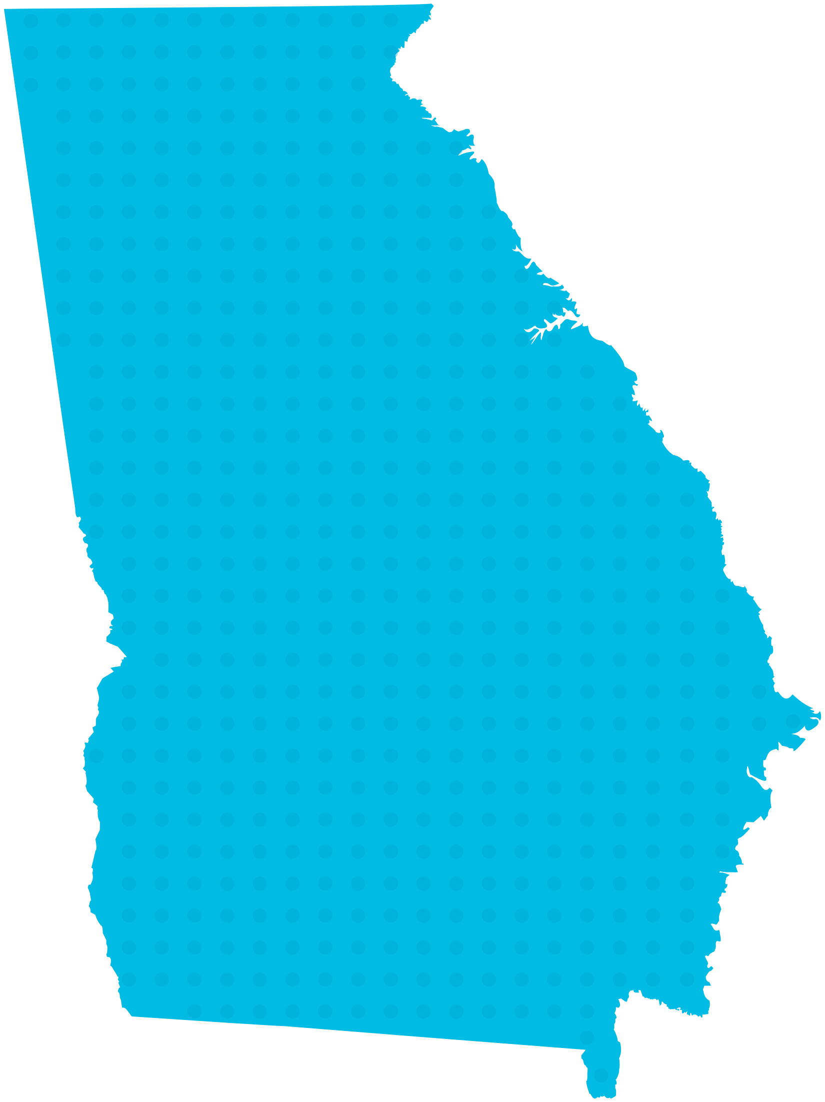


# GEORGIA STORMWATER MANAGEMENT MANUAL

2016 EDITION

**VOLUME 2:** TECHNICAL HANDBOOK

BACK TO VOL 1

PRINT ENTIRE VOL 2

# Volume 2: Technical Handbook

Click on any of the items in the Table of Contents to go directly to that section.

VOL2
TOC

# 1. Introduction

## 1.1 Objective of the Manual

The objective of the Georgia Stormwater Management Manual is to provide guidance on the best post-construction stormwater management practices available to Georgia communities to minimize the negative impacts of increasing stormwater runoff and its associated pollutants. The goal is to provide an effective tool for local governments and the development community to reduce both stormwater quality and quantity impacts and protect downstream areas and receiving waters.

This Manual does not cover construction site sediment and erosion control practices. Guidance on these practices can be found in the latest edition of the *Manual for Erosion and Sediment Control in Georgia.*

## 1.2 Organization of the Manual

The Georgia Stormwater Management Manual is organized as a three volume set, with each volume published as a separate document. You are currently reading Volume 2 of the Manual.

Volume 1 of the Manual, the *Local Government Guide to Stormwater Management*, is designed to provide guidance for local jurisdictions on the basic principles of effective urban stormwater management. Volume 1 covers the environmental, economic and social problems resulting from urban stormwater runoff and the need for local communities to address urban stormwater quantity and quality through recommended stormwater management standards and local stormwater programs. It also provides an overview of integrated stormwater management and technologies and tools for implementing stormwater management programs.

Volume 2 of the Manual, the *Technical Handbook*, provides guidance on the techniques and measures that can be implemented to meet a set of recommended stormwater management standards for new development and redevelopment. Volume 2 is designed to provide the site designer or engineer, as well as the local plan reviewer or inspector, with all of the information on best management practices (BMPs) required to effectively address and control both water quality and quantity on a development site. This includes guidance on site planning, better site design practices, hydrologic techniques, criteria for the selection and design of stormwater BMPs, and drainage system design, as well as construction and maintenance information.

Volume 3, the *Pollution Prevention Guidebook*, is a compendium of pollution prevention practices for stormwater quality for use by local jurisdictions, businesses and industry, and local citizens.

## 1.3 Target Audience

The users of Volume 2 will be site planners, engineers, contractors, plan reviewers, and inspectors from local government and the development community.

Local jurisdictions may adopt and apply the recommended standards for new development and redevelopment in this Manual directly as part of their local development code. Further, local jurisdictions may use Volume 2 to review stormwater site plans and provide technical advice, and may adopt any part of the guidance and design criteria for best management practices and drainage design contained in this Manual as their local engineering design requirements. Check with the local review authority for more information.

Those parties involved with site development will utilize Volume 2 for technical guidance and information on the preparation of stormwater site plans, the use of better site design techniques, hydrologic techniques, selection and design of appropriate best management practices, and drainage (hydraulic) design.

## 1.4 How to Use This Volume

The following provides a guide to the various chapters of Volume 2 of the Manual.

- **Chapter 1 (Introduction).** This chapter discusses the overall purpose and organization of the Manual, the intended use and other background information.

- **Chapter 2 (Stormwater Management Planning, Design, & Implementation).** This chapter provides the framework for addressing stormwater runoff on new development and redevelopment sites. This chapter includes the following sections:

» Section 2.1 – *The Need for Stormwater Management.* This section provides an overview of the impacts of stormwater runoff.

» Section 2.2 – *Stormwater Management Standards & Numerical Sizing Criteria.* This section contains the stormwater management recommended standards for new development and redevelopment sites, explains the four sizing criteria for water quality, channel protection, overbank flood protection, and extreme flood protection, and describes the approaches for meeting the criteria through the use of better site design practices and best management practices (BMPs).

» Section 2.3 – *Stormwater Better Site Design & Techniques.* This section covers the toolkit of better site design practices and techniques that can be used to reduce the amount of stormwater runoff and pollutants generated from a site.

» Section 2.4 – *Stormwater Site Planning & Design.* This section outlines the typical contents and procedures for preparing a stormwater site plan.

- **Chapter 3 (Stormwater Hydrology).** This chapter presents engineering topics and methods used in stormwater drainage, conveyance and facility design.

» Section 3.1 – *Methods for Estimating Stormwater Runoff.* This section provides an overview of the different hydrologic methods and their application.

» Section 3.2 – *Methods for Estimating Stormwater Volume Reduction – Design Worksheet.* This section provides an overview of the spreadsheet developed to calculate runoff reduction and TSS removal.

» Section 3.3 – *Storage Design.* This section covers the criteria and general procedures for the design and evaluation of stormwater storage (detention and retention) facilities.

» Section 3.4 – *Outlet Structures.* This section outlines various stormwater facility outlet types and provides criteria and procedures for water quality outlet design.

- **Chapter 4 (Stormwater Best Management Practices).** This chapter contains the information and guidance for the selection and design of best management practices (BMPs) for managing stormwater quantity and quality. It is divided into the following sections:

» Section 4.1 – *Stormwater BMP Overview.* This section provides an overview of the best management practices that can be used to reduce and/or treat stormwater runoff and/or mitigate the effects of increased runoff peak rates, volumes, and velocities.

» Section 4.2 through 4.29 – *Individual BMP Sections.* These sections contain detailed information and design criteria for each best management practice and their applicability for sites with a demonstrated ability to meet stormwater management goals.

- **Chapter 5 (Stormwater Drainage System Design).** This chapter provides technical guidance on the various elements of stormwater drainage design. This chapter includes the following sections:

» Section 5.1 – *Stormwater Drainage Design Overview.*

» Section 5.2 – *Minor Drainage System Design.* This section provides guidelines and design criteria for gutter and inlet hydraulics, and provides an overview of storm drain pipe system design.

» Section 5.3 – *Culvert Design.* This section covers criteria and procedures for the design and evaluation of culverts.

» Section 5.4 – *Open Channel Design.* This section describes the criteria and calculations for the design of open stormwater drainage channels.

» Section 5.5 – *Energy Dissipation Design.* This section includes information and design criteria for a number of energy dissipators, including riprap aprons, riprap basins and baffled outlets.

- **Appendix A** – *Rainfall Tables for Georgia.* This appendix contains a weblink to the National Oceanic and Atmospheric Administration's website where users can access the most recent rainfall data available.

- **Appendix B** – *Best Management Practice Design Examples.* This appendix includes design examples for different best management practices: stormwater pond, bioretention area, surface sand filter, infiltration trench, vegetated filter strip, and enhanced (dry) swale.

- **Appendix C** – *Nomographs & Design Aids.* This appendix includes various nomographs and other design tools primarily referenced throughout Chapter 5.

- **Appendix D** – *Planting & Soil Guidance.* This appendix provides landscaping criteria and plant selection guidance for stormwater management facilities. A site specific landscaping plan for each BMP is recommended.
- **Appendix E** – *Best Management Practice Operations & Maintenance.* The appendix provides general information and checklists for the inspection and maintenance of post-construction stormwater best management practices. A site specific checklist regarding the inspection and maintenance needs for each BMP is recommended.

## 1.5 Regulatory Status of the Manual

The Georgia Stormwater Management Manual is designed to provide Georgia communities with comprehensive guidance on a Low Impact Development (LID)-based approach to natural resource protection, stormwater management and site design that they can use to better protect the state's valuable natural resources from the negative impacts of land development and nonpoint source pollution. Although communities may choose to use the information presented in this manual to regulate new development and redevelopment activities, the document itself has no independent regulatory authority. The approach to natural resource protection, stormwater management and site design detailed in the Georgia Stormwater Management Manual can only become required through:

(1) Codes and ordinances established by local governments

(2) Rules, regulations, and permits established by local, state and federal agencies

It is recommended that all Georgia communities use the information presented in this manual, or an equivalent post-construction stormwater management manual, to regulate new development and redevelopment activities. For those communities that are covered under the National Pollutant Discharge Elimination System (NPDES) Municipal Stormwater Program, adoption of portions of this manual (or an equivalent) are required.

Communities are encouraged to review and modify the contents of this manual, as necessary, to meet local watershed and stormwater management goals and objectives while still maintaining the essence of a Low Impact Development (LID)-based approach for stormwater management.

## 1.6 How to Find the Manual on the Internet

All three volumes of the Georgia Stormwater Management Manual are available in Adobe Acrobat PDF document format for download at the following Internet address:

http://www.georgiastormwater.com

## 1.7 Contact Information

If you have any technical questions or comments on the Manual, please send an email to:

info@georgiastormwater.com

# 2. Stormwater Management Planning, Design, & Implementation

## 2.1 The Need for Stormwater Management

### 2.1.1 Impacts of Development and Stormwater Runoff

Land development changes not only the physical, but also the chemical and biological conditions of Georgia's waterways and water resources. This chapter describes the changes that occur due to development and the resulting stormwater runoff impacts.

#### 2.1.1.1 DEVELOPMENT CHANGES LAND AND RUNOFF

When land is developed, the hydrology, or the natural cycle of water is disrupted and altered. Clearing removes the vegetation that intercepts, slows and returns rainfall to the air through evaporation and transpiration. Grading flattens hilly terrain and fills in natural depressions that slow and provide temporary storage for rainfall. The topsoil and sponge-like layers of humus are scraped and removed and the remaining subsoil is compacted. Rainfall that once seeped into the ground now runs off the surface. The addition of buildings, roadways, parking lots and other surfaces that are impervious to rainfall further reduces infiltration and increases runoff.

Depending on the magnitude of changes to the land surface, the total runoff volume can increase dramatically. These changes not only increase the total volume of runoff, but also accelerate the rate at which runoff flows across the land. This effect is further exacerbated by drainage systems such as gutters, storm sewers and lined channels that are designed to quickly carry runoff to rivers and streams.

**Figure 2.1.1-1** Clearing and Grading Alter the Hydrology of the Land

Development and impervious surfaces also reduce the amount of water that infiltrates into the soil and groundwater, thus reducing the amount of water that can recharge aquifers and feed streamflow during periods of dry weather.

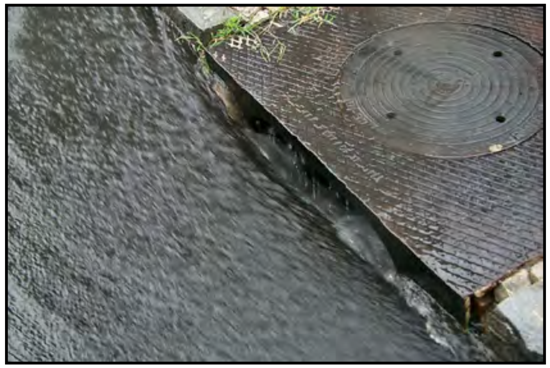


**Figure 2.1.1-2** Impervious Cover Increases Runoff Peak Flows and Volumes While Reducing Recharge

Finally, development and urbanization affect not only the quantity of stormwater runoff, but also its quality. Development increases both the concentration and types of pollutants carried by runoff. As it runs over rooftops and lawns, parking lots and industrial sites, stormwater picks up and transports a variety of contaminants and pollutants to downstream waterbodies. The loss of the original topsoil and vegetation removes a valuable filtering mechanism for stormwater runoff.

The cumulative impact of development and urban activities, and the resultant changes to both stormwater quantity and quality in the entire land area that drains to a stream, river, lake or estuary determines the conditions of the waterbody. This land area that drains to the waterbody is known as its watershed. Urban development within a watershed has a number of direct impacts on downstream waters and waterways. These impacts include:

- Changes to stream flow
- Changes to stream geometry
- Degradation of aquatic habitat
- Water quality impacts

The remainder of this section discusses these impacts and why effective stormwater management is needed to address and mitigate them.

BACK TO TOC

### 2.1.2 Addressing Stormwater Impacts

The focus of this Manual is how to effectively deal with the impacts of urban stormwater runoff through effective and comprehensive *stormwater management*. Stormwater management involves the reduction, prevention, and mitigation of stormwater runoff quantity and quality impacts as described in this chapter through a variety of methods and mechanisms.

Volume 2 of this Manual deals with ways that developers in Georgia can effectively implement stormwater management to address the impacts of new development and redevelopment, and reduce, prevent, and mitigate problems associated with stormwater runoff. This is accomplished by:

- ❑ Developing land in a way that minimizes its impact on a watershed, and reduces both the amount of runoff and pollutants generated through infiltration, evapotranspiration, and reuse.
- ❑ Using the most current and effective erosion and sedimentation control practices during the *construction* phase of development
- ❑ Reducing and controlling stormwater runoff peaks, volumes, and velocities to prevent both downstream flooding and streambank channel erosion
- ❑ Treating *post-construction* stormwater runoff before it is discharged to a waterway
- ❑ Implementing pollution prevention practices to prevent stormwater from becoming contaminated in the first place
- ❑ Using various techniques to maintain groundwater recharge

The remainder of Chapter 2 outlines a technical approach for incorporating all of these stormwater management approaches into the development process. The next section discusses stormwater management standards and numerical sizing criteria for new development and redevelopment in Georgia that aim to meet the objectives above.

## 2.2 Stormwater Management Standards & Numerical Sizing Criteria

### 2.2.1 Overview

This section presents a comprehensive set of recommended performance standards for stormwater management for development activities in the state of Georgia. The overall aim is to provide an integrated approach to address both the water quality and quantity problems associated with stormwater runoff due to development.

The goal of a set of recommended stormwater management standards for areas of new development and significant redevelopment is to reduce the impact of post-construction stormwater runoff on the watershed. This can be achieved by:

1. Maximizing the use of site design and nonstructural methods such as canopy interception, infiltration, evapotranspiration and reuse to reduce the generation of runoff and pollutants;
2. Managing and treating stormwater runoff though the use of best management practices (BMPs);
3. Implementing pollution prevention practices to limit potential stormwater contaminants.

It should be noted that the standards presented here are recommended for all communities in Georgia. They may be adopted by local jurisdictions as stormwater management development **requirements** and/or may be modified to meet local or watershed-specific stormwater management goals and objectives. Please consult your local review authority for more information.

The recommended standards for development are designed to assist local governments in complying with regulatory and programmatic requirements for various state and Federal programs including the National Pollutant Discharge Elimination System (NPDES) Municipal Separate Storm Sewer System (MS4) permit program and the National Flood Insurance Program under FEMA.

BACK TO TOC

## 2.2.2 Recommended Standards for Development

### 2.2.2.1 APPLICABILITY

It is recommended that the stormwater management standards listed below be required for any new development or redevelopment site that meets one or more of the following criteria:

1. New development that includes the creation or addition of 5,000 square feet or greater of new impervious surface area, or that involves land disturbing activity of 1 acre of land or greater.
2. Redevelopment that includes the creation or addition of 5,000 square feet or greater of new impervious surface area, or that involves land disturbing activity of 1 acre or more.
3. Any commercial or industrial new development or redevelopment, regardless of size, with a Standard Industrial Classification (SIC) code that falls under the NPDES Industrial Stormwater Permit program, or is a hotspot land use as defined below.

Since runoff from smaller developments can cause water quality and quantity impacts as well, an individual community may choose to adopt more stringent area criteria, especially if it determines that a significant amount of development in the community falls below these thresholds. In addition, a community may choose to apply stormwater management standards on a case-by-case basis to smaller developments if it determines that the quantity, quality, and or rate of stormwater runoff coming from the site will cause significant impacts to the receiving waters due to the type or location of the development site or other circumstances.

**Definitions**

» **New development** is defined as land disturbing activities, structural development (construction, installation or expansion of a building or other structure), and/or creation of impervious surfaces on a previously undeveloped site.

» **Redevelopment** is defined as structural development (construction, installation, or expansion of a building or other structure), creation or addition of impervious surfaces, replacement of impervious surfaces not as part of routine maintenance, and land disturbing activities associated with structural or impervious development on a previously developed site. Redevelopment does not include such activities as exterior remodeling.

» **Previously developed site** is defined as a site that has been altered by paving, construction, and/or land use that would typically have required regulatory permitting to have been initiated (alterations may exist now or in the past).

» A **hotspot** is defined as a land use or activity on a site that produces higher concentrations of trace metals, hydrocarbons, or other priority pollutants than are normally found in urban stormwater runoff. Examples of hotspots include gas stations, vehicle service and maintenance areas, industrial facilities such as salvage yards (both permitted under the Industrial General Permit and others), material storage sites, garbage transfer facilities, and commercial parking lots with high-intensity use.

The goals and policies of individual communities should determine the specific definition of **pre-development**. It is recommended that pre-development be defined as "natural, undisturbed conditions." This can be simplified to a set type of vegetative condition, such as "woods in good condition," if appropriate. However, where redevelopment incentives are desired, or where flooding concerns do not currently exist, pre-development may be defined as the condition of the site immediately prior to the implementation of the proposed project.

**Exemptions**

In order to avoid excessive regulation on individual residential lots, maintenance and repair efforts, and environmental projects, the following development activities are recommended to be exempted from the stormwater management standards:

1. Individual single family residential lots (single family lots that are part of a subdivision or phased development project should not be exempt from the standards);
2. Additions or modifications to existing single-family structures;

BACK TO TOC

3. Duplex residential units that do not meet the criteria listed above.
4. Land disturbing activity conducted by local, state, authority, or federal agencies, solely to respond to an emergency need to protect life, limb, or property or conduct emergency repairs;
5. Land disturbing activity that consists solely of cutting a trench for utility work and related pavement replacement; and
6. Land disturbing activity conducted by local, state, authority, or federal agencies, whose sole purpose is to implement stormwater management or environmental restoration.

**Additional Requirements**

New development or redevelopment in critical or sensitive areas, or as identified through a watershed study or plan, may be subject to additional performance and/or regulatory criteria. Furthermore, these sites may need to utilize or restrict certain structural practices in order to protect a special resource or address certain water quality or drainage problems identified for a drainage area.

**Off-Site Compliance Alternatives**

Where site conditions do not permit the achievement of the minimum runoff reduction goals for development or redevelopment onsite, alternatives exist for compliance as follows:

1. **Off-Site Mitigation** – Runoff reduction practices at a redevelopment or retrofit site are implemented at another location within the same watershed. The off-site project would likely be initiated by the site developer, and the MS4 plays a coordinating and/or project approval role.
2. **Fee in Lieu** – The developer pays the MS4 (or its assigned entity) an appropriate fee. Fees from multiple sites are aggregated by the MS4 to construct public stormwater projects. This requires an active role for the MS4.

These alternatives are discussed in further detail in Section 5.7 of Vol. 1.

#### 2.2.2.2 RECOMMENDED STORMWATER MANAGEMENT STANDARDS

It is recommended that the following stormwater management performance standards be adopted for new development or redevelopment sites falling under the applicability criteria above. It is further recommended that these twelve (12) standards be adopted in whole to create a comprehensive stormwater management approach. However, an individual community may choose to adopt some of the standards rather than the entire set, or modify individual standards, depending upon its regulatory requirements and specific local approach to stormwater management. *Specific required criteria for communities covered by a Municipal Separate Storm Sewer System (MS4) permit are delineated in the permit.*

❑ **Standard #1 – Natural Resource Inventory**

Prior to the start of any land disturbing activities (including any clearing or grading activities), acceptable site reconnaissance and surveying techniques shall be used to complete a thorough assessment of the natural resources, both terrestrial and aquatic, found on a development site.

The site's critical natural features and drainage patterns shall be identified early in the site planning process. The natural resources inventory shall be used to identify and map the natural resources on site, as they exist prior to the start of any land disturbing activities. The identification, and subsequent preservation and/or restoration of these natural resources, through the use of better site design practices, helps reduce the negative impacts of the land development process "by design".

Resources to be identified and mapped during the natural resources inventory, include, at a minimum (as applicable):

- Topography and Steep Slopes (i.e., Areas with Slopes Greater Than 15%)
- Natural Drainage Divides and Patterns
- Natural Drainage Features (e.g., swales, basins, depressional areas)
- Wetlands
- Water Bodies
- Floodplains
- Aquatic Buffers
- Shellfish Harvesting Areas

BACK TO TOC

» Soils
» Erodible Soils
» Groundwater Recharge Areas
» Wellhead Protection Areas
» Trees and Other Existing Vegetation
» High Quality Habitat Areas
» Protected River Corridors
» Protected Mountains
» Karst Areas

All relevant resources shall be shown on the Stormwater Management Site Plan (Standard #12).

❑ **Standard #2 –Better Site Design Practices for Stormwater Management**

All site designs shall implement a combination of approaches collectively known as *stormwater better site design practices* to the maximum extent practicable. Through the use of these practices and techniques, the impacts of urbanization on the natural hydrology of the site and water quality can be significantly reduced. The goal is to reduce the amount of stormwater runoff and pollutants that are generated, provide for natural on-site control and treatment of runoff, and optimize the location of stormwater management facilities. Better site design concepts can be viewed as both water quantity and water quality management tools and can reduce the size and cost of required BMPs.

Site designs shall preserve the natural drainage and treatment systems and reduce the generation of additional stormwater runoff and pollutants to the maximum extent practicable. More information on Better Site Design is provided in Section 2.3.

The use of certain better site design practices that provide water quality benefits allows for a reduction (known as a "credit") of the water quality volume. The applicable design practices and stormwater site design credits are covered in Section 2.3.2.

❑ **Standard #3 – Runoff Reduction**

Runoff reduction practices shall be sized and designed to retain the first 1.0 inch of rainfall on the site to the maximum extent practicable. Runoff reduction practices are stormwater BMPs used to disconnect impervious and disturbed pervious surfaces from the storm drain system, thereby reducing post-construction stormwater runoff rates, volumes, and pollutant loads. Since runoff reduction practices actually eliminate stormwater runoff (and the pollutants associated with it), rather than simply treating or detaining runoff, they can contribute to several of the other performance standards, while providing many additional benefits. If the entire 1.0 inch of rainfall can be retained onsite using runoff reduction methods, the community may choose to waive the water quality treatment volume in Standard #4. If the entire 1.0-inch runoff reduction standard cannot be achieved, the remaining runoff from the 1.2-inch rainfall event must be treated by BMPs to remove at least 80% of the calculated average annual post-development TSS loading from the site per Standard #4 Water Quality.

Runoff reduction percentages are assigned to applicable BMPs that reduce the amount of stormwater required for treatment, and subsequently reduce the other stormwater management volumes, incentivizing their use. Runoff reduction practices inherently reduce TSS and other pollutants to provide water quality treatment (i.e. 100% pollutant removal for stormwater retention, infiltration, evaporation, transpiration, or rainwater harvesting and reuse). This standard is quantified and expressed in terms of engineering design criteria through the specification of the runoff reduction volume ($RR_V$), which is equal to the runoff generated on a site from 1.0 inches of rainfall. Individual runoff reductions specific to each practice are described in detail in Chapter 4.

While runoff reduction practices provide important water quality benefits, as described in Volume 1, Chapter 2, certain conditions, such as karst topography, soils with very low infiltration rates, high groundwater, or shallow bedrock, may lead a community to choose to waive or reduce the runoff reduction requirement. Alternatively, these conditions can be addressed on a site-specific basis. If the $RR_V$ of 1.0 inches of rainfall cannot be achieved, adequate documentation should be provided to the local development review authority to show that no additional runoff reduction practices can be used on the development site.

❑ **Standard #4 – Water Quality**

Stormwater management systems shall be designed to retain or treat the runoff from 85% of the storms that occur in an average year, and reduce average annual post-development total suspended solids loadings by 80%. Averaged from rainfall events across the state of Georgia, this equates to treating storm events of 1.2 inches or less, as well as the first 1.2 inches of runoff for all larger storm events.

Communities that choose to adopt runoff reduction may choose to waive the water quality treatment volume from this standard if 100% of the 1.0 inch runoff reduction volume is achieved. If the entire 1.0-inch runoff reduction standard cannot be achieved, the remaining runoff from the 1.2-inch rainfall event must be treated by BMPs to remove at least 80% of the calculated average annual post-development TSS loading from the site.

This standard is quantified and expressed in terms of engineering design criteria through specification of the water quality volume ($WQ_v$), which is equal to the runoff generated on a site from 1.2 inches of rainfall. The $WQ_v$ must be treated to the 80% TSS removal performance goal.

This standard assumes that BMPs will be designed, constructed and maintained according to the criteria in this Manual. Stormwater discharges from land uses or activities with higher or special potential pollutant loadings may require the use of specific structural practices and pollution prevention practices. A detailed overview of BMPs is provided in Chapter 4.

❑ **Standard #5 – Stream Channel Protection**

Stream channel protection shall be provided by using all of the following three approaches:

1. 24-hour extended detention storage of the 1-year, 24-hour return frequency storm event
2. Erosion prevention measures, such as energy dissipation and velocity control
3. Preservation of the applicable stream buffer.

Stream channel protection requirements are further described in Section 2.2.4.2.

The first method of providing stream bank protection is the extended detention of the 1-year, 24-hour storm for a period of 24 hours using BMPs. It is known that the increase in runoff due to development can dramatically increase stream channel erosion. This standard is intended to reduce the frequency, magnitude and duration of post-development bankfull flow conditions. The volume to be detained is also known as the channel protection volume ($CP_v$). The use of nonstructural site design practices and runoff reduction BMPs that reduce the total amount of runoff may also reduce $CP_v$ by a proportional amount. Refer to **Table 4.1.3-1** (BMP Selection Guide) for applicable BMPs. Where runoff reduction practices are used, an adjusted curve number (CN) is computed that is lower than the original CN based on an actual stormwater volume removed from the total runoff, see Section 3.1.7.5 for additional information.

This requirement may be waived by a local jurisdiction for sites that discharge directly or through piped stormwater drainage systems into larger streams, rivers, wetlands, lakes, estuaries, tidal waters, or other situations where the reduction in the smaller flows will not have an impact on stream bank or channel integrity.

The second stream bank protection method is to implement velocity control, energy dissipation, stream bank stabilization, and erosion prevention practices and structures as necessary in the stormwater management system to prevent downstream erosion and stream bank damage. Energy dissipation and velocity control methods are discussed in Section 5.5.

The third method of providing for stream channel protection is through the establishment of riparian stream buffers on the development site. Stream buffers not only provide channel protection but also water quality benefits and protection of streamside properties from flooding. It is recommended that 100-foot buffers be established where feasible. Additional stream buffer guidelines are presented in Section 2.3.

❑ **Standard #6 – Overbank Flood Protection**

Overbank flood protection shall be provided by controlling the post-development peak discharge rate to the pre-development rate (natural or existing condition, as applicable) for the 25-year, 24-hour return frequency storm event. If control of the 1-year, 24-hour storm (Standard #5) is exempted, then overbank flood protection shall be provided by controlling the post-development peak discharge rate to the pre-development rate (natural or existing condition, as applicable) for the 2-year through the 25-year return frequency storm events. Overbank flood protection requirements are further described in Section 2.2.4.3.

The use of nonstructural site design practices and runoff reduction BMPs that reduce the total amount of runoff will also reduce $Q_{p25}$ by a proportional amount. Refer to **Table 4.1.3-1** (BMP Selection Guide) for applicable BMPs. Where runoff reduction practices are used, an adjusted curve number (CN) is computed that is lower than the original CN based on an actual stormwater volume removed from the total runoff, see Section 3.1.7.5 for additional information.

Smaller storm events (e.g., 2-year and 10-year) are effectively controlled through a combination of extended detention for the 1-year, 24-hour event (channel protection) and control of the 25-year peak rate for overbank flood protection. These design standards, therefore, are intended to be used in unison.

This standard may be adjusted by a local jurisdiction for areas where all downstream conveyances and receiving waters have the natural capacity to handle the full build-out 25-year storm through a combination of channel capacity and overbank flood storage without increasing flood stages above pre-development flood levels.

❑ **Standard #7 – Extreme Flood Protection**

Extreme flood protection shall be provided by controlling and/or safely conveying the 100-year, 24-hour storm event (denoted $Q_f$). This is accomplished either by (1) controlling $Q_f$ through BMPs to maintain the existing 100-year floodplain, or (2) by sizing the on-site conveyance system to safely pass $Q_f$ and allowing it to discharge into a receiving water whose protected floodplain is sufficiently sized to account for extreme flow increases without causing damage. In this case, the extreme flood protection criterion may be waived by a local jurisdiction in lieu of provision of safe and effective conveyance to receiving waters that have the capacity to handle flow increases to maintain 100-year level.

The use of nonstructural site design practices and runoff reduction BMPs that reduce the total amount of runoff will also reduce $Q_f$ by a proportional amount. Refer to **Table 4.1.3-1** (BMP Selection Guide) for applicable BMPs. Where runoff reduction practices are used, an adjusted curve number (CN) is computed that is lower than the original CN based on an actual stormwater volume removed from the total runoff, see Section 3.1.7.5 for additional information.

Existing and future floodplain areas shall be preserved to the extent possible. Extreme flood protection requirements are further described in Section 2.2.3.

❑ **Standard #8 – Downstream Analysis**

Due to peak flow timing and runoff volume effects, some structural practices fail to reduce discharge peaks to pre-development levels downstream from the development site. A downstream peak flow analysis shall be provided to the point in the watershed downstream of the site or the stormwater management system where the area of the site comprises 10% of the total drainage area. This is to help ensure that there are minimal downstream impacts from the developed site. The downstream analysis may result in the need to resize BMPs, or may allow the waiving of some unnecessary peak flow controls altogether. The use of a downstream analysis and the "ten-percent" rule are discussed in Section 3.1.9.

❑ **Standard #9 – Construction Erosion and Sedimentation Control**

Erosion and sedimentation control practices shall be utilized during the construction phase of development or during any land disturbing activities.

All new development and redevelopment sites must meet the regulatory requirements for land disturbance activities under the Georgia Erosion and Sedimentation Control Act and/or the NPDES General Permit for Construction Activities. This involves the preparation and implementation of an approved erosion and sedimentation control plan, including appropriate best management practices, during the construction phase of development.

Further guidance on practices for construction site erosion and sedimentation control can be found in the latest version of the *Manual for Erosion and Sediment Control in Georgia*.

Better site design practices and techniques that can reduce the total amount of area that needs to be cleared and graded should be implemented wherever possible. It is essential that erosion and sedimentation control be considered and implemented in stormwater concept plans and throughout the construction phase to prevent damage to natural stormwater drainage systems and previously constructed best management practices and conveyance facilities.

❑ **Standard #10 – Stormwater Management System Operation and Maintenance**

The stormwater management system, including all best management practices and conveyances, shall have an operation and maintenance plan to ensure that it continues to function as designed. See Appendix E for more information on stormwater operation and maintenance.

All new development and redevelopment sites are to prepare a comprehensive operation and maintenance plan for the on-site stormwater management system. This is to include all of the stormwater management system components, including drainage facilities, BMPs, and conveyance systems. To ensure that stormwater management systems function as they were designed and constructed, the operation and maintenance plan must provide:

1. A clear assignment of stormwater inspection and maintenance responsibilities

2. The routine and non-routine maintenance tasks to be undertaken

3. A schedule for inspection and maintenance

4. Any necessary legally binding maintenance agreements

❑ **Standard #11 – Pollution Prevention**

To the maximum extent practicable, the development or redevelopment project shall implement pollutant prevention practices and have a stormwater pollution prevention plan.

All new development and redevelopment sites are to consider pollution prevention in the design and operation of the site, and prepare a formal stormwater pollution prevention plan if circumstances warrant it. Specific land use types and hotspots may need to implement more rigorous pollution prevention practices. The preparation of pollution prevention plans and the full set of pollution prevention practices are covered in Volume 3 of this Manual.

❑ **Standard #12 – Stormwater Management Site Plan**

The development project shall prepare a stormwater management site plan for local government review that addresses Standard #1 through Standard #11.

All new development and redevelopment sites will require the preparation of a stormwater management site plan for development activities. A stormwater site plan is a comprehensive report that contains the technical information and analysis to allow a local review authority to determine whether a proposed new development or redevelopment project meets local stormwater regulatory requirements. See, Volume 1, Section 4.3 and other local stormwater regulatory requirements for specific guidance.

BACK TO TOC

### 2.2.3 Numerical Sizing Criteria Overview

This section presents an integrated approach for meeting the volume-based stormwater runoff quality and quantity management standards for development (see Section 2.2.2) by addressing the key adverse impacts of stormwater runoff from a development site. The purpose is to provide a framework for designing a stormwater management system to:

- Improve water quality through runoff reduction and/or water quality treatment (Recommended Standard #3 and #4);
- Prevent downstream stream bank and channel erosion (Recommended Standard #5);
- Reduce downstream overbank flooding (Recommended Standard #6); and
- Safely pass or reduce the runoff from extreme storm events (Recommended Standard #7).

For these objectives, an integrated set of engineering criteria, known as the *Unified Stormwater Sizing Criteria*, have been developed which are used to size and design best management practices. **Table 2.2.3-1** briefly summarizes the criteria. Recommended standard #3 and #4 (water quality) can be achieved by using one of the two standards listed below. The local community will select and require either Runoff Reduction (Standard #3) and/or Water Quality Treatment (Standard #4). See Section 4.2.3 of Volume 1 for more information.

Table 2.2.3-1 Summary of the Statewide Stormwater Sizing Criteria for Stormwater Control and Mitigation

| Sizing Criteria | | Description |
|---|---|---|
| Water Quality | Runoff Reduction, $RR_v$ (Standard #3) | Retain or reduce the runoff for the first 1.0 inch of rainfall, or to the maximum extent practicable. Since runoff reduction practices eliminate stormwater runoff, and the pollutants associated with it, rather than treating or detaining, they can contribute to other stormwater management standards. If the entire 1.0 inch runoff reduction cannot be achieved, the remaining runoff from the 1.2 inch rainfall must be treated, as described in Standard #4. |
| Water Quality | Treatment, $WQ_v$ (Standard #4) | Retain or treat the runoff from 85% of the storms that occur in an average year. For Georgia, this equates to providing water quality treatment for the runoff resulting from a rainfall depth of 1.2 inches. The water quality treatment goal is to reduce average annual post-development total suspended solids loadings by 80%. |
| Channel Protection | | Provide extended detention of the 1-year, 24 hour storm event released over a period of 24 hours to reduce bankfull flows and protect downstream channels from erosive velocities and unstable conditions. |
| Overbank Flood Protection | | Provide peak discharge control of the 25-year, 24 hour storm event such that the post-development peak rate does not exceed the predevelopment rate to reduce overbank flooding. |
| Extreme Flood Protection | | Evaluate the effects of the 100-year, 24 hour storm on the stormwater management system, adjacent property, and downstream facilities and property. Manage the impacts of the extreme storm event through detention controls and/or floodplain management. |

BACK TO TOC

Each of the unified stormwater sizing criteria are intended to be used in conjunction with the others to address the overall stormwater impacts from a development site. When used as a set, the unified criteria control the entire range of hydrologic events, from the smallest runoff producing rainfalls to the 100-year, 24 hour storm. **Figure 2.2.3-1** graphically illustrates the relative volume requirements of each of the unified stormwater sizing criteria as well as demonstrates that the criteria are "nested" within one another, i.e., the extreme flood protection volume requirement also contains the overbank flood protection volume, the channel protection volume and the water quality treatment volume. **Figure 2.2.3-2** shows how these volumes would be nested in a typical stormwater wet pond designed to handle all four criteria.

Also shown is the sediment storage volume. The pond must be dredged when the sediment storage volume is full to maintain its sediment removal effectiveness, and its channel, overbank flood and extreme flood protection capabilities.

The following pages describe the four sizing criteria in detail and present guidance on how to properly compute and apply the required storage volumes.

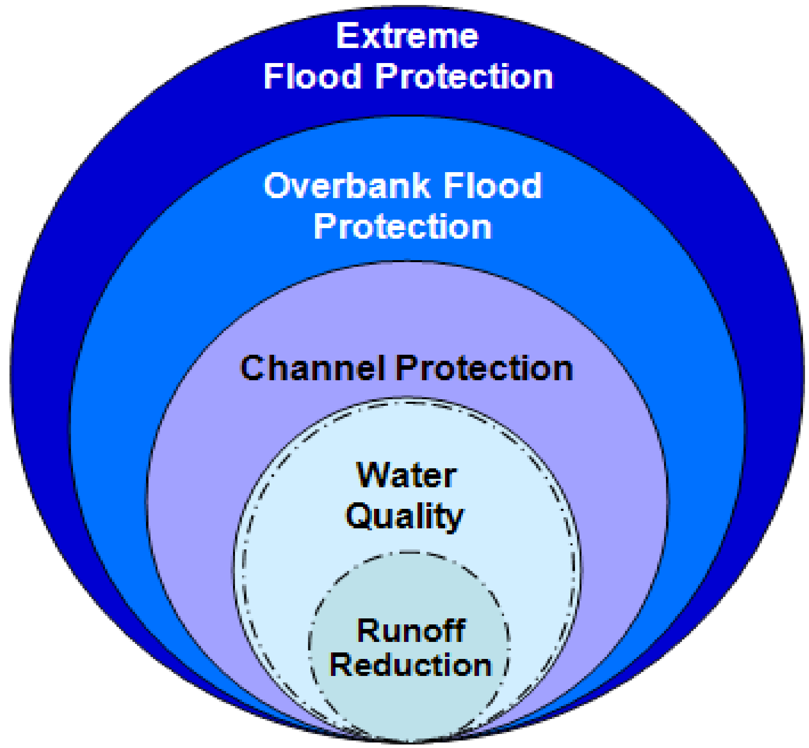



**Figure 2.2.3-1** Representation of the Unified Stormwater Sizing Criteria

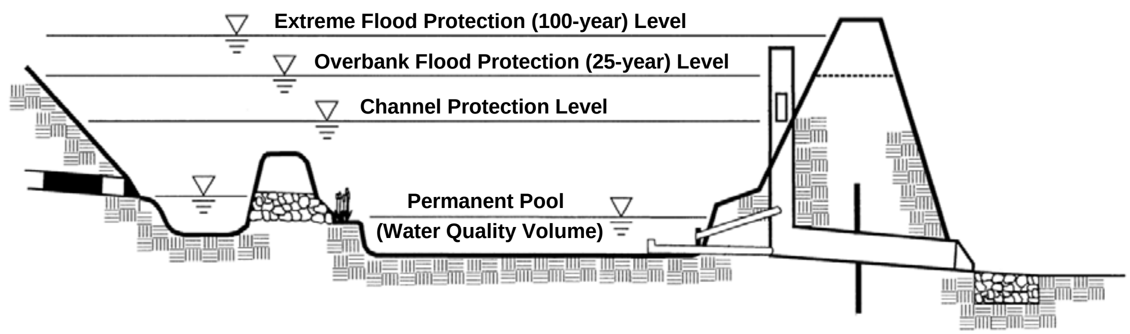



**Figure 2.2.3-2** Unified Sizing Criteria Water Surface Elevations in a Stormwater (Wet) Pond

BACK TO TOC

## 2.2.4 Description of Unified Stormwater Sizing Criteria

### 2.2.4.1 WATER QUALITY VOLUME ($WQ_v$)

As noted in Recommended Standard #3 and #4, the water quality goal can be accomplished either through runoff reduction or water quality treatment or some combination of the two. For more information on either water quality approach, see Section 2.2.2.2.

The Runoff Reduction approach to the Water Quality sizing criterion, denoted $RR_v$, specifies the reduction or elimination of the total pollution load inherent in stormwater runoff by intercepting and reducing or eliminating the first 1.0 inch of rainfall, or to the maximum extent practicable. The Runoff Reduction Volume is a runoff volume that is directly related to the amount of impervious cover at a site.

In numerical terms, it is equivalent to a rainfall depth of 1.0 inches (or other target rainfall amount specified by a local community) multiplied by the volumetric runoff coefficient ($R_v$) and the site area, and is calculated using the formula below:

$$RR_v = \frac{P \times R_v \times A}{12}$$

*Where*:

**$RR_v$** = runoff reduction volume (cubic ft)

**P** = target runoff reduction rainfall (often 1.0 inches)

**$R_v$** = 0.05 + 0.009(I) where I is *percent* impervious cover

**A** = site area in square feet

The Water Quality sizing criterion, denoted $WQ_v$, specifies the retention and/or treatment required to remove a significant percentage of the total pollution load inherent in stormwater runoff by intercepting and retaining or treating the 85th percentile storm event, which is equal to 1.2 inches (i.e., all the runoff from 85% of the storms that occur on average during the course of a year and a portion of the runoff from all storms greater than 1.2 inches). The Water Quality Volume is a runoff volume that is directly related to the amount of impervious cover at a site.

In numerical terms, it is equivalent to a rainfall depth of 1.2 inches multiplied by the volumetric runoff coefficient ($R_v$) and the site area, and is calculated using the formula below:

$$WQ_v = \frac{1.2R_vA}{12}$$

*Where*:

**$WQ_v$** = water quality volume (in cubic ft)

**$R_v$** = 0.05 + 0.009(I) where I is *percent* impervious cover

**A** = site area in square feet

**Discussion**

Hydrologic studies show that small-sized, frequently occurring storms account for the majority of rainfall events that generate stormwater runoff. Consequently, the runoff from these storms also accounts for a major portion of the annual pollutant loadings. Therefore, by eliminating, retaining, or treating these frequently occurring smaller rainfall events and a portion of the stormwater runoff from larger events, it is possible to effectively mitigate the water quality impacts from a developed area.

A water quality volume ($WQ_v$) is specified to size best management practices (BMPs) to reduce/eliminate or treat these small storms up to a maximum runoff depth and the "first flush" of all larger storm events. For Georgia, this maximum depth was determined to be the runoff generated from the 85th percentile storm event (i.e., the storm event that is greater than 85% of the storms that occur within an average year). The 85th percentile volume was considered the point of optimization between pollutant removal ability and cost-effectiveness. Capturing and treating a larger percentage of the annual stormwater runoff would provide only a small increase in additional pollutant removal, but would considerably increase the required size (and cost) of the best management practices.

BACK TO TOC

Based on a rainfall analysis, a value of 1.2 inches for the 85th percentile storm was selected as an average for the entire state. **Thus, the statewide water quality treatment volume criterion is equal to the runoff from the first 1.2 inches of rainfall**. A stormwater management system designed for the $WQ_v$ treatment will retain or treat the runoff from all storm events of 1.2 inches or less, as well as the first 1.2 inches of runoff for all larger storm events. For the runoff reduction approach, a best management practice eliminating or reducing the stormwater runoff from all storm events of 1.0 inches or less, as well as the first 1.0 inches of runoff for all larger storm events will also meet the Water Quality criterion. The correlation between the 1.2 inch treatment requirement for water quality volume and the 1.0 inch volume based reduction is the following:

*Given that an 80% TSS removal rate for the 1.2 inch rainfall event is the standard for addressing water quality, 100% TSS removal through volume reduction of the 1.0 inch rainfall event will address the same requirement. In another method of describing total TSS removal, 80% of 1.2 inches (0.96) approximately equates to 100% of 1.0 inches.*

The volumetric runoff coefficient ($R_v$) was derived from a regression analysis performed on rainfall runoff volume data from a number of cities nationwide and is a shortcut method considered adequate for runoff volume calculation for the type of small storms normally considered in stormwater quality calculations.

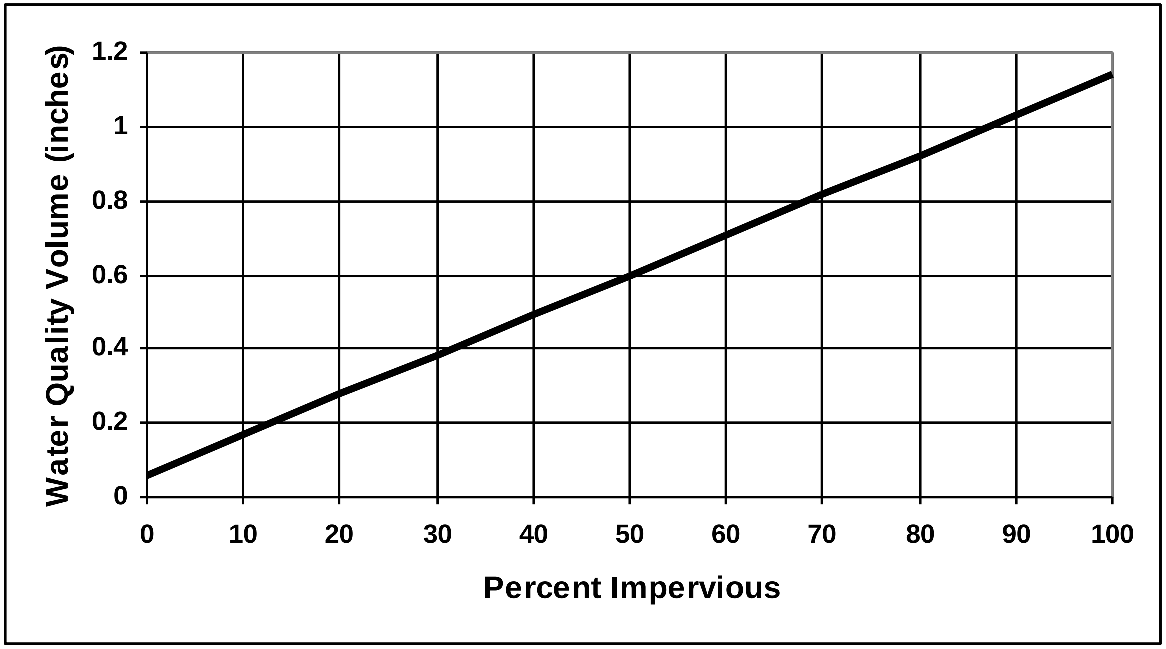



**Figure 2.2.4-1** Water Quality Volume versus Percent Impervious Area

**Figure 2.2.4-1** shows a plot of the Water Quality Volume versus impervious area percentage.

**TSS Reduction Goal**

This Manual follows the philosophy of removing pollutants to the "maximum extent practicable" through the use of a percentage removal performance goal. The approach taken in this Manual is to require treatment of the $WQ_v$ from a site to reduce post-development total suspended solids (TSS) loadings by 80%, as measured on an average annual basis. TSS was chosen as the representative stormwater pollutant for measuring treatment effectiveness for several reasons:

1. The use of TSS as an "indicator" pollutant is well established.
2. Sediment and turbidity, as well as other pollutants of concern that adhere to suspended solids, are a major source of water quality impairment due to urban development in Georgia watersheds.
3. A large fraction of many other pollutants of concern are either removed along with TSS, or at rates proportional to the TSS removal.
4. The 80% TSS removal level is reasonably attainable using well-designed best management practices (for typical ranges of TSS concentration found in stormwater runoff).

TSS is a good indicator for many stormwater pollutants. However, the removal performance for pollutants that are soluble or that cannot be removed by settling will vary depending on the best management practice. For pollutants of specific concern, individual analyses of specific pollutant sources and the appropriate removal mechanisms should be performed.

Determining the Runoff Reduction ($RR_v$) and Water Quality Volumes ($WQ_v$)

» **Measuring Impervious Area**: The area of impervious cover can be taken directly off a set of plans or appropriate mapping. Where this is impractical, NRCS TR-55 land use/impervious cover relationships can be used to estimate impervious cover. I is expressed as a percent value not a fraction (e.g., I = 30 for 30% impervious cover).

» **Multiple Drainage Areas**: When a development project contains or is divided into multiple drainage areas, $RR_v$ and $WQ_v$ should be calculated and addressed separately for each drainage area.

- Calculate the target $RR_v$ for the entire site. To receive the runoff reduction credit for the project, the target $RR_v$ should be met for all drainage areas within the site. Should a portion of the site not be managed by a BMP or a drainage area not providing the target $RR_v$, the other drainage areas will be required to have $RR_v$ values greater than the target to ensure the entire site $RR_v$ requirement is met.

*OR*

- Calculate the $WQ_v$ for the entire site. To satisfy the $WQ_v$ for the project, the $WQ_v$ should be met for all drainage areas within the site. Should a portion of the site not be managed by a BMP or a drainage area not providing the $WQ_v$, the other drainage areas will be required to provide greater than the $WQ_v$ to ensure the entire site $WQ_v$ goal is met.

» **Off-site Drainage Areas**: Off-site drainage may be excluded from the calculation of the $RR_v$ and $WQ_v$ if the drainage is routed around the site.

» **Credits for Site Design Practices**: The use of certain better site design practices may allow the $WQ_v$ to be reduced through the subtraction of a site design "credit." These site design credits are described in Section 2.3.

» **Determining the Peak Discharge for the Water Quality Storm**: When designing off-line structural control facilities, the peak discharge of the water quality storm ($Q_{wq}$) can be determined using the method provided in Section 3.1.

» **Extended Detention of the Water Quality Volume**: The water quality treatment requirement can be met by providing a 24-hour drawdown of a portion of $WQ_v$ in a wet stormwater pond or wetland system (as described in Section 4.25). Referred to as water quality ED (extended detention), it is different than providing extended detention of the 1-year, 24 hour storm for the channel protection volume ($CP_v$). The ED portion of the $WQ_v$ may be included when routing the $CP_v$.

» $RR_v$ and $WQ_v$ can be expressed in cubic feet by multiplying by 43,560.

» $RR_v$ and $WQ_v$ can also be expressed in watershed-inches by removing the area (A) and the "12" in the denominator.

### 2.2.4.2 CHANNEL PROTECTION ($CP_v$)

The Channel Protection sizing criterion specifies that *24 hours of extended detention be provided for runoff generated by the 1-year, 24-hour rainfall event* to protect downstream channels. The required volume needed for 1-year extended detention, denoted $CP_v$, is roughly equivalent to the required volume needed for peak discharge control of the 5- to 10-year storm.

- $CP_v$ control is not required for post-development discharges less than 2.0 cfs at each individual discharge location
- The use of nonstructural site design practices and runoff reduction practices that reduce the total amount of runoff will also reduce the channel protection volume by a proportional amount.
- The channel protection criteria may be waived by a local jurisdiction for sites that discharge directly into larger streams, rivers, wetlands, lakes, estuaries, or tidal waters where the reduction in the smaller flows will not have an impact on stream bank or channel integrity.

BACK TO TOC

#### Discussion

The increase in the frequency and duration of bankfull flow conditions in stream channels due to urban development is the primary cause of stream bank erosion and the widening and down-cutting of stream channels. Therefore, channel erosion downstream of a development site can be significantly reduced by storing and releasing stormwater runoff from the channel-forming runoff events (which corresponds approximately to the 1-year, 24 hour storm event) in a gradual manner to ensure that critical erosive velocities and flow volumes are not exceeded.

#### Determining the Channel Protection Volume ($CP_v$)

- **$CP_v$ Calculation Methods**: Several methods can be used to calculate the $CP_v$ storage volume required for a site. Subsection 3.1.5 and Appendix B illustrate the recommended average outflow method for volume calculation.
- **Hydrograph Generation**: The NRCS TR-55 hydrograph methods provided in Section 3.1 can be used to compute the runoff hydrograph for the 1-year, 24-hour storm.
- **Rainfall Depths**: The rainfall depth of the 1-year, 24-hour storm will vary depending on location and can be determined from rainfall data found in the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 publication, or online using the *Precipitation Frequency Data Server* database for any location across Georgia (http://hdsc.nws.noaa.gov/hdsc/pfds/).
- **Multiple Drainage Areas**: When a development project contains or is divided into multiple drainage areas, $CP_v$ may be distributed proportionally to each drainage area.
- **Off-site Drainage Areas**: Off-site drainage areas should be modeled as “present condition” for the 1-year storm event. If there are adequate upstream channel protection controls, then the off-site area can be modeled as “forested” or “natural” condition. A best management practice located “on-line” will need to safely bypass any off-site flows.
- **Routing/Storage Requirements**: The required storage volume for the $CP_v$ may be provided above the $WQ_v$ storage in stormwater ponds and wetlands with appropriate hydraulic control structures for each storage requirement.
- **Control Orifices**: Orifice diameters for $CP_v$ control of less than 3 inches are not recommended without adequate clogging protection (see Section 3.3).

### 2.2.4.3 OVERBANK FLOOD PROTECTION ($Q_{p25}$)

The Overbank Flood Protection criterion specifies that the *post-development 25-year, 24-hour storm peak discharge rate, denoted $Q_{p25}$, not exceed the pre-development (natural or existing condition, as applicable) discharge rate*. This is achieved through detention of runoff from the 25-year event.

- Smaller storm events (e.g., 2-year and 10-year) are effectively controlled through the combination of the extended detention for the 1-year event (channel protection $CP_v$ control) and the control of $Q_{p25}$ for overbank channel protection.
- Larger storms (> 25-year) are partially attenuated through the control of $Q_{p25}$.
- The use of nonstructural site design practices and runoff reduction practices that reduce the total amount of runoff will also reduce $Q_{p25}$ by a proportional amount.

*Control of $Q_{p25}$ is not intended to serve as a stand-alone design standard, but is intended to be used in conjunction with the channel protection AND extreme flood protection criteria.* If detention is designed for only the 25-year storm, smaller runoff events will simply pass through the outlet structure with little attenuation. If the channel protection criterion is not used, then for overbank flood protection, peak flow attenuation of the 2-year ($Q_{p2}$) through the 25-year ($Q_{p25}$) return frequency storm events **must** be provided.

#### Discussion

The purpose of overbank flood protection is to prevent an increase in the frequency and magnitude of damaging out-of-bank flooding (i.e., flow events that exceed the capacity of the channel and enter the floodplain). It is intended to protect downstream properties from flooding at middle-frequency storm events.

This criterion may be adjusted by a local jurisdiction for areas where all downstream conveyances are designed to handle runoff from the full buildout

25-year storm, or where it can be demonstrated that no downstream flooding will occur as a result of a proposed development (see Section 3.1.9). In this case, the overbank flood protection criterion may be waived by a local jurisdiction in lieu of provision of safe and effective conveyance to a major river system, lake, wetland, estuary, or tidal waters that have capacity to handle flow increases at the 25-year level.

**Determining the Overbank Flood Protection Volume ($Q_{p25}$)**

- **Peak-Discharge and Hydrograph Generation**: The NRCS TR-55 or USGS hydrograph methods provided in Section 3.1 can be used to compute the peak discharge rate and runoff for the 25-year, 24-hour storm.
- **Rainfall Depths**: The rainfall depth of the 25-year, 24-hour storm will vary depending on location and can be determined from rainfall data found in the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 publication, or online using the *Precipitation Frequency Data Server* database for any location across Georgia (http://hdsc.nws.noaa.gov/hdsc/pfds/).
- **Off-site Drainage Areas**: Off-site drainage areas should be modeled as "present condition" for the 25-year storm event and do not need to be included in $Q_{p25}$ estimates, but can be routed through a best management practice.
- **Downstream Analysis**: Downstream areas should be checked to ensure there is no peak flow increase above pre-development conditions to the point where the site area is 10% of the total drainage to that point.

### 2.2.4.4 EXTREME FLOOD PROTECTION ($Q_f$)

The Extreme Flood Protection criterion specifies that all stormwater management facilities be designed to *safely handle the runoff from the 100-year, 24-hour return frequency storm event, denoted $Q_f$*. This is accomplished either by:

1. Controlling $Q_f$ through on-site or regional best management practices to maintain the existing 100-year floodplain. This is done where residences or other structures have already been constructed within the 100-year floodplain fringe area; or
2. By sizing the on-site conveyance system to safely pass $Q_f$ and allowing it to discharge into a receiving water whose protected full buildout floodplain is sufficiently sized to account for extreme flow increases without causing damage.

Local flood protection (levees, floodwalls, floodproofing, etc.) and/or channel enlargements may be substituted as appropriate, as long as adequate conveyance and structural safety is ensured through the measure used, and stream environmental integrity is adequately maintained.

**Discussion**

The intent of the extreme flood protection is to prevent flood damage from infrequent but large storm events, maintain the boundaries of the mapped 100-year floodplain, and protect the physical integrity of the best management practices as well as downstream stormwater and flood control facilities.

It is recommended that $Q_f$ be used in the routing of runoff through the drainage system and stormwater management facilities to determine the effects on the facilities, adjacent property, and downstream. Emergency spillways of best management practices should be designed appropriately to safely pass the resulting flows.

**Determining the Extreme Flood Protection Criteria ($Q_{p25}$)**

- **Peak-Discharge and Hydrograph Generation**: The NRCS TR-55 or USGS hydrograph methods provided in Section 3.1 can be used to compute the peak discharge rate and runoff for the 100-year, 24-hour storm.
- **Rainfall Depths**: The rainfall depth of the 100-year, 24-hour storm will vary depending on location and can be determined from rainfall data found in the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 publication, or online using the *Precipitation Frequency Data Server* database for any location across Georgia (http://hdsc.nws.noaa.gov/hdsc/pfds/).
- **Off-site Drainage Areas**: Off-site drainage areas should be modeled as "full buildout condition" for the 100-year storm event to ensure safe passage of future flows.
- **Downstream Analysis**: If $Q_f$ is being detained, downstream areas should be checked to ensure there is no peak flow increase above pre-development conditions to the point where the site area is 10% of the total drainage to that point.

## 2.2.5 Meeting the Unified Stormwater Sizing Criteria Requirements

### 2.2.5.1 INTRODUCTION

There are two primary approaches for managing stormwater runoff and addressing the unified stormwater sizing criteria requirements on a development site:

1. The use of better site design practices to reduce the amount of stormwater runoff and pollutants generated and/or provide for natural runoff reduction, treatment, and control of runoff; and
2. The use of best management practices to provide runoff reduction, treatment, and control of stormwater runoff

This subsection introduces both of these approaches. Stormwater better site design practices are discussed in-depth in Section 2.3, while best management practices are covered in Chapter 4.

### 2.2.5.2 SITE DESIGN AS THE FIRST STEP IN ADDRESSING UNIFIED STORMWATER SIZING CRITERIA REQUIREMENTS

Using the site design process to reduce stormwater runoff and pollutants should always be the first consideration of the site designer and engineer in the planning of the stormwater management system for a development.

Through the use of a combination of approaches collectively known as stormwater better site design practices and techniques, it is possible to reduce the amount of runoff and pollutants that are generated, as well as provide for at least some nonstructural on-site treatment and control of runoff. Better site design concepts can be viewed as both water quantity and water quality management tools and can reduce the size and cost of required best management practices—sometimes eliminating the need for them entirely. The site design approach can result in a more natural and cost-effective stormwater management system that better mimics the natural hydrologic conditions of the site, has a lower maintenance burden and provides for more sustainability.

» Better site design includes:
  - Conserving natural features and resources
  - Using lower impact site design techniques
  - Reducing impervious cover
  - Utilizing natural features for stormwater management

For each of the above categories, there are a number of practices and techniques that aim to reduce the impact of urban development and stormwater runoff from the site. These better site design practices are described in detail in Section 2.3.

For several of the better site design practices, there is a direct economic benefit to their implementation for both stormwater quality and quantity through the application of site design "credits." In terms of the unified stormwater sizing criteria, **Table 2.2.5-1** shows how the use of nonstructural site design practices can provide a reduction in the amount of stormwater runoff that is required to be treated and/or controlled through the application of site design credits.

Table 2.2.5-1 Reductions or "Credits" to the Unified Stormwater Sizing Criteria through the Use of Better Site Design Practices

| Sizing Criteria | Potential Benefits of the Use of Better Site Design Practices |
|---|---|
| Water Quality ($RR_v$ & $WQ_v$) | • Better site design practices that reduce the total amount of runoff will also reduce $RR_v$ & $WQ_v$ by a proportional amount, through the overall volume and TSS removed.<br>• Certain site design practices will allow for a further reduction to the $RR_v$ and $WQ_v$. The site design credits are discussed in Section 2.3. |
| Channel Protection, Overbank Flood Protection, and Extreme Flood Protection ($CP_v$, $Q_{p25}$, $Q_f$) | • The use of better site design practices that reduce the total amount of runoff will also reduce $CP_v$, $Q_{p25}$, and $Q_f$ by a proportional amount.<br>• Floodplain preservation may allow waiving of overbank flood and/or extreme flood protection requirements. |

BACK TO TOC

### 2.2.5.3 RECOMMENDED BEST MANAGEMENT PRACTICES

Best management practices (sometimes referred to as structural practices or BMPs) are constructed stormwater management facilities designed to retain or treat stormwater runoff and/or mitigate the effects of increased stormwater runoff peak rate, volume, and velocity due to urbanization.

This Manual recommends a number of best management practices for meeting unified stormwater sizing criteria. The recommended practices are divided into three categories: **runoff reduction ($RR_v$), stormwater treatment (water quality volume, $WQ_v$),** and **detention control**s. Some of the best management practices provide multiple benefits and may even fit into more than one of the previously mentioned categories.

Some stormwater practices are best suited to address runoff quality, while others are best suited to address runoff quantity. However, better site design and runoff reduction practices address both simultaneously. As discussed in Volume 1, Chapter 3, by reducing the impervious cover associated with a development, better site design techniques reduce the amount of runoff being generated by a development in the first place. Runoff reduction practices, on the other hand eliminate some of the runoff after it is generated. Instead of treating the runoff like a typical water quality practice, or detaining it like a typical water quality practice, they remove it – removing the pollutants along with it.

In Georgia, the state requirement for Water Quality is to remove 80% TSS from the 1.2" rainfall event. To comply with this requirement, there are several ways to incorporate better site design, runoff reduction practices, and best management practices in a design.

### 2.2.5.4 USING BEST MANAGEMENT PRACTICES TO MEET UNIFIED STORMWATER SIZING CRITERIA REQUIREMENTS

Best management practices should be considered after all reasonable attempts have been made to minimize stormwater runoff and maximize its control and treatment through the better site design methods. Once the need for structural practices has been established, one or more appropriate best management practices will need to be selected to handle the stormwater runoff storage and treatment requirements calculated using the unified stormwater sizing criteria. Guidance for choosing the appropriate best management practice(s) for a site is provided in Section 4.1.

### 2.2.5.5 TYPICAL STEPS IN ADDRESSING THE UNIFIED STORMWATER SIZING CRITERIA

Each development site is unique in how stormwater management objectives are met. The type of development, physical site conditions, location in the watershed, and other factors determine how the recommended stormwater management standards and unified stormwater sizing criteria are addressed.

**Figure 2.2.5-2** provides a flowchart for the typical steps in stormwater management system design using the unified stormwater sizing criteria. This is a subset of the stormwater site planning process detailed in Section 2.4.

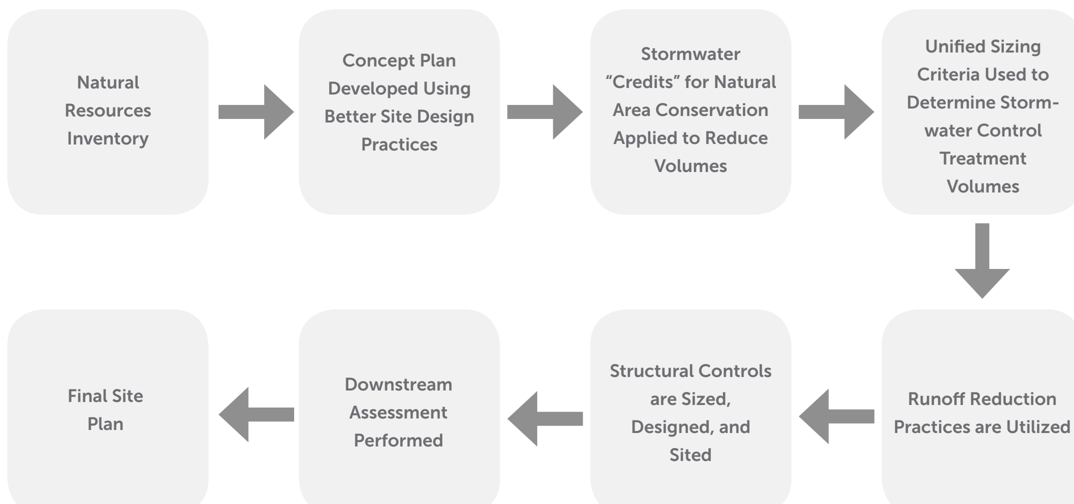



**Figure 2.2.5-2** Typical Stormwater Management System Design Process

BACK TO TOC

# 2.3 Stormwater Better Site Design & Techniques

## 2.3.1 Overview

### 2.3.1.1 INTRODUCTION

The first step in addressing stormwater management begins with the site planning and design process. Development projects can be designed to reduce their impact on watersheds when careful efforts are made to conserve natural areas, reduce impervious cover and better integrate stormwater treatment. By implementing a combination of these nonstructural approaches collectively known as *stormwater better site design practices*, it is possible to reduce the amount of runoff and pollutants that are generated from a site and provide for some nonstructural on-site treatment and control of runoff. The goals of better site design include:

- Managing stormwater (quantity and quality) as close to the point of origin as possible and minimizing collection and conveyance
- Preventing stormwater impacts rather than mitigating them
- Utilizing simple, nonstructural methods for stormwater management that are lower cost and lower maintenance than structural controls (best management practices)
- Creating a multifunctional landscape
- Using hydrology as a framework for site design

Better site design for stormwater management includes a number of site design techniques such as preserving natural features and resources, effectively laying out the site elements to reduce impact, reducing the amount of impervious surfaces, and utilizing natural features on the site for stormwater management. The aim is to reduce the environmental impact "footprint" of the site while retaining and enhancing the owner/developer's purpose and vision for the site. Many of the better site design concepts can reduce the cost of infrastructure while maintaining or even increasing the value of the property.

Reduction of adverse stormwater runoff impacts through the use of better site design should be the first consideration of the design engineer. Operationally, economically, and aesthetically, the use of better site design practices offers significant benefits over treating and controlling runoff downstream. Therefore, feasible opportunities for using these methods should be explored and exhausted before considering best management practices.

The reduction in runoff and pollutants using better site design can reduce the required runoff peak and volumes that need to be conveyed and controlled on a site and, therefore, the size and cost of necessary drainage infrastructure and best management practices. In some cases, the use of better site design practices may eliminate the need for structural controls entirely. Hence, better site design concepts can be viewed as both a water quantity and water quality management tool.

One of the site design practices described in this section provide a calculable reduction or site design "credit" which can be applied to the unified stormwater sizing criteria requirements. Section 2.3.2 discusses these practices and provide examples of their application.

The use of stormwater better site design can also have a number of other ancillary benefits including:

- Reduced construction costs
- Increased property values
- More open space for recreation
- More pedestrian friendly neighborhoods
- Protection of sensitive forests, wetlands and habitats
- More aesthetically pleasing and naturally attractive landscape
- Easier compliance with wetland and other resource protection regulations

BACK TO TOC

#### 2.3.1.2 LIST OF STORMWATER BETTER SITE DESIGN PRACTICES AND TECHNIQUES

The stormwater better site design practices and techniques covered in this Manual are grouped into four categories and are listed below:

❑ **Conservation of Natural Features and Resources**
- Preserve Undisturbed Natural Areas
- Preserve Riparian Buffers
- Avoid Developing in Floodplains
- Avoid Developing on Steep Slopes
- Minimize Siting on Porous or Erodible Soils

❑ **Lower Impact Site Design Techniques**
- Fit Design to the Terrain
- Locate Development in Less Sensitive Areas
- Reduce Limits of Clearing and Grading
- Utilize Open Space Development
- Consider Creative Development Design

❑ **Reduction of Impervious Cover**
- Reduce Roadway Lengths and Widths
- Reduce Building Footprints
- Reduce the Parking Footprint
- Reduce Setbacks and Frontages
- Use Fewer or Alternative Cul-de-Sacs
- Create Parking Lot Stormwater "Islands"

❑ **Utilization of Natural Features for Stormwater Management**
- Use Buffers and Undisturbed Areas
- Use Natural Drainageways Instead of Storm Sewers
- Use Vegetated Swale Instead of Curb and Gutter
- Use soil restoration practices to improve native soils
- Drain Rooftop Runoff to Pervious Areas

More detail on each site design practice is provided in the Stormwater Better Site Design Practice Summary Sheets in Subsection 2.3.2. These summaries provide the key benefits of each practice, examples and details on how to apply them in site design.

#### 2.3.1.3 USING STORMWATER BETTER SITE DESIGN PRACTICES

Site design should be done in unison with the design and layout of stormwater infrastructure in attaining stormwater management goals. **Figure 2.3.1-1** illustrates the stormwater better site design process that utilizes the four better site design categories.

The first step in stormwater better site design involves identifying significant natural features and resources on a site such as undisturbed forest areas, stream buffers and steep slopes that should be preserved to retain some of the original hydrologic function of the site.

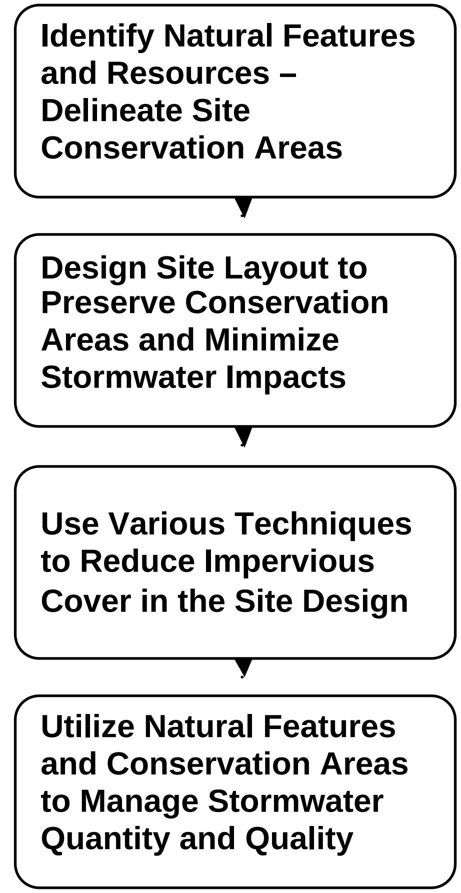



**Figure 2.3.1-1** Stormwater Better Site Design Process

Next, the site layout is designed such that these conservation areas are preserved and the impact of the development is minimized. A number of techniques can then be used to reduce the overall imperviousness of the development site.

Finally, natural features and conservation areas can be utilized to serve stormwater quantity and quality management purposes.

## 2.3.2 Better Site Design Practices

### 2.3.2.1 CONSERVATION OF NATURAL FEATURES AND RESOURCES

Conservation of natural features is integral to better site design. The first step in the better site design process is to identify and preserve the natural features and resources that can be used in the protection of water resources by reducing stormwater runoff, providing runoff storage, reducing flooding, preventing soil erosion, promoting infiltration, and removing stormwater pollutants. Some of the natural features that should be taken into account include:

- Areas of undisturbed vegetation
- Floodplains and riparian areas
- Ridgetops and steep slopes
- Natural drainage pathways
- Intermittent and perennial streams
- Wetlands / tidal marshes
- Aquifers and recharge areas
- Soils
- Shallow bedrock or high water table
- Other natural features or critical areas

Some of the ways used to conserve natural features and resources described over the next several pages include the following methods:

- Preserve Undisturbed Natural Areas
- Preserve Riparian Buffers
- Avoid Floodplains
- Avoid Steep Slopes
- Minimize Siting on Porous or Erodible Soils

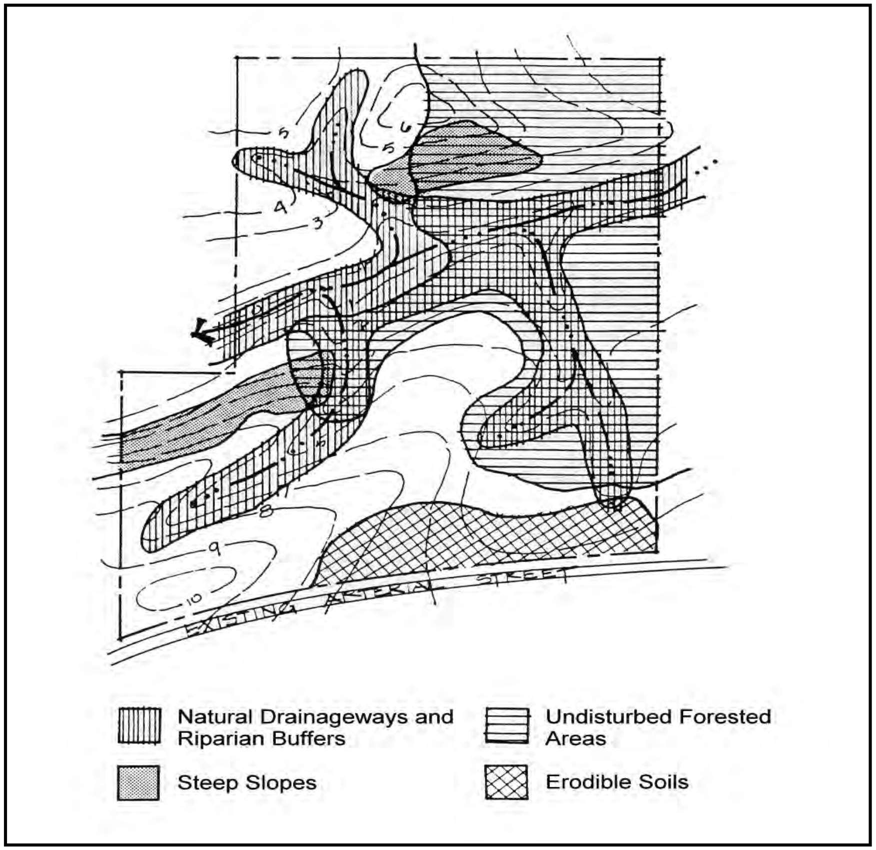



**Figure 2.3.2-1** Example of Natural Feature Delineation
(Source: MPCA, 1989)

Delineation of natural features is typically done through a comprehensive site analysis and inventory before any site layout design is performed (see Section 2.4). From this site analysis, a concept plan for a site can be prepared that provides for the conservation and protection of natural features. **Figure 2.3.2-1** shows an example of the delineation of natural features on a base map of a development parcel.

BACK TO TOC

### Better Site Design Practice #1: Preserve Undisturbed Natural Areas

» **Description**: Important natural features and areas such as undisturbed forested and vegetated areas, natural drainageways, stream corridors, wetlands and other important site features should be delineated and placed into conservation areas.

#### KEY BENEFITS

- Conserving undisturbed natural areas helps to preserve a portion of the site's natural pre-development hydrology
- Can be used as nonstructural stormwater filtering and infiltration zones
- Helps to preserve the site's natural character and aesthetic features
- May increase the value of the developed property
- A stormwater site design credit can be taken if allowed by the local review authority (see subsection 2.3.2)

#### USING THIS PRACTICE

- Delineate natural areas before performing site layout and design
- Ensure that conservation areas and native vegetation are protected in an undisturbed state throughout construction and occupancy

**Discussion**

Preserving natural conservation areas such as undisturbed forested and vegetated areas, natural drainageways, stream corridors and wetlands on a development site helps to preserve the original hydrology of the site and aids in reducing the generation of stormwater runoff and pollutants. Undisturbed vegetated areas also promote soil stabilization and provide for filtering, infiltration and evapotranspiration of runoff.

Natural conservation areas are typically identified through a site analysis using maps and aerial/satellite photography, or by conducting a site visit. These areas should be delineated before any site design, clearing or construction begins. When done before the concept plan phase, the planned conservation areas can be used to guide the layout of the site. **Figure 2.3.2-2** shows a site map with undisturbed natural areas delineated.

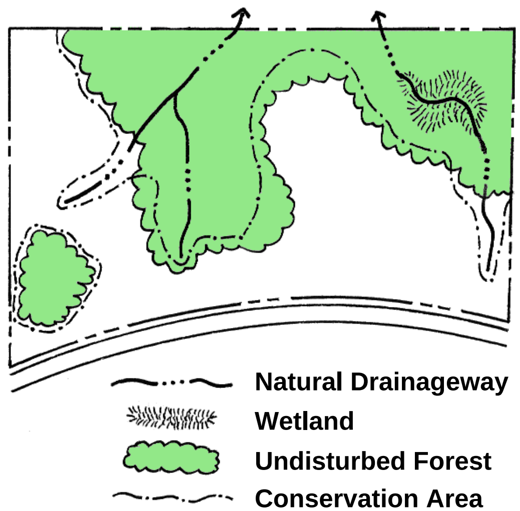



**Figure 2.3.2-2** Delineation of Natural Conservation Areas

Conservation areas should be incorporated into site plans and clearly marked on all construction and grading plans to ensure that equipment is kept out of these areas and that native vegetation is kept in an undisturbed state. The boundaries of each conservation area should be mapped by carefully determining the limit which should not be crossed by construction activity.

Once established, natural conservation areas must be protected during construction and managed after occupancy by a responsible party able to maintain the areas in a natural state in perpetuity. Typically, conservation areas are protected by legally enforceable deed restrictions, conservation easements, and maintenance agreements.

**Better Site Design Practice #2:**

## Preserve Riparian Buffers

» **Description**: Naturally vegetated buffers should be delineated and preserved along perennial streams, rivers, lakes, and wetlands.

### KEY BENEFITS

- Riparian buffers can be used as nonstructural stormwater filtering and infiltration zones
- Keeps structures out of the floodplain and provides a right-of-way for large flood events
- Helps to preserve riparian ecosystems and habitats

### USING THIS PRACTICE

- Delineate and preserve naturally vegetated riparian buffers
- Ensure that buffers and native vegetation are protected throughout construction and occupancy

**Discussion**

A riparian buffer is a special type of natural conservation area along a stream, wetland or shoreline where development is restricted or prohibited. The primary function of buffers is to protect and physically separate a stream, lake or wetland from future disturbance or encroachment. If properly designed, a buffer can provide stormwater management functions, can act as a right-of-way during floods, and can sustain the integrity of stream ecosystems and habitats. An example of a riparian stream buffer is shown in **Figure 2.3.2-3**.

**Figure 2.3.2-3** Riparian Stream Buffer

Forested riparian buffers should be maintained and reforestation should be encouraged where no wooded buffer exists. Proper restoration should include all layers of the forest plant community, including understory, shrubs and groundcover, not just trees. A riparian buffer can be of fixed or variable width, but should be continuous and not interrupted by impervious areas that would allow stormwater to concentrate and flow into the stream without first flowing through the buffer.

Ideally, riparian buffers should be sized to include the 100-year floodplain as well as steep banks and freshwater wetlands. The buffer width needed to perform properly will depend on the size of the stream and the surrounding conditions, but a minimum 25-foot undisturbed vegetative buffer is needed for even the smallest perennial streams and a 50-foot or larger undisturbed buffer is ideal. Even with a 25-foot undisturbed buffer, additional zones can be added to extend the total buffer to at least 75 feet from the edge of the stream. The three distinct zones within the 75-foot width are shown in **Figure 2.3.2-4**. The function, vegetative target and allowable uses vary by zone as described in **Table 2.3.2-1**.

These recommendations are minimum standards to apply to most streams. Some streams and watershed may require additional measures to achieve protection. In some areas, specific state laws or local ordinances already require stricter buffers than are described here. The buffer widths discussed are not intended to modify or supersede deeper or more restrictive buffer requirements that are already in place.

BACK TO TOC

As stated above, the streamside or inner zone should consist of a minimum of 25 feet of undisturbed mature forest. In addition to runoff protection, this zone provides bank stabilization as well as shading and protection for the stream. This zone should also include wetlands and any critical habitats, and its width should be adjusted accordingly. The middle zone provides a transition between upland development and the inner zone and should consist of managed woodland that allows for infiltration and filtration of runoff. An outer zone allows more clearing and acts as a further setback for impervious surfaces. It also functions to prevent encroachment and filter runoff. It is here that flow into the buffer should be transformed from concentrated flow into sheet flow to maximize ground contact with the runoff.

Development within the riparian buffer should be limited only to those structures and facilities that are absolutely necessary. Such limited development should be specifically identified in any codes or ordinances enabling the buffers. When construction activities do occur within the riparian corridor, specific mitigation measures should be required, such as deeper buffers or riparian buffer improvements.

Generally, the riparian buffer should remain in its natural state. However, some maintenance is periodically necessary, such as planting to minimize concentrated flow, the removal of exotic plant species when these species are detrimental to the vegetated buffer and the removal of diseased or damaged trees.

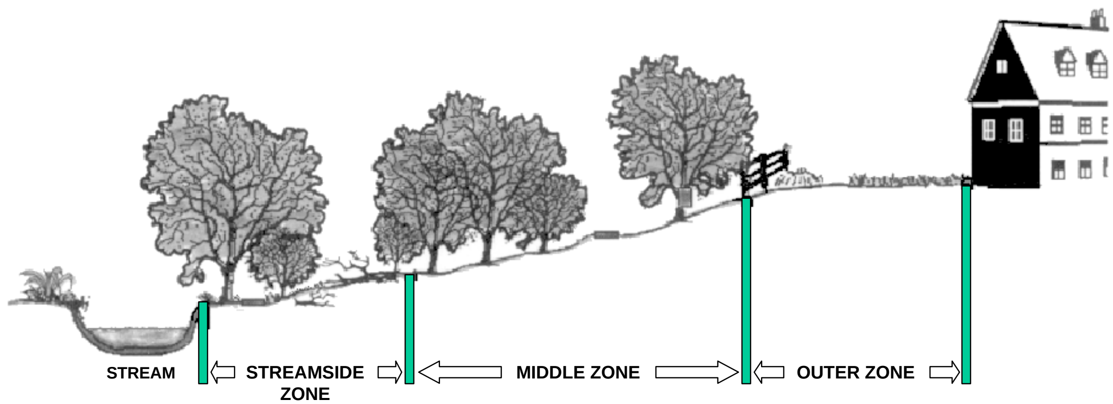



**Figure 2.3.2-4** Three-Zone Stream Buffer System

Table 2.3.2-1 Riparian Buffer Management Zones

| | Streamside Zone | Middle Zone | Outer Zone |
|---|---|---|---|
| Width | Minimum 25 feet plus wetlands and critical habitat | Variable depending on stream order, slope, and 100-year floodplain (min. 25 ft) | 25-foot minimum setback from structures |
| Vegetative Target | Undisturbed mature forest. Reforest if necessary. | Managed forest, some clearing allowed. | Forest encouraged, but usually turf grass. |
| Allowable Uses | Very Restricted<br>e.g., flood control, utility easements, footpaths | Restricted<br>e.g., some recreational uses, some stormwater controls, bike paths | Unrestricted<br>e.g., residential uses including lawn, garden, most stormwater controls |

BACK TO TOC

### Better Site Design Practice #3:
## Avoid Floodplains

» **Description**: Floodplain areas should be avoided for homes and other structures to minimize risk to human life and property damage, and to allow the natural stream corridor to accommodate flood flows.

### KEY BENEFITS

- Preserving floodplains provides a natural right-of-way and temporary storage for large flood events
- Keeps people and structures out of harm's way
- Helps to preserve riparian ecosystems and habitats
- Can be combined with riparian buffer protection to create linear greenways

### USING THIS PRACTICE

- Obtain maps of the 100-year floodplain from the local review authority
- Ensure that all development activities do not encroach on the designated floodplain areas

**Discussion**

Floodplains are the low-lying flat lands that border streams and rivers. When a stream reaches its capacity and overflows its channel after storm events, the floodplain provides for storage and conveyance of these excess flows. In their natural state they reduce flood velocities and peak flow rates by the passage of flows through dense vegetation. Floodplains also play an important role in reducing sedimentation and filtering runoff, and provide habitat for both aquatic and terrestrial life. Development in floodplain areas can reduce the ability of the floodplain to convey stormwater, potentially causing safety problems or significant damage to the site in question, as well as to both upstream and downstream properties. Most communities regulate the use of floodplain areas to minimize the risk to human life as well as to avoid flood damage to structures and property.

As such, floodplain areas should be avoided on a development site. Ideally, the entire 100-year full-buildout floodplain should be avoided for clearing or building activities, and should be preserved in a natural undisturbed state where possible. Floodplain protection is complementary to riparian buffer preservation. Both of these better site design practices preserve stream corridors in a natural state and allow for the protection of vegetation and habitat. Depending on the site topography, 100-year floodplain boundaries may lie inside or outside of a preserved riparian buffer corridor, as shown in **Figure 2.3.2-5**.

Maps of the 100-year floodplain can typically be obtained through the local review authority, through FEMA's website, or through the Georgia Flood M.A.P. website. Developers and builders should also ensure that their site design comply will any other relevant local floodplain and FEMA requirements.

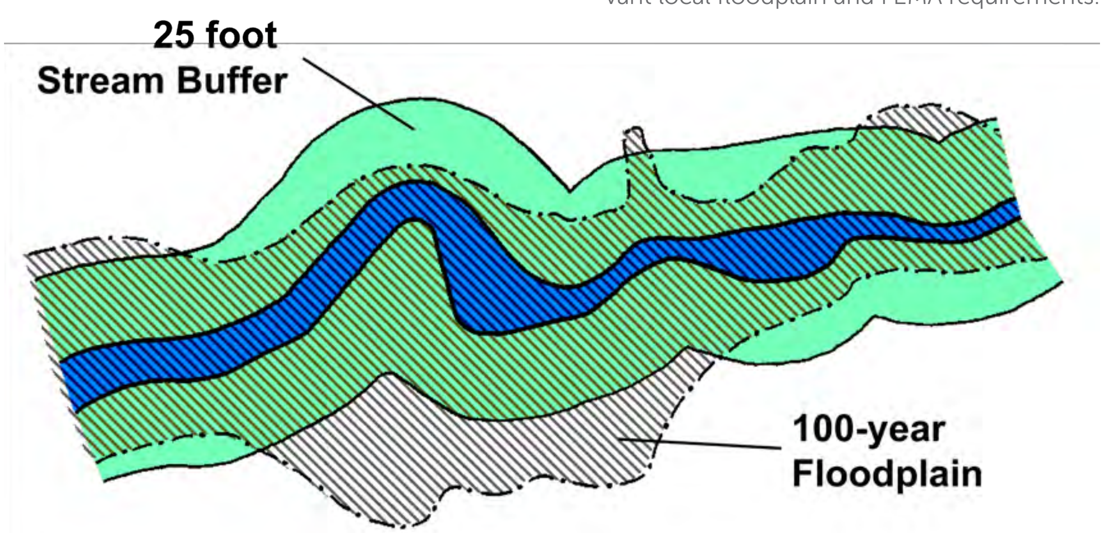



**Figure 2.3.2-5** Floodplain Boundaries in Relation to a Riparian Buffer

BACK TO TOC

Better Site Design Practice #4:

## Avoid Steep Slopes

» **Description**: Steep slopes should be avoided due to the potential for soil erosion and increased sediment loading. Excessive grading and flattening of hills and ridges should be minimized.

### KEY BENEFITS

- Preserving steep slopes helps to prevent soil erosion and degradation of stormwater runoff
- Steep slopes can be kept in an undisturbed natural condition to help stabilize hillsides and soils
- Building on flatter areas will reduce the need for cut-and-fill and grading

### USING THIS PRACTICE

- Avoid development on steep slope areas, especially those with a grade of 15% or greater
- Minimize grading and flattening of hills and ridges

**Discussion**

Developing on steep slope areas has the potential to cause excessive soil erosion and stormwater runoff during and after construction. Past studies by the NRCS and others have shown that soil erosion is significantly increased on slopes of 15% or greater. In addition, the nature of steep slopes means that greater areas of soil and land area are disturbed to locate facilities on them compared to flatter slopes as demonstrated in **Figure 2.3.2-6**.

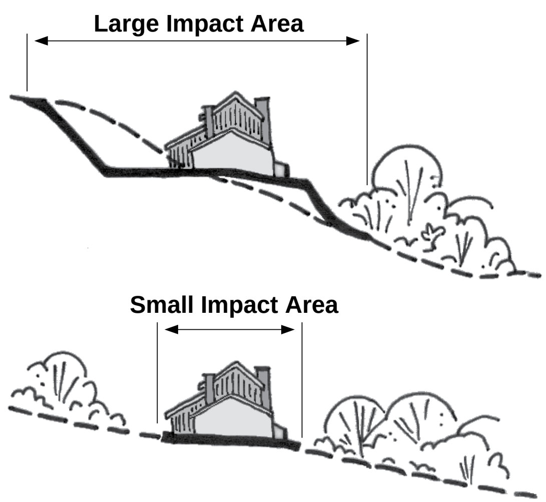



**Figure 2.3.2-6** Flattening Steep Slopes for Building Sites Uses More Land Area Than Building on Flatter Slopes
(Source: MPCA, 1989)

Therefore, development on slopes with a grade of 15% or greater should be avoided if possible to limit soil loss, erosion, excessive stormwater runoff, and the degradation of surface water.

Excessive grading should be avoided on all slopes, as should the flattening of hills and ridges. Steep slopes should be kept in an undisturbed natural condition to help stabilize hillsides and soils

On slopes greater than 25%, no development, regrading, or stripping of vegetation should be considered unless the disturbance is for roadway crossings or utility construction and it can be demonstrated that the roadway or utility improvements are absolutely necessary in the sloped area.

**Better Site Design Practice #5:**

## Minimize Siting on Porous or Erodible Soils

» **Description**: Porous soils such as sand and gravels provide an opportunity for groundwater recharge of stormwater runoff and should be preserved as a potential stormwater management option. Unstable or easily erodible soils should be avoided due to their greater erosion potential.

### KEY BENEFITS

- Areas with highly permeable soils can be used as nonstructural stormwater infiltration zones.
- Avoiding high erodible or unstable soils can prevent erosion and sedimentation problems and water quality degradation

### USING THIS PRACTICE

- Use soil surveys to determine site soil types
- Leave areas of porous or highly erodible soils as undisturbed conservation areas

**Discussion**

Infiltration of stormwater into the soil reduces both the volume and peak discharge of runoff from a given rainfall event, and also provides for water quality treatment and groundwater recharge. Soils with maximum permeabilities (hydrologic soil group A and B soils such as sands and sandy loams) allow for the most infiltration of runoff into the subsoil. Thus, areas of a site with these soils should be conserved as much as possible and these areas should ideally be incorporated into undisturbed natural or open space areas. Conversely, buildings and other impervious surfaces should be located on those portions of the site with the least permeable soils.

Similarly, areas on a site with highly erodible or unstable soils should be avoided for land disturbing activities and buildings to prevent erosion and sedimentation problems as well as potential future structural problems. These areas should be left in an undisturbed and vegetated condition.

Soils on a development site should be mapped in order to preserve areas with porous soils, and to identify those areas with unstable or erodible soils as shown in **Figure 2.3.2-7**. Soil surveys can provide a considerable amount of information relating to all relevant aspects of soils. Appendix D of this Manual provides permeability, shrink-swell potential and hydrologic soils group information for all Georgia soil series. General soil types should be delineated on concept site plans to guide site layout and the placement of buildings and impervious surfaces.

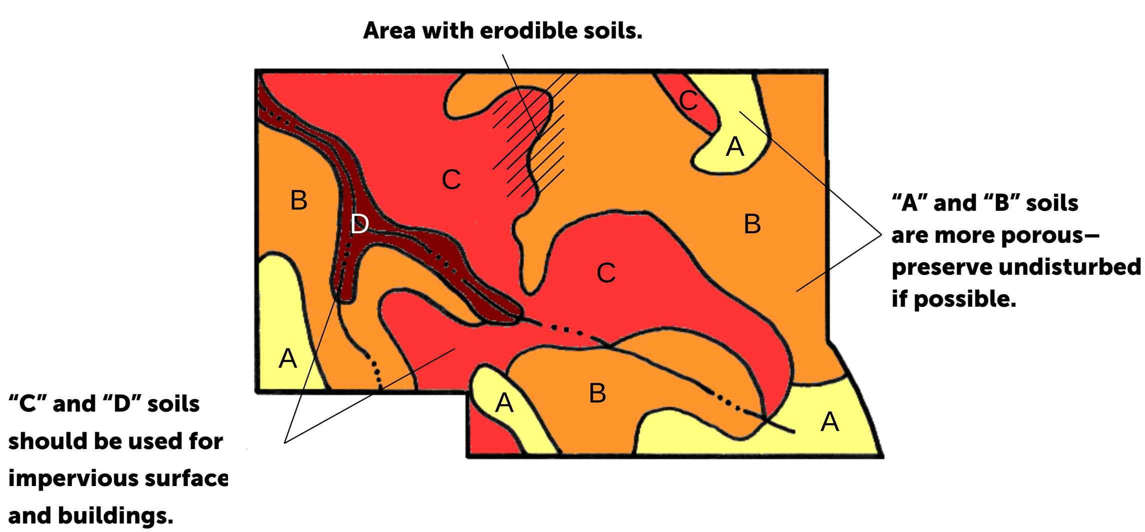



**Figure 2.3.2-7** Soil Mapping Information Can Be Used to Guide Development

## 2.3.2.2 LOWER IMPACT SITE DESIGN TECHNIQUES

After a site analysis has been performed and conservation areas have been delineated, there are numerous opportunities in the site design and layout phase to reduce both water quantity and quality impacts of stormwater runoff. These primarily deal with the location and configuration of impervious surfaces or structures on the site and include the following practices and techniques covered over the next several pages:

- Fit the Design to the Terrain
- Locate Development in Less Sensitive Areas
- Reduce Limits of Clearing and Grading
- Utilize Open Space Development
- Consider Creative Development Design

The goal of lower impact site design techniques is to lay out the elements of the development project in such a way that the site design (i.e. placement of buildings, parking, streets and driveways, lawns, undisturbed vegetation, buffers, etc.) is optimized for effective stormwater management. That is, the site design takes advantage of the site's natural features, including those placed in conservation areas, as well as any site constraints and opportunities (topography, soils, natural vegetation, floodplains, shallow bedrock, high water table, etc.) to prevent both on-site and downstream stormwater impacts.

**Figure 2.3.2-8** shows a development that has utilized several lower impact site design techniques in its overall layout and design.

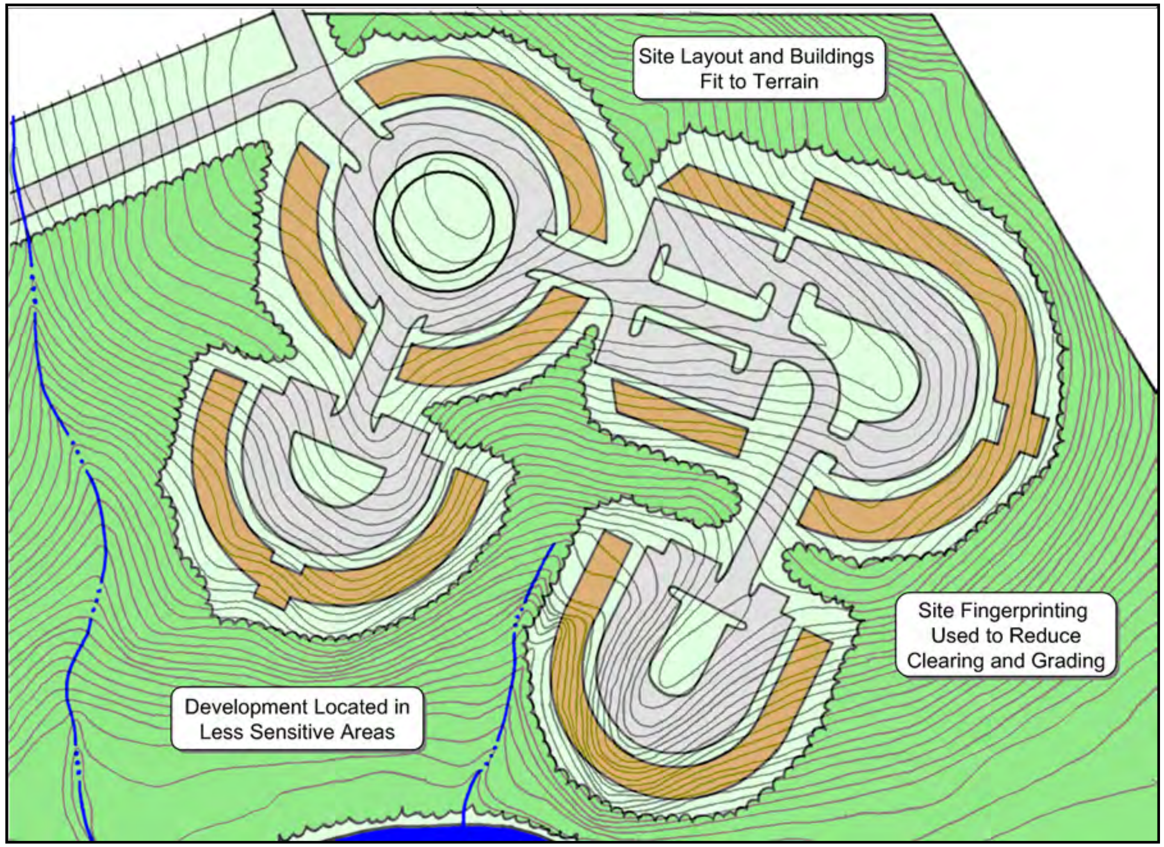



**Figure 2.3.2-8** Development Design Utilizing Several Lower Impact Site Design Techniques

BACK TO TOC

Better Site Design Practice #6:

## Fit Design to the Terrain

» **Description**: The layout of roadways and buildings on a site should generally conform to the landforms on a site. Natural drainageways and stream buffer areas should be preserved by designing road layouts around them. Buildings should be sited to utilize the natural grading and drainage system and avoid the unnecessary disturbance of vegetation and soils.

### KEY BENEFITS

- Helps to preserve the natural hydrology and drainageways of a site
- Reduces the need for grading and land disturbance
- Provides a framework for site design and layout

### USING THIS PRACTICE

- Develop roadway patterns to fit the site terrain. Locate buildings and impervious surfaces away from steep slopes, drainageways and floodplains

**Discussion**

All site layouts should be designed to conform with or "fit" the natural landforms and topography of a site. This helps to preserve the natural hydrology and drainageways on the site, as well as reduces the need for grading and disturbance of vegetation and soils. **Figure 2.3.2-9** illustrates the placement of roads and homes in a residential development.

Roadway patterns on a site should be chosen to provide access schemes which match the terrain. In rolling or hilly terrain, streets should be designed to follow natural contours to reduce clearing and grading. Street hierarchies with local streets branching from collectors in short loops and cul-de-sacs along ridgelines help to prevent the crossing of streams and drainageways as shown in **Figure 2.3.2-10**.

In flatter areas, a traditional grid pattern of streets or "fluid" grids which bend and may be interrupted by natural drainageways may be more appropriate (see **Figure 2.3.2-11**). In either case, buildings and impervious surfaces should be kept off of steep slopes, away from natural drainageways, and out of floodplains and other lower lying areas. In addition, the major axis of buildings should be oriented parallel to existing contours.

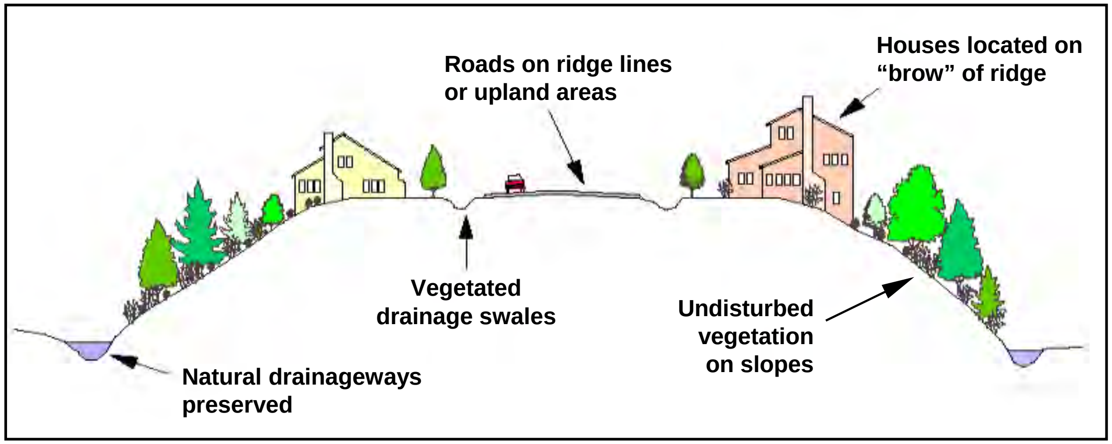
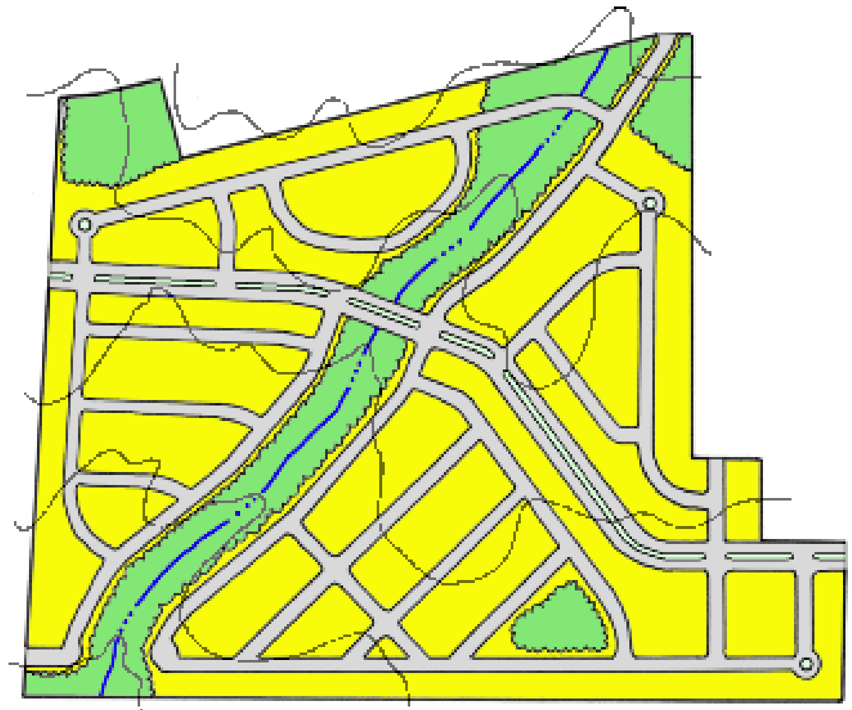



**Figure 2.3.2-9** Preserving the Natural Topography of the Site
(Adapted from Sykes, 1989)

**Figure 2.3.2-10** Subdivision Design for Hilly or Steep Terrain Utilizes Branching Streets From Collectors that Preserves Natural Drainageways and Stream Corridors



**2.3.2-11** A Subdivision Design for Flat Terrain Uses a Fluid Grid Layout that is Interrupted by the Stream Corridor



BACK TO TOC

**Better Site Design Practice #7:**
## Locate Development in Less Sensitive Areas

» **Description**: To minimize the hydrologic impacts on the existing site land cover, the area of development should be located in areas of the site that are less sensitive to disturbance or have a lower value in terms of hydrologic function.

### KEY BENEFITS

- Helps to preserve the natural hydrology and drainageways of a site
- Makes most efficient use of natural site features for preventing and mitigating stormwater impacts
- Provides a framework for site design and layout

### USING THIS PRACTICE

- Lay out the site design to minimize the hydrologic impact of structures and impervious surfaces

### Discussion

In much the same way that a development should be designed to conform to terrain of the site, a site layout should also be designed so that the areas of development are placed in the locations of the site that minimize the hydrologic impact of the project. This is accomplished by steering development to areas of the site that are less sensitive to land disturbance or have a lower value in terms of hydrologic function using the following methods:

- Locate buildings and impervious surfaces away from stream corridors, wetlands and natural drainageways. Use buffers to preserve and protect riparian areas and corridors.
- Areas of the site with porous soils should left in an undisturbed condition and/or used as stormwater runoff infiltration zones. Buildings and impervious surfaces should be located in areas with less permeable soils.
- Avoid land disturbing activities or construction on areas with steep slopes or unstable soils.
- Minimize the clearing of areas with dense tree canopy or thick vegetation, and ideally preserve them as natural conservation areas
- Ensure that natural drainageways and flow paths are preserved, where possible. Avoid the filling or grading of natural depressions and ponding areas.

**Figure 2.3.2-12** shows a development site where the natural features have been mapped in order to delineate the hydrologically sensitive areas. Through careful site planning, sensitive areas can be set aside as natural open space areas (see Better Site Design Practice #9). In many cases, such areas can be used as buffer spaces between land uses on the site or between adjacent sites. contours.

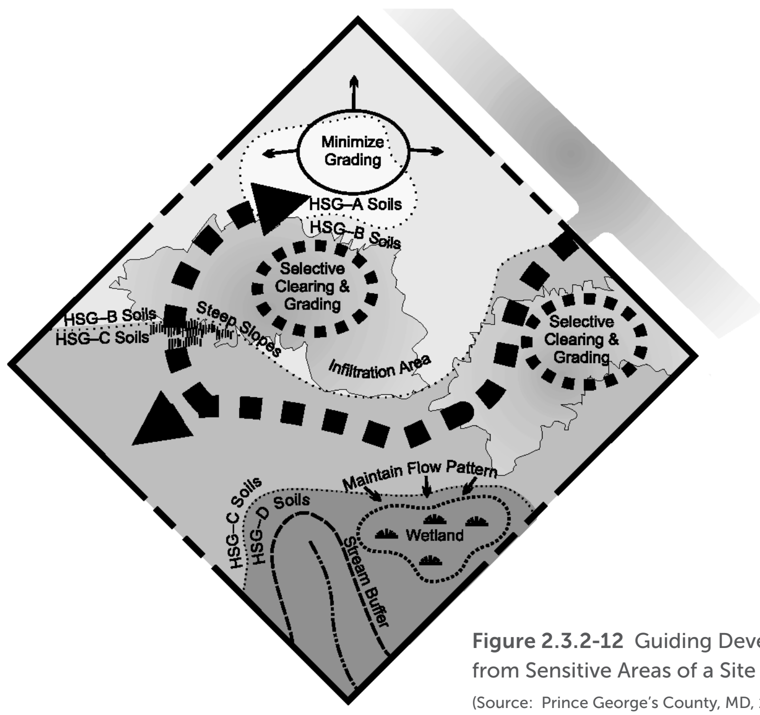



**Figure 2.3.2-12** Guiding Development Away from Sensitive Areas of a Site
(Source: Prince George's County, MD, 1999)

BACK TO TOC

**Better Site Design Practice #8:**
## Reduce Limits of Clearing and Grading

» **Description**: Clearing and grading of the site should be limited to the minimum amount needed for the development and road access. Site footprinting should be used to disturb the smallest possible land area on a site.

### KEY BENEFITS

- Preserves more undisturbed natural areas on a development site
- Techniques can be used to help protect natural conservation areas and other site features

### USING THIS PRACTICE

- Establish limits of disturbance for all development activities
- Use site footprinting to minimize clearing and land disturbance

**Discussion**

Minimal disturbance methods should be used to limit the amount of clearing and grading that takes place on a development site, preserving more of the undisturbed vegetation and natural hydrology of a site. These methods include:

- Establishing a limit of disturbance (LOD) based on maximum disturbance zone radii/lengths. These maximum distances should reflect reasonable construction techniques and equipment needs together with the physical situation of the development site such as slopes or soils. LOD distances may vary by type of development, size of lot or site, and by the specific development feature involved.
- Using site "footprinting" which maps all of the limits of disturbance to identify the smallest possible land area on a site which requires clearing or land disturbance. Examples of site footprinting is illustrated in **Figures 2.3.2-13** and **2.3.2-14**.
- Fitting the site design to the terrain.
- Using special procedures and equipment which reduce land disturbance.

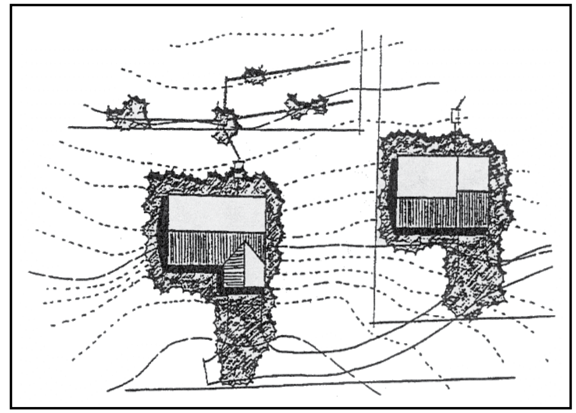


**Figure 2.3.2-13**
Establishing Limits of Clearing
(Source: DDNREC, 1997)

**Figure 2.3.2-14** Example of Site Footprinting

BACK TO TOC

**Better Site Design Practice #9:**
## Utilize Open Space Development

» **Description**: Open space site designs incorporate smaller lot sizes to reduce overall impervious cover while providing more undisturbed open space and protection of water resources.

### KEY BENEFITS

- Preserves conservation areas on a development site
- Can be used to preserve natural hydrology and drainageways
- Can be used to help protect natural conservation areas and other site features
- Reduces the need for grading and land disturbance
- Reduces infrastructure needs and overall development costs

### USING THIS PRACTICE

- Use a site design which concentrates development and preserves open space and natural areas of the site

**Discussion**

Open space development, also known as *conservation development* or *clustering*, is a better site design technique that concentrates structures and impervious surfaces in a compact area in one portion of the development site in exchange for providing open space and natural areas elsewhere on the site. Typically smaller lots and/or nontraditional lot designs are used to cluster development and create more conservation areas on the site.

Open space developments have many benefits compared with conventional commercial developments or residential subdivisions: they can reduce impervious cover, stormwater pollution, construction costs, and the need for grading and landscaping, while providing for the conservation of natural areas. **Figures 2.3.2-15** and **2.3.2-16** show examples of open space developments.

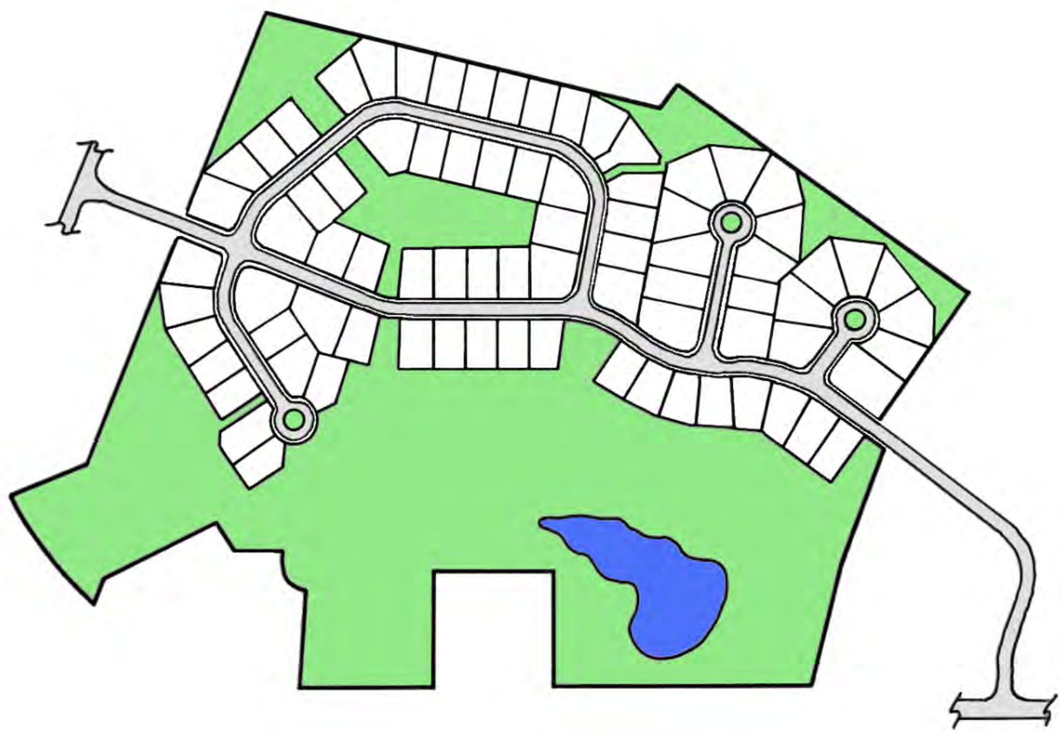


**Figure 2.3.2-15** Open Space Subdivision Site Design Example



BACK TO TOC

Along with reduced imperviousness, open space designs provide a host of other environmental benefits lacking in most conventional designs. These developments reduce potential pressure to encroach on conservation and buffer areas because enough open space is usually reserved to accommodate these protection areas. As less land is cleared during the construction process, alteration of the natural hydrology and the potential for soil erosion are also greatly diminished. Perhaps most importantly, open space design reserves 25 to 50 percent of the development site in conservation areas that would not otherwise be protected. Open space developments can also be significantly less expensive to build than conventional projects. Most of the cost savings are due to reduced infrastructure cost for roads and stormwater management controls and conveyances. While open space developments are frequently less expensive to build, developers find that these properties often command higher prices than those in more conventional developments.

**Figure 2.3.2-16** Aerial View of an Open Space Subdivision

Several studies estimate that residential properties in open space developments garner premiums that are higher than conventional subdivisions and moreover, sell or lease at an increased rate.

Once established, common open space and natural conservation areas must be managed by a responsible party able to maintain the areas in a natural state in perpetuity. Typically, the conservation areas are protected by legally enforceable deed restrictions, conservation easements, and maintenance agreements.

**Better Site Design Practice #10:**

## Consider Creative Development Design

» **Description**: Planned Unit Developments (PUDs) allow a developer or site designer the flexibility to design a residential, commercial, industrial, or mixed-use development in a fashion that best promotes effective stormwater management and the protection of environmentally sensitive areas.

### KEY BENEFITS

- Allows flexibility to developers to implement creative site designs which include stormwater better site design practices
- May be useful for implementing an open space development

### USING THIS PRACTICE

- Check with your local review authority to determine if the community supports PUDs
- Determine the type and nature of deviations allowed and other criteria for receiving PUD approval

**Discussion**

A Planned Unit Development (PUD) is a type of planning approval available in some communities which provides greater design flexibility by allowing deviations from the typical development standards required by the local zoning code with additional variances or zoning hearings.

The intent is to encourage better designed projects through the relaxation of some development requirements, in exchange for providing greater benefits to the community. PUDs can be used to implement many of the other stormwater better site design practices covered in this Manual and to create site designs that maximize natural nonstructural approaches to stormwater management.

Examples of the types of zoning deviations which are often allowed through a PUD process include:

- Allowing uses not listed as permitted, conditional or accessory by the zoning district in which the property is located
- Modifying lot size and width requirements
- Reducing building setbacks and frontages from property lines
- Altering parking requirements
- Increasing building height limits

A developer or site designer should consult their local review authority to determine whether the community supports PUD approvals. If so, the type and nature of deviations allowed from individual development requirements should be obtained from the review authority in addition to any other criteria that must be met to obtain a PUD approval.

BACK TO TOC

## 2.3.3 Site Design Stormwater Credits

### 2.3.3.1 INTRODUCTION

Non-structural stormwater control practices are increasingly recognized as a critical feature in every site design. As such, a set of stormwater "credits" has been developed to provide developers and site designers an incentive to implement better site design practices that can reduce the volume of stormwater runoff and minimize the pollutant loads from a site. The credit system directly translates into cost savings to the developer by reducing the size of best management practices and conveyance facilities.

The basic premise of the credit system is to recognize the water quality benefits of certain site design practices by allowing for a reduction in the water quality treatment volume ($WQ_v$). If a developer incorporates one or more of the credited practices in the design of the site, the requirement for capture and treatment of the water quality volume will be reduced.

The Natural Area Conservation better site design practice provides a stormwater credit, given certain site-specific conditions are met. For example, natural area conservation credits **must** be protected by a conservation easement. Natural conservation areas are defined as undisturbed natural areas that are conserved on a site, thereby retaining their pre-development hydrologic and water quality characteristics.

It should be noted that better site design practices and techniques that reduce the overall impervious area on a site already implicitly reduce the total amount of stormwater runoff generated by a site (and thus reduce $WQ_v$) and are not further credited under this system.

For each potential credit, there is a minimum set of criteria and requirements which identify the conditions or circumstances under which the credit may be applied. The intent of the suggested numeric conditions (e.g., flow length, contributing area, etc.) is to avoid situations that could lead to a credit being granted without the corresponding reduction in pollution attributable to an effective site design modification.

Site designers are encouraged to utilize credits on a site. Greater reductions in stormwater storage volumes can be achieved when many credits are combined (e.g., disconnecting rooftops and protecting natural conservation areas). However, credits cannot be claimed twice for an identical area of the site (e.g. claiming credit for natural conservation areas and disconnecting rooftops over the same site area).

Due to local safety codes, soil conditions, and topography, some of these site design credits may be restricted. Designers are encouraged to consult with the appropriate approval authority to ensure if and when a credit is applicable and to determine restrictions on non-structural strategies.

### 2.3.3.2 STORMWATER CREDITS AND THE SITE PLANNING PROCESS

During the site planning process described in Section 2.4 there are several steps involved in site layout and design, each more clearly defining the location and function of the various components of the stormwater management system. The integration of site design credits can be integrated with this process as shown in **Table 2.3.3-2**.

BACK TO TOC

Table 2.3.3-2  Integration of Site Design Credits with Site Development Process

| Site Development Phase | Site Design Credit Activity |
|---|---|
| Feasibility Study | • Determine stormwater management requirements<br>• Perform site reconnaissance to identify potential areas for and types of credits |
| Site Analysis | • Identify and delineate natural feature conservation areas (natural areas and stream buffers) |
| Concept Plan | • Preserve natural areas and stream buffers during site layout<br>• Reduce impervious surface area through various techniques<br>• Identify locations for use of vegetated channels and groundwater re-charge<br>• Look for areas to disconnect impervious surfaces<br>• Document the use of site design credits. |
| Preliminary and Final Plan | • Perform layout and design of credit areas – integrating them into treat-ment trains<br>• Ensure unified stormwater sizing criteria are satisfied<br>• Ensure appropriate documentation of site design credits according to local requirements. |
| Construction | • Ensure protection of key areas<br>• Ensure correct final construction of areas needed for credits |
| Final Inspection | • Develop maintenance requirements and documents<br>• Ensure long term protection and maintenance<br>• Ensure credit areas are identified on final plan and plat if applicable |

### 2.3.3.3 SITE DESIGN CREDIT - NATURAL AREA CONSERVATION

A stormwater credit can be taken when undisturbed natural areas are conserved on a site, thereby retaining their pre-development hydrologic and water quality characteristics. Under this credit, a designer would be able to subtract conservation areas from total site area when computing water quality volume requirements. An added benefit will be that the post-development peak discharges will be smaller, and hence water quantity control volumes ($CP_v$, $Q_{p25}$, and $Q_f$) will be reduced due to lower post-development curve numbers or rational formula "C" values.

*Rule: Subtract conservation areas from total site area when computing water quality volume requirements.*

**Criteria**:

- Conservation area cannot be disturbed during project construction
- Conservation area must remain undisturbed (if disturbed in the future, the credit is no longer valid and additional water quality measures will be required)
- Shall be protected by limits of disturbance clearly shown on all construction drawings
- Shall be located within an acceptable conservation easement instrument that ensures perpetual protection of the proposed area. The easement must clearly specify how the natural area vegetation shall be managed and boundaries will be marked

  *[Note: managed turf (e.g., playgrounds, regularly maintained open areas) is not an acceptable form of vegetation management]*
- Shall have a minimum contiguous area requirement of 10,000 square feet
- $R_v$ is kept constant when calculating $WQ_v$

**EXAMPLE**

**<u>Residential Subdivision</u>**

**Area** = 38 acres

**Natural Conservation Area** = 7 acres

**Impervious Area** = 13.8 acres

$$R_v = 0.05 + 0.009\,(I) = 0.05 + 0.009\,(36.3\%) = 0.37$$

*Credit*:

7.0 acres in natural conservation area

New drainage area = 38 – 7 = 31 acres

*Before credit*:

$$WQ_v = (1.2)(0.37)(38)/12 = 1.40 \text{ ac-ft}$$

*With credit*:

$$WQ_v = (1.2)(0.37)(31)/12 = 1.15 \text{ ac-ft}$$

**(18% reduction in water quality volume)**



BACK TO TOC

# 2.4 Stormwater Site Planning & Design

## 2.4.1 Stormwater Management and Site Planning

### 2.4.1.1 INTRODUCTION

In order to most effectively address stormwater management objectives, consideration of storm-water runoff needs to be fully integrated into the site planning and design process. This involves a more comprehensive approach to site planning and a thorough understanding of the physical characteristics and resources of the site. The purpose of this section is to provide a frame-work for including effective and environmentally sensitive stormwater management into the site development process and to encourage a greater uniformity in stormwater management site plan preparation.

When designing the stormwater management system for a site, a number of questions need to be answered by the site planners and design engineers, including:

- How can the stormwater management system be designed to most effectively meet the stormwater management recommended standards (and any additional needs or objectives)?
- What are the opportunities for utilizing better site design practices to minimize the need for best management practices?
- What are the development site constraints that preclude the use of certain best management practices?
- What best management practices are most suitable and cost-effective for the site?

### 2.4.1.2 PRINCIPLES OF STORMWATER MAN-AGEMENT SITE PLANNING

The following principles should be kept in mind in preparing a stormwater management plan for a development site:

1. **The site design should utilize an integrated approach to deal with stormwater quantity, quality and streambank (channel) protection requirements.**

   The stormwater management infrastructure for a site should be designed to integrate drainage and water quantity control, water quality protection, and downstream channel protection. Site design should be done in unison with the design and layout of stormwater infrastructure to attain stormwater management goals. Together, the combination of better site design practices and effective infrastructure layout and design can mitigate the worst stormwater impacts of most urban developments while preserving stream integrity and aesthetic attractiveness.

2. **Stormwater management practices should strive to utilize the natural drainage system and require as little maintenance as possible.**

   Almost all sites contain natural features which can be used to help manage and mitigate runoff from development. Features on a development site might include natural drainage patterns, depressions, permeable soils, wetlands, floodplains, and undisturbed vegetated areas that can be used to reduce runoff, provide infiltration and stormwater filtering of pollutants and sediment, recycle nutrients, and maximize on-site storage of stormwater. Site design should seek to improve the effectiveness of natural systems rather than to ignore or replace them. Further, natural systems typically require low or no maintenance, and will continue to function many years into the future.

3. **Best management practices should be implemented only after all site design and nonstructural options have been exhausted.**

   Operationally, economically, and aesthetically, stormwater better site design and the use of natural techniques offer significant benefits over best management practices. Therefore, all opportunities for utilizing these methods should be explored before implementing best management practices such as wet ponds and sand filters.

4. **Structural stormwater solutions should attempt to be multi-purpose and be aesthetically integrated into a site's design.**

. A structural stormwater facility need not be an afterthought or ugly nuisance on a development site. A parking lot, soccer field or city plaza can serve as a temporary storage facility for stormwater. In addition, water features such as ponds and lakes, when correctly designed and integrated into a site, can increase the aesthetic value of a development.

5. **"One size does not fit all" in terms of stormwater management solutions.**

   Although the basic problems of stormwater runoff and the need for its management remain the same, each site, project, and watershed presents different challenges and opportunities. For instance, an infill development in a highly urbanized town center or downtown area will require a much different set of stormwater management solutions than a low-density residential subdivision in a largely undeveloped watershed. Therefore, local stormwater management needs to take into account differences between development sites, different types of development and land use, various watershed conditions and priorities, the nature of downstream lands and waters, and community desires and preferences.

## 2.4.2 Preparation of Stormwater Management Site Plans

### 2.4.2.1 INTRODUCTION

A stormwater management site plan is a comprehensive report that contains the technical information and analysis to allow a local review authority to determine whether a proposed new development or redevelopment project meets the local stormwater regulatory requirements and/or the recommended stormwater management standards contained in this Manual. See Figure 2.2.5-2 for information on the stormwater design process.

This section describes the typical contents and general procedure for preparing a stormwater management site plan. The level of detail involved in the plan will depend on the project size and the individual site and development characteristics.

The preparation of a stormwater site plan ideally follows these steps: '

**(Step 1)** Pre-consultation Meeting and Joint Site Visit
**(Step 2)** Review of Local Requirements
**(Step 3)** Perform Site Analysis
**(Step 4)** Prepare Stormwater Concept Plan
**(Step 5)** Prepare Preliminary Stormwater Site Plan
**(Step 6)** Complete Final Stormwater Site Plan

### 2.4.2.2 PRE-CONSULTATION MEETING AND JOINT SITE VISIT

The most important action that can take place at the beginning of the development project is a pre-consultation meeting between the local review authority and the developer and their team to outline the stormwater management requirements and other regulations, and to assist the developer in assessing constraints, opportunities, and potential for stormwater design concepts utilizing low impact development and runoff reduction techniques.

This recommended step helps to establish a constructive partnership for the entire development process. A joint site visit, if possible, can yield a conceptual outline of the stormwater management plan and strategies. By walking the site, the two parties can identify and anticipate problems, define general expectations and establish general boundaries of natural feature protection and conservation areas. A major incentive for pre-consultation is that permitting and plan approval requirements will become clear at an early stage, increasing the likelihood that the approval process will proceed faster and more smoothly.

Prior to the joint site visit, it is helpful for the developer or the developer's design team to submit the following information for familiarity and review:

- Existing conditions plan (boundary information, topography, site features, etc.)
- Proposed site plan (proposed site features)
- Soil characteristics, infiltration rates, etc. (if available)
- Natural resources inventory
- Conceptual stormwater management plans

#### 2.4.2.3 REVIEW OF LOCAL REQUIREMENTS & PERMITTING GUIDANCE

The site developer should be made familiar with the local stormwater management and development requirements and design criteria that apply to the site. These requirements may include:

- The recommended standards for stormwater management included in this Manual (see Section 2.2)
- Design storm frequencies
- Conveyance design criteria
- Floodplain criteria
- Buffer/setback criteria
- Wetland provisions
- Watershed-based criteria
- Erosion and sedimentation control requirements
- Maintenance requirements
- Need for physical site evaluations (infiltration tests, geotechnical evaluations, etc.)

Much of this guidance can be obtained at the pre-consultation meeting with the local review authority and should be detailed in various local ordinances (e.g., subdivision codes, stormwater and drainage codes, etc.)

Current land use plans, comprehensive plans, zoning ordinances, road and utility plans, watershed or overlay districts, and public facility plans should all be consulted to determine the need for compliance with other local and state regulatory requirements.

Opportunities for special types of development (e.g., clustering) or special land use opportunities (e.g., conservation easements or tax incentives) should be investigated. There may also be an ability to partner with a local community for the development of greenways, or other riparian corridor or open space developments.

The checklist on the following page (**Table 2.4.2-1**) provides a condensed summary of current restrictions as they relate to common site features that may be regulated under local, state or federal law. These restrictions fall into one of three general categories:

- Locating a best management practice within an area that is expressly prohibited by law.
- Locating a best management practice within an area that is strongly discouraged, and is only allowed on a case by case basis. Local, state and/or federal permits shall be obtained, and the applicant will need to supply additional documentation to justify locating the stormwater practice within the regulated area.
- Best management practices must be setback a fixed distance from the site feature.

This checklist is only intended as a general guide to location and permitting requirements as they relate to siting of best management practices. Consultation with the appropriate regulatory agency is the **best** strategy.

BACK TO TOC

Table 2.4.2-1 Location and Permitting Checklist

| Site Feature | Location and Permitting Guidance |
|---|---|
| **Jurisdictional Wetland (Waters of the U.S)**<br>U.S. Army Corps of Engineers Section 404 Permit | • Jurisdictional wetlands should be delineated prior to siting structural practice.<br>• Use of natural wetlands for stormwater quality treatment is contrary to the goals of the Clean Water Act and should be avoided.<br>• Stormwater should be treated prior to discharge into a natural wetland.<br>• Structural practices may also be restricted in local buffer zones, although they may be utilized as a non-structural filter strip (i.e., accept sheet flow).<br>• Should justify that no practical upland treatment alternatives exist.<br>• Where practical, excess stormwater flows should be conveyed away from jurisdictional wetlands. |
| **Stream Channel (Waters of the U.S)**<br>U.S. Army Corps of Engineers Section 404 Permit | • All Waters of the U.S. (streams, ponds, lakes, etc.) should be delineated prior to design.<br>• Use of any Waters of the U.S. for stormwater quality treatment is contrary to the goals of the Clean Water Act and should be avoided.<br>• Stormwater should be treated prior to discharge into Waters of the U.S.<br>• In-stream ponds for stormwater quality treatment are highly discouraged.<br>• Must justify that no practical upland treatment alternatives exist.<br>• Temporary runoff storage preferred over permanent pools.<br>• Implement measures that reduce downstream warming. |
| **Georgia Planning Act**<br>Groundwater Recharge Areas | • Prevention of groundwater contamination<br>• Covers about 23% of State. Detailed mapping available at Regional Commissions of the Department of Community Affairs.<br>• Permanent stormwater infiltration devices are prohibited in areas having high pollution susceptibility. |
| **Georgia Planning Act**<br>Water Supply Watersheds | • Specific stream and reservoir buffer requirements.<br>• May be imperviousness limitations<br>• May be specific best management practice requirements. |
| **100 Year Floodplain**<br>Local Stormwater Review Authority | • Grading and fill for structural practice construction is generally discouraged within the ultimate 100 year floodplain, as delineated by FEMA flood insurance rate maps, FEMA flood boundary and floodway maps, or more stringent local floodplain maps.<br>• Floodplain fill cannot raise the floodplain water surface elevation by more than a tenth of a foot. |
| **Stream Buffer**<br><br>Check with appropriate review authority whether stream buffers are required | • Consult local authority for stormwater policy.<br>• Structural practices are discouraged in the streamside zone (within 25 feet or more of streambank, depending on the specific regulations).<br>• There are specific additional requirements related to River Corridor Protection, the Metropolitan River Protection Act, the Metropolitan North Georgia Water Planning District, and the Georgia Scenic Rivers Act (which include wider and more stringent buffers). |
| **Utilities**<br>Local Review Authority | • Call appropriate agency to locate existing utilities prior to design.<br>• Note the location of proposed utilities to serve development.<br>• Structural practices are discouraged within utility easements or rights of way for public or private utilities. |
| **Roads**<br>Local DOT, DPW, or State DOT | • Consult local DOT or DPW for any setback requirement from local roads.<br>• Consult DOT for setbacks from State maintained roads.<br>• Approval must also be obtained for any stormwater discharges to a local or state-owned conveyance channel. |
| **Structures**<br>Local Review Authority | • Consult local review authority for structural practice setbacks from structures. |
| **Septic Drain fields**<br>Local Health Authority | • Consult local health authority.<br>• Recommended setback is a minimum of 50 feet from drain field edge. |
| **Water Wells**<br>Local Health Authority | • 100-foot setback for stormwater infiltration.<br>• 50-foot setback for all other structural practices. |

### 2.4.2.4 PERFORM SITE ANALYSIS AND INVENTORY

Using approved field and mapping techniques, the site engineer should collect and review information on the existing site conditions and map the following site features:

- Topography
- Drainage patterns and basins
- Intermittent and perennial streams
- Soils
- Ground cover and vegetation
- Existing development
- Existing stormwater facilities
- Adjacent areas

In addition, the site engineer should identify and map all previously unmapped natural features such as:

- Wetlands
- Critical habitat areas
- Boundaries of wooded areas
- Floodplain boundaries
- Steep slopes
- Required buffers
- Proposed stream crossing locations
- Other required protection areas (e.g., well setbacks)

Some of this information may be available from previously performed studies or from the previous feasibility study. For example, if a development site requires a permit under the Erosion and Sedimentation Act, most of the resource protection features will likely have been mapped as part of the land disturbance activity plan. Other recommended site information to map or obtain includes utilities information, seasonal groundwater levels, and geologic mapping.

Individual map or geographic information system (GIS) layers can be designed to facilitate an analysis of the site through what is known as map overlay, or a composite analysis. Each layer (or group of related information layers) is placed on the map in such a way as to facilitate comparison and contrast with other layers. A composite layer is often developed to show all the layers at the same time (see **Figure 2.4.2-1**). This composite layer can be a useful tool for defining the best buildable areas and delineating and preserving natural feature conservation areas.

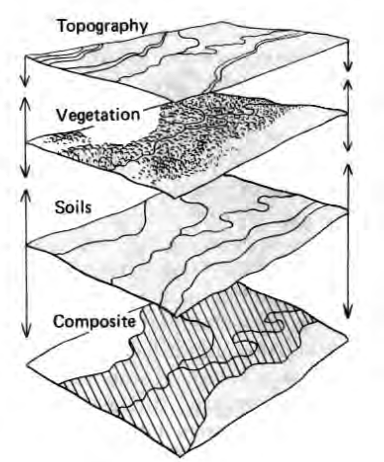



**Figure 2.4.2-1** Composite Analysis
(Source: Marsh, 1983)

### 2.4.2.5 PREPARE STORMWATER CONCEPT PLAN

Based upon the review of existing conditions and site analysis, the design engineer should develop a concept site layout plan for the project.

During the concept plan stage the site designer will perform most of the layout of the site including the preliminary stormwater management system design and layout. The stormwater concept plan allows the design engineer to propose a potential site layout and gives the developer and local review authority a "first look" at the stormwater management system for the proposed development. The stormwater concept plan should be submitted to the local plan reviewer before detailed preliminary site plans are developed.

The following steps should be followed in developing the stormwater concept plan:

(Step 1) Use better site design approaches (see Section 2.3) as applicable to develop the site layout, including:

- Preserving the natural feature conservation areas defined in the site analysis
- Fitting the development to the terrain and minimizing land disturbance
- Reducing impervious surface area through various techniques
- Preserving and utilizing the natural drainage system wherever possible

(Step 2) Calculate preliminary estimates of the unified stormwater sizing criteria requirements for water quality, channel protection, overbank flooding protection and extreme flood protection based on the concept plan site layout (Section 2.2)

(Step 3) Determine the site design stormwater credits to be accounted for in the design of best management practices handling the water quality volume (Section 2.3)

(Step 4) Perform screening and preliminary selection of appropriate best management practices and identification of potential siting locations (Section 4.1).

It is extremely important at this stage that stormwater design is integrated into the overall site design concept in order to best reduce the impacts of the development as well as provide for the most cost-effective and environmentally sensitive approach. Using hydrology calculations, the goal of mimicking pre-development (natural or existing condition, as applicable) conditions can serve a useful purpose in planning the stormwater management system.

For local review purposes, the stormwater concept plan should include the following elements:

1. **Common address and legal description of site**

2. **Vicinity map**

3. **Existing conditions and proposed site layout mapping and plans (recommended scale of 1" = 50'), which illustrate at a minimum:**
   - Existing and proposed topography (minimum of 2-foot contours recommended)
   - Perennial and intermittent streams
   - Mapping of predominant soils from USDA soil surveys
   - Boundaries of existing predominant vegetation and proposed limits of clearing and grading
   - Location and boundaries of other natural feature protection and conservation areas such as wetlands, lakes, ponds, floodplains, stream buffers and other setbacks (e.g., drinking water well setbacks, septic setbacks, etc.)
   - Location of existing and proposed roads, buildings, parking areas and other impervious surfaces
   - Existing and proposed utilities (e.g., water, sewer, gas, electric) and easements
   - Preliminary estimates of unified stormwater sizing criteria requirements
   - Identification and calculation of stormwater site design credits
   - Preliminary selection and location, size, and limits of disturbance of proposed best management practices
   - Location of existing and proposed conveyance systems such as grass channels, swales, and storm drains
   - Flow paths
   - Location of floodplain/floodway limits and relationship of site to upstream and downstream properties and drainages
   - Preliminary location and dimensions of proposed channel modifications, such as bridge or culvert crossings

4. **Identification of preliminary waiver requests**

#### 2.4.2.6 PREPARE PRELIMINARY STORMWATER SITE PLAN

The preliminary plan ensures that requirements and criteria are being complied with and that opportunities are being taken to minimize adverse impacts from the development.

The preliminary stormwater management site plan should consist of maps, narrative, and supporting design calculations (hydrologic and hydraulic) for the proposed stormwater management system, and should include the following sections:

1. **Natural Resources Inventory**
   - Natural Drainage Divides
   - Natural Drainage Features (e.g., swales, basins, depressional areas)
   - Wetlands
   - Water Bodies
   - Floodplains
   - Aquatic Buffers

BACK TO TOC

- Shellfish Harvesting Areas
- Soils
- Erodible Soils
- Steep Slopes (i.e., Areas with Slopes Greater Than 15%)
- Groundwater Recharge Areas
- Wellhead Protection Areas
- Trees and Other Existing Vegetation
- High Quality Habitat Areas

2. **Existing Conditions Hydrologic Analysis**
   - A topographic map of existing site conditions (minimum 2-foot contour interval recommended) with the basin boundaries indicated
   - Acreage, soil types, and land cover of areas for each sub-basin affected by the project
   - All perennial and intermittent streams and other surface water features
   - All existing stormwater conveyances and structural control facilities
   - Direction of flow and exits from the site
   - Analysis of runoff provided by off-site areas upstream of the project site
   - Infiltration rates of existing soils
   - Methodologies, assumptions, site parameters, and supporting design calculations used in analyzing the existing conditions and site hydrology

3. **Natural Conditions Hydrologic Analysis (where applicable)**
   - In communities where pre-development is defined as natural conditions rather than existing conditions, or where natural conditions are a more appropriate hydrologic standard, such as discharges to impaired streams or floodprone areas, a natural conditions hydrologic analysis will be necessary. The natural conditions hydrologic analysis should include all of the elements described for Existing Conditions Hydrologic Analysis above.
   - In some cases, the existing topography may not be representative of natural conditions, and the hydrologic analysis should be modified for leveling or grading that has occurred.
   - A set type of vegetative condition such as "woods in good condition" may be used in the natural conditions hydrologic analysis. The Georgia Stormwater Quality Site Development Review Tool is used to calculate these conditions and can be found at the following website: www.gastormwater.com

4. **Post-Development Hydrologic Analysis**
   - A topographic map of developed site conditions (minimum 2-foot contour interval recommended) with the post-development basin boundaries indicated
   - Total area of post-development impervious surfaces and other land cover areas for each sub-basin affected by the project
   - Unified stormwater sizing criteria runoff calculations for water quality, channel protection, overbank flooding protection, and extreme flood protection for each sub-basin
   - Location and boundaries of proposed natural feature protection areas
   - Documentation and calculations for any applicable site design credits that are being utilized
   - Methodologies, assumptions, site parameters and supporting design calculations used in analyzing the existing conditions site hydrology

5. **Stormwater Management System**
   - Drawing or sketch of the stormwater management system including the location of non-structural site design features and the placement of existing and proposed structural stormwater controls. This drawing should show design water surface elevations, storage volumes available from zero to maximum head, location of inlets and outlets, location of bypass and discharge systems, and all orifice/restrictor sizes.
   - Narrative describing that appropriate and effective structural stormwater controls have been selected
   - Cross-section and profile drawings and design details for each of the structural stormwater controls in the system. This should include supporting calculations to show that the facility is designed according to the applicable design criteria.

BACK TO TOC

- Hydrologic and hydraulic analysis of the stormwater management system for all applicable design storms (should include stage-storage or outlet rating curves, and inflow and outflow hydrographs)
- Documentation and supporting calculations to show that the stormwater management system adequately meets the unified stormwater sizing criteria
- Drawings, design calculations, and elevations for all existing and proposed stormwater conveyance elements including stormwater drains, pipes, culverts, catch basins, channels, swales, and areas of overland flow

6. **Downstream Analysis**
   - Supporting calculations for a downstream peak flow analysis using the ten-percent rule necessary to show safe passage of post-development design flows downstream
   - In some instances, the results from the downstream analysis may indicate a detention practice has an adverse impact on the watershed as a whole. In such cases, the local community may determine the detention practice is not warranted.

In calculating runoff volumes and discharge rates, consideration may need to be given to any planned future upstream land use changes. Depending on the site characteristics and given design criteria, upstream lands may need to be modeled as "existing condition" or "projected buildout/future condition" when sizing and designing on-site conveyances and stormwater practices.

#### 2.4.2.7 COMPLETE FINAL STORMWATER SITE PLAN

The final stormwater management site plan adds further detail to the preliminary plan and reflects changes that are requested or required by the local review authority. The final stormwater site plan should include all of the revised elements of the preliminary plan as well as the following items:

1. **Erosion and Sedimentation Control Plan**
   - Must contain all the elements specified in the Georgia Erosion and Sediment Control Act and local ordinances and regulations
   - Sequence/phasing of construction and temporary stabilization measures
   - Temporary structures that will be converted into permanent best management practices
2. **Landscaping Plan**
   - Arrangement of planted areas, natural areas and other landscaped features on the site plan
   - Information necessary to construct the landscaping elements shown on the plan drawings
   - Descriptions and standards for the methods, materials and vegetation that are to be used in the construction
3. **Operations and Maintenance Plan**
   - Description of maintenance tasks, responsible parties for maintenance, funding, access and safety issues
4. **Evidence of Acquisition of Applicable Local and Non-local Permits**
5. **Waiver Requests**

The completed final stormwater site plan should be submitted to the local review authority for final approval prior to any construction activities on the development site.

#### 2.4.2.8 OBTAIN NON-LOCAL PERMITS

The developer should obtain any applicable non-local environmental permit such as 404 wetland permits, 401 water quality certification, or construction NPDES permits prior to or in conjunction with final plan submittal. In some cases, a non-local permitting authority may impose conditions that require the original concept plan to be changed. Developers and engineers should be aware that permit acquisition can be a long, time-consuming process.

## 2.4.3 Stormwater Planning in the Development Process

### 2.4.3.1 GENERAL SITE DEVELOPMENT PROCESS

**Figure 2.4.3-1** depicts a typical site development process from the perspective of the land developer. After an initial site visit the developer assesses the feasibility of the project. If the project is deemed workable, a survey is completed. The design team prepares a concept plan (often called a sketch plan) for consultation with the local review authority. A preliminary plan is then prepared and submitted for necessary reviews and approvals. Federal, state and local permits are applied for at various stages in the process.

After review by the local authority and possible public hearings, necessary revisions are made and a final construction plan is prepared. There may be several iterations between plan submittal and plan approval. Bonds are set and placed, contractors are hired, and construction of the project takes place. During and after construction numerous types of inspections take place. At the end of construction, there is a final inspection and a use and occupancy permit is issued for the structure itself.

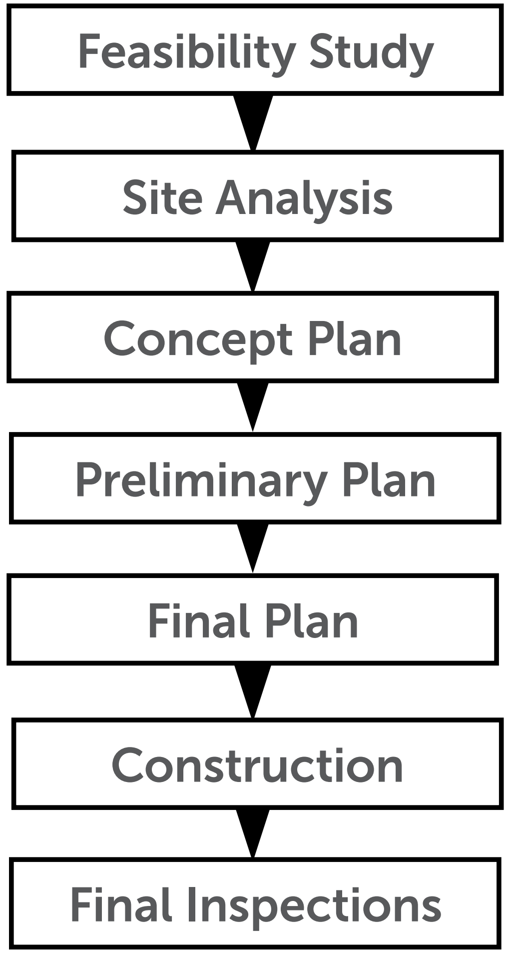



**Figure 2.4.3-1**
Typical Site Development Flowchart

### 2.4.3.2 STORMWATER SITE PLANNING AND DESIGN

Stormwater site planning and design is a subset of overall site development and must fit into the overall process if it is to be successful. **Table 2.4.3-1** on the next several pages shows how planning for the stormwater management system fits into the site development process from the perspective of the developer and site planner/engineer. For each step in the development process, the stormwater-related objectives are described, along with the key actions and major activities that are typically performed to meet those objectives.

Table 2.4.3-1  Stormwater Planning in the Site Development Process

| *Feasibility Study* |
| --- |
| Description:<br><br>A feasibility study is performed to determine the factors that may influence the decision to proceed with the site development, including the basic site characteristics, local and other governmental requirements, area information, surrounding developments, etc.<br><br>Stormwater-Related Objectives:<br><br>• Understand major site constraints and opportunities<br>• Understand local and other requirements<br><br>Key Actions:<br><br>• Initiate discussions with local review authority<br>• Pre-consultation between developer and plan reviewer<br>• Determine local stormwater management requirements<br><br>Major Activities:<br><br>• Base map development<br>• Review of project requirements<br>• Review of local development and stormwater management requirements<br>• Review of local stormwater master plans or comprehensive plans<br>• Joint site visit with local review authority<br>• Collection of secondary source information<br>• Determination of other factors or constraints impacting feasibility |

Table 2.4.3-1 Stormwater Planning in the Site Development Process (continued)

| *Site Analysis* |
|---|
| Description:<br><br>A site analysis is used to gain an understanding of the constraints and opportunities associated with the site through identification, mapping and assessment of natural features and resources. Potential conservation and resource protection areas are identified at this stage.<br><br>Stormwater-Related Objectives:<br><br>• Identify key site physical, environmental, and other significant resources<br>• Develop preliminary vision for stormwater management system<br><br>Key Actions:<br><br>• Site evaluation and delineation of natural feature protection areas<br><br>Major Activities:<br><br>• Mapping of natural resources: soils, vegetation, streams, topography, slope, wetlands, floodplains, aquifers<br>• Identification of other key cultural, historic, archaeological, or scenic features, orientation and exposure<br>• Identification of adjacent land uses<br>• Identification of natural feature protection and conservation areas<br>• Mapping of easements and utilities<br>• Integration of all layers – map overlay<br>• Other constraints and opportunities<br>• Identification of adjacent transportation and utility access |

BACK TO TOC

Table 2.4.3-1  Stormwater Planning in the Site Development Process (continued)

| *Concept Plan* |
| --- |
| Description:<br><br>A concept plan is used to provide both the developer and reviewer a preliminary look at the development and stormwater management concept.  Based on the site analysis, a concept plan should take into account the constraints and resources available on the site.  Several alternative "what if" concept plans can be created.<br><br>Stormwater-Related Objectives:<br><br>• Develop concept for stormwater management system<br>• Gain approval from developer and local review authority of concept plan<br><br>Key Actions:<br><br>• Develop site layout concept using better site design techniques where possible<br>• Perform initial runoff characterization based on site layout concept<br>• Determine necessary site design and/or best management practices needed to meet<br>• stormwater management requirements<br><br>Major Activities:<br><br>• Prepare sketches of functional land uses including conservation areas<br>• "What if" analysis of different design concepts<br>• Unified stormwater sizing criteria preliminary calculations<br>• Utilization of better site design concepts and crediting mechanisms in layout concept<br>• Preliminary selection and siting of best management practices<br>• Location of drainage/conveyance facilities |

## Table 2.4.3-1 Stormwater Planning in the Site Development Process (continued)

| *Preliminary and Final Plan* |
|---|
| Description:<br><br>A preliminary site plan is created for local review, which includes roadways, building and parking locations, conservation areas, utilities, and stormwater management facilities. Following local approval, a final set of construction plans are developed.<br><br>Stormwater-Related Objectives:<br><br>• Prepare preliminary and final stormwater management site plan<br>• Secure local and non-local permits<br><br>Key Actions:<br><br>• Perform runoff characterization based on preliminary/final site plan<br>• Design best management practices and conveyance systems<br>• Perform downstream analysis<br><br>Major Activities:<br><br>• Preliminary/final site layout plan<br>• Unified stormwater sizing criteria calculations<br>• Calculation of site design credit<br>• Selection, siting and design of best management practices<br>• Development of erosion and sedimentation control plan and landscaping plan<br>• Applications for needed permits and waivers<br>• Design of drainage and conveyance facilities |

BACK TO TOC

Table 2.4.3-1 Stormwater Planning in the Site Development Process (continued)

| *Construction* |
|---|
| Summary:<br><br>During the construction stage, the site must be inspected regularly to ensure that all elements are being built according to plan, and that all resource or conservation areas are suitably protected during construction.<br><br>Stormwater Objectives:<br><br>• Ensure that stormwater management facilities and site design practices are built as designed<br><br>Key Actions:<br><br>• Pre-construction meeting<br>• Inspection during construction<br><br>Major Activities:<br><br>• Execution of bonds<br>• Inspection during key phases or key installations<br>• Protection of best management practices<br>• Protection of conservation areas<br>• Erosion and sedimentation control<br>• Proper sequencing |

Table 2.4.3-1 Stormwater Planning in the Site Development Process (continued)

| *Final Inspection* |
| --- |
| Summary:<br><br>After construction, the site must be inspected to ensure that all elements are completed according to plan. Long-term maintenance agreements should be executed. |
| Stormwater Objectives:<br><br>• Ensure that stormwater management facilities and site design practices are built and operating as designed<br>• Ensure long-term maintenance of best management practices and conveyances<br>• Ensure long-term protection of conservation and resource protection areas |
| Key Actions:<br><br>• Final inspection and submission of record drawings<br>• Maintenance inspections |
| Major Activities:<br><br>• Final stabilization<br>• As-built survey<br>• Final inspection and use permit<br>• Execution of maintenance agreements |

## References

Center for Watershed Protection. 1998. *Better Site Design: A Handbook for Changing Development Rules in Your Community*. Center for Watershed Protection (CWP). Ellicott City, MD.

Center for Watershed Protection. 2000. *Maryland Stormwater Design Manual, Volumes I and II.* Center for Watershed Protection (CWP). Ellicott City, MD.

Delaware Department of Natural Resources and Environmental Control, 1997. *Conservation Design for Stormwater Management*. Prepared by the Brandywine Conservancy.

Department of Environmental Resources. 1999. Low-Impact Development Design Strategies, An Integrated Design Approach. Prince George's County, Maryland.

Marsh, W. 1983. *Landscape Planning: Environmental Applications*. John Wiley & Sons, New York.

Minnesota Pollution Control Agency. 1989. *Protecting Water Quality in Urban Areas*. Saint Paul, Minnesota.

Schueler, Thomas R. 1987. *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs*. Metropolitan Washington Council of Governments. Washington D.C.
Schueler, Thomas R. 1995. *Site Planning for Urban Stream Protection*. Prepared by Center for Watershed Protection (CWP). Metropolitan Washington Council of Governments. Washington D.C.

Schueler, Thomas R. and Heather K. Holland. 2000. *The Practice of Watershed Protection*. Center for Watershed Protection (CWP), Ellicott City, MD.

Sykes, R. D. 1989. *Chapter 31 – Site Planning*. University of Minnesota. Minneapolis, Minnesota.

# 3. Stormwater Hydrology

## 3.1 Methods for Estimating Stormwater Runoff

### 3.1.1 Introduction to Hydrologic Methods

Hydrology deals with estimating peak flows, volumes, and time distributions of stormwater runoff. The analysis of these parameters is fundamental to the design of stormwater management facilities, such as storm drainage systems and best management practices. In the hydrologic analysis of a development site, there are a number of factors that affect the nature of stormwater runoff from the site. Some of the factors that need to be considered include:

- Rainfall amount and storm distribution
- Drainage area size, shape and orientation
- Ground cover and soil type
- Slopes of terrain and stream channel(s)
- Antecedent moisture condition
- Storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Characteristics of the local drainage system

There are a number of empirical hydrologic methods that can be used to estimate runoff characteristics for a site or drainage subbasin; however, the following methods presented in this section have been selected to support hydrologic site analysis for the design methods and procedures included in the Manual:

❑ Rational Method
❑ NRCS TR-55 Unit Hydrograph Method
❑ U.S. Geological Survey (USGS) Regression Equations
❑ Water Quality Treatment Volume Calculation (which includes Runoff Reduction Calculations)
❑ Water Balance Calculations

These methods were selected based upon a verification of their accuracy in duplicating local hydrologic estimates for a range of design storms throughout the state and the availability of equations, nomographs, and computer programs to support the methods.

**Table 3.1.1-1** lists the hydrologic methods and the circumstances for their use in various analysis and design applications. **Table 3.1.1-2** provides some limitations on the use of several methods.

Table 3.1.1-1 Applications of the Recommended Hydrologic Methods

| Method | Manual Section | Rational Method | NRCS TR-55 Method | USGS Equations | Water Quality Volume |
|---|---|---|---|---|---|
| Water Quality Volume ($WQ_v$) | 2.2 | | | | ✓ |
| Channel Protection Volume ($CP_v$) | 2.2 | | ✓ | | |
| Overbank Flood Protection ($Q_{p25}$) | 2.2 | | ✓ | ✓ | |
| Extreme Flood Protection ($Q_f$) | 2.2 | | ✓ | ✓ | |
| Storage Facilities | 3.3 | | ✓ | ✓ | |
| Outlet Structures | 3.4 | | ✓ | ✓ | |
| Gutter Flow and Inlets | 5.2 | ✓ | | | |
| Storm Drain Pipes | 5.2 | ✓ | ✓ | ✓ | |
| Culverts | 5.3 | ✓ | ✓ | ✓ | |
| Small Ditches | 5.4 | ✓ | ✓ | ✓ | |
| Open Channels | 5,4 | | ✓ | ✓ | |
| Energy Dissipation | 5.5 | | ✓ | ✓ | |

BACK TO TOC

In general:

- The Rational Method is recommended for small highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters.
- The USGS regression equations are recommended for drainage areas with characteristics within the ranges given for the equations. The USGS equations should be used with caution when there are significant storage areas within the drainage basin or where other drainage characteristics indicate that general regression equations might not be appropriate.

If other hydrologic methods are to be considered and used by a local review authority or design engineer, the method should first be calibrated to local conditions and tested for accuracy and reliability. If local streamgage data are available, these data can be used to develop peak discharges and hydrographs. The user is referred to standard hydrology textbooks for statistical procedures that can be used to estimate design flood events from stream gage data.

*Note: It must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex and too little data are available on the factors influencing the rainfall-runoff relationship to expect exact solutions.*

Table 3.1.1-2 Constraints on Using Recommended Hydrologic Methods

| Method | Size Limitations[1] | Comments |
|---|---|---|
| Rational | 0-200 acres | Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems. Shall not be used for storage design. |
| NRCS TR-55[2] | 0-2,000 acres* | Method can be used for estimating peak flows and hydrographs for all design applications. |
| USGS | 25 acres to 25 $mi^2$ | Method can be used for estimating peak flows for all design applications. |
| USGS | 128 acres to 25 $mi^2$ | Method can be used for estimating hydrographs for all design applications. |
| Water Quality | Limits set for each BMP | Method used for calculating the Water Quality Volume ($WQ_v$) |

[1]Size limitation refers to the drainage basin for the stormwater management facility (e.g., culvert, inlet).
[2]There are many readily available programs (such as HEC-1) that utilize this methodology
*2,000-acre upper size limit applies to single basin simplified peak flow only.

### 3.1.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in **Table 3.1.2-1** will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3.1.2-1 Symbols and Definitions

| Symbol | Definition | Units | Symbol | Definition | Units |
|---|---|---|---|---|---|
| A | Drainage area | acres | $Q_f$ | Extreme Flood Protection Volume | $ft^3$ |
| $B_f$ | Baseflow | acres-feet | $Q_i$ | Peak inflow discharge | cfs |
| C | Runoff coefficient | - | $Q_o$ | Peak outflow discharge | cfs |
| $C_f$ | Frequency factor | - | $Q_p$ | Peak rate of discharge | cfs |
| CN | NRCS TR-55 runoff curve number | - | $Q_{p25}$ | Overbank Flood Protection Volume | $ft^3$ |
| $CP_v$ | Channel Protection Volume | $ft^3$ | $Q_{wq}$ | Water Quality peak rate of discharge | cfs |
| d | Time interval | hours | q | Storm runoff during a time interval | in |
| E | Evaporation | ft | $q_u$ | Unit peak discharge | cfs (or cfs/$mi^2$/inch) |
| $E_t$ | Evapotranspiration | ft | R | Hydraulic radius | ft |
| $F_p$ | Pond and swamp adjustment factor | - | $R_o$ | Runoff | $ft^3$ |
| $G_h$ | Hydraulic gradient | - | $R_v$ | Runoff Coefficient | |
| I or i | Runoff intensity | in/hr | $RR_v$ | Runoff Reduction Volume | $ft^3$ |
| I | Percent of impervious cover | % | S | Ground slope | ft/ft or % |
| I | Infiltration | ft | S | Potential maximum retention | in |
| $I_a$ | Initial abstraction from total rainfall | in | S | Slope of hydraulic grade line | ft/ft |
| $k_h$ | Infiltration rate | ft/day | T | Channel top width | ft |
| L | Flow length | ft | TL | Lag time | hours |
| n | Manning roughness coefficient | - | $T_p$ | Time to peak | hr |
| NRCS | Natural Resources Conservation Service | - | t | Time | min |
| $O_f$ | Overflow | $ft^3$ | $t_c$ | Time of concentration | min |
| P | Accumulated rainfall | in | TIA | Total impervious area | % |
| $P_2$ | 2-year, 24-hour rainfall | in | V | Velocity | ft/s |
| $P_w$ | Wetted perimeter | ft | V | Pond volume | $ft^3$ |
| PF | Peaking factor | - | $V_r$ | Runoff volume | $ft^3$ |
| Q | Rate of runoff | cfs (or inches) | $V_s$ | Storage volume | $ft^3$ |
| $Q_d$ | Developed runoff for the design storm | in | $WQ_v$ | Water Quality Volume | $ft^3$ |

BACK TO TOC

### 3.1.3 Rainfall Estimation

The first step in any hydrologic analysis is an estimation of the rainfall that will fall on the site for a given time period. The amount of rainfall can be quantified with the following characteristics:

- *Duration (hours)* – Length of time over which rainfall (storm event) occurs
- *Depth (inches)* – Total amount of rainfall occurring during the storm duration
- *Intensity (inches per hour)* – Depth divided by the duration

The Frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed either in terms of exceedence probability or return period.

- *Exceedence Probability* – Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically 1 year
- *Return Period* – Average length of time between events that have the same duration and volume

Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedence probability of 0.01 and a return period of 100 years.

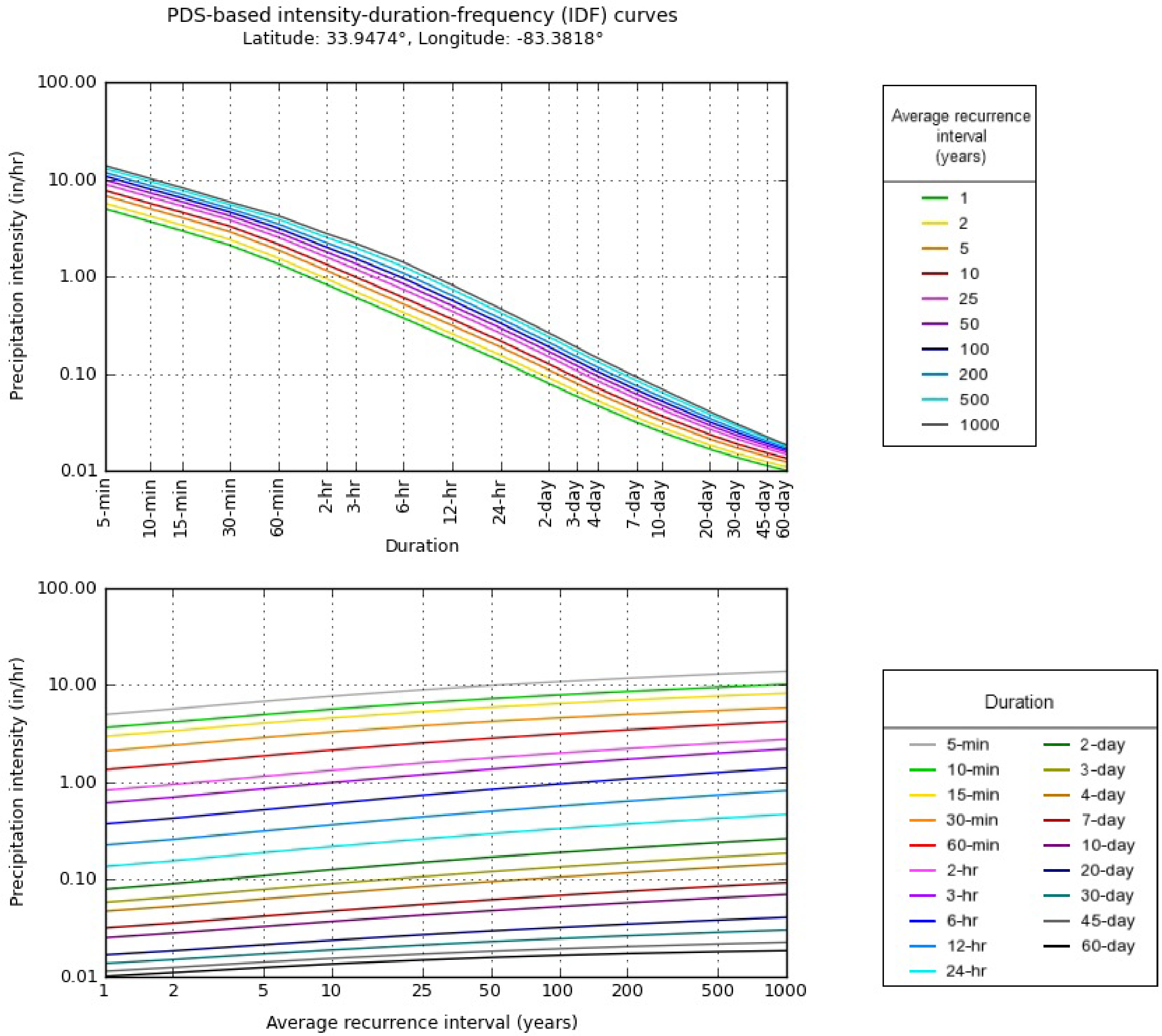



Figure 3.1.3-1 Example DDF Curve (Athens, Georgia)
(Source: NOAA Atlas 14, Volume 9, Version 2, 2013)

BACK TO TOC

Rainfall intensities for any location across Georgia can be obtained through the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 publication, or online using the Precipitation Frequency Data Server database (http://hdsc.nws.noaa.gov/hdsc/pfds). NOAA precipitation data should be used for all hydro¬logic analysis at the given locations. Additional information regarding how the values in this database were derived can be accessed using the link above.

The tabular precipitation data provided within the database are applicable for storm durations from 5 minutes to 60 days. In addition to the tabular data, the NOAA precipitation database also has a graphical display that shows the intensity duration frequency curve of any given precipitation data. **Figure 3.1.3-1** shows an example Intensity-Duration-Frequency (IDF) Curve for Athens, Georgia, for up to 10 storms (1-year – 1,000-year). These curves are plots of the tabular values. No values are given for times less than 5 minutes.

**Figure 3.1.3-2** (included as the 10-year 24-hour values from TP40) shows that the rainfall values vary south to north with generally constant values in a "V" pattern from east to west in central and south Georgia.


Figure 3.1.3-2 Rainfall Isohyetal Lines (10-year, 24-hour values)

## 3.1.4 Rational Method

### 3.1.4.1 INTRODUCTION

An important formula for determining the peak runoff rate is the Rational Formula. It is characterized by:

- Consideration of the entire drainage area as a single unit
- Estimation of flow at the most downstream point only
- The assumption that rainfall is uniformly distributed over the drainage area and is constant over time

The Rational Formula follows the assumption that:

- The predicted peak discharge has the same probability of occurrence (return period) as the used rainfall intensity (I)
- The runoff coefficient (C) is constant during the storm event

When using the Rational Method some precautions should be considered:

❑ In determining the C value (runoff coefficient based on land use) for the drainage area, hydro-logic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility.

❑ Since the Rational Method uses a composite C and a single $t_c$ value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), then basin should be divided into sub-drainage basins.

❑ The charts, graphs, and tables included in this section are given to assist the design engineer in applying the Rational Method. The design engineer should use sound engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate that these adjustments are appropriate.

### 3.1.4.2 APPLICATION

The Rational Method can be used to estimate stormwater runoff peak flows for the design of gutter flows, drainage inlets, storm drain pipe, culverts, and small ditches. It is most applicable to small, highly impervious areas. The recommended maximum drainage area that should be used with the Rational Method is 200 acres.

The Rational Method should not be used for storage design or any other application where a more detailed routing procedure is required. However, due to the popularity of the Modified Rational method among Georgia practitioners for design of small detention facilities, a method has been included in Section 3.3. The normal use of the Modified Rational method significantly under predicts detention volumes, but the improved method in Section 3.3 corrects this deficiency in the method and can be used for detention design for drainage areas up to 5 acres.

The Rational Method should also not be used for calculating peak flows downstream of bridges, culverts or storm sewers that may act as restrictions and impact the peak rate of discharge.

### 3.1.4.3 EQUATIONS

The Rational Formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration, $t_c$ (the time required for water to flow from the most remote point of the basin to the location being ana¬lyzed).

The Rational Formula is expressed as follows:

$$Q = CIA \qquad (3.1.1)$$

*Where:*

**Q** = maximum rate of runoff (cfs)
**C** = runoff coefficient representing a ratio of runoff to rainfall
**I** = average rainfall intensity for a duration equal to the $t_c$ (in/hr)
**A** = drainage area contributing to the design location (acres)

The coefficients given in **Table 3.1.4-2** are applicable for storms of 5 year to 10 year frequencies. Less frequent, higher intensity storms may require modification of the coefficient because infiltration and other losses have a proportion¬ally smaller effect on runoff (Wright ¬McLaughlin Engineers, 1969). The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor $C_f$. The Rational Formula now becomes:

$$Q = C_f CIA \qquad (3.1.2)$$

The $C_f$ values that can be used are listed in **Table 3.1.4-1**. The product of $C_f$ times C shall not exceed 1.0.

Table 3.1.4-1 Frequency Factors for Rational Formula

| Recurrence Interval (years) | $C_f$ |
|---|---|
| 10 or less | 1.0 |
| 25 | 1.1 |
| 50 | 1.2 |
| 100 | 1.25 |

BACK TO TOC

### 3.1.4.4 TIME OF CONCENTRATION

Use of the Rational Formula requires the time of concentration ($t_c$) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). The time of concentration consists of an overland flow time to the point where the runoff is concentrated or enters a defined drainage feature (e.g., open channel) plus the time of flow in a closed conduit or open channel to the design point.

**Figure 3.1.4-1** can be used to estimate overland flow time. For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient (C) is determined by the procedure described in a subsequent section of this chapter.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the nomograph shown in **Figure 3.1.4-2**. Note: time of concentration cannot be less than 5 minutes.

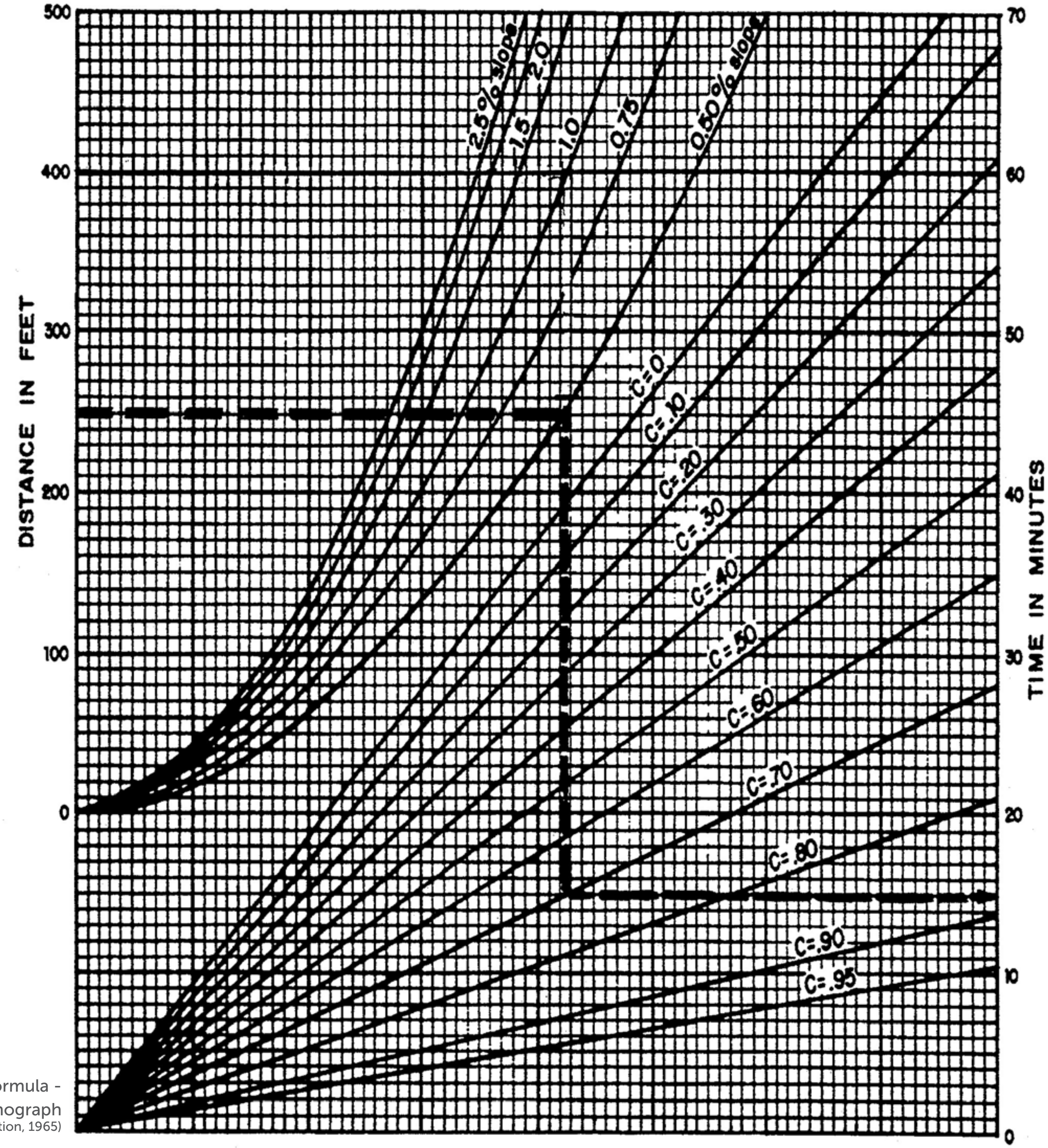
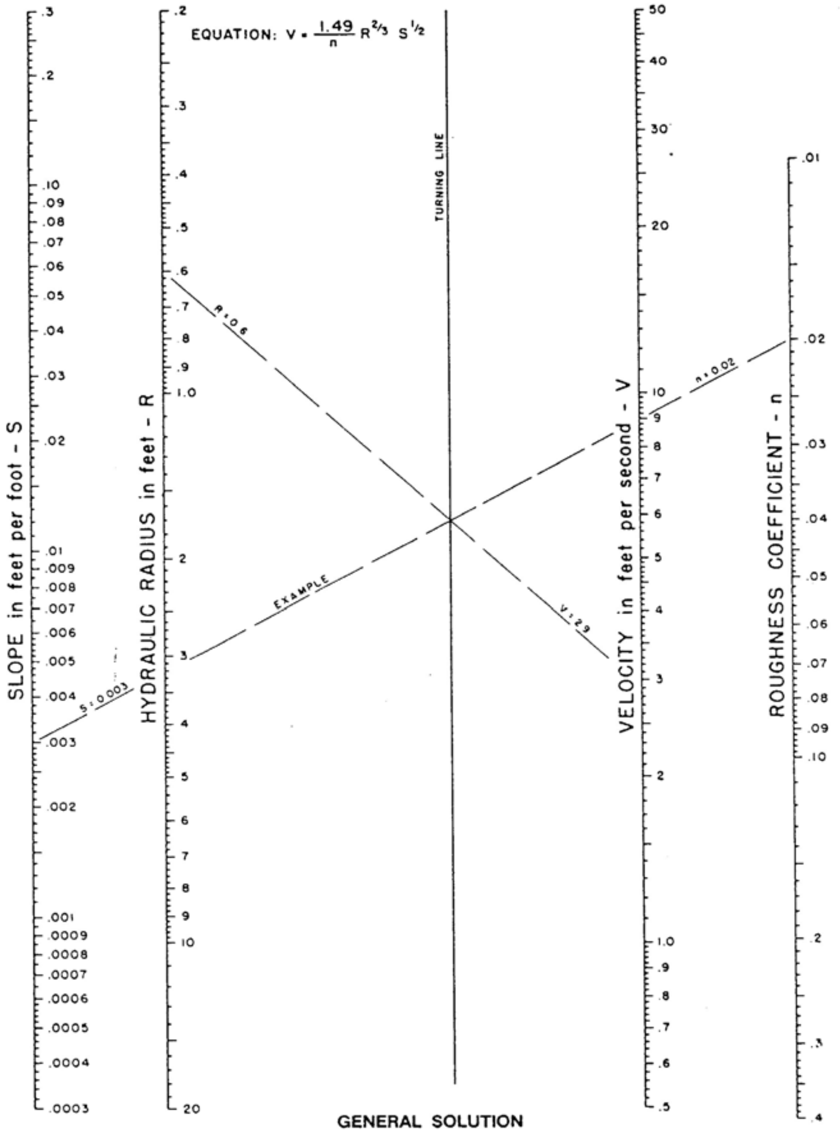



Figure 3.1.4-1 Rational Formula - Overland Time of Flow Nomograph
(Source: Airport Drainage, Federal Aviation Administration, 1965)



BACK TO TOC




Reference: USDOT, FHWA, HDS-3 (1961).

Figure 3.1.4-2 Manning's Equation Nomograph
(Source: USDOT, FHWA, HDS-3 (1961))

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the nomograph shown in **Figure 3.1.4-2.** Note: time of concentration cannot be less than 5 minutes.

Another method that can be used to determine the overland flow portion of the time of concentration is the "Kinematic Wave Nomograph" (**Figure 3.1.4-3**). The kinematic wave method incorporates several variables including rainfall intensity and Manning's "n". In using the nomograph, the design engineer has two unknowns starting the computations: the time of concentration and the rainfall intensity. A value for the rainfall intensity "I" must be assumed. The travel time is determined iteratively.

If one has determined the length, slope and roughness coefficient, and selected a rainfall intensity table, the steps to use **Figure 3.1.4-3** are as follows:

(Step 1) Assume a rainfall intensity.

(Step 2) Use **Figure 3.1.4-3** (or the equation given in the figure) to obtain the first estimate of time of concentration.

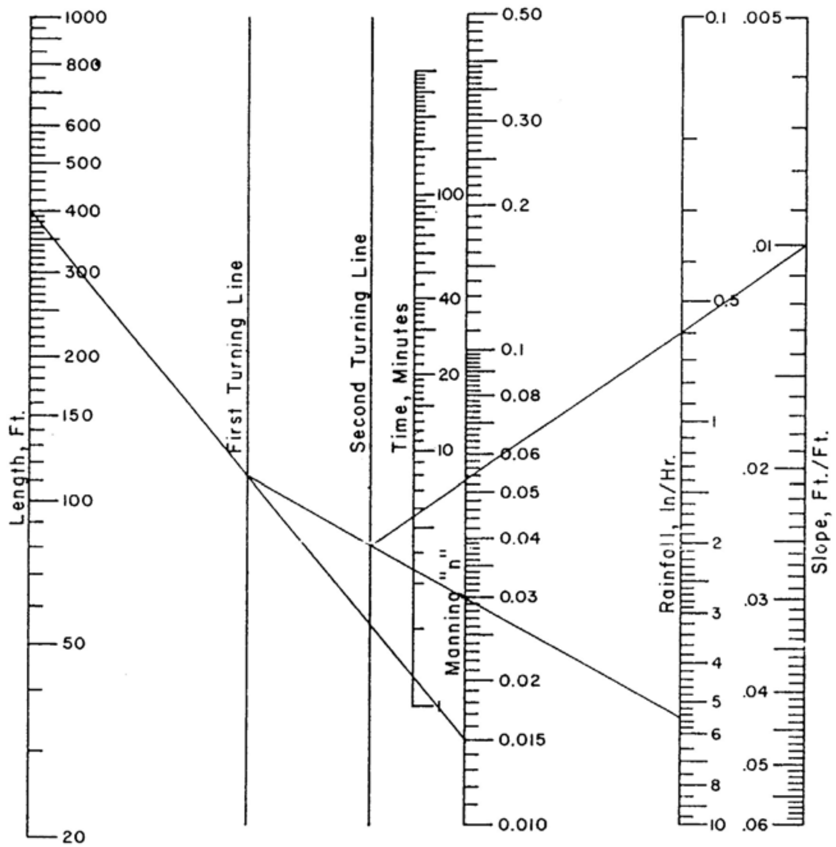
(Step 3) Using the time of concentration obtained from Step 2, use the appropriate rainfall intensity from NOAA Atlas 14 and find the rainfall intensity corresponding to the computed time of concentration. If this rainfall intensity corresponds with the assumed intensity, the problem is solved. If not, proceed to Step 4.

(Step 4) Assume a new rainfall intensity that is between that assumed in Step 1 and that determined in Step 3.

(Step 5) Repeat Steps 1 through 3 until there is good agreement between the assumed rainfall intensity and that obtained from the rainfall intensity tables.

BACK TO TOC

Equation solved by nomograph: $t_c\,(\text{Sec}) = 56\,\dfrac{L_o^{.6}\,n^{.6}}{i^{.4}\,S_o^{.3}}$




Example:
$L_o$ = 400 ft.
n = 0.015
i = 5.5 in./hr.
$S_o$ = 0.01
t = 5.5 min.

ONE INCH is 25.4mm
ONE FOOT is 0.3048m

Figure 3.1.4-3 Kinematic Wave Nomograph
(Source: Manual For Erosion And Sediment Control In Georgia)

Generally, the time of concentration for overland flow is only a part of the overall design problem. Often one encounters swale flow, confined channel flow, and closed conduit flow-times that must be added as part of the overall time of concentration. When this situation is encountered, it is best to compute the confined flow-times as the first step in the overall determination of the time of concentration. This will give the designer a rough estimate of the time involved for the overland flow, which will give a better first start on the rainfall intensity assumption. For example, if the flow time in a channel is 15 minutes and the overland flow time from the ridge line to the channel is 10 minutes, then the total time of concentration is 25 minutes.

Other methods and charts may be used to calculate overland flow time if approved by the local review authority.

Two common errors should be avoided when calculating time of concentration. First, in some cases runoff from a portion of the drainage area which is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. Second, when designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland flow paths in excess of 50 feet for impervious areas should be done only after careful consideration.

### 3.1.4.5 RAINFALL INTENSITY (I)

The rainfall intensity (I) is the average rainfall rate in in/hr for a duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from data given from NOAA Atlas 14.



BACK TO TOC

### 3.1.4.6 RUNOFF COEFFICIENT (C)

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer.

While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. **Table 3.1.4-2** gives the recommended runoff coefficients for the Rational Method.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from **Table 3.1.4-2** by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as rooftops, asphalt, and concrete streets and sidewalks. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

It should be remembered that the Rational Method assumes that all land uses within a drainage area are uniformly distributed throughout the area. If it is important to locate a specific land use within the drainage area then another hydrologic method should be used where hydrographs can be generated and routed through the drainage system.

It may be that using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) will yield a higher peak runoff value than by using the whole site. This should be checked particularly in areas where the overland portion is grassy (yielding a long $t_c$) to avoid underestimating peak runoff.

Table 3.1.4-2 Recommended Runoff Coefficient Values

| Description of Area | Runoff Coefficients (C) | Description of Area | Runoff Coefficients (C) |
|---|---|---|---|
| **Lawns** | | **Industrial** | |
| Sandy soil, flat, 2% | 0.10 | Light areas | 0.70 |
| Sandy soil, average, 2-7% | 0.15 | Heavy areas | 0.80 |
| Sandy soil, steep, >7% | 0.20 | **Parks, cemeteries** | **0.25** |
| Clay soil, flat, 2% | 0.17 | **Playgrounds** | 0.35 |
| Clay soil, average, 2-7% | 0.22 | **Railroad yard areas** | 0.40 |
| Clay soil, steep, >7% | 0.35 | **Streets** | |
| **Unimproved areas (forest)** | **0.15** | Asphalt and concrete | 0.95 |
| **Business** | | Brick | 0.85 |
| Downtown areas | 0.95 | **Drives, walks, and roofs** | **0.95** |
| Neighborhood areas | 0.70 | **Gravel areas** | **0.50** |
| **Residential** | | **Graded or no plant cover** | |
| Single-family areas | 0.50 | Sandy soil, flat, 0-5% | 0.30 |
| Multi-units, detached | 0.60 | Sandy soil, flat, 5-10% | 0.40 |
| Multi-units, attached | 0.70 | Clay soil, flat, 0-5% | 0.50 |
| Suburban | 0.40 | Clay soil, average, 5-10% | 0.60 |
| Apartment dwelling areas | 0.70 | | |

BACK TO TOC

### 3.1.4.7 EXAMPLE PROBLEM

Following is an example problem that illustrates the application of the Rational Method to estimate peak discharges.

Estimates of the maximum rate of runoff are needed at the inlet to a proposed culvert for a 25-year return period.

**Site Data**

From a topographic map of the City of Roswell and a field survey, the area of the drainage basin upstream from the point in question is found to be 23 acres. In addition the following data were measured:

- Average overland slope = 2.0%
- Length of overland flow = 50 ft
- Length of main basin channel = 2,250 ft
- Slope of channel .018 ft/ft = 1.8%
- Roughness coefficient (n) of channel was estimated to be 0.090
- Roughness coefficient (n) of channel was estimated to be 0.090
- From existing land use maps, land use for the drainage basin was estimated to be:
  - » Residential (single family) - 80%
  - » Graded - sandy soil, 3% slope - 20%

From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be: Lawn - sandy soil, 2% slope

**Overland Flow**

A runoff coefficient (C) for the overland flow area is determined from **Table 3.1.4-2** to be 0.10.

**Time of Concentration**

From **Figure 3.1.4-1** with an overland flow length of 50 ft, slope of 2% and a C of 0.10, the overland flow time is 10 min. Channel flow velocity is determined from Figure 3.1.4-2 to be 3.1 ft/s (n = 0.090, R = 1.62 (from channel dimensions) and S = .018). Therefore,

$$\text{Flow Time} = \frac{2{,}250 \text{ feet}}{(3.1 \text{ ft/s})/(60 \text{ s/min})} = 12.1 \text{ minutes}$$

and $t_c = 10 + 12.1 = 22.1$ min (use 22 min)

BACK TO TOC

**Rainfall Intensity**

From NOAA Atlas 14, and using a duration equal to 22 minutes,

$I_{25}$ (25 yr return period) = 4.88 in/hr

**Runoff Coefficient**

A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the values from **Table 3.1.4-2**.

| Land Use | Percent of Total Land Area | Runoff Coefficient | Weighted Runoff Coefficient* |
|---|---|---|---|
| Residential *(single family)* | 0.80 | 0.50 | 0.40 |
| Graded area | 0.20 | 0.30 | 0.06 |
| *Total Weighted Runoff Coefficient - 0.46* | | | |
| *Column 3 equals Column 1 multiplied by Column 2 | | | |

**Peak Runoff**

The estimate of peak runoff for a 25-yr design storm for the given basin is:

$$Q_{25} = C_f CIA = (1.10)(.46)(4.88)(23) = 57 \text{ cfs}$$

## 3.1.5 NRCS TR-55 Hydrologic Method

### 3.1.5.1 INTRODUCTION

The Soil Conservation Service (NRCS TR-55) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The NRCS

TR-55 approach, however, is more sophisticated in that it also considers the time distribution of the rain¬fall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the *NRCS National Engineering Handbook,* Part 630, Hydrology.

A typical application of the NRCS TR-55 method includes the following basic steps:

(Step 1) Determination of curve numbers that represent different land uses within the drainage area.

(Step 2) Calculation of time of concentration to the study point.

(Step 3) Using the Type II or Type III rainfall distribution, total and excess rainfall amounts are determined. Note: See **Figure 3.1.5-1** for the geographic boundaries for the different NRCS TR-55 rainfall distributions.

(Step 4) Using the unit hydrograph approach, the hydro¬graph of direct runoff from the drainage basin can be developed.

### 3.1.5.2 APPLICATION

The NRCS TR-55 method can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows. The simplified method of Subsection 3.1.5.7 can be used for drainage areas up to 2,000 acres. Thus, the NRCS TR-55 method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches and open channels, and energy dissipators.

### 3.1.5.3 EQUATIONS AND CONCEPTS

The hydrograph of outflow from a drainage basin is the sum of the elemental hydrographs from all the sub-areas of the basin, modified by the effects of transit time through the basin and storage in the stream channels. Since the physical characteristics of the basin including shape, size and slope are constant, the unit hydrograph approach assumes that there is considerable similarity in the shape of hydrographs from storms of similar rainfall characteristics. Thus, the unit hydrograph is a typical hydrograph for the basin with a runoff volume under the hydrograph equal to one (1.0) inch from a storm of specified duration. For a storm of the same duration but with a different amount of runoff, the hydrograph of direct runoff can be expected to have the same time base as the unit hydrograph and ordinates of flow proportional to the runoff volume. Therefore, a storm that produces 2 inches of runoff would have a hydrograph with a flow equal to twice the flow of the unit hydrograph. With 0.5 inches of runoff, the flow of the hydrograph would be one-half of the flow of the unit hydrograph.

The following discussion outlines the equations and basin concepts used in the NRCS TR-55 method.

**Drainage Area** - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, combine hydrographs from different sub-basins as applicable, and/or route flows to points of interest.

**Rainfall -** The NRCS TR-55 method applicable to the State of Georgia is based on a storm event that has a Type II or Type III time distribution. These distributions are used to distribute the 24-hour volume of rainfall for the different storm frequencies (**Figure 3.1.5-1)**.

**Rainfall-Runoff Equation -** A relationship between accumulated rainfall and accumulated runoff was derived by NRCS TR-55 from ex¬perimental plots for numerous soils and vegetative cover conditions. The following NRCS TR-55 runoff equation is used to estimate direct runoff from 24 hour or 1 day storm rainfall. The equation is:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad (3.1.3)$$

*Where:*

**Q** = accumulated direct runoff (in)

**P** = accumulated rainfall (potential maximum runoff) (in)

**$I_a$** = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)

**S** = potential maximum soil retention (in)

An empirical relationship used in the NRCS TR-55 method for estimating $I_a$ is:

$$I_a = 0.2S \qquad (3.1.4)$$

This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment.

Substituting 0.2S for $I_a$ in **equation 3.1.3**, the equation becomes:

$$Q = \frac{(P-0.2S)^2}{(P + 0.8S)} \qquad (3.1.5)$$

*Where:*

**S** = 1000/CN - 10

**CN** = NRCS TR-55 curve number.

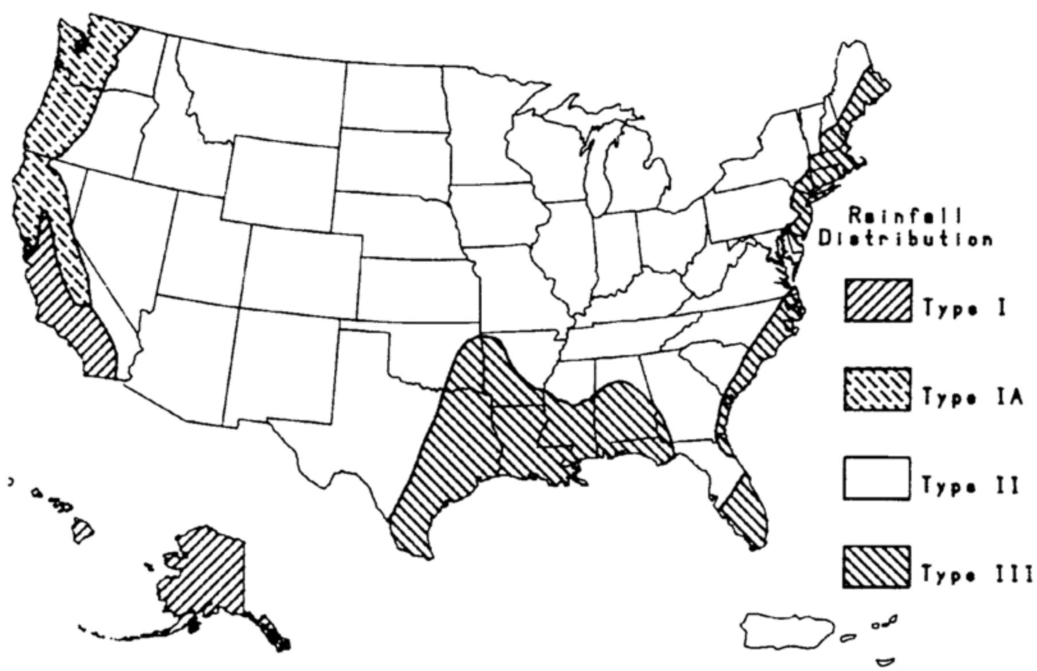



Figure 3.1.5-1 Approximate Geographic Boundaries for NRCS TR-55 Rainfall Distributions

**Figure 3.1.5-2** shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurs on a watershed with a curve number of 85.

**Equation 3.1.5** can be rearranged so that the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

$$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}] \qquad (3.1.6)$$

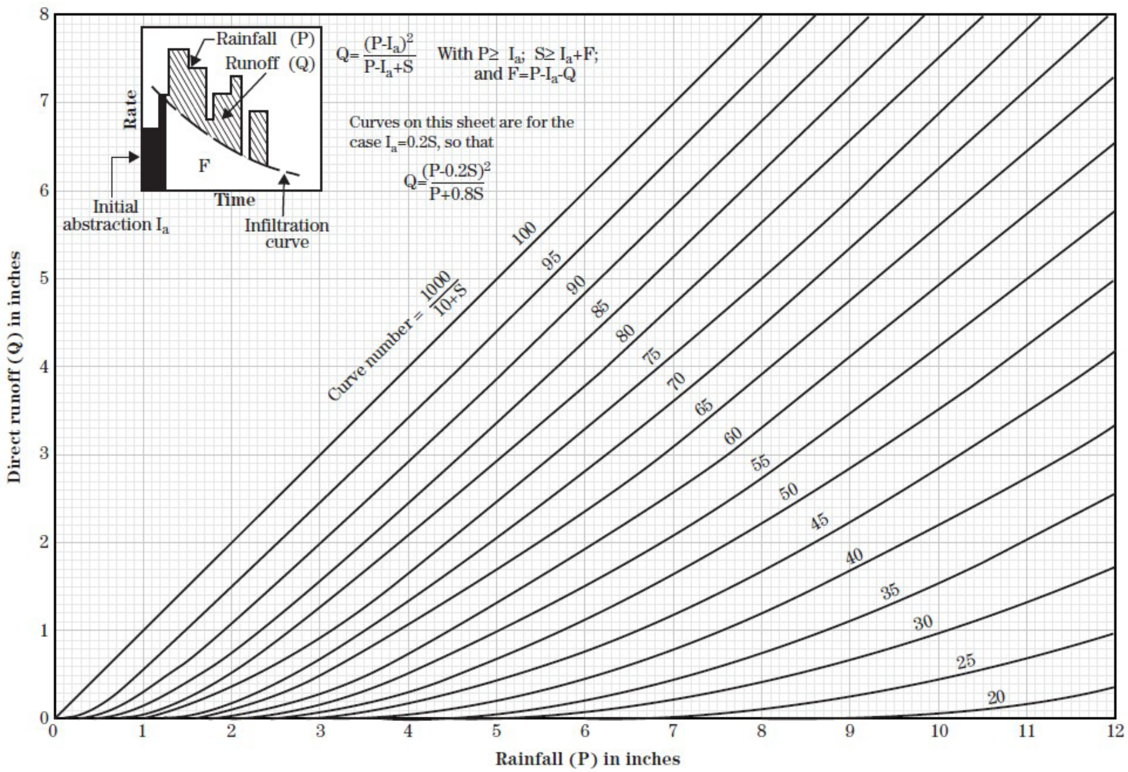


Figure 3.1.5-2 NRCS TR-55 Solution of the Runoff Equation
(Source: NRCS TR-55, NEH630, 2004)

**3.1.5.4 RUNOFF FACTOR**

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The NRCS TR-55 method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration.

Based on infiltration rates, the NRCS TR-55 has divided soils into four hydrologic soil groups.

Group A: Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B: Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderate¬y coarse textures.

Group C: Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the down¬ward movement of water or soils with moderately fine to fine texture.

Group D: Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

A list of soils throughout the State of Georgia and their hydrologic classification can be found in the publication *Urban Hydrology for Small Watersheds, 2nd Edition, Technical Release* Number 55, 1986. Soil Survey maps can be obtained online at the United States Department of Agriculture (USDA) Natural Resource Conservation Commission's (NRCS) web soil survey online tool to classify the soil type. (http://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm)

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions. Average antecedent soil moisture conditions (AMC II) are recommended for most hydrologic analysis, except in the design of state-regulated Category I dams where AMC III may be required. Areas with high water table conditions may want to consider using AMC III antecedent soil moisture conditions. This should be considered a calibration parameter for modeling against real calibration data. **Table 3.1.5-1** gives recommended curve number values for a range of different land uses.

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but sees the drainage area as a uniform land use represented by the composite curve number.

Composite curve numbers for a drainage area can be calculated by using the weighted method as presented below.

Composite Curve Number Calculation Example

| Land Use | Percent of Total Land Area | Curve Number | Weighted Curve Number (%area x CN) |
|---|---|---|---|
| Residential<br>1/8 acre<br>Soil group B | 80% | 85 | 68 |
| Meadow<br>Good conditions<br>Soil group C | 20% | 71 | 14 |
| *Total Weighted Curve Number = 68 + 14 = 82* | | | |

The different land uses within the basin should reflect a uniform hydrologic group represented by a single curve number. Any number of land uses can be included, but if their spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

### 3.1.5.5 URBAN MODIFICATIONS OF THE NRCS TR-55 METHOD

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for developed areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in **Table 3.1.5-1** are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system. It is possible that curve number values from urban areas could be reduced by not directly connecting impervious surfaces to the drainage system, but allowing runoff to flow as sheet flow over significant pervious areas.

BACK TO TOC

The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.

### Connected Impervious Areas

The CNs provided in **Table 3.1.5-1** for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

(a) Pervious urban areas are equivalent to pasture in good hydrologic condition, and

(b) Impervious areas have a CN of 98 and are directly connected to the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in **Table 3.1.5-1** are not applicable, use **Figure 3.1.5-3** to compute a composite CN. For example, **Table 3.1.5-1** gives a CN of 70 for a 1/2-acre lot in hydrologic soil group B, with an assumed impervious area of 25%. However, if the lot has 20% impervious area and a pervious area CN of 61, the composite CN obtained from **Figure 3.1.5-3** is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

### Unconnected Impervious Areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use **Figure 3.1.5-4** if total impervious area is less than 30% or (2) use **Figure 3.1.5-3** if the total impervious area is equal to or greater than 30%, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

Table 3.1.5-1 Runoff Curve Numbers[1]

| Cover description | | | Curve numbers for hydrological soil groups | | | |
|---|---|---|---|---|---|---|
| *Cover type and hydrologic condition* | | *Average percent impervious area*[2] | A | B | C | D |
| **Cultivated land:** | without conservation treatment | | 72 | 81 | 88 | 91 |
| | with conservation treatment | | 62 | 71 | 78 | 81 |
| **Pasture or range land:** | poor condition | | 68 | 79 | 86 | 89 |
| | good condition | | 39 | 61 | 74 | 80 |
| **Meadow:** | good | | 30 | 58 | 71 | 78 |
| **Wood or forest land:** | thin stand, poor cover | | 45 | 66 | 77 | 83 |
| | good cover | | 25 | 55 | 70 | 77 |
| **Open space (laws, parks, golf courses, cemeteries, etc)[3]:** | Poor condition (grass cover <50%) | | 68 | 79 | 86 | 89 |
| | Fair condition (grass cover 50% to 75%) | | 49 | 69 | 79 | 84 |
| | Good condition (grass cover >75%) | | 39 | 61 | 74 | 80 |
| **Impervious areas:** | Paved parking lots, roofs, driveways, etc (excluding right-of-way) | | 98 | 98 | 98 | 98 |
| **Streets and roads:** | Paved; curbs and storm drains (excluding right-of-way) | | 98 | 98 | 98 | 98 |
| | Paved; open ditches (including right-of-way) | | 83 | 89 | 92 | 93 |
| | Gravel (including right-of-way) | | 76 | 85 | 89 | 91 |
| | Dirt (including right-of-way) | | 72 | 82 | 87 | 89 |
| **Urban Districts:** | Commercial and business | 85% | 89 | 92 | 94 | 95 |
| | Industrial | 72% | 81 | 88 | 91 | 93 |
| **Residential districts by average lot size:** | 1/8 acre or less (townhouses) | 65% | 77 | 85 | 90 | 92 |
| | 1/4 acre | 38% | 61 | 75 | 83 | 87 |
| | 1/3 acre | 30% | 57 | 72 | 81 | 86 |
| | 1/2 acre | 25% | 54 | 70 | 80 | 85 |
| | 1 acre | 20% | 51 | 68 | 79 | 84 |
| | 2 acres | 12% | 46 | 65 | 77 | 82 |
| **Developing urban areas and newly graded areas (pervious areas only, no vegetation)** | | | 77 | 86 | 91 | 94 |

[1]Average runoff condition, and $I_a = 0.2S$
[2]The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the NRCS TR-55 method has an adjustment to reduce the effect.
[3]CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

When impervious area is less than 30%, obtain the composite CN by entering the right half of **Figure 3.1.5-4** with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1/2-acre lot with 20% total impervious area (75% of which is unconnected) and pervious CN of 61, the composite CN from **Figure 3.1.5-4** is 66. If all of the impervious area is connected, the resulting CN (from **Figure 3.1.5-3**) would be 68.

### 3.1.5.6 TRAVEL TIME ESTIMATION

Travel time ($T_t$) is the time it takes water to travel from one location to another within a watershed, through the various components of the drainage system. Time of concentration ($t_c$) is computed by summing all the travel times for consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the point of interest within the watershed. Following is a discussion of related procedures and equations (USDA, 1986).

#### Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection..

Travel time is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600V} \tag{3.1.7}$$

*Where:*

**$T_t$** = travel time (hr)
**L** = flow length (ft)
**V** = average velocity (ft/s)
**3600** = conversion factor from seconds to hours

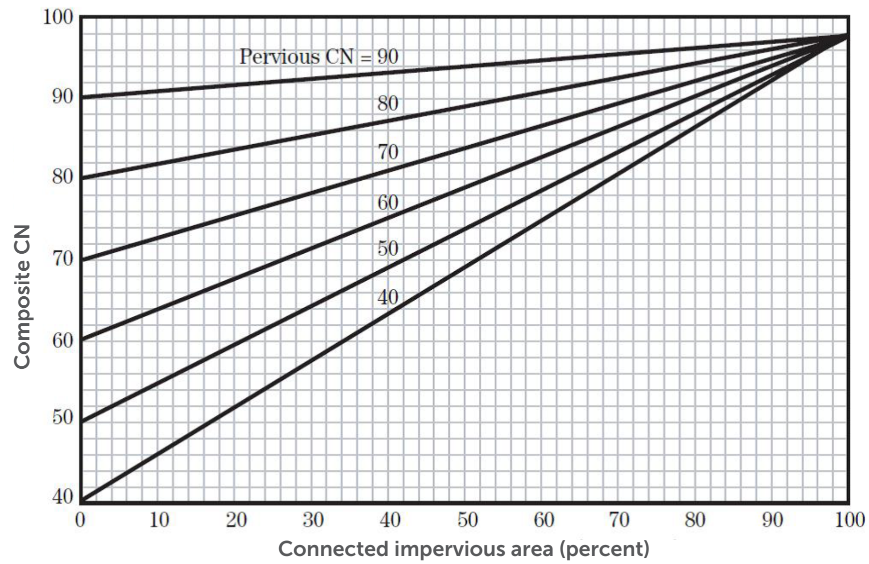



Figure 3.1.5-3 Composite CN with Connected Impervious Areas
(Source: NRCS TR-55, NEH630, 2004)

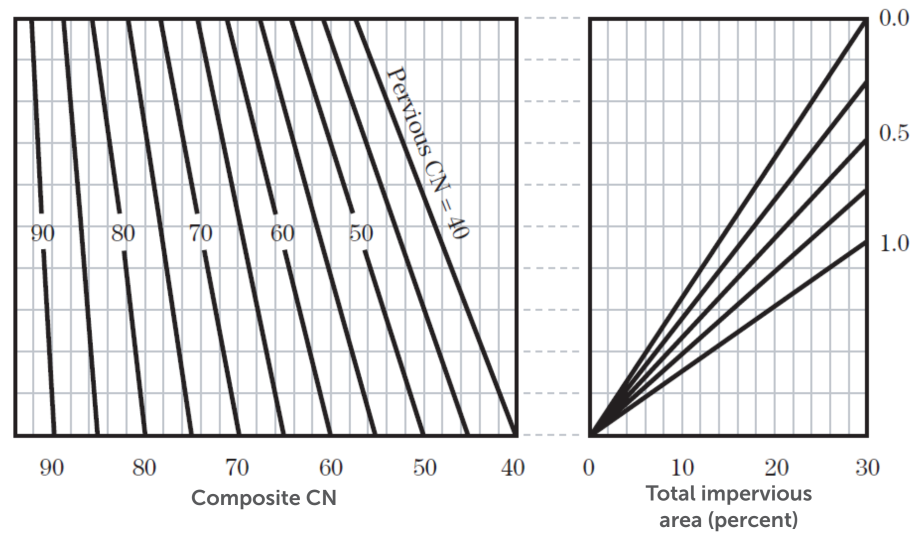



Figure 3.1.5-4 Composite CN with Unconnected Impervious Areas
(Total Impervious Area Less Than 30%)
(Source: NRCS TR-55, NEH630, 2004)

BACK TO TOC

### Sheet Flow

Sheet flow can be calculated using the following formula:

$$T_t = \frac{0.42(nL)^{0.8}}{60(P_2)^{0.5}(S)^{0.4}} \qquad (3.1.8)$$

*Where:*

**$T_t$** = travel time (hr)

**n** = Manning roughness coefficient (see **Table 3.1.5-2**)

**L** = flow length (ft)

**$P_2$** = 2-year, 24-hour rainfall

**S** = land slope (ft/ft)

Table 3.1.5-2 Roughness Coefficients (Manning's n) for Sheet Flow[1]

| Surface description | | n |
|---|---|---|
| Smooth surfaces (concrete, asphalt, gravel, or bare soil): | | 0.011 |
| Fallow (no residue): | | 0.05 |
| Cultivated soils: | Residue cover <20% | 0.06 |
| | Residue cover >20% | 0.17 |
| Grass: | Short grass prairie | 0.15 |
| | Dense grasses[2] | 0.24 |
| | Bermuda grass | 0.41 |
| Range (natural): | | 0.13 |
| Woods[3]: | Light underbrush | 0.40 |
| | Dense underbrush | 0.80 |

[1]The n values are a composite of information by Engman (1986).
[2]Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
[3]When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Source: NRCS TR-55, TR-55, Second Edition, June 1986.

### Shallow Concentrated Flow

After a maximum of 50 to 100 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from **Figure 3.1.5-5**, in which average velocity is a function of water¬course slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be computed from using **Figure 3.1.5-5,** or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

$$\text{Unpaved: } V = 16.13(S)^{0.5} \qquad (3.1.9)$$

$$\text{Paved: } V = 20.33(S)^{0.5} \qquad (3.1.10)$$

*Where:*

**V** = average velocity (ft/s)

**S** = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity using **Figure 3.1.5-5** or **Equations 3.1.9** or **3.1.10**, use **Equation 3.1.7** to estimate travel time for the shallow concentrated flow segment.

### Open Channels

Velocity in channels should be calculated from the Manning equation. Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where channels have been identified by the local municipality, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity.

Average flow velocity for travel time calculations is usually determined for bank-full elevation assuming low vegetation winter conditions.

Manning's equation is:

$$V = \frac{1.49(R)^{2/3}(S)^{1/2}}{n} \qquad (3.1.11)$$

*Where:*

**V** = average velocity (ft/s)

**R** = hydraulic radius (ft) and is equal to $A/P_w$

**A** = cross sectional flow area ($ft^2$)

$\mathbf{P_w}$ = wetted perimeter (ft)

**S** = slope of the hydraulic grade line (ft/ft)

**n** = Manning's roughness coefficient for open channel flow

After average velocity is computed using **Equation 3.1.11**, $T_t$ for the channel segment can be estimated using **Equation 3.1.7**.

**Limitations**

- ❏ Equations in this section should not be used for sheet flow longer than 50 feet for impervious land uses.
- ❏ In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate $t_c$.
- ❏ A culvert or bridge can act as detention structure if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert or bridge.

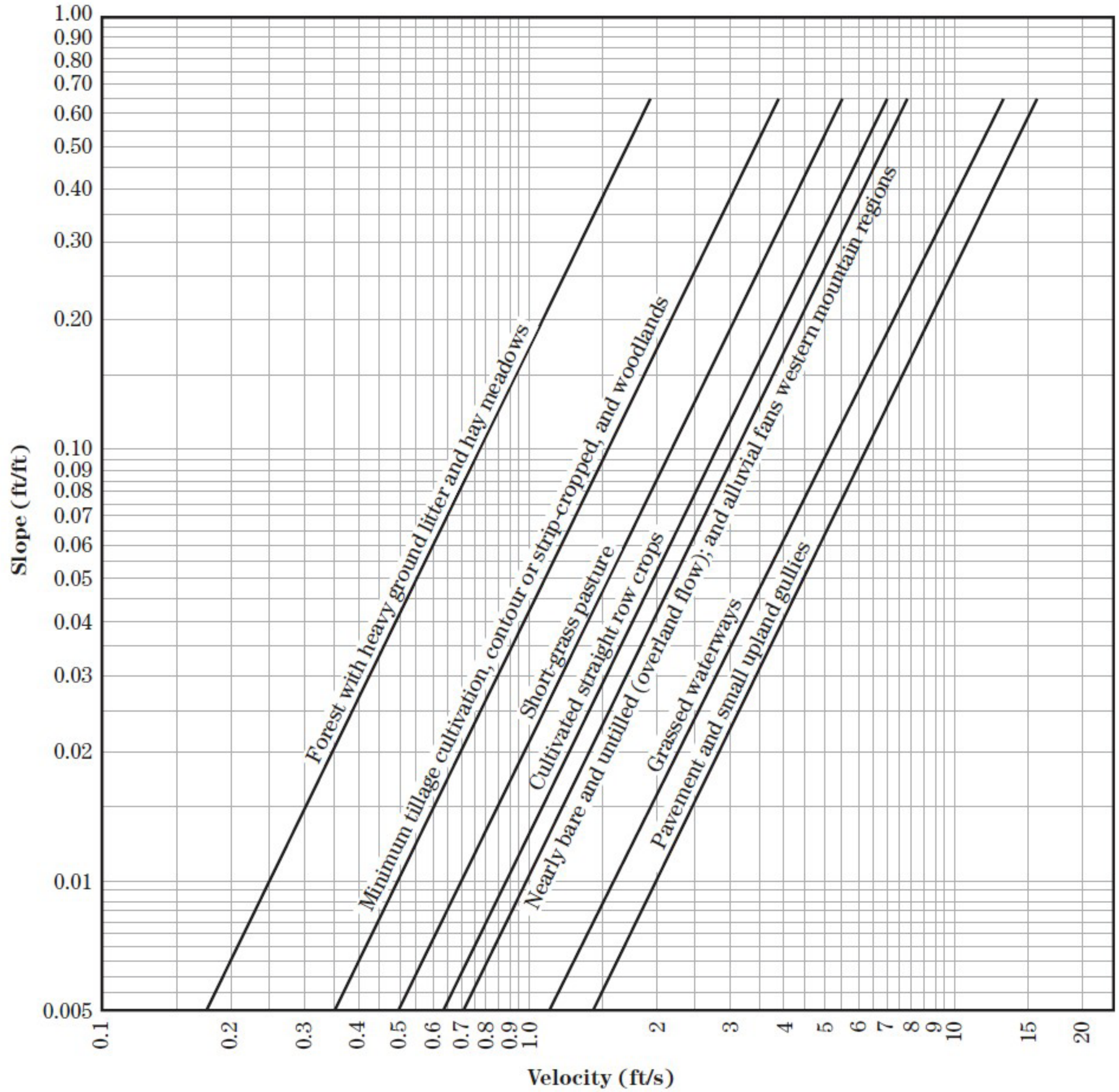



Figure 3.1.5-5 Average Velocities - Shallow Concentrated Flow
(Source: NRCS TR-55, NEH630, 2004)

BACK TO TOC

#### 3.1.5.7 SIMPLIFIED NRCS TR-55 PEAK RUNOFF RATE ESTIMATION

The following NRCS TR-55 procedures were taken from the NRCS TR-55 Technical Release 55 (USDA, 1986) which presents simplified procedures to calculate storm runoff volume and peak rate of discharges. These procedures are applicable to small drainage areas (typically less than 2,000 acres) with homogeneous land uses that can be described by a single CN value. The peak discharge equation is:

$$Q_p = q_u A Q F_p \qquad (3.1.12)$$

*Where:*

$\mathbf{Q_p}$ = peak discharge (cfs)
$\mathbf{q_u}$ = unit peak discharge (cfs/mi$^2$/in)
**A** = drainage area (mi$^2$)
**Q** = runoff (in)
$\mathbf{F_p}$ = pond and swamp adjustment factor

The input requirements for this method are as follows:

- $t_c$ – hours
- Drainage area – mi$^2$
- Type II or type III rainfall distribution
- 24-hour design rainfall
- CN value
- Pond and Swamp adjustment factor (If pond and swamp areas are spread throughout the watershed and are not considered in the $t_c$ computation, an adjustment is needed.)

Computations for the peak discharge method proceed as follows:

(Step 1) The 24-hour rainfall depth is determined from the precipitation data in the NOAA Atlas 14 publication, or online using the *Precipitation Frequency Data Server* database (http://hdsc.nws.noaa.gov/hdsc/pfds/).

(Step 2) The runoff curve number, CN, is estimated from **Table 3.1.5-1** and direct runoff, $Q_p$, is calculated using **Equation 3.1.12**.

(Step 3) The CN value is used to determine the initial abstraction, $I_a$, from **Table 3.1.5-3**, and the ratio $I_a/P$ is then computed (P = accumulated 24-hour rainfall).

(Step 4) (The watershed time of concentration is computed using the procedures in Subsection 3.1.4.4 and is used with the ratio Ia/P to obtain the unit peak discharge, $q_{up}$, from **Figure 3.1.5-6** for the Type II rainfall distribution and **Figure 3.1.5-7** for the Type III rainfall distribution. If the ratio $I_a/P$ lies outside the range shown in the figures, either the limiting values or another peak discharge method should be used. Note: **Figures 3.1.5-6** and **3.1.5-7** are based on a peaking factor of 484. If a peaking factor of 300 is needed, these figures are not applicable and the simplified NRCS TR-55 method should not be used.

(Step 5) The pond and swamp adjustment factor, $F_p$, is estimated from below:

| Pond and Swamp Areas (%)* | $F_p$ |
|---|---|
| 0 | 1.00 |
| 0.2 | 0.97 |
| 1.0 | 0.87 |
| 3.0 | 0.75 |
| 5.0 | 0.72 |
| *Percent of entire drainage basin | |

(Step 6) The peak runoff rate is computed using **Equation 3.1.12**.

Table 3.1.5-3 $I_a$ Values for Runoff Curve Numbers

| Curve Number | $I_a$ (in) | Curve Number | $I_a$ (in) |
|---|---|---|---|
| 40 | 3.000 | 70 | 0.857 |
| 41 | 2.878 | 71 | 0.817 |
| 42 | 2.762 | 72 | 0.778 |
| 43 | 2.651 | 73 | 0.740 |
| 44 | 2.545 | 74 | 0.703 |
| 45 | 2.444 | 75 | 0.667 |
| 46 | 2.348 | 76 | 0.632 |
| 47 | 2.255 | 77 | 0.597 |
| 48 | 2.167 | 78 | 0.564 |
| 49 | 2.082 | 79 | 0.532 |
| 50 | 2.000 | 80 | 0.500 |
| 51 | 1.922 | 81 | 0.469 |
| 52 | 1.846 | 82 | 0.439 |
| 53 | 1.774 | 83 | 0.410 |
| 54 | 1.704 | 84 | 0.381 |
| 55 | 1.636 | 85 | 0.353 |
| 56 | 1.571 | 86 | 0.326 |
| 57 | 1.509 | 87 | 0.299 |
| 58 | 1.448 | 88 | 0.273 |
| 59 | 1.390 | 89 | 0.247 |
| 60 | 1.333 | 90 | 0.222 |
| 61 | 1.279 | 91 | 0.198 |
| 62 | 1.226 | 92 | 0.174 |
| 63 | 1.175 | 93 | 0.151 |
| 64 | 1.125 | 94 | 0.128 |
| 65 | 1.077 | 95 | 0.105 |
| 66 | 1.030 | 96 | 0.083 |
| 67 | 0.985 | 97 | 0.062 |
| 68 | 0.941 | 98 | 0.041 |
| 69 | 0.899 | | |

Source: NRCS TR-55, TR-55, Second Edition, June 1986

BACK TO TOC

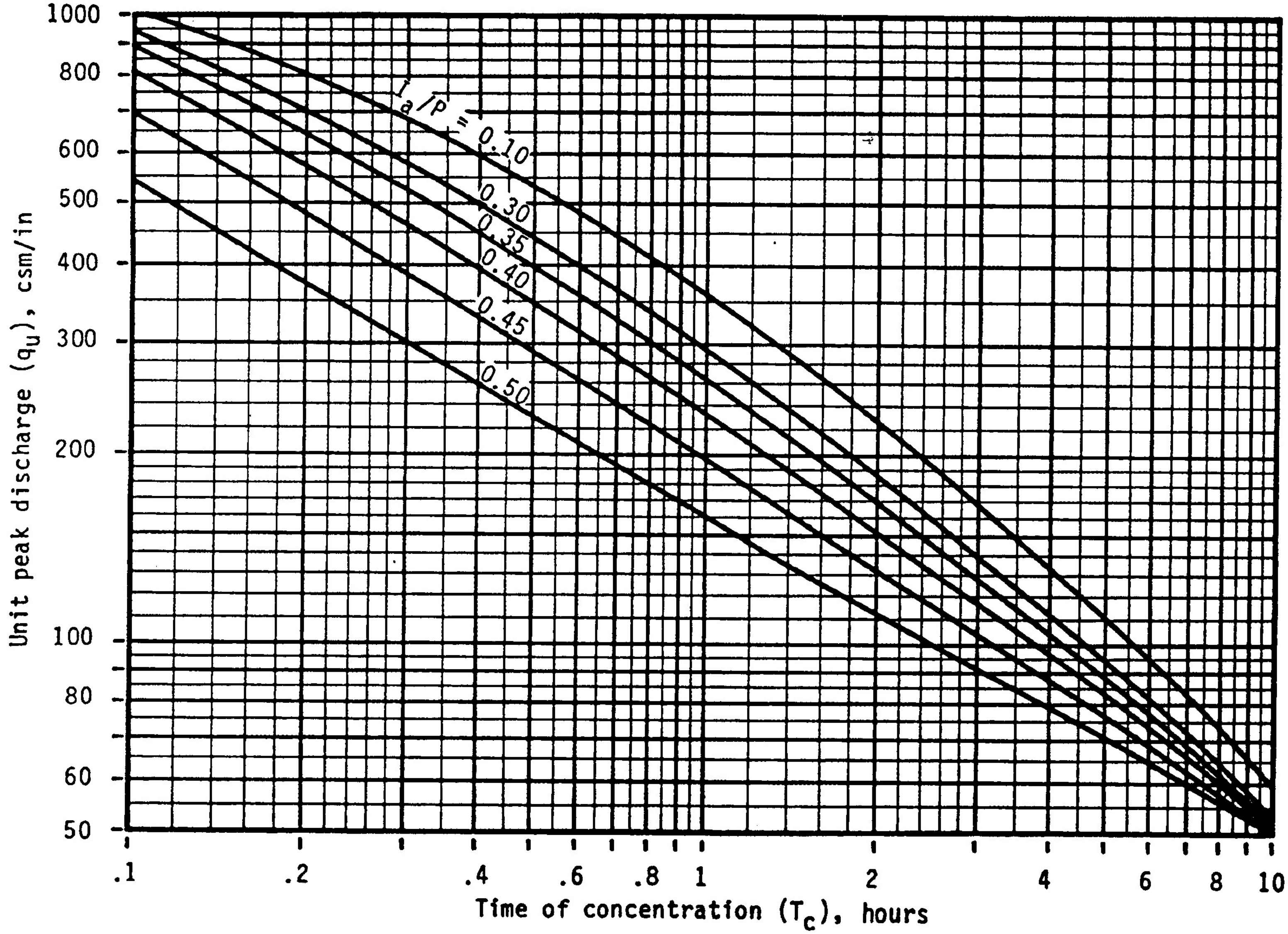



Figure 3.1.5-6 NRCS TR-55 Type II Unit Peak Discharge Graph
(Source: NRCS TR-55, TR-55, Second Edition, June 1986)

BACK TO TOC

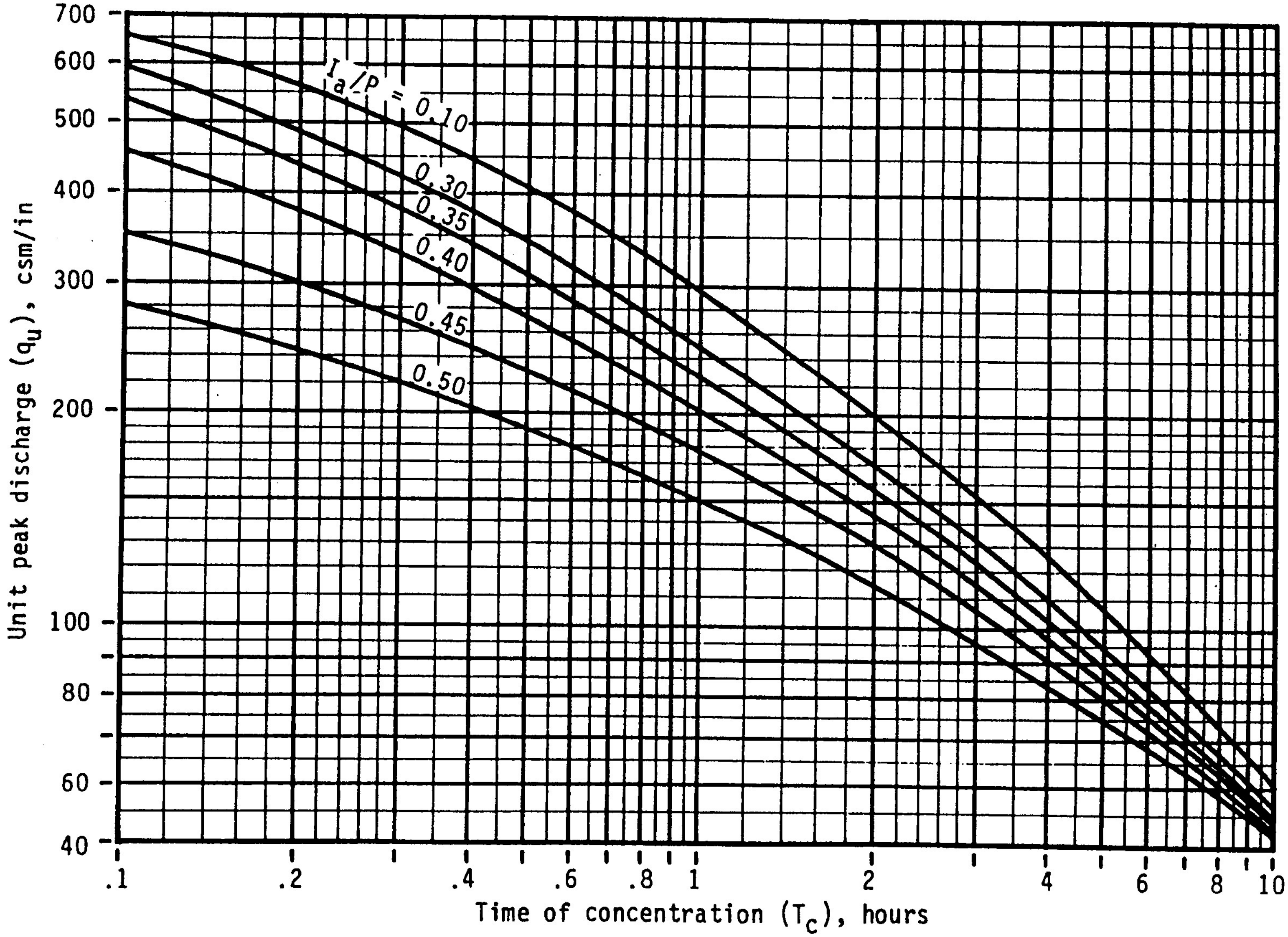



Figure 3.1.5-7 RCS TR-55 Type III Unit Peak Discharge Graph
(Source: NRCS TR-55, TR-55, Second Edition, June 1986)

### 3.1.5.8 EXAMPLE PROBLEM 1

Compute the 100-year peak discharge for a 50-acre wooded watershed located in Peachtree City, which will be developed as follows:

- Forest land - good cover (hydrologic soil group B) = 10 ac
- Forest land - good cover (hydrologic soil group C) = 10 ac
- 1/3 acre residential (hydrologic soil group B) = 20 ac
- Industrial development (hydrological soil group C) = 10 ac

Other data include the following: Total impervious area = 18 acres, % of pond / swamp area = 0

**Computations**

1. Calculate rainfall excess:

- The 100-year, 24-hour rainfall is 8.22 inches (From NOAA Atlas 14).
- The 1-year, 24-hour rainfall is 3.37 inches (From NOAA Atlas 14).
- Composite weighted runoff coefficient is:

| Dev # | Area | % Total | CN | Composite CN |
|---|---|---|---|---|
| 1 | 10 ac | 20% | 55 | 11.0 |
| 2 | 10 ac | 20% | 70 | 14.0 |
| 3 | 20 ac | 40% | 72 | 28.8 |
| 4 | 10 ac | 20% | 91 | 18.2 |
| Total | 50 ac | 100% | | 72 |

*from Equation 3.1.5, Q (100-year) = 4.89 inches
$Q_d$ (1-year developed) = 1.0 inches

2. Calculate time of concentration

The hydrologic flow path for this watershed = 1,890 ft

| Segment | Type of Flow | Length (ft) | Slope (%) |
|---|---|---|---|
| 1 | Overland n=0.24 | 40 | 2.0 |
| 2 | Shallow channel | 750 | 1.7 |
| 3 | Main channel* | 1100 | 0.5 |

*For the main channel, n = .06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

Segment 1 - Travel time from **Equation 3.1.8** with P2 = 3.84 inches

(From NOAA Atlas 14)

$T_t = [0.42(0.24 \text{ X } 40)_{0.8}] / [(3.84)^{0.5} (.020)^{0.4}] =$ 6.26 minutes

Segment 2 - Travel time from **Figure 3.1.5-5** or **Equation 3.1.9**

V = 2.1 ft/sec (from **equation 3.1.9**)

$T_t$ = 750 / 60(2.1) = 5.95 minutes

Segment 3 - Using e**quation 3.1.11**

V = (1.49/.06) (1.43)0.67 (.005)0.5 = 2.23 ft/sec

Tt = 1100 / 60 (2.23) = 8.22 minutes

$t_c$ = 6.26 + 5.95 + 8.22 = 20.43 minutes (.34 hours)

3. Calculate $I_a$/P for Cn = 72 (**Table 3.1.5-1**), $I_a$ = .778 (**Table 3.1.5-3**)
   $I_a$/P = (.778 / 8.23) = .095 (Note: Use $I_a$/P = .10 to facilitate use of **Figure 3.1.5-6**. Straight line interpolation could also be used.)

4. Unit discharge $q_u$ (100-year) from **Figure 3.1.5-6** = 650 csm/in, $q_u$ (1-year) = 580 csm/in

5. Calculate peak discharge with $F_p$ = 1 using equation 3.1.12
   $Q_{100}$ = 650 (50/640)(4.89)(1) = 248 cfs

### 3.1.5.9 HYDROGRAPH GENERATION

In addition to estimating the peak discharge, the NRCS TR-55 method can be used to estimate the entire hydrograph from a drainage area. The NRCS TR-55 has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for small drainage areas (less than 2,000 acres). The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, NRCS TR-55 has developed hydrograph procedures to be used to generate composite flood hydrographs. For the development of a hydrograph from a homogeneous developed drainage area and drainage areas that are not homoge¬neous, where hydro¬graphs need to be generated from sub-areas and then routed and combined at a point downstream, the design engineer is referred to the procedures outlined by the NRCS TR-55 in the 1986 version of TR-55 available from the *National Technical Information Service in Springfield, Virginia 22161*. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

BACK TO TOC

The unit hydrograph equations used in the NRCS TR-55 method for generating hydrographs includes a constant to account for the general land slope in the drainage area. This constant, called a peaking factor, can be adjusted when using the method. A default value of 484 for the peaking factor represents rolling hills – a medium level of relief. NRCS TR-55 indicates that for mountainous terrain the peaking factor can go as high as 600, and as low as 300 for flat (coastal) areas. Referring to **Figure 3.1.6-1**, which shows the different hydrologic regions developed by the USGS for the state of Georgia, Region 3 represents the primary region of the state where modification of the peaking factor from 484 to 300 is most often warranted if the individual watershed possesses flat terrain.

As a result of hydrologic/hydraulic studies completed in the development of this Manual, the following are recommendations related to the use of different peaking factors:

- The NRCS TR-55 method can be used without modification (peaking factor left at 484) in Regions 1, 2 and 4 generally when performing modeling analysis.
- The NRCS TR-55 method can be modified in that a peaking factor of 484 to 300 can be used for modeling generally in Region 3 when watersheds are flat and have significant storage in the overbanks. These watersheds would be characterized by:
  - ❑ Mild Slopes (less than 2% slope)
  - ❑ Significant surface storage throughout the watershed in the form of standing water during storm events or inefficient drainage system

The NRCS TR-55 method can be similarly adjusted for any watershed that has flow and storage characteristics similar to a typical Region 3 stream

The development of a runoff hydrograph from a watershed is a laborious process not normally done by hand. For that reason only an overview of the process is given here to assist the designer in reviewing and understanding the input and output from a typical computer program. There are choices of computational interval, storm length (if the 24-hour storm is not going to be used), and other "administrative" parameters that are peculiar to each computer program.

The development of a runoff hydrograph for a watershed or one of many sub-basins within a more complex model involves the following steps:

(Step 1) Development or selection of a design storm hyetograph. Often the NRCS TR-55 24-hour storm described in Subsection 3.1.5.3 is used. This storm is recommended for use in Georgia.

(Step 2) Development of curve numbers and lag times for the watershed using the methods described in Subsections 3.1.5.4,3.1.5.5, and 3.1.5.6.

(Step 3) Development of a unit hydrograph from either the standard (peaking factor of 484) or coastal area (peaking factor of 300) dimensionless unit hydrographs. See discussion below.

(Step 4) Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the NRCS TR-55 rainfall-runoff equation (**Equation 3.1.5**)

(Step 5) Application of each increment of excess rainfall to the unit hydrograph to develop a series of runoff hydrographs, one for each increment of rainfall (this is called "convolution."

(Step 6) Summation of the flows from each of the small incremental hydrographs (keeping proper track of time steps) to form a runoff hydrograph for that watershed or sub-basin.

To assist the designer in using the NRCS TR-55 unit hydrograph approach with a peaking factor of 300, **Figure 3.1.5-8** and **Table 3.1.5-4** have been developed. The unit hydrograph is used in the same way as the unit hydrograph with a peaking factor of 484.

The procedure to develop a unit hydrograph from the dimensionless unit hydrographs in the table below is to multiply each time ratio value by the time-to-peak ($T_p$) and each value of $q/q_u$ by $q_u$ calculated as:

$$q_u = (PF\ A) / (T_p) \qquad (3.1.13)$$

Where:

**$q_u$** = unit hydrograph peak rate of discharge (cfs)
**PF** = peaking factor (either 484 or 300)
**A** = area (mi²)
**d** = rainfall time increment (hr)
**$T_p$** = time to peak = d/2 + 0.6 $T_c$ (hr)

For ease of spreadsheet calculations, the dimensionless unit hydrographs for 484 and 300 can be approximated by the equation:

$$\frac{q}{q_u} = \left[\frac{t}{T_p} e^{\left(1-\frac{t}{T_p}\right)}\right]^X \qquad (3.1.14)$$

Where X is 3.79 for the PF=484 unit hydrograph and 1.50 for the PF=300 unit hydrograph.

Table 3.1.5-4 Dimensionless Unit Hydrographs

| | 484 | | 300 | |
|---|---|---|---|---|
| $t/T_t$ | $q/q_u$ | $Q/Q_p$ | $q/q_u$ | $Q/Q_p$ |
| 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 0.1 | 0.005 | 0.000 | 0.122 | 0.006 |
| 0.2 | 0.046 | 0.004 | 0.296 | 0.019 |
| 0.3 | 0.148 | 0.015 | 0.469 | 0.041 |
| 0.4 | 0.301 | 0.038 | 0.622 | 0.070 |
| 0.5 | 0.481 | 0.075 | 0.748 | 0.105 |
| 0.6 | 0.657 | 0.125 | 0.847 | 0.144 |
| 0.7 | 0.807 | 0.186 | 0.918 | 0.186 |
| 0.8 | 0.916 | 0.255 | 0.966 | 0.231 |
| 0.9 | 0.980 | 0.330 | 0.992 | 0.277 |
| 1.0 | 1.000 | 0.406 | 1.000 | 0.324 |
| 1.1 | 0.982 | 0.481 | 0.993 | 0.370 |
| 1.2 | 0.935 | 0.552 | 0.974 | 0.415 |
| 1.3 | 0.867 | 0.618 | 0.945 | 0.459 |
| 1.4 | 0.786 | 0.677 | 0.909 | 0.501 |
| 1.5 | 0.699 | 0.730 | 0.868 | 0.541 |
| 1.6 | 0.611 | 0.777 | 0.823 | 0.579 |
| 1.7 | 0.526 | 0.817 | 0.775 | 0.615 |
| 1.8 | 0.447 | 0.851 | 0.727 | 0.649 |
| 1.9 | 0.376 | 0.879 | 0.678 | 0.680 |
| 2.0 | 0.312 | 0.903 | 0.631 | 0.710 |
| 2.1 | 0.257 | 0.923 | 0.584 | 0.737 |
| 2.2 | 0.210 | 0.939 | 0.539 | 0.762 |
| 2.3 | 0.170 | 0.951 | 0.496 | 0.785 |
| 2.4 | 0.137 | 0.962 | 0.455 | 0.806 |
| 2.5 | 0.109 | 0.970 | 0.416 | 0.825 |
| 2.6 | 0.087 | 0.977 | 0.380 | 0.843 |
| 2.7 | 0.069 | 0.982 | 0.346 | 0.859 |
| 2.8 | 0.054 | 0.986 | 0.314 | 0.873 |
| 2.9 | 0.042 | 0.989 | 0.285 | 0.886 |
| 3.0 | 0.033 | 0.992 | 0.258 | 0.898 |
| 3.1 | 0.025 | 0.994 | 0.233 | 0.909 |

| | 484 | | 300 | |
|---|---|---|---|---|
| $t/T_t$ | $q/q_u$ | $Q/Q_p$ | $q/q_u$ | $Q/Q_p$ |
| 3.2 | 0.020 | 0.995 | 0.211 | 0.919 |
| 3.3 | 0.015 | 0.996 | 0.190 | 0.928 |
| 3.4 | 0.012 | 0.997 | 0.171 | 0.936 |
| 3.5 | 0.009 | 0.998 | 0.153 | 0.943 |
| 3.6 | 0.007 | 0.998 | 0.138 | 0.949 |
| 3.7 | 0.005 | 0.999 | 0.124 | 0.955 |
| 3.8 | 0.004 | 0.999 | 0.111 | 0.960 |
| 3.9 | 0.003 | 0.999 | 0.099 | 0.965 |
| 4.0 | 0.002 | 1.000 | 0.089 | 0.969 |
| 4.1 | | | 0.079 | 0.972 |
| 4.2 | | | 0.071 | 0.976 |
| 4.3 | | | 0.063 | 0.979 |
| 4.4 | | | 0.056 | 0.981 |
| 4.5 | | | 0.050 | 0.984 |
| 4.6 | | | 0.044 | 0.986 |
| 4.7 | | | 0.039 | 0.987 |
| 4.8 | | | 0.035 | 0.989 |
| 4.9 | | | 0.031 | 0.990 |
| 5.0 | | | 0.028 | 0.992 |
| 5.1 | | | 0.024 | 0.993 |
| 5.2 | | | 0.022 | 0.994 |
| 5.3 | | | 0.019 | 0.995 |
| 5.4 | | | 0.017 | 0.996 |
| 5.5 | | | 0.015 | 0.996 |
| 5.6 | | | 0.013 | 0.997 |
| 5.7 | | | 0.012 | 0.997 |
| 5.8 | | | 0.010 | 0.998 |
| 5.9 | | | 0.009 | 0.998 |
| 6.0 | | | 0.008 | 0.999 |
| 6.1 | | | 0.007 | 0.999 |
| 6.2 | | | 0.006 | 0.999 |
| 6.3 | | | 0.006 | 1.000 |

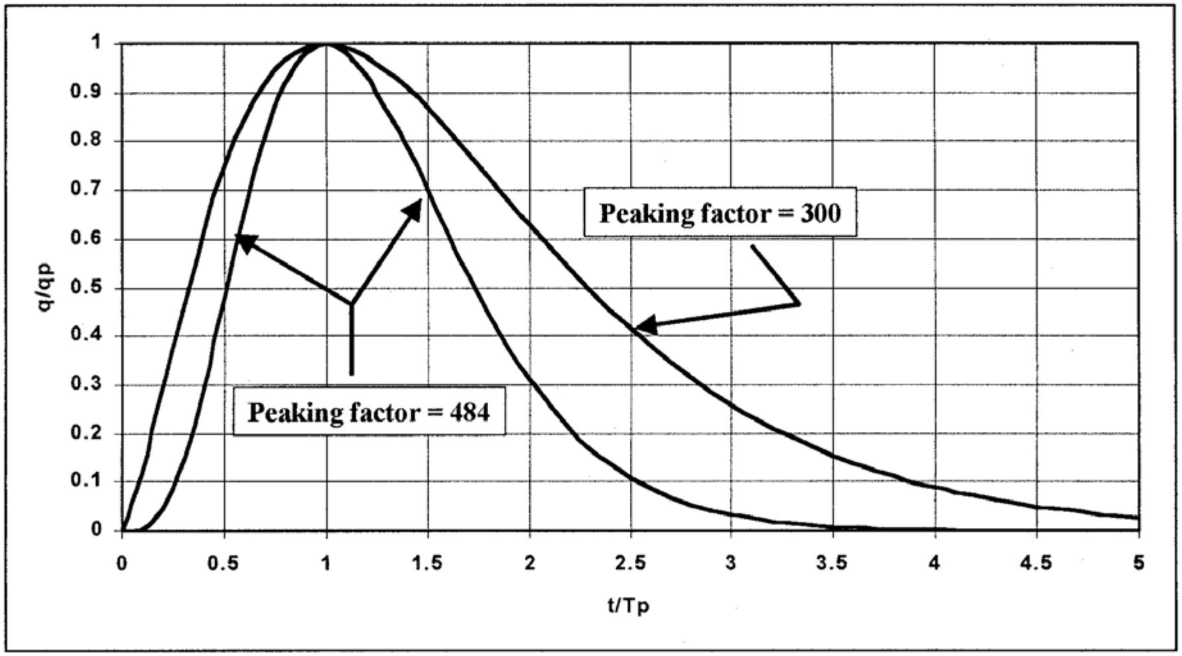



Figure 3.1.5-8 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

### 3.1.5.10 EXAMPLE PROBLEM 2

Compute the unit hydrograph for the 50-acre wooded watershed in Subsection 3.1.5.8.

#### Computations

1. Calculate $T_p$ and time increment

The time of concentration ($T_c$) is calculated to be 20.24 minutes for this watershed. If we assume a computer calculation time increment (d) of 3 minutes then:

$T_p$ = d/2 + 0.6$T_c$ = 3/2 + 0.6 * 20.24 = 13.64 minutes (0.227 hrs)

2. Calculate $q_{pu}$

$q_{pu}$ = PF A/Tp = (484 * 50/640)/(0.227) = 166 cfs

For a PF of 300 $q_{pu}$ would be:

$q_{pu}$ = PF A/$T_p$ = (300 * 50/640)/(0.227) = 103 cfs

3. Calculate unit hydrograph for both 484 and 300.

Based on spreadsheet calculations using **Equations 3.1.13 and 3.1.14**, the table to the right has been derived.

| Time | | 484 | | 300 | |
|---|---|---|---|---|---|
| $t/T_p$ | time *(min)* | $q/q_{pu}$ | q | $q/q_{pu}$ | q |
| 0 | 0 | 0 | 0.00 | 0 | 0.00 |
| 0.22 | 3.0 | 0.06 | 10.26 | 0.33 | 34.18 |
| 0.44 | 6.0 | 0.37 | 61.74 | 0.68 | 69.60 |
| 0.66 | 9.0 | 0.75 | 124.79 | 0.89 | 91.99 |
| 0.88 | 12.0 | 0.97 | 161.37 | 0.99 | 101.85 |
| 1 | 13.64 | 1 | 166 | 1 | 103 |
| 1.10 | 15.0 | 0.98 | 163.39 | 0.99 | 102.35 |
| 1.32 | 18.0 | 0.85 | 141.70 | 0.94 | 96.74 |
| 1.54 | 21.0 | 0.66 | 110.45 | 0.85 | 87.64 |
| 1.76 | 24.0 | 0.48 | 79.61 | 0.75 | 76.98 |
| 1.98 | 27.0 | 0.33 | 54.06 | 0.64 | 66.03 |
| 2.20 | 30.0 | 0.21 | 35.02 | 0.54 | 55.59 |
| 2.42 | 33.0 | 0.13 | 21.84 | 0.45 | 46.10 |
| 2.64 | 36.0 | 0.08 | 13.19 | 0.37 | 37.76 |
| 2.86 | 39.0 | 0.05 | 7.77 | 0.30 | 30.60 |
| 3.08 | 42.0 | 0.03 | 4.47 | 0.24 | 24.58 |
| 3.30 | 45.0 | 0.02 | 2.52 | 0.19 | 19.60 |
| 3.52 | 48.0 | 0.01 | 1.40 | 0.15 | 15.52 |
| 3.74 | 51.0 | 0.00 | 0.76 | 0.12 | 12.21 |
| 3.96 | 54.0 | 0.00 | 0.41 | 0.09 | 9.57 |
| 4.18 | 57.0 | 0.00 | 0.22 | 0.07 | 7.46 |
| 4.40 | 60.0 | 0.00 | 0.12 | 0.06 | 5.79 |
| 4.62 | 63.0 | 0.00 | 0.06 | 0.04 | 4.48 |
| 4.84 | 66.0 | 0.00 | 0.03 | 0.03 | 3.45 |
| 5.06 | 69.0 | 0.00 | 0.02 | 0.03 | 2.65 |
| 5.28 | 72.0 | 0.00 | 0.01 | 0.02 | 2.03 |
| 5.50 | 75.0 | 0.00 | 0.00 | 0.02 | 1.55 |
| 5.72 | 78.0 | | | 0.01 | 1.18 |
| 5.94 | 81.0 | | | 0.01 | 0.90 |
| 6.16 | 84.0 | | | 0.01 | 0.68 |
| 6.38 | 87.0 | | | 0.01 | 0.52 |
| 6.60 | 90.0 | | | 0.00 | 0.39 |
| 6.82 | 93.0 | | | 0.00 | 0.30 |
| 7.04 | 96.0 | | | 0.00 | 0.22 |
| 7.26 | 99.0 | | | 0.00 | 0.17 |
| 7.48 | 102.0 | | | 0.00 | 0.13 |
| 7.70 | 105.0 | | | 0.00 | 0.09 |
| 7.92 | 108.0 | | | 0.00 | 0.07 |
| 8.14 | 111.0 | | | 0.00 | 0.05 |
| 8.36 | 114.0 | | | 0.00 | 0.04 |
| 8.58 | 117.0 | | | 0.00 | 0.03 |
| 8.80 | 120.0 | | | 0.00 | 0.02 |
| 9.01 | 123.0 | | | 0.00 | 0.02 |
| 9.23 | 126.0 | | | 0.00 | 0.01 |
| 9.45 | 129.0 | | | 0.00 | 0.01 |
| 9.67 | 132.0 | | | 0.00 | 0.01 |
| 9.89 | 135.0 | | | 0.00 | 0.01 |
| 10.11 | 138.0 | | | 0.00 | 0.00 |

BACK TO TOC

## 3.1.6 U.S. Geological Survey Peak Flow and Hydrograph Method

### 3.1.6.1 INTRODUCTION

For the past 30 years the USGS has been collecting rain and streamflow data at various sites throughout the state of Georgia. The data from these efforts have been used to calibrate a USGS rainfall-runoff model. The U.S. Geological Survey Model was then used to develop peak dis-charge regression equations for the 2-, 5-, 10-, 25-, 50- and 100-year floods. In addition, the USGS used the statewide database to develop a dimensionless hydrograph that can be used to simulate flood hydrographs from rural and urban streams in Georgia. This USGS information is specific to geographical regions of Georgia. **Figure 3.1.6-1** shows the locations of these different regions.

### 3.1.6.2 APPLICATION

The USGS regression method is used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows for larger drainage areas:

- 25 acres and larger for peak flow estimation
- 128 acres and larger for hydrograph generation

The USGS method can be used for most design applications, including the design of storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches and open channels, and energy dissipators.

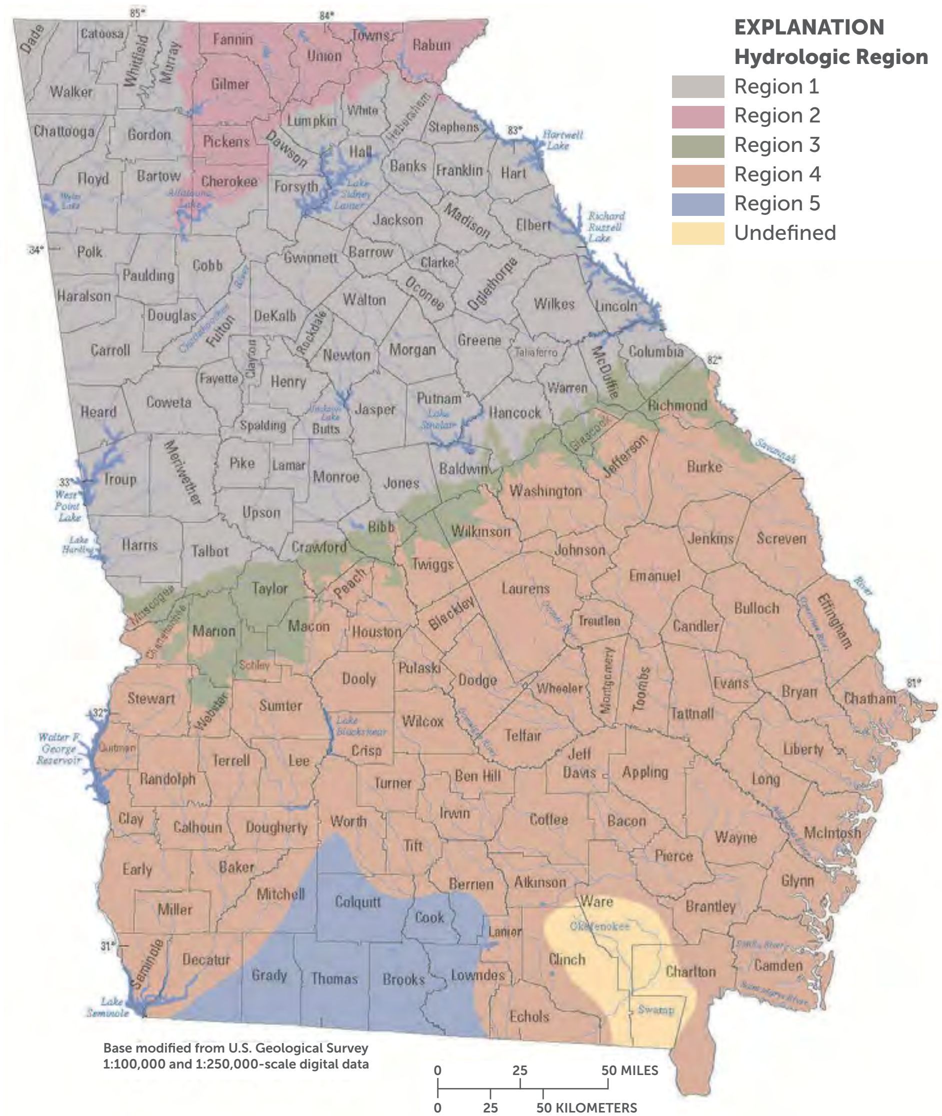



Figure 3.1.6-1 USGS Hydrologic Regions in Georgia
(Source: USGS, 2011)

BACK TO TOC

### 3.1.6.3 PEAK DISCHARGE EQUATIONS

For a complete description of the USGS regression equations presented below, consult the latest USGS publications regarding both rural and urban flood frequencies. Based on the current USGS publications, a watershed is determined to be urban if 10% or more of the watershed basin is impervious. USGS regression equations have been removed from this Manual due to their periodic update. Check the USGS publications website for the most recent publications and regression equations. At the time of this Manual update, the following publications were available:

- **Urban Method:** Methods for Estimating the Magnitude and Frequency of Floods for Urban and Small, Rural Streams in Georgia, South Carolina, and North Carolina, 2011 (http://pubs.usgs.gov/sir/2014/5030/)
- **Rural Method**: Magnitude and Frequency of Rural Floods in the Southeastern United States, 2006: Volume 1, Georgia (http://pubs.usgs.gov/sir/2009/5043/)

In addition to the publications, USGS has also developed a spreadsheet tool to assist the designer in computing flood-frequency characteristics, for both the urban and rural methods. The spreadsheets are downloadable, using the links provided above, as Microsoft Excel documents

### 3.1.6.4 PEAK DISCHARGE LIMITATIONS FOR URBAN AND RURAL BASINS

Each USGS regression equation uses variables that represent the following:

- Drainage area (DRNAREA, DA, or A)
- Percent impervious cover; and
- Percent developed land

The most recent version of the USGS publications should also be used to verify the limitations of these variables within the peak discharge equations. These equations should not be used on any variables which have physical characteris¬tics outside of their appropriate range.

### 3.1.6.5 HYDROGRAPHS

The USGS has developed a dimensionless hydrograph for Georgia streams having drainage areas of less than 500 $mi^2$. This dimensionless hydrograph can be used to simulate flood hydrographs for rural and urban streams throughout the State of Georgia. For a complete description of the USGS dimension¬less hydrograph, consult the USGS publication *Simulation of Flood Hydrographs for Georgia Streams, Water-Resources Investigation Report 86-4004*. **Table 3.1.6-1** lists the time and discharge ratios for the dimensionless hydrograph.

Table 3.1.6-1 Dimensionless USGS Hydrograph

| Time Ratio ($t/T_L$) | Discharge Ratio ($Q/Q_p$) | Time Ratio ($t/T_L$) | Discharge Ratio ($Q/Q_p$) |
|---|---|---|---|
| 0.25 | 0.12 | 1.35 | 0.62 |
| 0.30 | 0.16 | 1.40 | 0.56 |
| 0.35 | 0.21 | 1.45 | 0.51 |
| 0.40 | 0.26 | 1.50 | 0.47 |
| 0.45 | 0.33 | 1.55 | 0.43 |
| 0.50 | 0.40 | 1.60 | 0.39 |
| 0.55 | 0.49 | 1.65 | 0.36 |
| 0.60 | 0.58 | 1.70 | 0.33 |
| 0.65 | 0.67 | 1.75 | 0.30 |
| 0.70 | 0.76 | 1.80 | 0.28 |
| 0.75 | 0.84 | 1.85 | 0.26 |
| 0.80 | 0.90 | 1.90 | 0.24 |
| 0.85 | 0.95 | 1.95 | 0.22 |
| 0.90 | 0.98 | 2.00 | 0.20 |
| 0.95 | 1.00 | 2.05 | 0.19 |
| 1.00 | 0.99 | 2.10 | 0.17 |
| 1.05 | 0.96 | 2.15 | 0.16 |
| 1.10 | 0.92 | 2.20 | 0.15 |
| 1.15 | 0.86 | 2.25 | 0.14 |
| 1.20 | 0.80 | 2.30 | 0.13 |
| 1.25 | 0.74 | 2.35 | 0.12 |
| 1.30 | 0.68 | 2.40 | 0.11 |

Source: USGS, 1986

BACK TO TOC

The lag time equations for calculating the dimensionless hydrograph are:

North of the Fall Line (rural):

$$T_L = 4.64A^{0.49}S^{-0.21} \quad (3.1.15)$$

South of the Fall Line (rural):

$$T_L = 13.6A^{0.43}S^{-0.31} \quad (3.1.16)$$

Regions 1, 2 and 3 (urban):

$$TL = 7.86A^{0.35}TIA^{-0.22}S^{-0.31} \quad (3.1.17)$$

Region 4 (urban):

$$TL = 6.10A^{0.35}TIA^{-0.22}S^{-0.31} \quad (3.1.18)$$

*Where:*

$T_L$ = lag time (hours)
**A** = drainage area (mi²)
**S** = main channel slope (ft/mi)
**TIA** = total impervious area (percent)

Using these lag time equations and the dimensionless hydrograph, a runoff hydrograph can be determined after the peak discharge is calculated.

### 3.1.6.6 HYDROGRAPH LIMITATIONS

List in the table shown at the right are the limitations of the variables within the lag time equations. The lag time equation should not be used for drainage areas that have physical characteristics outside the limits listed on this page.

### 3.1.6.7 EXAMPLE PROBLEM

For the 100-year flood, calculate the peak discharge for rural and developed conditions for the following drainage area located in Region 1 in the Atlanta metro area. For the developed conditions, develop the flood hydrograph for this drainage area.

- Drainage Area = 175 acres = 0.273 mi²
- Main Channel Slope = 117 ft/mi
- Percent Impervious Area = 32%

**Peak Discharge Calculations**

100-year Rural Peak Discharge:

$Q_{100} = 776(DA)^{0.594}$ (*Taken from the most recent publication*)

$Q_{100} = 776(0.273)0^{.594} = 359$ cfs

100-year Developed (Urban) Peak Flow:

$Q_{100} = 753(DRNAREA)^{0.8038}10^{(0.0024*IMPNLCD06)}$ (*Taken from the most recent publication*)

$Q_{100}$ = 753(0.273)0.803810(0.0024*32)= 317 cfs

**Lag Time Calculations**

$T_L = 7.86A^{0.35}$TIA-0.22S-0.31 = 7.86 (0.273)0.35 (32)-0.22 (117)-0.31 = 0.53 hours

**Hydrograph Calculations**

Using the dimensionless USGS hydrograph given in **Table 3.1.6-1**, the following calculations are done to determine the coordinates of the flood hydrograph.

Time (t) = $t/T_L$ x 0.53 $t/T_L$ from **Table 3.1.6-1 on previous page**

Discharge (Q) = $Q/Q_p$ x 561 $Q/Q_p$ from T**able 3.1.6-1 on previous page**

Coordinates for the flood hydrograph are given in **Table 3.1.6-2** on the next page.

| Physical Characteristics | | Minimum | Maximum | Units |
|---|---|---|---|---|
| North of the Fall Line | A - Drainage Area | 0.3 | 500 | mi² |
| (rural) | S - Main Channel Slope | 5.0 | 200 | feet per mile |
| South of the Fall Line | A - Drainage Area | 0.2 | 500 | mi² |
| (rural) | S - Main Channel Slope | 1.3 | 60 | feet per mile |
| Regions 1, 2, & 3 | A - Drainage Area | 0.04 | 19.1 | mi² |
| (urban) | S - Main Channel Slope | 9.4 | 772 | feet per mile |
| | TIA - Total Impervious Area | 1.0 | 61.6 | percent |
| Region 4 (urban) | A - Drainage Area | 0.12 | 2.9 | mi² |
| | S - Main Channel Slope | 19.4 | 110 | feet per mile |
| | TIA - Total Impervious Area | 6.1 | 42.4 | percent |

BACK TO TOC

Table 3.1.6-2  Flood Hydrograph

| Time Ratio ($t/T_L$) | Time (t) (Hours) | Discharge Ratio ($Q/Q_p$) | Discharge (cfs) | Time Ratio ($t/T_L$) | Time (t) (Hours) | Discharge Ratio ($Q/Q_p$) | Discharge (cfs) |
|---|---|---|---|---|---|---|---|
| 0.25 | 0.13 | 0.12 | 67 | 1.35 | 0.72 | 0.62 | 348 |
| 0.30 | 0.16 | 0.16 | 90 | 1.40 | 0.74 | 0.56 | 314 |
| 0.35 | 0.19 | 0.21 | 118 | 1.45 | 0.77 | 0.51 | 286 |
| 0.40 | 0.21 | 0.26 | 146 | 1.50 | 0.80 | 0.47 | 264 |
| 0.45 | 0.24 | 0.33 | 185 | 1.55 | 0.82 | 0.43 | 241 |
| 0.50 | 0.27 | 0.40 | 224 | 1.60 | 0.85 | 0.39 | 219 |
| 0.55 | 0.29 | 0.49 | 275 | 1.65 | 0.87 | 0.36 | 202 |
| 0.60 | 0.32 | 0.58 | 325 | 1.70 | 0.90 | 0.33 | 185 |
| 0.65 | 0.34 | 0.67 | 376 | 1.75 | 0.93 | 0.30 | 168 |
| 0.70 | 0.37 | 0.76 | 426 | 1.80 | 0.95 | 0.28 | 157 |
| 0.75 | 0.40 | 0.84 | 471 | 1.85 | 0.98 | 0.26 | 146 |
| 0.80 | 0.42 | 0.90 | 505 | 1.90 | 1.01 | 0.24 | 135 |
| 0.85 | 0.45 | 0.95 | 533 | 1.95 | 1.03 | 0.22 | 123 |
| 0.90 | 0.48 | 0.98 | 550 | 2.00 | 1.06 | 0.20 | 112 |
| 0.95 | 0.50 | 1.00 | 561 | 2.05 | 1.09 | 0.19 | 107 |
| 1.00 | 0.53 | 0.99 | 555 | 2.10 | 1.11 | 0.17 | 95 |
| 1.05 | 0.56 | 0.96 | 539 | 2.15 | 1.14 | 0.16 | 90 |
| 1.10 | 0.58 | 0.92 | 516 | 2.20 | 1.17 | 0.15 | 84 |
| 1.15 | 0.61 | 0.86 | 482 | 2.25 | 1.19 | 0.14 | 79 |
| 1.20 | 0.64 | 0.80 | 449 | 2.30 | 1.22 | 0.13 | 73 |
| 1.25 | 0.66 | 0.74 | 415 | 2.35 | 1.25 | 0.12 | 67 |
| 1.30 | 0.69 | 0.68 | 381 | 2.40 | 1.27 | 0.11 | 62 |

Source:  U.S.G.S., 1986



BACK TO TOC

## 3.1.7 Water Quality Volume and Peak Flow

In addition to the discussion of the calculations provided below, full examples for five stormwater best management practices have been provided in Appendix B.

### 3.1.7.1 WATER QUALITY VOLUME CALCULATION

The Water Quality Volume ($WQ_v$) is the retention or treatment volume required to remove a significant percentage of the stormwater pollution load, defined in this Manual as an 80% removal of the average annual post-development total suspended solids (TSS) load. This is achieved by intercepting and retaining or treating a portion of the runoff from all storms and all the runoff from 85% of the storms that occur on average during the course of a year.

The water quality treatment volume is calculated by multiplying the 85th percentile annual rainfall event by the volumetric runoff coefficient ($R_v$) and the site area. $R_v$ is defined as:

$$R_v = 0.05 + 0.009(I) \qquad (3.1.19)$$

Where: I = percent of impervious cover (%)

For the state of Georgia, the average 85th percentile annual rainfall event is 1.2 inches. Therefore, $WQ_v$ is calculated using the following formula:

$$WQ_v = \frac{1.2\ R_v\ A}{12} \qquad (3.1.20)$$

*Where:*

**$WQ_v$** = water quality volume (acre-feet)
**$R_v$** = volumetric runoff coefficient
**A** = total drainage area (acres)

$WQ_v$ can be expressed in inches simply as $1.2(R_v) = Q_{wv}$

### 3.1.7.2 WATER QUALITY VOLUME PEAK FLOW CALCULATION

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as for sand filters and infiltration trenches. An arbitrary storm would need to be chosen using the Rational Method, and conventional NRCS TR-55 methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2 inches. This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the treatment practice due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

The following procedure can be used to estimate peak discharges for small storm events.
It relies on the Water Quality Volume and the simplified peak flow estimating method above. A brief description of the calculation procedure is presented below:

1. Using $WQ_v$, a corresponding Curve Number (CN) is computed utilizing the following equation:

$$CN = 1000/[10 + 5P + 10Q_{wv} - 10(Q_{wv}^2 + 1.25Q_{wv}P)^{1/2}]$$

*Where:*

**P** = rainfall, in inches (use 1.2 inches for the Water Quality Storm in Georgia)
**$Q_{wv}$** = Water Quality Volume, in inches ($1.2R_v$)

2. Once a CN is computed, the time of concentration ($t_c$) is computed (based on the methods described in this section).
3. Using the computed CN, tc and drainage area (A), in acres; the peak discharge ($Q_{wq}$) for the water quality storm event is computed using a slight modification of the Simplified NRCS TR-55 Peak Runoff Rate Estimation technique of Subsection 3.1.5.7. Use appropriate rainfall distribution type (either Type II or Type III in Georgia).
   - Read initial abstraction ($I_a$), compute $I_a/P$
   - Read the unit peak discharge ($q_u$) for appropriate $t_c$
   - Using $WQ_v$, compute the peak discharge ($Q_{wq}$)

$$Q_{wq} = q_u * A * Q_{wv}$$

Where:

**$Q_{wq}$** = the water quality peak discharge (cfs)
**$q_u$** = the unit peak discharge (cfs/mi²/inch)
**A** = drainage area (mi²)
**$Q_{wv}$** = Water Quality Volume, in inches ($1.2R_v$)

#### 3.1.7.3 EXAMPLE PROBLEM

Using the data and information from the example problem in Subsection 3.1.5.8 calculate the water quality volume and the water quality peak flow.

**Calculate water quality volume ($WQ_v$)**

Compute volumetric runoff coefficient, $R_v$

$R_v$ = 0.05 + (0.009) = 0.05 + (0.009)(18/50 x 100%) = 0.37

Compute water quality volume, $WQ_v$

$WQ_v$ = 1.2(RV)(A)/12 = 1.2(.37)(50)/12 = 1.85 acre-feet

**Calculate water quality peak flow**

Compute runoff volume in inches, $Q_{wv}$:

$Q_{wv}$ = 1.2$R_v$ = 1.2 * 0.37 = 0.44 inches

Computer curve number:

CN = 1000/[10 + 5P +10Q - 10($Q_{wv}^2$ + 1.25$Q_{wv}$ P)$^{½}$]

CN = 1000/[10 + 5*1.2 +10*0.252 - 10(0.252$^2$ + 1.25*0.252*1.2)$^{½}$] = 84

$t_c$ = 0.34 (computed previously)

S = 1000/CN – 10 = 1000/84 – 10 = 1.90 inches

0.2S = $I_a$ = 0.38 inches

$I_a$/P = 0.38/1.2 = 0.317

Find $q_u$:

From **Figure 3.1.5-6** for $I_a$/P = 0.317

$q_u$ = 535 cfs/mi²/in

Compute water quality peak flow:

$Q_{wq}$ = $q_u$ * A * $Q_{wv}$ = 535 * 50/640 * 0.44 = 18.4 cfs

#### 3.1.7.4 RUNOFF REDUCTION VOLUME CALCULATION

The Runoff Reduction Volume ($RR_v$) is the retention volume calculated to infiltrate, evapotranspirate, or otherwise removed from a post-developed condition to more naturally mimic the natural hydrologic conditions. For additional information, see Section 2.2.2.2 (Standard #3).

The runoff reduction volume is calculated by multiplying the target runoff reduction rainfall event (typically 1.0 inches) by the volumetric runoff coefficient (Rv) and the site area. Rv is again defined as:

$$R_v = 0.05 + 0.009(I) \qquad (3.1.19)$$

*Where:*

I = percent of impervious cover (%)

For the state of Georgia, the recommended target runoff reduction rainfall is 1.0 inches. Therefore, $RR_v$ is calculated using the following formula:

$$RR_v = \frac{P\, R_v\, A}{12}$$

*Where:*

**$RR_v$** = runoff reduction volume (cubic feet)

**P** = target runoff reduction rainfall

**$R_v$** = volumetric runoff coefficient

**A** = total drainage area (square feet)

#### 3.1.7.5 ADJUSTED CURVE NUMBER PROCEDURE FOR PEAK FLOW REDUCTION

The following method utilizes the Natural Resource Conservation Service runoff equations originally provided in *Urban Hydrology for Small Watersheds* (USDA 1986) to compute a curve number adjustment that effectively reduces the peak flow of other storm events. A simplified approach has been provided that combines these runoff equations. The NRCS runoff equations are discussed in Subsection 3.1.5 and include **Equations 3.1.3**, **3.1.4**, and **3.1.**5. The following modified equation is discussed in the 2010 Center for Watershed Protection journal article titled, T*he Runoff Reduction Method*:

$$Q - R = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \text{(Modified Eq. 3.1.5)}$$

*Where:*

**Q** = runoff depth (in),

**P** = rainfall depth (in),

**$I_a$** = Initial abstraction (in),

**S** = potential maximum retention after runoff begins (in),

**CN** = Runoff Curve Number, and

**R** = Retention storage provided by runoff reduction practices (in).

To calculate "R", the provided $RR_v$ of the practice(s) can be calculated by the following:

R = (VP)(RR%) / Area

*Where:*

**VP** = Total Volume Provided by the BMP (See Chapter 4 for BMP sizing information)

**RR%** = runoff reduction credit provided by the BMP (See **Table 4.1.3-2** for RR%)

By solving the modified equation above for a new potential maximum retention value, S, the adjusted curve number can be back-calculated as a representation of the runoff reduction achieved in any particular storm event.

**Runoff Reduction/Adjusted Curve Number Example**

Using the given data and information provided below, calculate the runoff reduction volume and the adjusted curve number for channel protection assuming a best management practice is used that provides a runoff reduction removal percentage (RR%) of 50%.

Given Information:

- Site Area: 3.0 ac (130,680 ft²)
- Impervious Area: 1.9 ac; or I=1.9/3.0 = 63.3%
- Pre-developed CN: 70
- Post-developed CN: 88
- $P_{1YR}$: 3.4 inches
- S: 1.36 (1000/CN – 10)
- Post-$Q_{1YR}$:2.18 inches (See **Equation 3.1.5**)

**Calculate water runoff reduction volume ($RR_v$)**

Compute volumetric runoff coefficient, $R_v$

RV = 0.05 + (0.009)(I)

= 0.05 + (0.009)(1.9/3 x 100%)

= 0.62

Compute runoff reduction volume, $RR_v$

$RR_v$ = 1.0(RV)(A)/12

= 1.0(.62)(3)/12

= 0.155 acre-feet (6,752 cubic feet)

**Calculate the minimum volume of the practice ($VP_{MIN}$)**

($VP_{MIN}$) ≥ $RR_v$ / (RR%)

= (6,752 ft³) / (50%)

= 13,504 ft³

*Note: The Volume Provided (VP) of this practice must be a minimum of 13,504 ft³.*

**Calculate the amount of Runoff Reduction ($RR_v$ provided) by the practice. This information will be needed in the adjusted curve number calculation (converted to the variable "R"):**

$RR_v$ (provided) = (RR%) (VP)

= (50%) (13,504 ft³)

= 6,752 ft³

**Adjusted Curve Number Procedure for Peak Flow Reduction of $CP_v$**

Given Q = 2.18 in. and P = 3.4 in., Find "R" and "S" to back calculate an adjusted CN

$$Q - R = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$ (Modified Equation 3.1.5)

Retention storage (expressed in inches) for this basin is calculated by the following formula:

R = $RR_v$ (provided) / Basin Area

= (6,752 ft³) / A

= 6,752 ft³ / 130,680 ft² (12 in / 1 ft)

= 0.62 inches

Solve for "S" to back calculate CN: S = 2.49

S = 1000/CN-10: therefore, Adjusted CN = 80.1

## 3.1.8 Water Balance Calculations

### 3.1.8.1 INTRODUCTION

Water balance calculations help determine if a drainage area is large enough, or has the right characteristics, to support a permanent pool of water during average or extreme conditions. When in doubt, a water balance calculation may be advisable for any best management practice that maintains a permanent volume of stormwater.

The details of a rigorous water balance are beyond the scope of this manual. However, a simplified procedure is described herein that will provide an estimate of pool viability and point to the need for more rigorous analysis. Water balance can also be used to help establish planting zones in a wetland design.

### 3.1.8.2 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):

$$\Delta V = \Sigma I - \Sigma O \qquad (3.1.21)$$

*Where:*

$\Delta$ = "change in"
**V** = permanent pool volume (ac-ft)
$\Sigma$ = "sum of"
**I** = Inflows (ac-ft)
**O** = Outflows (ac-ft)

The inflows consist of rainfall, runoff and baseflow into the pond. The outflows consist of infiltration, evaporation, evapotranspiration, and surface overflow out of the best management practice. **Equation 3.1.21** can be changed to reflect these factors.

$$\Delta V = P + Ro + Bf - I - E - Et - Of \qquad (3.1.22)$$

*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 at: http://hdsc.nws.noaa.gov/hdsc/pfds/

Monthly values are commonly used for calculations of values over a season. Rainfall is then the direct amount that falls on the permanent pool surface for the period in question. 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, Q/P can be estimated one of several ways. The best method would be to perform long-term simulation modeling using rainfall records and a watershed model. Two other methods have been proposed.

**Equation 3.1.19** 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. 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 3.1.23** reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

$$Q = 0.9 PR_v \qquad (3.1.23)$$

*Where:*

**P** = precipitation (in)
**Q** = runoff volume (in)
**$R_v$** = volumetric runoff coefficient [see **Equation 3.1.19**]

Ferguson (1996) has performed simulation modeling in an attempt to quantify an average ratio on a monthly basis. For the Atlanta area he has developed the following equation:

$$Q = 0.235P/S^{0.64} - 0.161 \qquad (3.1.24)$$

*Where:*

**P** = precipitation (in)

**Q** = runoff volume (in)

**S** = potential maximum retention (in) [see **Equation 3.1.5**]

**Baseflow (Bf)** – Most stormwater ponds and wetlands have little, if any, baseflow, as they are rarely placed across perennial streams. 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.

**Infiltration (I)** – Infiltration is a very complex subject and cannot be covered in detail here. The amount of infiltration depends on soils, water table depth, rock layers, surface disturbance, the presence or absence of a liner in the pond, and other factors. The infiltration rate is governed by the Darcy equation as:

$$I = Ak_hG_h \qquad (3.1.25)$$

Where:

**I** = infiltration (ac-ft/day)

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

$\mathbf{K_h}$ = saturated hydraulic conductivity or infiltration rate (ft/day)

$\mathbf{G_h}$ = 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. Infiltration rate can be established through testing, though not always accurately. As a first cut estimate **Table 3.1.8-1** can be used.

Table 3.1.8-1 Saturated Hydraulic Conductivity

| 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 |
| Clay loam | 0.09 | 0.18 |
| Silty clay loam | 0.06 | 0.12 |
| Sandy clay | 0.05 | 0.10 |
| Silty clay | 0.04 | 0.08 |
| Clay | 0.02 | 0.04 |

Source: Ferguson and Debo, "On-Site Stormwater Management," 1990

**Evaporation (E)** – Evaporation is from an open water surface. Evaporation rates are dependent on differences in vapor pressure, which, in turn, depend on temperature, wind, atmospheric pressure, water purity, and shape and depth of the pond. It is estimated or measured in a number of ways, which can be found in most hydrology textbooks. Pan evaporation methods are also used 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 3.1.8-2 | Evaporation Monthly Distribution | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| J | F | M | A | M | J | J | A | S | O | N | D |
| 3.2% | 4.4% | 7.4% | 10.3% | 12.3% | 12.9% | 13.4% | 11.8% | 9.3% | 7.0% | 4.7% | 3.2% |

**Table 3.1.8-2** gives pan evaporation rate distributions for a typical 12-month period based on pan evaporation information from five stations in and around Georgia. Figure 3.1.8-1 depicts a map of annual free water surface (FWS) evaporation averages for Georgia based on a National Oceanic and Atmospheric Administration (NOAA) assessment done 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 3.1.8-1**.

**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. However, for wetlands the estimating methods are not documented, nor are there consistent studies to assist the designer in estimating the demand wetland plants would put on water volumes. Literature values for various places in the United States vary around the free water surface lake evaporation values. Estimating Et only becomes important when wetlands are being designed and emergent vegetation covers a significant portion of the pond surface. In these cases conservative estimates of lake evaporation should be compared to crop-based Et estimates and a decision made. Crop-based Et estimates can be obtained from typical hydrology textbooks or from the web sites mentioned above.

**Overflow (Of)** – Overflow is considered as excess runoff, and in water balance design is either not considered, since the concern is for average values of precipitation, or is considered lost for all volumes above the maximum pond storage. Obviously, for long-term simulations of rainfall-runoff, large storms would play an important part in pond design.

BACK TO TOC

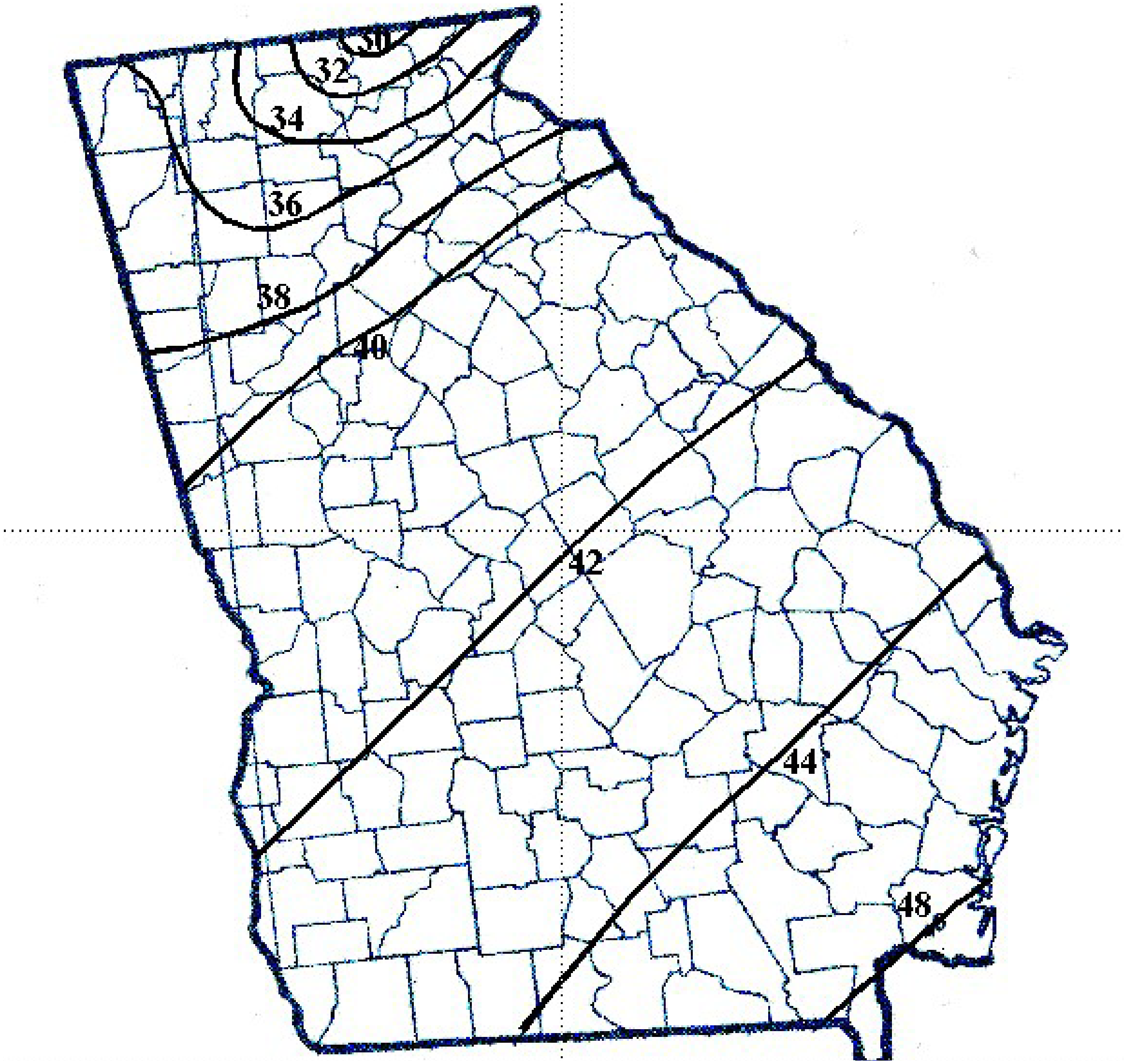


Figure 3.1.8-1 Average Annual Free Water Surface Evaporation (in inches)
(Source: NOAA, 1982)

### 3.1.8.3 EXAMPLE PROBLEM

Austin Acres, a 26-acre site in Augusta, is being developed along with an estimated 0.5-acre surface area pond. There is no baseflow. The desired pond volume to the overflow point is 2 acre-feet. Will the site be able to support the pond volume? From the basic site data we find that the site is 75% impervious with sandy clay loam soil.

- From **Equation 3.1.19**, $R_v = 0.05 + 0.009\ (75) = 0.73$. With the correction factor of 0.9 the watershed efficiency is 0.65.
- The annual lake evaporation from **Figure 3.1.8-1** is about 42 inches.
- For a sandy clay loam the infiltration rate is I = 0.34 ft/day (**Table 3.1.8-1**).
- From a grading plan it is known that about 10% of the total pond area is sloped greater than 1:4.
- Monthly rainfall for Augusta was found from the Web site provided above.

Table 3.1.8-3  Summary Calculations for Each Month of the Year

| 1 | | J | F | M | A | M | J | J | A | S | O | N | D |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | Days/mo | 31 | 28 | 31 | 30 | 31 | 30 | 31 | 31 | 30 | 31 | 30 | 31 |
| 3 | Precipitation (in) | 4.05 | 4.27 | 4.65 | 3.31 | 3.77 | 4.13 | 4.24 | 4.5 | 3.02 | 2.84 | 2.48 | 3.40 |
| 4 | 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% |
| 5 | $R_o$ (ac-ft) | 5.70 | 6.01 | 6.55 | 4.66 | 5.31 | 5.82 | 5.97 | 6.34 | 4.25 | 4.00 | 3.49 | 4.79 |
| 6 | P (ac-ft) | 0.17 | 0.18 | 0.19 | 0.14 | 0.16 | 0.17 | 0.18 | 0.19 | 0.18 | 0.12 | 0.10 | 0.14 |
| 7 | E (ac-ft) | 0.06 | 0.08 | 0.13 | 0.18 | 0.22 | 0.29 | 0.23 | 0.21 | 0.16 | 0.12 | 0.08 | 0.05 |
| 8 | I (ac-ft) | 5.01 | 4.52 | 5.01 | 4.85 | 5.01 | 4.85 | 5.01 | 5.01 | 4.85 | 5.01 | 4.85 | 5.01 |
| 9 | | | | | | | | | | | | | |
| 10 | Balance (ac-ft) | 0.81 | 1.59 | 1.61 | -0.23 | 0.24 | 0.92 | 0.91 | 1.31 | -0.63 | -1.01 | -1.33 | -0.13 |
| 11 | Running Balance (ac-ft) | 0.81 | 2.00 | 2.00 | 1.77 | 2.00 | 2.00 | 2.00 | 2.00 | 1.87 | 0.36 | 0.00 | 0.00 |

Explanation of Table:

1. Months of year
2. Days per month
3. Monthly precipitation from web site is shown in **Figure 3.1.8-2**.
4. Distribution of evaporation by month from **Table 3.1.8-2**.
5. Watershed efficiency of 0.65 times the rainfall and converted to acre-feet.
6. Precipitation volume directly into pond equals precipitation depth times pond surface area divided by 12 to convert to acre-feet
7. Evaporation equals monthly percent of 42 inches from line 4 converted to acre-feet
8. Infiltration equals infiltration rate times 90% of the surface area plus infiltration rate times 0.5 (banks greater than 1:4) times 10% of the pond area converted to acre-feet
9. Lines 5 and 6 minus lines 7 and 8
10. Accumulated total from line 10 keeping in mind that all volume above 2 acre-feet overflows and is lost in the trial design

It can be seen that for this example the pond has potential to go dry in winter months.  This can be remedied in a number of ways including compacting the pond bottom, placing a liner of clay or geosynthetics, and changing the pond geometry to decrease surface area.

```
Station: (90495) AUGUSTA_WSO_AIRPORT      From Year=1961 To Year=1990
Missing Data:   0.0%
                         Total Precipitation                        Snow     #Days Precip
      Mean    High--Yr    Low--Yr      1-Day Max     Mean  High--Yr =>.10 =>.50 =>1.
-----------------------------------------------------------------------------------------
Ja   4.05    8.91  87    0.75  81    2.78  25/1978    0.3    2.3  88      7     3    1
Fe   4.27    7.67  61    0.69  68    3.50   5/1985    1.0   14.0  73      7     3    1
Ma   4.65   11.92  80    0.88  68    5.31  10/1967    0.0    1.1  80      7     3    1
Ap   3.31    8.43  61    0.60  70    2.71  27/1961    0.0    0.0   0      5     2    1
Ma   3.77    9.61  79    1.57  87    4.44  31/1981    0.0    0.0   0      7     2    1
Jn   4.13    8.84  89    0.68  84    2.95  12/1964    0.0    0.0   0      6     3    1
Jl   4.24   11.43  67    1.02  87    3.67  21/1979    0.0    0.0   0      8     2    1
Au   4.50   11.34  86    0.65  80    5.95  29/1964    0.0    0.0   0      6     3    1
Se   3.02    9.51  75    0.31  84    4.55  20/1975    0.0    0.0   0      5     2    1
Oc   2.84   14.82  90    0.01  63    5.32  12/1990    0.0    0.0   0      4     2    1
No   2.48    7.76  85    0.57  73    3.43  22/1985    0.0    0.0   0      4     2    1
De   3.40    8.65  81    0.96  80    2.89  16/1970    0.0    0.4  71      6     3    1

An  44.66   66.04  64   32.96  78    5.95 29/08/64    1.4   14.4  73     71    30   13
Wi  11.71   20.26  87    5.62  86    3.50  5/02/85    1.3   14.4  73     20     8    3
Sp  11.73   19.93  84    4.00  85    5.31 10/03/67    0.0    1.1  80     18     8    3
Su  12.87   24.89  64    7.08  80    5.95 29/08/64    0.0    0.0   0     20     9    4
Fa   8.35   18.50  90    1.96  84    5.32 12/10/90    0.0    0.0   0     12     5    2
```

Figure 3.1.8-2  Augusta Precipitation Information

### 3.1.9 Downstream Hydrologic Assessment

The purpose of the overbank flood protection and extreme flood protection criteria is to protect downstream properties from flood increases due to upstream development. These criteria require the designer to control peak flow at the outlet of a site such that post-development peak discharge equals pre-development peak discharge. It has been shown that in certain cases this does not always provide effective water quantity control downstream from the site and may actually exacerbate flooding problems downstream. The reasons for this have to do with (1) the timing of the flow peaks, and (2) the total increase in volume of runoff. Further, due to a site's location within a watershed, there may be very little reason for requiring overbank flood control from a particular site. This section outlines a suggested procedure for determining the impacts of post-development stormwater peak flows and volumes on downstream flows that a community may require as part of a developer's stormwater management site plan. In summary, a downstream analysis may warrant a development to over-detain to protect downstream properties or may even warrant a reduction/elimination of detention because of the timing of peak discharges within the watershed.

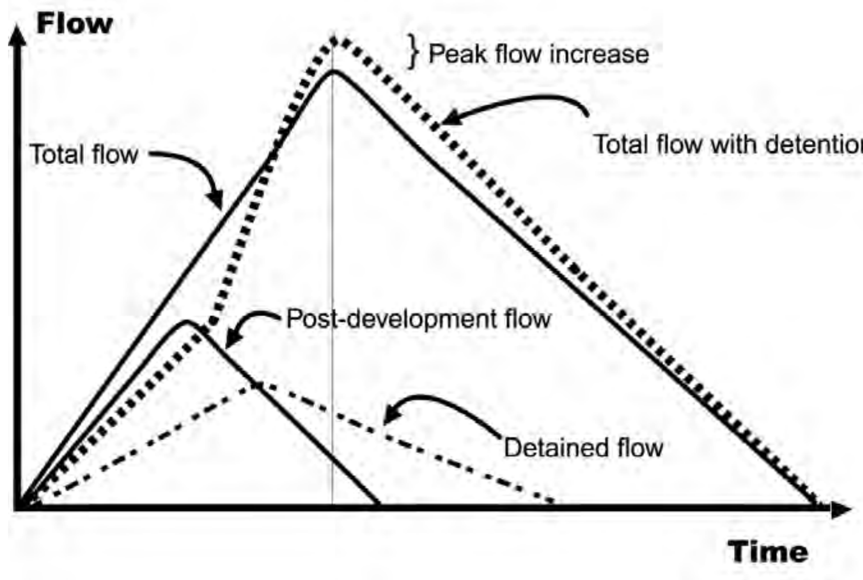



Figure 3.1.9-1 Detention Timing Example

#### 3.1.9.1 REASONS FOR DOWNSTREAM PROBLEMS

**Flow Timing**

If water quantity control (detention) structures are indiscriminately placed in a watershed and changes to the flow timing are not considered, the structural control may actually increase the peak discharge downstream. The reason for this may be seen in **Figure 3.1.9-1**. The peak flow from the site is reduced appropriately, but the timing of the flow is such that the combined detained peak flow (the larger dashed triangle) is actually higher than if no detention were required. In this case, the shifting of flows to a later time brought about by the detention pond actually makes the downstream flooding worse than if the post-development flows were not detained.

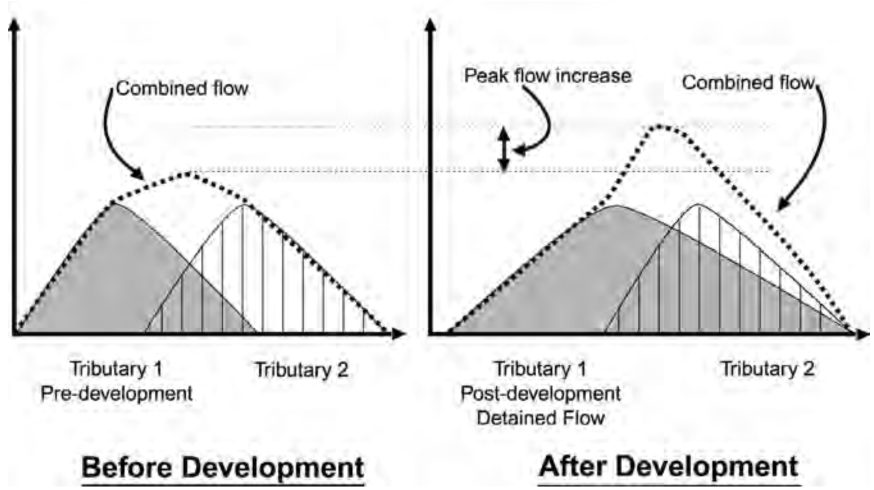



Figure 3.1.9-2 Effect of Increased Post-Development Runoff Volume with Detention on a Downstream Hydrograph

BACK TO TOC

### Increased Volume

An important impact of new development is an increase in the total runoff volume of flow. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with downstream tributaries to increase the downstream peak flows.

**Figure 3.1.9-2** illustrates this concept. The figure shows the pre- and post-development hydrographs from a development site (Tributary 1). The post-development runoff hydrograph meets the flood protection criteria (i.e., the post-development peak flow is equal to the pre-development peak flow at the outlet from the site). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than pre-development combined flow. This is because the increased volume and timing of runoff from the developed site increases the combined flow and flooding downstream. 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.

### 3.1.9.2 THE TEN-PERCENT RULE

In this Manual the "ten percent" criterion has been adopted as the most flexible and effective approach for ensuring that stormwater quantity detention ponds actually attempt to maintain pre-development peak flows throughout the system downstream.

The ten-percent rule recognizes the fact that a structural control providing detention has a "zone of influence" downstream where its effectiveness can be felt. Beyond this zone of influence the structural control becomes relatively small and insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, that zone of influence is considered to be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. For example, if the structural control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater.

Typical steps in the application of the ten-percent rule are:

1. Determine the target peak flow for the site for predevelopment conditions.
2. Using a topographic map determine the lower limit of the zone of influence (10% point).
3. Using a hydrologic model determine the pre-development peak flows and timing of those peaks at each tributary junction beginning at the pond outlet and ending at the next tributary junction beyond the 10% point.
4. Change the land use on the site to post-development and rerun the model.
5. Design the structural control facility such that the overbank flood protection (25-year) post-development flow does not increase the peak flows at the outlet and the determined tributary junctions.
6. If it does increase the peak flow, the structural control facility must be redesigned or one of the following options considered:
   - Control of the overbank flood volume ($Q_{p25}$) may be waived by the local authority saving the developer the cost of sizing a detention basin for overbank flood control. In this case the ten-percent rule saved the construction of an unnecessary structural control facility that would have been detrimental to the watershed flooding problems. In some communities this situation may result in a fee being paid to the local government in lieu of detention. That fee would go toward alleviating downstream flooding or making channel or other conveyance improvements.
   - Work with the local government to reduce the flow elevation through channel or flow conveyance structure improvements downstream.
   - Obtain a flow easement from downstream property owners to the 10% point.

Even if the overbank flood protection requirement is eliminated, the water quality treatment ($WQ_v$), channel protection ($CP_v$), and extreme flood protection ($Q_f$) criteria will

BACK TO TOC

### 3.1.9.3 EXAMPLE PROBLEM

**Figure 3.1.9-3** illustrates the concept of the ten-percent rule for two sites in a watershed.

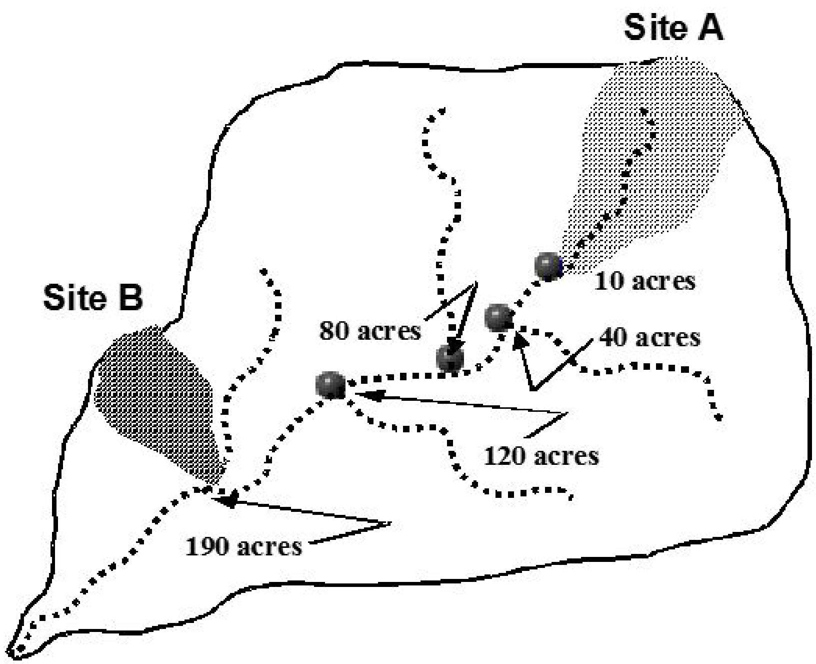



Figure 3.1.9-3 Example of the Ten-Percent Rule

**Discussion**

Site A is a development of 10 acres, all draining to a wet ED stormwater pond. The overbank flooding and extreme flood portions of the design are going to incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked "80 acres." The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The assumption is that if there is no peak flow increase at the 80-acre point then there will be no increase through the next stream reach downstream through the 10% point (100 acres) to the 120-acre point. The designer constructs a simple HEC-1 model of the 80-acre areas using single existing condition sub-watersheds for each tributary. Key detention structures existing in other tributaries must be modeled. An approximate curve number is used since the actual peak flow is not key for initial analysis; only the increase or decrease is important. The accuracy in curve number determination is not as significant as an accurate estimate of the time of concentration. Since flooding is an issue downstream, the pond is designed (through several iterations) until the peak flow does not increase at junction points downstream to the 80-acre point.

Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin with care taken to estimate the time of concentration for input into the TR-20 model of the watershed. The model shows that a detention facility, in this case, will actually increase the peak flow in the stream.

## 3.2 Methods for Estimating Stormwater Volume Reduction

### 3.2.1 Introduction

The runoff reduction approach to addressing Water Quality requirements is discussed in detail in Volume 1 and Subsections 2.2.2 and 2.2.3 of Volume 2. Best management practices that incorporate stormwater runoff reduction are provided in Chapter 4. Within each BMP section of the manual, design steps have been provided for each unique application and practice, and detailed design examples are provided in Appendices B-2 (bioretention area), B-4 (infiltration trench), and B-5 (enhanced swale). Refer to **Table 4.1.3-1** (*BMP Selection Guide*) for applicable BMPs that can provide runoff reduction.

Where runoff reduction practices are used, an adjusted curve number (CN) is computed that is lower than the original CN based on an actual stormwater volume removed from the total runoff, see Subsection 3.1.7.5 for additional information. This adjusted CN calculation is performed for each storm event being analyzed after a portion of the stormwater runoff has been quantified and removed from the site. In other words, an adjusted CN can be computed separately for stream channel protection, overbank flood protection, and extreme flood protection storm events since the volume of runoff changes for any given rainfall event. The modified equation discussed in Subsection 3.1.7.5 was originally presented in the 2010 Center for Watershed Protection journal article titled, *The Runoff Reduction Method*.

As a Georgia Stormwater Management Manual design aid, a site development review tool has been created to aid in the design and documentation of stormwater management requirements. This tool can be accessed and downloaded at the following location: www.georgiastormwater.com.

Within the tool, an "Instructions" and "Tool Flowchart" tab have been created to provide specific input requirements and responsibilities of the end user. These same instructions and workflow process are particularly beneficial for local communities who will ultimately review the information as part of their stormwater management review process.

# 3.3 Storage Design

## 3.3.1 General Storage Concepts

### 3.3.1.1 INTRODUCTION

This section provides general guidance on stormwater runoff storage for meeting stormwater management control requirements (i.e., water quality treatment, downstream channel protection, overbank flood protection, and extreme flood protection).

Storage of stormwater runoff within a stormwater management system is essential to providing the extended detention of flows for water quality treatment and downstream channel protection, as well as for peak flow attenuation of larger flows for overbank and extreme flood protection. Runoff storage can be provided within an on-site system through the use of best management practices and/or nonstructural features and landscaped areas. **Figure 3.3.1-1** illustrates various storage facilities that can be considered for a development site.

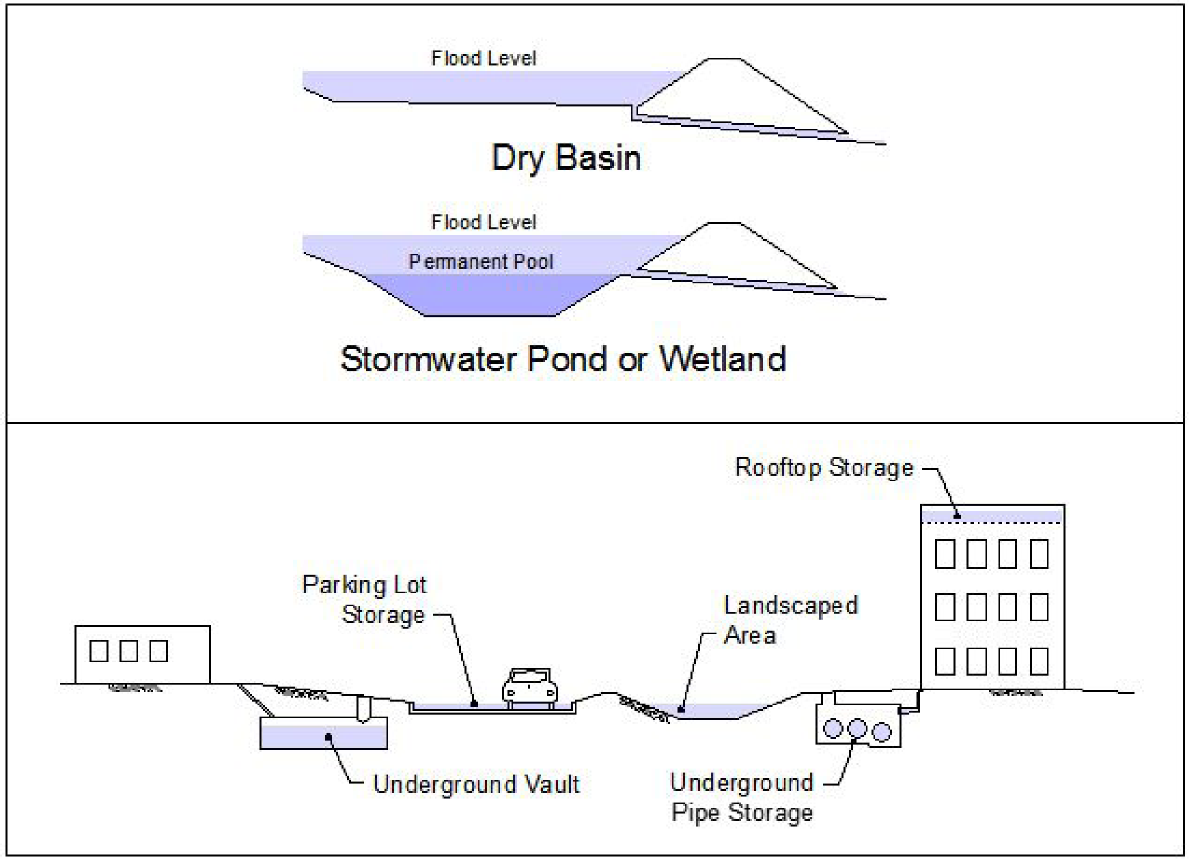



Figure 3.3.1-1 Examples of Typical Stormwater Storage Facilities



BACK TO TOC

### 3.3.1.2 STORAGE CLASSIFICATION

Stormwater storage(s) can be classified as detention, extended detention or retention. Some facilities include one or more types of storage.

Stormwater *detention* is used to reduce the peak discharge and detain runoff for a specified short period of time. Detention volumes are designed to completely drain after the design storm has passed. Detention is used to meet overbank flood protection criteria, and extreme flood criteria where required.

*Extended detention* (ED) is used to drain a runoff volume over a specified period of time, typically 24 hours, and is used to meet channel protection criteria. Some best management practices (wet ED pond, micropool ED pond, and wetlands) also include extended detention storage of a portion of the water quality volume.

*Retention* facilities are designed to contain a permanent pool of water, such as stormwater ponds and wetlands, which is used for water quality treatment.

Storage facilities are often classified on the basis of their location and size. On-site storage is constructed on individual development sites. *Regional* storage facilities are constructed at the lower end of a subwatershed and are designed to manage stormwater runoff from multiple projects and/or properties. A discussion of regional stormwater controls is found in Section 4.1.

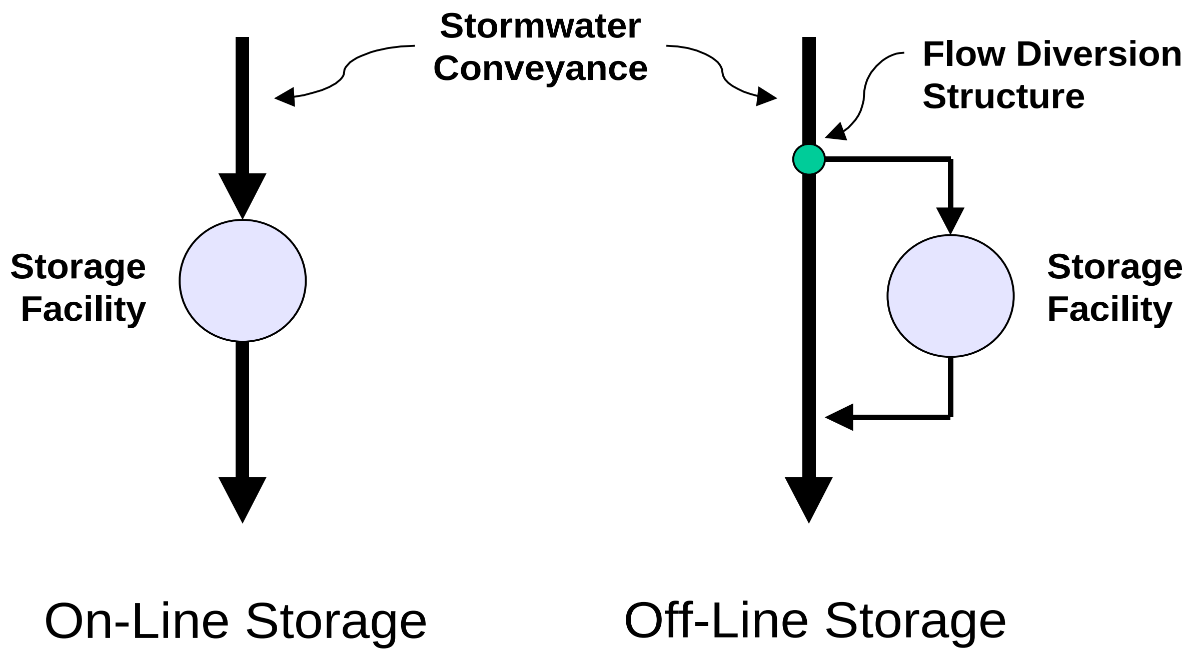



Figure 3.3.1-2 On-Line versus Off-Line Storage

Storage can also be categorized as on-line or off-line. On-line storage uses a structural control facility that intercepts flows directly within a conveyance system or stream. Off-line storage is a separate storage facility to which flow is diverted from the conveyance system. **Figure 3.3.1-2** illustrates on-line versus off-line storage.

### 3.3.1.3 STAGE-STORAGE RELATIONSHIP

A stage-storage curve defines the relationship between the depth of water and storage volume in a storage facility (see **Figure 3.3.1-3**). The volume of storage can be calculated by using simple geometric formulas expressed as a function of depth.

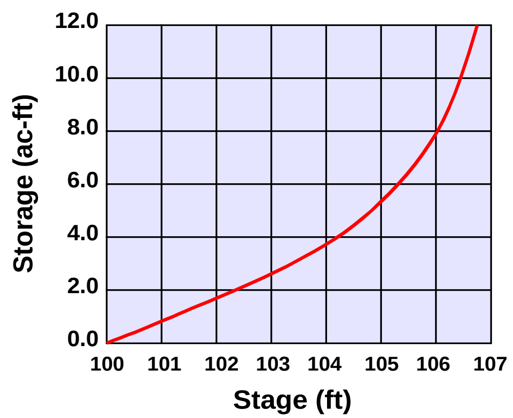



Figure 3.3.1-3 Stage-Storage Curve

The storage volume for natural basins may be developed using a topographic map and the double-end area, frustum of a pyramid, prismoidal or circular conic section formulas.

The double-end area formula (see **Figure 3.3.1-4**) is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad (3.3.1)$$

*Where:*

**$V_{1,2}$** = storage volume ($ft^3$) between elevations 1 and 2

**$A_1$** = surface area at elevation 1 ($ft^2$)

**$A_2$** = surface area at elevation 2 ($ft^2$)

**d** = change in elevation between points 1 and 2 (ft)

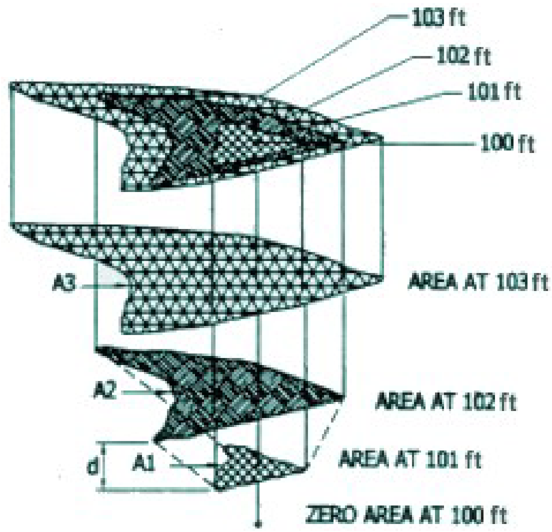



Figure 3.3.1-4 Double-End Area Method

The frustum of a pyramid formula is expressed as:

$$V = d/3\ [A_1 + (A_1 \times A_2)^{0.5} + A2]/3 \qquad (3.3.2)$$

*Where:*

**V** = volume of frustum of a pyramid (ft3)

**d** = change in elevation between points 1 and 2 (ft)

**$A_1$** = surface area at elevation 1 ($ft^2$)

**$A_2$** = surface area at elevation 2 ($ft^2$)

The prismoidal formula for trapezoidal basins is expressed as:

$$V = LWD + (L + W)\ ZD^2 + 4/3\ Z^2\ D^3 \qquad (3.3.3)$$

*Where:*

**V** = volume of trapezoidal basin ($ft^3$)

**L** = length of basin at base (ft)

**W** = width of basin at base (ft)

**D** = depth of basin (ft)

**Z** = side slope factor, ratio of horizontal to vertical

The circular conic section formula is:

$$V = 1.047\ D(R_1^2 + R_2^2 + R_1R_2) \qquad (3.3.4)$$

$$V = 1.047\ D(3\ R_1^2 + 3ZDR_1 + Z_2D^2) \qquad (3.3.5)$$

*Where:*

**$R_1$ , $R_2$** = bottom and surface radii of the conic section (ft)

**D** = depth of basin (ft)

**Z** = side slope factor, ratio of horizontal to vertical

#### 3.3.1.4 STAGE-DISCHARGE RELATIONSHIP

AA stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility (see **Figure 3.3.1-5**). A typical storage facility has two outlets or spillways: a principal outlet and a secondary (or emergency) outlet. The principal outlet is usually designed with a capacity sufficient to convey the design flows without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. This spillway should be designed taking into account the potential threat to downstream areas if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway. For more details, see Section 3.4, *Outlet Structures*.

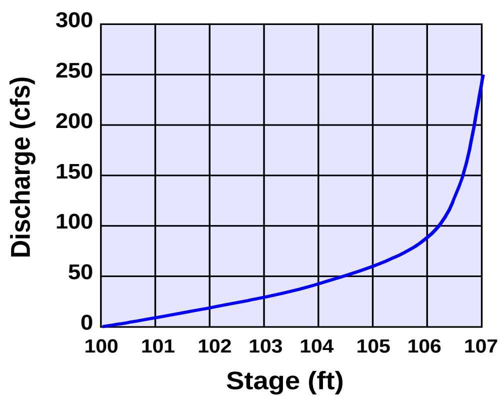



Figure 3.3.1-5 Stage-Discharge Curve

### 3.3.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in **Table 3.3.2-1** will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3.3.2-1 Symbols and Definitions

| Symbol | Definition | Units |
|---|---|---|
| A | Cross sectional or surface area | $ft^2$ |
| $A_m$ | Drainage area | $mi^2$ |
| C | Weir coefficient | - |
| d | Change in elevation | ft |
| D | Depth of basin or diameter of pipe | ft |
| t | Routing time period | sec |
| g | Acceleration due to gravity | $ft/s^2$ |
| H | Head on structure | ft |
| $H_c$ | Height of weir crest above channel bottom | ft |
| K | Coefficient | - |
| l | Inflow rate | cfs |
| L | Length | ft |
| Q, q | Peak inflow or outflow | cfs, in |
| R | Surface radii | ft |
| S, $V_s$ | Storage volume | $ft^3$ |
| $t_b$ | Time base on hydrograph | hrs |
| $T_l$ | Duration of basin inflow | hrs |
| $t_p$ | Time to peak | hrs |
| $V_s$, S | Storage Volume | $ft^3$, in, acre-ft |
| $V_r$ | Volume of runoff | $ft^3$, in, acre-ft |
| W | Width of basin | ft |
| Z | Side slope factor | - |

## 3.3.3 General Storage Design Procedures

### 3.3.3.1 INTRODUCTION

This section discusses the general design procedures for designing storage to provide standard detention of stormwater runoff for overbank and extreme flood protection ($Q_{p25}$ and $Q_f$).

The design procedures for all structural control storage facilities are the same whether or not they include a permanent pool of water. In the latter case, the permanent pool elevation is taken as the "bottom" of storage and is treated as if it were a solid basin bottom for routing purposes.

It should be noted that the location of structural stormwater controls is very important as it relates to the effectiveness of these facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different down¬stream locations. Therefore, a downstream peak flow analysis should be performed as part of the storage facility design process (see Subsection 3.1.9).

In multi-purpose multi-stage facilities such as stormwater ponds, the design of storage must be integrated with the overall design for water quality treatment objectives. See Chapter 4 for further guidance and criteria for the design of structural stormwater controls.

An important consideration in these designs is the sediment volume that the system is capable of storing before performance and/or capacity are reduced. For larger watersheds, or regional detention facilities, the sediment volume is estimated from the sediment produced per year, times the years between dredging or similar maintenance. Provisions should be made in the layout to facilitate access for dredging equipment to the storage area and the maximum sediment depth should be defined. Maintenance plans should discuss dredging and set a time interval for evaluation such as once per year.

For smaller watersheds, and all other stormwater detention facilities, sedimentation can be addressed by a local community requirement for an as-built pond survey and/or certification process. During the construction phase of a development or redevelopment, sedimentation occurs and can reduce the storage capacity of the post-construction stormwater detention basin drastically. Once final stabilization of a site has occurred, the accumulated sediment should be removed and the detention basin surveyed to comply with the originally approved design volume. It is understood that sedimentation of a fully stabilized site over time is minimal for smaller watersheds, when compared to a larger watershed during active construction activities.

### 3.3.3.2 DATA NEEDS

The following data are needed for storage design and routing calculations:

- Inflow hydrograph for all selected design storms
- Stage-storage curve for proposed storage facility
- Stage-discharge curve for all outlet control structures
- Estimate of sediment deposited per year and the number of years desired before dredging.

#### 3.3.3.3 DESIGN PROCEDURE

A general procedure for using the above data in the design of storage facilities is presented below.

(Step 1) Compute inflow hydrograph for run¬off from the 25- ($Qp_{25}$), and 100-year ($Q_f$) design storms using the hydrologic methods outlined in Section 3.1. Both existing- and post-development hydrographs are required for 25-year design storm.

(Step 2) Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see Subsection 3.3.4).

(Step 3) Determine the physical dimensions necessary to hold the estimated volume from Step 2, including free¬board. Include the estimated volume of sediment storage (if appropriate). The maximum storage requirement calculated from Step 2 should be used. From the selected shape determine the maximum depth in the pond.

(Step 4) Select the type of outlet and size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allow¬able discharge at this stage.

(Step 5) Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model. If the routed post-development peak discharges from the 25-year design storm exceed the existing-development peak discharges, then revise the available storage volume, outlet device, etc., and return to Step 3.

(Step 6) Perform routing calculations using the 100-year hydrograph to determine if any increases in downstream flows from this hydrograph will cause damages and/or drainage and flooding problems. If problems will be created (e.g., flooding of habitable dwellings, property damage, or public access and/or utility interruption) then the storage facility must be designed to control the increased flows from the 100-year storm. If not then consider emergency overflow from runoff due to the 100-year (or larger) design storm and established freeboard requirements.

(Step 7) Evaluate the downstream effects of detention outflows for the 25- and 100-year storms to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed though the downstream channel system until a confluence point is reached where the drainage area being analyzed represents 10% of the total drainage area (see Subsection 3.1.9).

(Step 8) Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

Routing of hydrographs through storage facilities is critical to the proper design of these facilities. Although storage design procedures using inflow/outflow analysis without routing have been developed, their use in designing detention facilities has not produced acceptable results in many areas of the country, including Georgia.

Although hand calculation procedures are available for routing hydrographs through storage facilities, they are very time consuming, especially when several different designs and iterations are evaluated. Many standard hydrology and hydraulics textbooks give examples of hand-routing techniques. For this Manual, it assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given here.

BACK TO TOC

## 3.3.4 Preliminary Detention Calculations

### 3.3.4.1 INTRODUCTION

Procedures for preliminary detention calculations are included here to provide a simple method that can be used to estimate storage needs and also provide a quick check on the results of using different computer programs. Standard routing should be used for actual (final) storage facility calculations and design.

### 3.3.4.2 STORAGE VOLUME

For small drainage areas, a preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in **Figure 3.3.4-1**.

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

$$V_S = 0.5T_I\,(Q_i - Q_O) \qquad (3.3.6)$$

*Where:*

**$V_S$** = storage volume estimate (ft³)
**$Q_i$** = peak inflow rate (cfs)
**$Q_O$** = peak outflow rate (cfs)
**$T_i$** = duration of basin inflow (s)

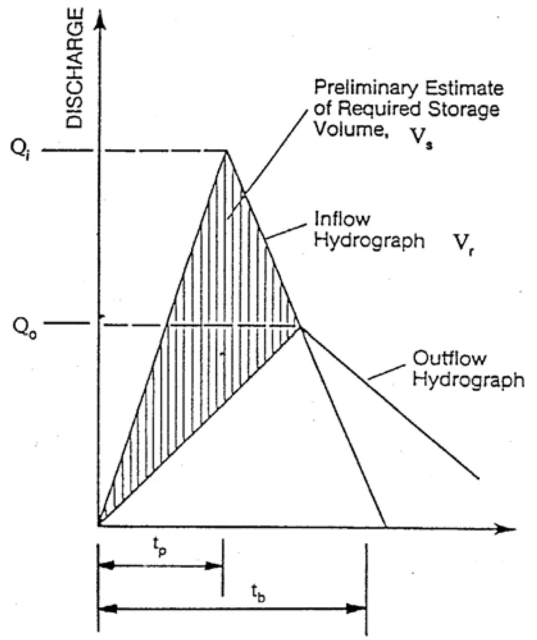



Figure 3.3.4-1 Triangular-Shaped Hydrographs
(For Preliminary Estimate of Required Storage Volume)

### 3.3.4.3 ALTERNATIVE METHOD

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff and Singh, 1976).

(Step 1) Determine input data, including the allowable peak outflow rate, $Q_O$, the peak flow rate of the inflow hydrograph, $Q_i$, the time base of the inflow hydro¬graph, $t_b$, and the time to peak of the inflow hydrograph, $t_p$.

(Step 2) Calculate a preliminary estimate of the ratio $V_S/V_r$ using the input data from Step 1 and the following equation:

$$\frac{V_S}{V_R} = \frac{1.291\times\left(1-\dfrac{Q_O}{Q_I}\right)^{0.753}}{\left(\dfrac{t_p}{t_b}\right)^{0.411}} \qquad (3.3.7)$$

Where:

$V_s$ = volume of storage (in)
$V_r$ = volume of runoff (in)
$Q_O$ = outflow peak flow (cfs)
$Q_i$ = inflow peak flow (cfs)
$t_b$ = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak]
$t_p$ = time to peak of the inflow hydrograph (hr)

(Step 3) Multiply the volume of runoff, $V_r$, times the ratio $V_S/V_r$, calculated in Step 2 to obtain the estimated storage volume $V_S$.

### 3.3.4.4 PEAK FLOW REDUCTION

A preliminary es¬timate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

(Step 1) Determine volume of runoff, $V_r$, peak flow rate of the inflow hydrograph, $Q_i$, time base of the inflow hydrograph, $t_b$, time to peak of the inflow hydrograph, $t_p$, and storage volume $V_S$.

(Step 2) Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the follow-ing equation (Wycoff and Singh, 1976):

$$Q_O/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546} \qquad (3.3.8)$$

*Where:*

**$Q_O$** = outflow peak flow (cfs)
**$Q_i$** = inflow peak flow (cfs)
**$V_S$** = volume of storage (in)
**$V_r$** = volume of runoff (in)
**$t_b$** = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak]
**$t_p$** = time to peak of the inflow hydrograph (hr)

(Step 3) Multiply the peak flow rate of the inflow hydrograph, $Q_i$, times the potential peak flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, $Q_O$, for the selected storage volume.

BACK TO TOC

## 3.3.5 Channel Protection Volume Estimation

### 3.3.5.1 INTRODUCTION

The Simplified NRCS TR-55 Peak Runoff Rate Estimation approach (see Subsection 3.1.5.7) can be used for estimation of the Channel Protection Volume ($CP_v$) for storage facility design.

This method should not be used for standard detention design calculations. See either Subsection 3.3.4 or the modified rational method in Subsection 3.3.6 for preliminary detention calculations without formal routing.

### 3.3.5.2 BASIC APPROACH

For $CP_v$ estimation, using **Figures 3.1.5-6** and **3.1.5-7** in Section 3.1, the unit peak discharge ($q_U$) can be determined based on $I_a/P$ and time of concentration ($t_C$). Knowing $q_U$ and T (extended detention time, typically 24 hours), the $q_O/q_I$ ratio (peak outflow discharge/peak inflow discharge) can be estimated from **Figure 3.3.5-1**.

Using the following equation from TR-55 for a Type II or Type III rainfall distribution, $V_S/V_r$ can be calculated.

Note: **Figure 3.3.4-1** can also be used to estimate VS/Vr.

$$V_S/V_r = 0.682 - 1.43(q_O/q_I) + 1.64\,(q_O/q_I)^2 - 0.804(q_O/q_I)^3 \quad (3.3.9)$$

Where:

**$V_S$** = required storage volume (acre-feet)
**$V_r$** = runoff volume (acre-feet)
**$q_O$** = peak outflow discharge (cfs)
**$q_I$** = peak inflow discharge (cfs)

The required storage volume can then be calculated by:

$$V_S = \frac{(V_S/V_r)(Q_d)(A)}{12} \quad (3.3.10)$$

Where:

**$V_S$** and **$V_r$** are defined above
**$Q_d$** = the developed runoff for the design storm (inches)
**A** = total drainage area (acres)

While the TR-55 short-cut method reports to incorporate multiple stage structures, experience has shown that an additional 10-15% storage is required when multiple levels of extended detention are provided inclusive with the 25-year storm.

**3.3.5.3 EXAMPLE PROBLEM**

Compute the 100-year peak discharge for a 50-acre wooded watershed located in Peachtree City, which will be developed as follows:

- Forest land - good cover (hydrologic soil group B) = 10 ac
- Forest land - good cover (hydrologic soil group C) = 10 ac
- 1/3 Acre residential (hydrologic soil group B) = 20 ac

Industrial development (hydrological soil group C) = 10 ac

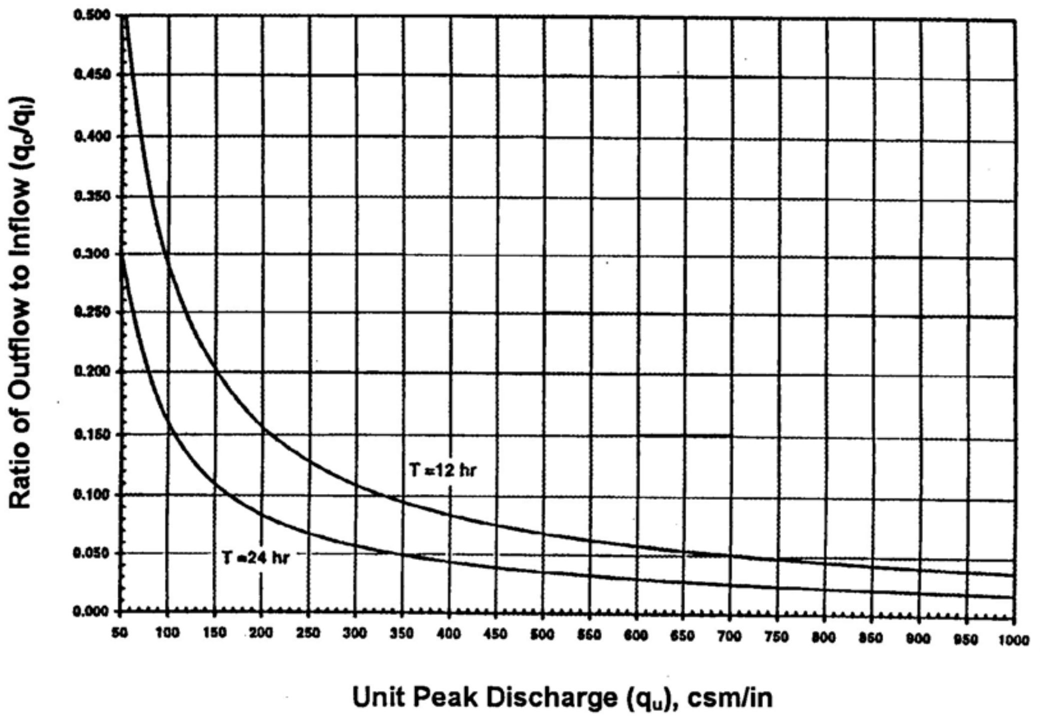



Figure 3.3.5-1 Detention Time vs. Discharge Ratios
(Source: MDE, 1998)

Other data include the following:

- Total impervious area = 18 acres
- % of pond and swamp area = 0

**Computations**

1. Calculate rainfall excess:

   » The 100-year, 24-hour rainfall is 8.22 inches (From NOAA Atlas 14).

   » The 1-year, 24 hour rainfall is 3.37 inches (From NOAA Atlas 14).

   » Composite weighted runoff coefficient is:

| Dev # | Area | % Total | CN | Composite CN |
|---|---|---|---|---|
| 1 | 10 ac | 20% | 55 | 11.0 |
| 2 | 10 ac | 20% | 70 | 14.0 |
| 3 | 20 ac | 40% | 72 | 28.8 |
| 4 | 10 ac | 20% | 91 | 18.2 |
| Total | 50 ac | 100% | | 72 |

*from Equation 3.1.6, Q (100-year) = 4.89 inches
$Q_d$ (1-year developed) = 1.0 inches

2. Calculate time of concentration

| Segment | Type of Flow | Length (ft) | Slope (%) |
|---|---|---|---|
| 1 | Overland n=0.24 | 40 | 2.0% |
| 2 | Shallow channel | 750 | 1.7% |
| 3 | Main channel | 1100 | 0.5% |

*For the main channel, n = .06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

The hydrologic flow path for this watershed = 1,890 ft

**Segment 1** - Travel time from **equation 2.1.9** with $P_2$ = 3.84 in
(From NOAA Atlas 14)

$T_t = [0.42(0.24 \text{ X } 40)^{0.8}] / [(3.84)^{0.5} (.020)^{0.4}] = 6.26$ minutes

BACK TO TOC

**Segment 2** - Travel time from **Figure 3.1.5-5** or **equation 3.1.9**

V = 2.1 ft/sec (from **equation 3.1.9**)

$T_t$ = 750 / 60 (2.1) = 5.95 minutes

**Segment 3** - Using **equation 3.1.11**

V = (1.49/.06) $(1.43)^{0.67}$ $(.005)^{0.5}$ = 2.23 ft/sec

$T_t$ = 1100 / 60 (2.23) = 8.22 minutes

$t_c$ = 6.26 + 5.95 + 8.22 = 20.43 minutes (.34 hours)

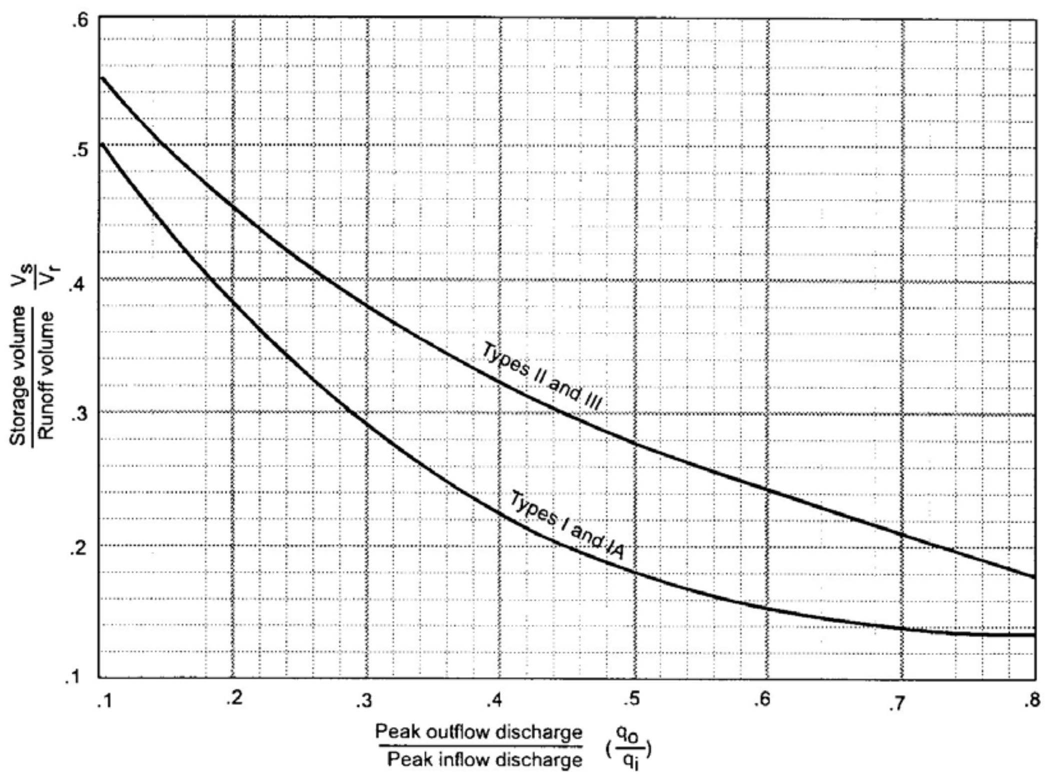



Figure 3.3.5-2 Approximate Detention Basin Routing for Rainfall Types I, IA, II, and III
(Source: TR-55, 1986)

3. Calculate $I_a$/P for $C_n$ = 72 (**Table 3.1.5-1**), $I_a$ = .778 (**Table 3.1.5-3**)

$I_a$/P = (.778 / 8.23 = .095

Note: Use $I_a$/P = .10 to facilitate use of **Figure 3.1.5-6.** Straight line interpolation could also be used.

4. Unit discharge $q_u$ (100-year) from **Figure 3.1.5-6** = 650 csm/in, $q_u$ (1-year) = 580 csm/in

5. Calculate peak discharge with $F_p$ = 1 using **equation 3.1.12**

$Q_{100}$ = 650 (50/640)(4.89)(1) = 248 cfs

6. Calculate water quality volume ($WQ_v$)

Compute runoff coefficient, $R_v$

$R_v$ = 0.50 + (IA)(0.009)
= 0.50 + (18)(0.009)
= 0.21

Compute water quality volume, $WQ_v$

$WQ_v$ = 1.2($R_v$)(A)/12
= 1.2(.21)(50)/12
= 1.05 acre-feet

7. (Calculate channel protection volume ($CP_v$ = $V_s$)

Knowing $q_u$ (1-year) = 580 csm/in from Step 3 and T (extended detention time of 24 hours), find $q_O/q_I$ from **Figure 3.3.5-1**.

$q_O/q_I$ = 0.03

For a Type II rainfall distribution:

$V_S/V_r$ = 0.682 – 1.43 ($q_O/q_I$) + 1.64 $(q_O/q_I)^2$ – 0.804 $(q_O/q_I)^3$
= 0.682 – 1.43 (0.03) + 1.64 (0.03) – 0.804 (0.03)
= 0.64

Therefore, stream channel protection volume with $Q_d$ (1-year developed) = 1.0 inches, from Step 1, is $CP_v$ = $V_S$ = ($V_S/V_r$)($Q_d$)(A)/12 = (0.64)(1.0)(50)/12 = 2.67 acre-feet

BACK TO TOC

## 3.3.6 The Modified Rational Method

### 3.3.6.1 INTRODUCTION

For drainage areas of *less than 5 acres*, a modification of the Rational Method can be used for the estimation of storage volumes for detention calculations.

The Modified Rational Method uses the peak flow calculating capability of the Rational Method paired with assumptions about the inflow and outflow hydrographs to compute an approximation of storage volumes for simple detention calculations. There are many variations on the approach. **Figure 3.3.6-1** illustrates one application. The rising and falling limbs of the inflow hydrograph have a duration equal to the time of concentration ($t_c$). An allowable target outflow is set ($Q_a$) based on pre-development conditions. The storm duration is $t_d$, and is varied until the storage volume (shaded gray area) is maximized. It is normally an iterative process done by hand or on a spreadsheet. Downstream analysis is not possible with this method as only approximate graphical routing takes place.

### 3.3.6.2 DESIGN EQUATIONS

The design of detention using the Modified Rational Method is presented as a noniterative approach suitable for spreadsheet calculation (Debo & Reese, 1995).

The allowable release rate can be determined from:

$$Q_a = C_a \, i \, A \qquad (3.3.11)$$

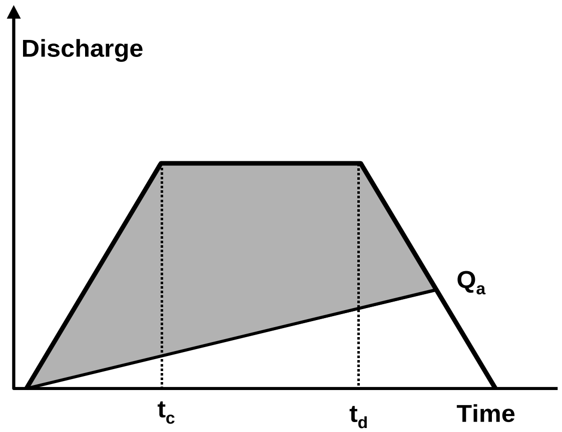



Figure 3.3.6-1 Modified Rational Definitions

*Where:*

**$Q_a$** = allowable release rate (cfs)
**$C_a$** = predevelopment Rational Method runoff coefficient
**i** = rainfall intensity for the corresponding time of concentration (in/hr)
**A** = area (acres)

The critical duration of storm, the time value to determine rainfall intensity, at which the storage volume is maximized is:

$$T_d = \sqrt{\frac{2CAab}{Q_a}} - b \qquad (3.3.12)$$

*Where:*

**$T_d$** = critical storm duration (min)
**$Q_a$** = allowable release rate (cfs)

**C** = developed condition Rational Method runoff coefficient
**A** = area (acres)
**a, b** = rainfall factors dependent on location and return period taken from **Table 3.3.6-1.**

The required storage volume, in cubic feet can be obtained from **Equation 3.3.13**.

$$Vp_{reliminary} = 60\,[CAa - (2CabAQ_a)^{1/2} + (Q_a/2)\,(b-t_c)] \qquad (3.3.13a)$$

$$V_{max} = V_{preliminary} * P_{180}/P_{td} \qquad (3.3.13b)$$

*Where:*

**$V_{preliminary}$** = preliminary required storage ($ft^3$)
**$V_{max}$** = required storage ($ft^3$)
**$t_c$** = time of concentration for the developed condition (min)
**$P_{180}$** = 3-hour (180-minute) storm depth (in)
**$P_{td}$** = storm depth for the critical duration (in)
All other variables are as defined above

The equations above include the use of an adjustment factor to the calculated storage volume to account for undersizing. The factor ($P_{180}/P_{td}$) is the ratio of the 3-hour storm depth for the return frequency divided by the rainfall depth for the critical duration calculated in **Equation 3.3.12**. The Modified Rational Method also often undersizes storage facilities in flat and more sandy areas where the target discharge may be set too large, resulting in an oversized orifice. In these locations a C factor of 0.05 to 0.1 should be used.

BACK TO TOC

#### 3.3.6.3 EXAMPLE PROBLEM

A 5-acre site is to be developed in Atlanta. Based on site and local information, it is determined that channel protection is not required and that limiting the 25-year and 100-year storm is also not required. The local government has determined that the development must detain the 2-year and 10-year storms. Rainfall values are taken from NOAA Atlas 14. The following key information is obtained:

**A** = 5 acres
Slope is about 5%
**Pre-development $t_c$** = 21 minutes and C factor = 0.22
**Post-development $t_c$** = 10 minutes and C factor = 0.80

| Steps | 2 - year | 10 - year |
|---|---|---|
| $t_c$ (min) | 21 | 21 |
| i (in/hr) | 3.34 | 4.51 |
| $Q_a$ (**Equation 3.3.11**) (cfs) | 3.67 | 4.96 |
| a (from **Table 3.3.6-1**) | 123.19 | 184.23 |
| b (from **Table 3.3.6-1**) | 15.91 | 19.96 |
| $V_{max}$ (**Equation 3.3.13**) ($ft^3$) | 16,017 | 23,199 |
| $P_{180}$ (from NOAA Atlas 14) (in) | 2.43 | 3.42 |
| $T_d$ (**Equation 3.3.12**) (min) | 49 | 57 |
| $P_{td}$ (from NOAA Atlas 14) (in) | 1.62 | 2.70 |
| $V_{max}$ (**Equation 3.3.13**) ($ft^3$) | 24,025 | 29,385 |

Table 3.3.6-1 Rainfall Factors "a" and "b" for the Modified Rational Method (1-year through 100-year return periods)

| City | | Return Interval | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 5 | 10 | 25 | 50 | 100 |
| Albany | a | 126.72 | 159.17 | 198.14 | 230.00 | 271.84 | 305.29 | 341.98 |
| | b | 16.02 | 19.72 | 22.52 | 24.49 | 26.00 | 26.97 | 28.23 |
| Atlanta | a | 97.05 | 123.19 | 157.99 | 184.23 | 219.21 | 249.86 | 278.71 |
| | b | 12.88 | 15.91 | 18.44 | 19.96 | 21.13 | 22.28 | 23.01 |
| Athens | a | 106.01 | 126.29 | 162.23 | 187.80 | 224.41 | 253.05 | 281.69 |
| | b | 15.41 | 16.95 | 19.57 | 20.87 | 22.19 | 22.99 | 23.68 |
| Augusta | a | 119.32 | 142.78 | 171.04 | 192.10 | 221.48 | 247.98 | 271.24 |
| | b | 17.05 | 19.12 | 20.34 | 20.96 | 21.40 | 22.10 | 22.32 |
| Bainbridge | a | 128.79 | 171.90 | 215.02 | 245.38 | 291.64 | 329.59 | 367.38 |
| | b | 16.39 | 21.13 | 24.33 | 25.87 | 27.73 | 29.12 | 30.26 |
| Brunswick | a | 177.81 | 191.06 | 233.75 | 266.24 | 314.79 | 352.59 | 367.38 |
| | b | 26.30 | 24.13 | 27.51 | 29.49 | 31.77 | 33.16 | 34.22 |
| Columbus | a | 113.09 | 142.00 | 177.92 | 205.63 | 246.52 | 273.92 | 306.45 |
| | b | 15.67 | 17.87 | 20.34 | 21.88 | 23.63 | 24.11 | 25.13 |
| Macon | a | 111.40 | 139.06 | 176.78 | 203.43 | 242.56 | 272.93 | 306.45 |
| | b | 15.48 | 17.68 | 20.55 | 21.94 | 23.47 | 24.38 | 25.59 |
| Metro Chattanooga | a | 93.15 | 116.20 | 148.58 | 171.22 | 201.95 | 227.07 | 254.06 |
| | b | 14.25 | 15.97 | 18.00 | 18.91 | 19.60 | 20.12 | 20.84 |
| Peachtree City | a | 101.63 | 125.43 | 160.73 | 185.58 | 219.86 | 250.95 | 277.86 |
| | b | 13.72 | 15.94 | 18.64 | 19.91 | 21.02 | 22.25 | 22.81 |
| Rome | a | 88.91 | 120.41 | 159.75 | 188.99 | 229.97 | 264.15 | 292.64 |
| | b | 12.10 | 16.05 | 19.06 | 20.82 | 22.51 | 23.81 | 24.21 |
| Roswell | a | 93.33 | 126.28 | 159.12 | 182.23 | 219.74 | 246.68 | 273.06 |
| | b | 12.28 | 16.92 | 19.00 | 19.96 | 21.54 | 22.17 | 22.67 |
| Savannah | a | 135.97 | 178.06 | 230.29 | 266.68 | 325.90 | 373.89 | 418.97 |
| | b | 19.41 | 23.22 | 28.28 | 30.80 | 34.41 | 36.82 | 38.60 |
| Toccoa | a | 114.77 | 124.54 | 164.15 | 192.50 | 234.48 | 266.57 | 299.01 |
| | b | 19.58 | 17.40 | 20.33 | 21.85 | 23.67 | 24.65 | 25.51 |
| Valdosta | a | 132.93 | 165.35 | 203.32 | 229.47 | 269.41 | 301.00 | 333.57 |
| | b | 16.72 | 19.94 | 22.63 | 23.79 | 25.20 | 26.10 | 26.98 |
| Vidalia | a | 120.40 | 161.23 | 201.42 | 230.71 | 272.84 | 310.23 | 343.58 |
| | b | 15.00 | 20.17 | 23.69 | 25.24 | 26.80 | 28.32 | 29.15 |



BACK TO TOC

# 3.4 Outlet Structures

## 3.4.1 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in **Table 3.4.1-1** will be used. These symbols were selected because of their wide use in technical publications.

In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3.4.1-1 Symbols and Definitions

| Symbol | Definition | Units |
|---|---|---|
| A, a | Cross sectional or surface area | ft² |
| $A_m$ | Drainage area | mi² |
| D | Breadth of weir | ft |
| C | Weir coefficient | - |
| d | Change in elevation | ft |
| D | Depth of basin or diameter of pipe | ft |
| g | Acceleration due to gravity | ft/s² |
| H | Head on structure | ft |
| $H_c$ | Height of weir crest above channel bottom | ft |
| K, k | Coefficient | - |
| L | Length | ft |
| n | Manning's n | - |
| Q, q | Peak inflow or outflow | cfs, in |
| $V_u$ | Approach velocity | ft/s |
| $WQ_v$ | Water quality volume | ac-ft |
| w | Maximum cross sectional bar width facing the flow | in |
| x | Minimum clear spacing between bars | in |
| Θ | Angle of v-notch | degrees |
| $\Theta_g$ | Angle of the grate with respect to the horizontal | degrees |

## 3.4.2 Primary Outlets

### 3.4.2.1 INTRODUCTION

Primary outlets provide the critical function of the regulation of flow for structural stormwater controls. There are several different types of outlets that may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.

For a single stage system, the stormwater facility can be designed as a simple pipe or culvert. For multistage control structures, the inlet is designed considering a range of design flows.

A stage-discharge curve is developed for the full range of flows that the structure would experience. The outlets are housed in a riser structure connected to a single outlet conduit. An alternative approach would be to provide several pipe or culvert outlets at different levels in the basin that are either discharged separately or are combined to discharge at a single location.

This section provides an overview of outlet structure hydraulics and design for stormwater storage facilities. The design engineer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

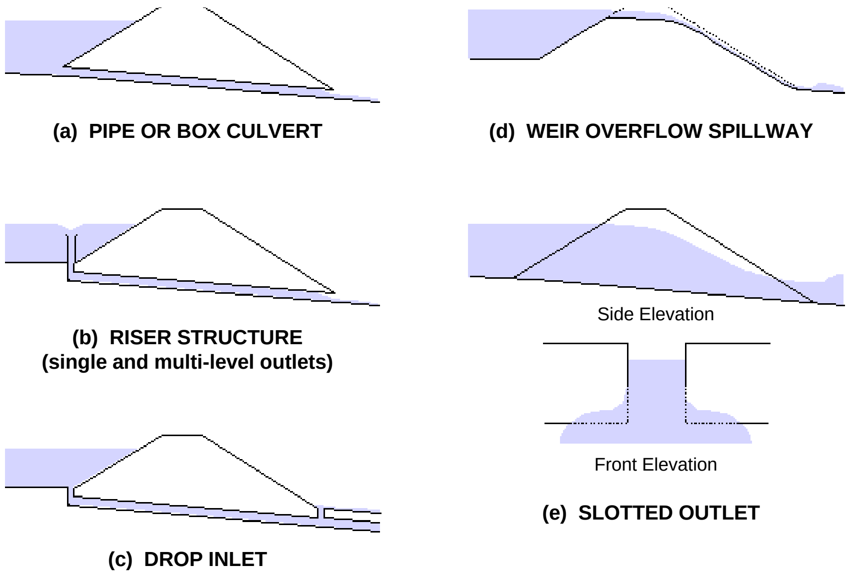



Figure 3.4.2-1 Typical Primary Outlets

### 3.4.2.2 OUTLET STRUCTURE TYPES

There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in stormwater facility design:

- Orifices
- Perforated risers
- Pipes / Culverts
- Sharp-crested weirs
- Broad-crested weirs
- V-notch weirs
- Proportional weirs
- Combination outlets

Each of these outlet types has a different design purpose and application:

- Water quality and channel protection flows are normally handled with smaller, more protected outlet structures such as reverse slope pipes, hooded orifices, orifices located within screened pipes or risers, perforated plates or risers, and V-notch weirs.
- Larger flows, such as overbank protection and extreme flood flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway through the embankment. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.



BACK TO TOC

### 3.4.2.3 ORIFICES

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice.

For a single orifice, as illustrated in **Figure 3.4.2-2(a)**, the orifice discharge can be determined using the standard orifice equation below.

$$Q = CA\,(2gH)^{0.5} \qquad (3.4.1)$$

*Where:*

**Q** = the orifice flow discharge (cfs)
**C** = discharge coefficient
**A** = cross-sectional area of orifice or pipe ($ft^2$)
**g** = acceleration due to gravity (32.2 $ft/s^2$)
**D** = diameter of orifice or pipe (ft)
**H** = effective head on the orifice, from orifice center to the water surface

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces as shown in **Figure 3.4.2-2(b).**

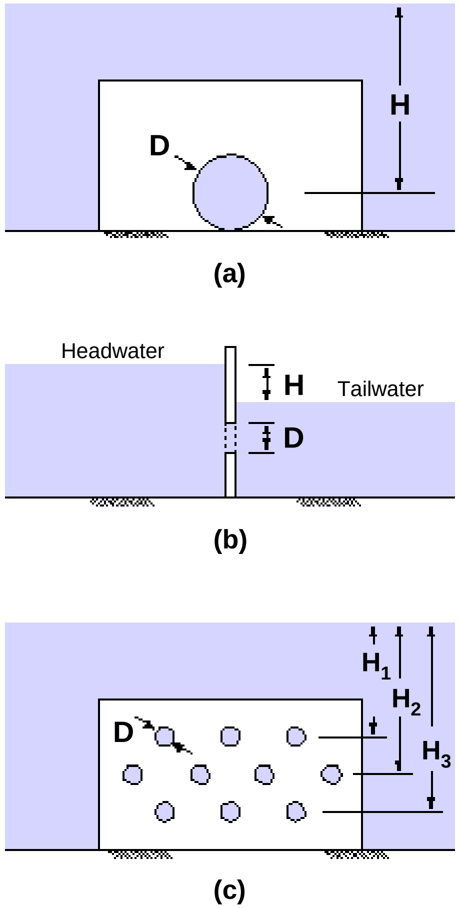



Figure 3.4.2-2 Orifice Definition

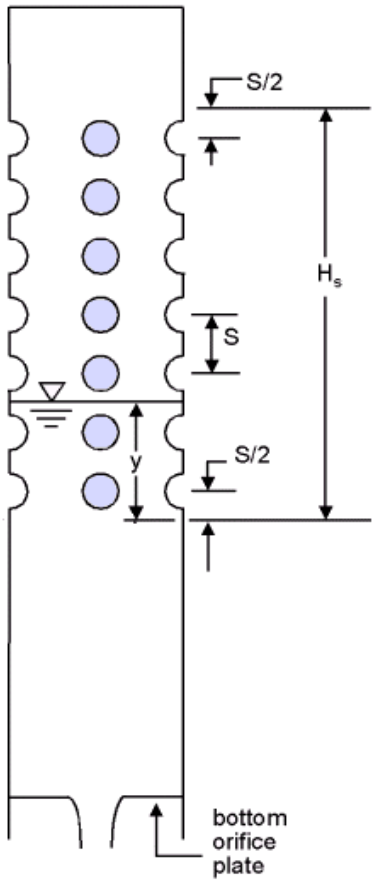



Figure 3.4.2-3 Perforated Riser

When the material is thinner than the orifice diameter, with sharp edges, a coefficient of 0.6 should be used. For square-edged entrance conditions the generic orifice equation can be simplified:

$$Q = 0.6A\,(2gH)^{0.5} = 3.78D^2H^{0.5} \quad (3.4.2)$$

When the material is thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges are rounded, a coefficient of 0.92 can be used.

Flow through multiple orifices, such as the perforated plate shown in **Figure 3.4.2-2(c)**, 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.

Perforated orifice plates for the control of discharge can be of any size and configuration. However, the Denver Urban Drainage and Flood Control District has developed standardized dimensions that have worked well. **Table 3.4.2-1** gives appropriate dimensions. The vertical spacing between hole centerlines is always 4 inches.

For rectangular slots the height is normally 2 inches with variable width. Only one column of rectangular slots is allowed.

Table 3.4.2-1 Circular Perforation Sizing

| Hole Diameter (in) | Minimum Column Hole Centerline Spacing (in) | Flow Area per Row ($in^2$) | | |
|---|---|---|---|---|
| | | 1 column | 2 columns | 3 columns |
| 1/4 | 1 | 0.05 | 0.1 | 0.15 |
| 5/16 | 2 | 0.08 | 0.15 | 0.23 |
| 3/8 | 2 | 0.11 | 0.22 | 0.33 |
| 7/16 | 2 | 0.15 | 0.3 | 0.45 |
| 1/2 | 2 | 0.2 | 0.4 | 0.6 |
| 9/16 | 3 | 0.25 | 0.5 | 0.75 |
| 5/8 | 3 | 0.31 | 0.62 | 0.93 |
| 11/16 | 3 | 0.37 | 0.74 | 1.11 |
| 3/4 | 3 | 0.44 | 0.88 | 1.32 |
| 13/16 | 3 | 0.52 | 1.04 | 1.56 |
| 7/8 | 3 | 0.6 | 1.2 | 1.8 |
| 15/16 | 3 | 0.69 | 1.38 | 2.07 |
| 1 | 4 | 0.79 | 1.58 | 2.37 |
| 1 1/16 | 4 | 0.89 | 1.78 | 2.67 |
| 1 1/8 | 4 | 0.99 | 1.98 | 2.97 |
| 1 3/16 | 4 | 1.11 | 2.22 | 3.33 |
| 1 1/4 | 4 | 1.23 | 2.46 | 3.69 |
| 1 5/16 | 4 | 1.35 | 2.7 | 4.05 |
| 1 3/8 | 4 | 1.48 | 2.96 | 4.44 |
| 1 7/16 | 4 | 1.62 | 3.24 | 4.86 |
| 1 1/2 | 4 | 1.77 | 3.54 | 5.31 |
| 1 9/16 | 4 | 1.92 | 3.84 | 5.76 |
| 1 5/8 | 4 | 2.07 | 4.14 | 6.21 |
| 1 11/16 | 4 | 2.24 | 4.48 | 6.72 |
| 1 3/4 | 4 | 2.41 | 4.82 | 7.23 |
| 1 13/16 | 4 | 2.58 | 5.16 | 7.74 |
| 1 7/8 | 4 | 2.76 | 5.52 | 8.28 |
| 1 15/16 | 4 | 2.95 | 5.9 | 8.85 |
| 2 | 4 | 3.14 | 6.28 | 9.42 |
| Number of columns refers to parallel columns of holes | | | | |
| Minimum steel plate thickness | | 1/4" | 5/16" | 3/8" |

Source: Urban Drainage and Flood Control District, Denver, CO

BACK TO TOC

**Figure 3.4.2-4** provides a schematic of an orifice plate outlet structure for a wet ED pond showing the design pool elevations and the flow control mechanisms.

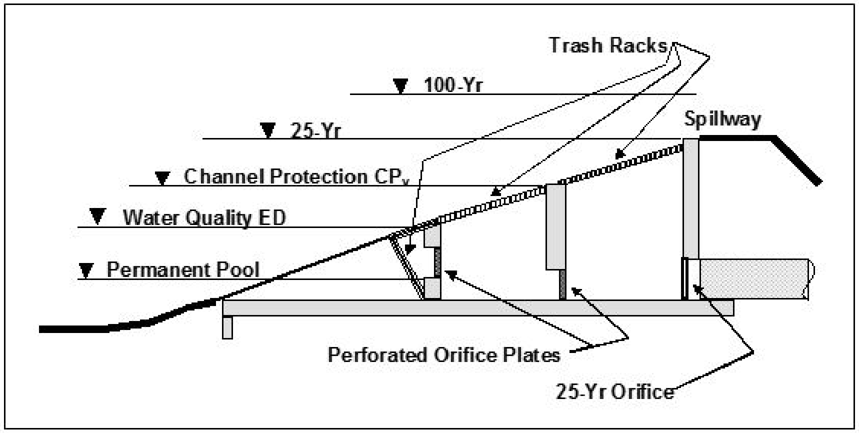



Figure 3.4.2-4  Schematic of Orifice Plate Outlet Structure

### 3.4.2.4 PERFORATED RISERS

A special kind of orifice flow is a perforated riser as illustrated in **Figure 3.4.2-3**. In the perforated riser, an orifice plate at the bottom of the riser, or in the outlet pipe just downstream from the elbow at the bottom of the riser, controls the flow. It is important that the perforations in the riser convey more flow than the orifice plate so as not to become the control.

Referring to Figure 3.4.2-3, a shortcut formula has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988):

$$Q = C_p \frac{2A_p}{3H_s} \sqrt{2g} H^{3/2} \qquad (3.4.3)$$

*Where:*

- **Q** = discharge (cfs)
- **C$_p$** = discharge coefficient for perforations (normally 0.61)
- **A$_p$** = cross-sectional area of all the holes (ft$^2$)
- **H$_s$** = distance from S/2 below the lowest row of holes to S/2 above the top row (ft)

### 3.4.2.5 PIPES AND CULVERTS

Discharge pipes are often used as outlet structures for stormwater control facilities. The design of these pipes can be for either single or multi-stage discharges. A reverse-slope underwater pipe is often used for water quality or channel protection outlets.

Pipes smaller than 12 inches in diameter may be analyzed as a submerged orifice as long as H/D is greater than 1.5. *Note: For low flow conditions when the flow reaches and begins to overflow the pipe, weir flow controls (see Subsection 3.4.2.6). As the stage increases the flow will transition to orifice flow.*

Pipes greater than 12 inches in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet control culvert nomographs and procedures given in Section 5.3, *Culvert Design*, or by using **Equation 3.4.4** (NRCS, 1984).

The following equation is a general pipe flow equation that is derived through the use of the Bernoulli and continuity principles.

$$Q = a[(2gH) / (1 + k_m + k_pL)]0.5 \qquad (3.4.4)$$

Where:

- **Q** = discharge (cfs)
- **a** = pipe cross sectional area (ft$^2$)
- **g** = acceleration of gravity (ft/s$^2$)
- **H** = elevation head differential (ft)
- **k$_m$** = coefficient of minor losses (use 1.0)
- **k$_p$** = pipe friction coefficient = 5087n$^2$/D$^{4/3}$
- **L** = pipe length (ft)

BACK TO TOC

### 3.4.2.6 SHARP-CRESTED WEIRS

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear as it overflows, the overflow is termed a sharp-crested weir. If the sides of the weir also cause the through flow to contract, it is termed an end-contracted sharp-crested weir. Sharp-crested weirs have stable stage-discharge relations and are often used as a measurement device. A sharp-crested weir with no end contractions is illustrated in **Figure 3.4.2-5(a)**. The discharge equation for this configuration is (Chow, 1959):

$$Q = [(3.27 + 0.4(H/H_c)]\ LH^{1.5} \qquad (3.4.5)$$

*Where:*

**Q** = discharge (cfs)
**H** = head above weir crest excluding velocity head (ft)
**$H_c$** = height of weir crest above channel bottom (ft)
**L** = horizontal weir length (ft)

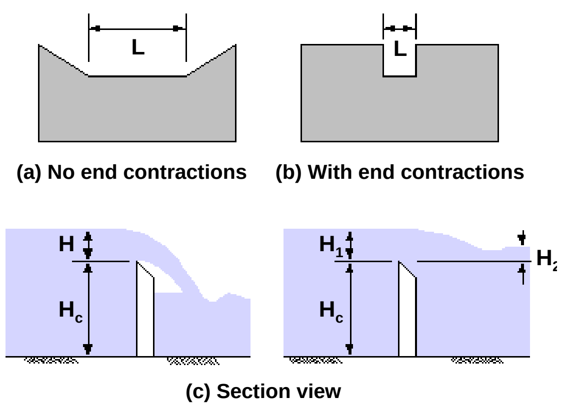



Figure 3.4.2-5 Sharp-Crested Weir

A sharp-crested weir with two end con¬tractions is illustrated in **Figure 3.4.2-5(b)**. The discharge equation for this configuration is (Chow, 1959):

$$Q = [(3.27 + 0.04(H/H_c)]\ (L - 0.2H)\ H^{1.5} \qquad (3.4.6)$$

*Where:*

**Q** = discharge (cfs)
**H** = head above weir crest excluding velocity head (ft)
**$H_c$** = height of weir crest above channel bottom (ft)
**L** = horizontal weir length (ft)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 197

$$Q_s = Q_f\ (1 - (H_2/H_1)^{1.5})^{0.385} \qquad (3.4.7)$$

*Where:*

**$Q_s$** = submergence flow (cfs)
**$Q_f$** = free flow (cfs)
**$H_1$** = upstream head above crest (ft)
**$H_2$** = downstream head above crest (ft)

BACK TO TOC

### 3.4.2.7 BROAD-CRESTED WEIRS

A weir in the form of a relatively long raised channel control crest section is a broad-crested weir. The flow control section can have different shapes, such as triangular or circular. True broad-crested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. For example, a thick wall or a flat stop log can act like a sharp-crested weir when the approach head is large enough that the flow springs from the upstream corner.  If upstream head is small enough relative to the top profile length, the stop log can act like a broad-crested weir (USBR, 1997).

The equation for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{1.5} \qquad (3.4.8)$$

*Where:*

**Q** = discharge (cfs)
**C** = broad-crested weir coefficient
**L** = broad-crested weir length perpendicular to flow (ft)
**H** = head above weir crest (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087.  For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used.  Information on C values as a function of weir crest breadth and head is given in **Table 3.4.2-2**.

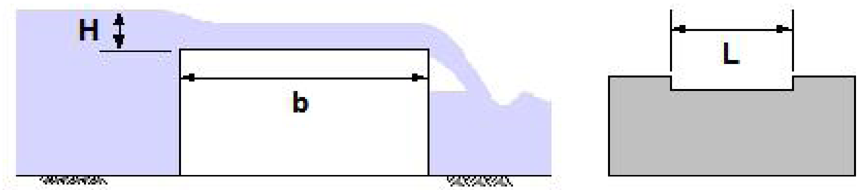



Figure 3.4.2-6  Broad-Crested Weir

Table 3.4.2-2  Broad-Crested Weir Coefficient (C) Values

| Measured Head (H)* | Weir Crest Breadth (b) in feet | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| In feet | 0.50 | 0.75 | 1.00 | 1.50 | 2.00 | 2.50 | 3.00 | 4.00 | 5.00 | 10.00 | 15.00 |
| 0.2 | 2.80 | 2.75 | 2.69 | 2.62 | 2.54 | 2.48 | 2.44 | 2.38 | 2.34 | 2.49 | 2.68 |
| 0.4 | 2.92 | 2.80 | 2.72 | 2.64 | 2.61 | 2.60 | 2.58 | 2.54 | 2.50 | 2.56 | 2.70 |
| 0.6 | 3.08 | 2.89 | 2.75 | 2.64 | 2.61 | 2.60 | 2.68 | 2.69 | 2.70 | 2.70 | 2.70 |
| 0.8 | 3.30 | 3.04 | 2.85 | 2.68 | 2.60 | 2.60 | 2.67 | 2.68 | 2.68 | 2.69 | 2.64 |
| 1.0 | 3.32 | 3.14 | 2.98 | 2.75 | 2.66 | 2.64 | 2.65 | 2.67 | 2.68 | 2.68 | 2.63 |
| 1.2 | 3.32 | 3.20 | 3.08 | 2.86 | 2.70 | 2.65 | 2.64 | 2.67 | 2.66 | 2.69 | 2.64 |
| 1.4 | 3.32 | 3.26 | 3.20 | 2.92 | 2.77 | 2.68 | 2.64 | 2.65 | 2.65 | 2.67 | 2.64 |
| 1.6 | 3.32 | 3.29 | 3.28 | 3.07 | 2.89 | 2.75 | 2.68 | 2.66 | 2.65 | 2.64 | 2.63 |
| 1.8 | 3.32 | 3.32 | 3.31 | 3.07 | 2.88 | 2.74 | 2.68 | 2.66 | 2.65 | 2.64 | 2.63 |
| 2.0 | 3.32 | 3.31 | 3.30 | 3.03 | 2.85 | 2.76 | 2.27 | 2.68 | 2.65 | 2.64 | 2.63 |
| 2.5 | 3.32 | 3.32 | 3.31 | 3.28 | 3.07 | 2.89 | 2.81 | 2.72 | 2.67 | 2.64 | 2.63 |
| 3.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.20 | 3.05 | 2.92 | 2.73 | 2.66 | 2.64 | 2.63 |
| 3.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.19 | 2.97 | 2.76 | 2.68 | 2.64 | 2.63 |
| 4.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.07 | 2.79 | 2.70 | 2.64 | 2.63 |
| 4.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 2.88 | 2.74 | 2.64 | 2.63 |
| 5.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.07 | 2.79 | 2.64 | 2.63 |
| 5.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 2.88 | 2.64 | 2.63 |

* Measured at least 2.5H upstream of the weir.    Source: Brater and King (1976)

### 3.4.2.8 V-NOTCH WEIRS

The discharge through a V-notch weir (**Figure 3.4.2-7**) can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5 \tan(\theta/2)\, H^{2.5} \qquad (3.4.9)$$

Where:

**Q** = discharge (cfs)
**θ** = angle of V-notch (degrees)
**H** = head on apex of notch (ft)

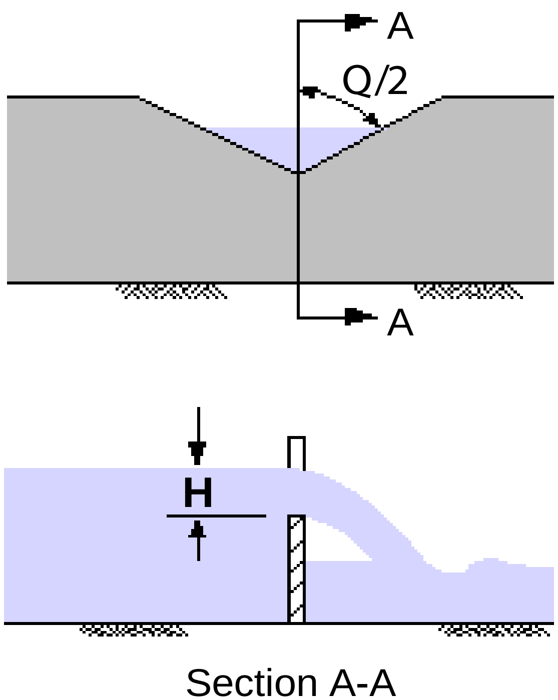



Figure 3.4.2-7 V-Notch Weir

### 3.4.2.9 PROPORTIONAL WEIRS

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head. A typical proportional weir is shown in **Figure 3.4.2-8**. Design equations for proportional weirs are (Sandvik, 1985):

$$Q = 4.97\, a^{0.5}\, b\, (H - a/3) \qquad (3.4.10)$$

$$x/b = 1 - (1/3.17)\,(\arctan\,(y/a)^{0.5} \qquad (3.4.11)$$

Where:

**Q** = discharge (cfs)
Dimensions a, b, H, x, and y are shown in **Figure 3.4.2-8**

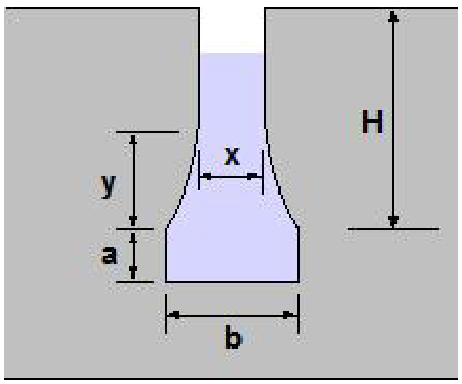



Figure 3.4.2-8 Proportional Weir Dimensions

### 3.4.2.10 COMBINATION OUTLETS

Combinations of orifices, weirs and pipes can be used to provide multi-stage outlet control for different control volumes within a storage facility (i.e., water quality volume, channel protection volume, overbank flood protection volume, and/or extreme flood protection volume).

They are generally of two types of combination outlets: shared outlet control structures and separate outlet controls. Shared outlet control is typically a number of individual outlet openings (orifices), weirs or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. **Figure 3.4.2-9** shows an example of a riser designed for a wet ED pond. The orifice plate outlet structure in **Figure 3.4.2-4** is another example of a combination outlet.

Separate outlet controls are less common and may consist of several pipe or culvert outlets at different levels in the storage facility that are either discharged separately or are combined to discharge at a single location.

The use of a combination outlet requires the construction of a composite stage-discharge curve (as shown in **Figure 3.4.2-10**) suitable for control of multiple storm flows. The design of multi-stage combination outlets is discussed later in this section.

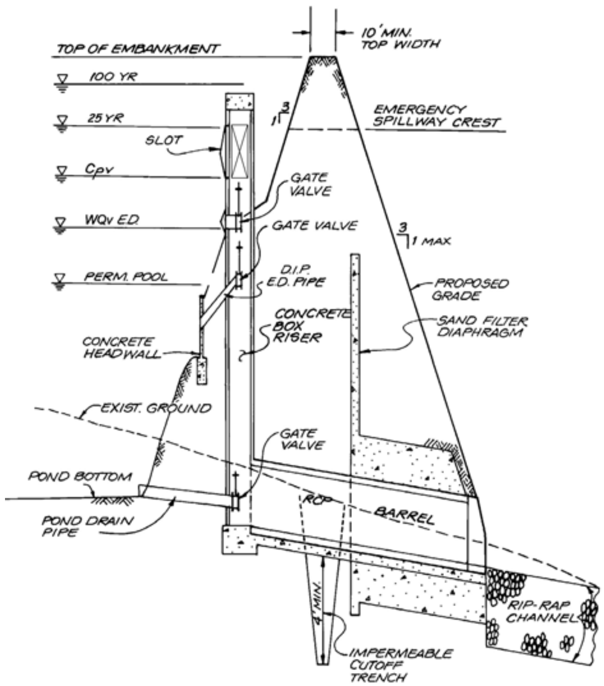



Figure 3.4.2-9 Schematic of Combination Outlet Structure

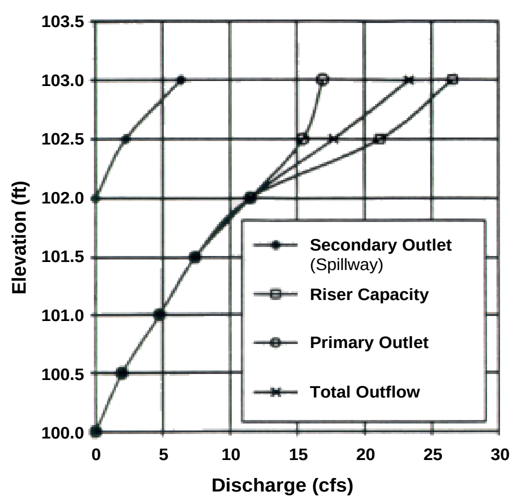



Figure 3.4.2-10 Composite Stage-Discharge Curve

## 3.4.3 Extended Detention (Water Quality and Channel Protection) Outlet Design

### 3.4.3.1 INTRODUCTION

Extended detention orifice sizing is required in design applications that provide extended detention for downstream channel protection or the ED portion of the water quality volume. In both cases an extended detention orifice or reverse slope pipe can be used for the outlet. For a structural control facility providing both $WQ_v$ extended detention and $CP_v$ control (wet ED pond, micropool ED pond, and shallow ED wetland), there will be a need to design two outlet orifices – one for the water quality control outlet and one for the channel protection

*(The following procedures are based on the water quality outlet design procedures included in the Virginia Stormwater Management Handbook, 1999)*

The outlet hydraulics for peak control design (overbank flood protection and extreme flood protection) is usually straightforward in that an outlet is selected that will limit the peak flow to some predetermined maximum. Since volume and the time required for water to exit the storage facility are not usually considered, the outlet design can easily be calculated and routing procedures used to determine if quantity design criteria are met.

In an extended detention facility for water quality treatment or downstream channel protection, however, the storage volume is detained and released over a specified amount of time (e.g., 24-hours). The release period is a brim drawdown time, beginning at the time of peak storage of the water quality volume until the entire calculated volume drains out of the basin. This assumes that the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention outlet can be sized using either of the following methods:

1. Use the maximum hydraulic head associated with the storage volume and maximum flow, and calculate the orifice size needed to achieve the required drawdown time, and route the volume through the basin to verify the actual storage volume used and the drawdown time.
2. Approximate the orifice size using the average hydraulic head associated with the storage volume and the required drawdown time.

These two procedures are outlined in the examples below and can be used to size an extended detention orifice for water quality and/or channel protection.

BACK TO TOC

### 3.4.3.2 METHOD 1: MAXIMUM HYDRAULIC HEAD WITH ROUTING

A wet ED pond sized for the required water quality volume will be used here to illustrate the sizing procedure for an extended-detention orifice.

Given the following information, calculate the required orifice size for water quality design.

Given: Water Quality Volume ($WQ_v$) = 0.76 ac ft = 33,106 $ft^3$
Maximum Hydraulic Head ($H_{max}$) = 5.0 ft (from stage vs. storage data)

(Step 1) Determine the maximum discharge resulting from the 24-hour drawdown requirement. It is calculated by dividing the Water Quality Volume (or Channel Protection Volume) by the required time to find the average discharge, and then multiplying by two to obtain the maximum discharge.

$Q_{avg}$ = 33,106 $ft^3$ / (24 hr)(3,600 s/hr) = 0.38 cfs
$Q_{max}$ = 2 * $Q_{avg}$ = 2 * 0.38 = 0.76 cfs

(Step 2) Determine the required orifice diameter by using the orifice **Equation (3.4.8)** and $Q_{max}$ and $H_{max}$:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$
$A = 0.76 / 0.6[(2)(32.2)(5.0)]^{0.5} = 0.071\ ft^3$

Determine pipe diameter from
$A = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$
$D = [4(0.071)/3.14]^{0.5} = 0.30\ ft = 3.61\ in$

*Use a 3.6-inch diameter water quality orifice.*

Routing the water quality volume of 0.76 ac ft through the 3.6-inch water quality orifice will allow the designer to verify the drawdown time, as well as the maximum hydraulic head elevation. The routing effect will result in the actual drawdown time being less than the calculated 24 hours. Judgment should be used to determine whether the orifice size should be reduced to achieve the required 24 hours or if the actual time achieved will provide adequate pollutant removal.

### 3.4.3.3 METHOD 2: AVERAGE HYDRAULIC HEAD AND AVERAGE DISCHARGE

Using the data from the previous example (3.4.3.2) use Method 2 to calculate the size of the outlet orifice.

Given: Water Quality Volume ($WQ_v$) = 0.76 ac ft = 33,106 $ft^3$
Average Hydraulic Head ($h_{avg}$) = 2.5 ft (from stage vs storage data)

(Step 1) Determine the average release rate to release the water quality volume over a 24-hour time period.

Q = 33,106 $ft^3$ / (24 hr)(3,600 s/hr) = 0.38 cfs

(Step 2) Determine the required orifice diameter by using the orifice **Equation (3.4.8)** and the average head on the orifice:

$Q = CA(2gH)^{0.5}$, or $A = Q / C(2gH)^{0.5}$
$A = 0.38 / 0.6[(2)(32.2)(2.5)]^{0.5} = 0.05\ ft^3$

Determine pipe diameter from
$A = 3.14r^2 = 3.14d^2/4$, then $d = (4A/3.14)^{0.5}$
$D = [4(0.05)/3.14]^{0.5} = 0.252\ ft = 3.03\ in$

*Use a 3-inch diameter water quality orifice.*

Use of Method 1, utilizing the maximum hydraulic head and discharge and routing, results in a 3.6-inch diameter orifice (though actual routing may result in a changed orifice size) and Method 2, utilizing average hydraulic head and average discharge, results in a 3.0-inch diameter orifice.

BACK TO TOC

## 3.4.4 Multi-Stage Outlet Design

### 3.4.4.1 INTRODUCTION

A combination outlet such as a multiple orifice plate system or multi-stage riser is often used to provide adequate hydraulic outlet controls for the different design requirements (e.g., water quality, channel protection, overbank flood protection, and/or extreme flood protection) for stormwater ponds, stormwater wetlands and detention-only facilities. Separate openings or devices at different elevations are used to control the rate of discharge from a facility during multiple design storms. As discussed in the previous section, **Figures 3.4.2-4** and **3.4.2-9** are examples of multi-stage combination outlet systems.

A design engineer may be creative to provide the most economical and hydraulically efficient outlet design possible in designing a multi-stage outlet. Many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. The stage-discharge table or rating curve is a composite of the different outlets that are used for different elevations within the multi-stage riser (see **Figure 3.4.2-10**).

### 3.4.4.2 MULTI-STAGE OUTLET DESIGN PROCEDURE

Below are the steps for designing a multi-stage outlet. Note that if a structural control facility will not control one or more of the required storage volumes ($WQ_v$, $CP_v$, $Q_{p25}$, and $Q_f$), then that step in the procedure is skipped

(Step 1) Determine Stormwater Control Volumes.
Using the procedures from Sections 3.1 and 3.3, estimate the required storage volumes for water quality treatment ($WQ_v$), channel protection ($CP_v$), and overbank flood control ($Q_{p25}$) and extreme flood control ($Q_f$).

(Step 2) Develop Stage-Storage Curve.
Using the site geometry and topography, develop the stage-storage curve for the facility in order to provide sufficient storage for the control volumes involved in the design.

(Step 3) Design Water Quality Outlet.
Design the water quality extended detention (WQv-ED) orifice using either Method 1 or Method 2 outlined in Subsection 3.4.3. If a permanent pool is incorporated into the design of the facility, a portion of the storage volume for water quality will be above the elevation of the permanent pool. The outlet can be protected using either a reverse slope pipe, a hooded protection device, or another acceptable method (see Subsection 3.4.5).

(Step 4) Design Channel Protection Outlet.
Design the stream channel protection extended detention outlet ($CP_v$-ED) using either method from Subsection 3.4.3. For this design, the storage needed for channel protection will be “stacked” on top of the water quality volume storage elevation determined in Step 3. The total stage-discharge rating curve at this point will include water quality control orifice and the outlet used for stream channel protection. The outlet should be protected in a manner similar to that for the water quality orifice.

(Step 5) Design Overbank Flood Protection Outlet.
The overbank protection volume is added above the water quality and channel protection storage. Establish the Qp25 maximum water surface elevation using the stage-storage curve and subtract the $CP_v$ elevation to find the 25-year maximum head. Select an outlet type and calculate the initial size and geometry based upon maintaining the predevelopment 25-year peak discharge rate. Develop a stage-discharge curve for the combined set of outlets ($WQ_v$, $CP_v$ and $Q_{p25}$).

This procedure is repeated for control (peak flow attenuation) of the 100-year storm ($Q_f$), if required.

(Step 6) **Check Performance of the Outlet Structure.**

Perform a hydraulic analysis of the multi-stage outlet structure using reservoir routing to ensure that all outlets will function as designed. Several iterations may be required to calibrate and optimize the hydraulics and outlets that are used. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This usually requires that the structure have a larger cross-sectional area than the outlet conduit.

The hydraulic analysis of the design must take into account the hydraulic changes that will occur as depth of storage changes for the different design storms. As shown in **Figure 3.4.4-1**, as the water passes over the rim of a riser, the riser acts as a weir. However, when the water surface reaches a certain height over the rim of a riser, the riser will begin to act as a submerged orifice. The designer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place for an outlet where this change in hydraulic conditions will change. Also note in **Figure 3.4.4-1** that as the elevation of the water increases further, the control can change from barrel inlet flow control to barrel pipe flow control. **Figure 3.4.4-2** shows another condition where weir flow can change to orifice flow, which must be taken into account in the hydraulics of the rating curve as different design conditions results in changing water surface elevations.

(Step 7) **Size the Emergency Spillway.**

It is recommended that all stormwater impoundment structures have a vegetated emergency spillway (see Subsection 3.4.6). An emergency spillway provides a degree of safety to prevent overtopping of an embankment if the primary outlet or principal spillway should become clogged, or otherwise inoperative. The 100-year storm should be routed through the outlet devices and emergency spillway to ensure the hydraulics of the system will operate as designed.

(Step 8) **Design Outlet Protection.**

Design necessary outlet protection and energy dissipation facilities to avoid erosion problems downstream from outlet devices and emergency spillway(s). See Section 5.5, *Energy Dissipation Design*, for more information.

(Step 9) **Perform Buoyancy Calculations.**

Perform buoyancy calculations for the outlet structure and footing. Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water.

(Step 10) **Provide Seepage Control.**

Seepage control should be provided for the outflow pipe or culvert through an embankment. The two most common devices for controlling seepage are (1) filter and drainage diaphragms and (2) anti-seep collars.

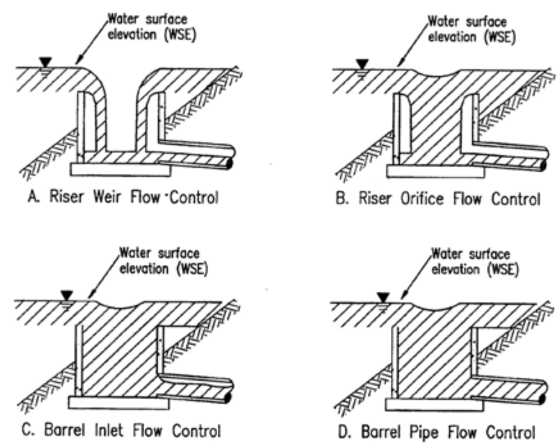



Figure 3.4.4-1 Riser Flow Diagrams
(Source: VDCR, 1999)

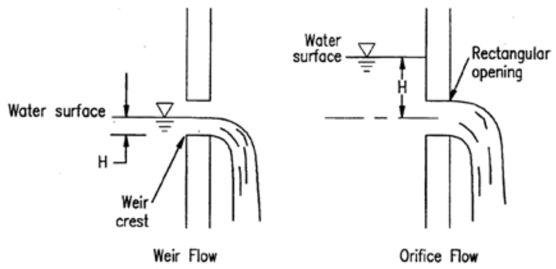



Figure 3.4.4-2 Weir and Orifice Flow
(Source: VDCR, 1999)

### 3.4.5 Extended Detention Outlet Protection

Small, low flow orifices such as those used for extended detention applications can easily clog, preventing the structural control from meeting its design purpose(s) and potentially causing adverse impacts. Therefore, extended detention orifices need to be adequately protected from clogging. There are a number of different anti-clogging designs, including:

- The use of a reverse slope pipe attached to a riser for a stormwater pond or wetland with a permanent pool (see **Figure 3.4.5-1**). The inlet is submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond.
- The use of a hooded outlet for a stormwater pond or wetland with a permanent pool (see **Figures 3.4.5-2** and **3.4.5-3**).
- Internal orifice protection through the use of an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket (see **Figure 3.4.5-4**).
- Internal orifice protection through the use of an adjustable gate valves can to achieve an equivalent orifice diameter.



BACK TO TOC

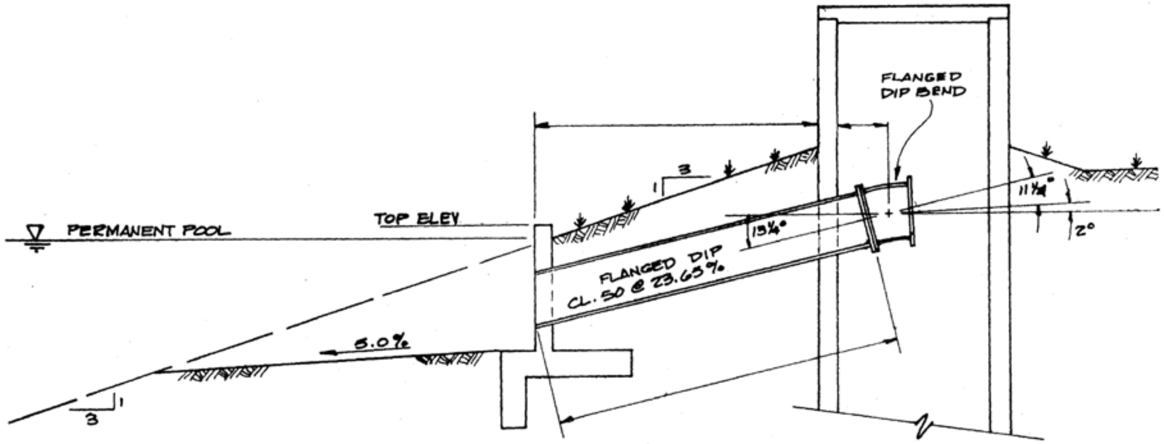



Figure 3.4.5-1  Reverse Slope Pipe Outlet

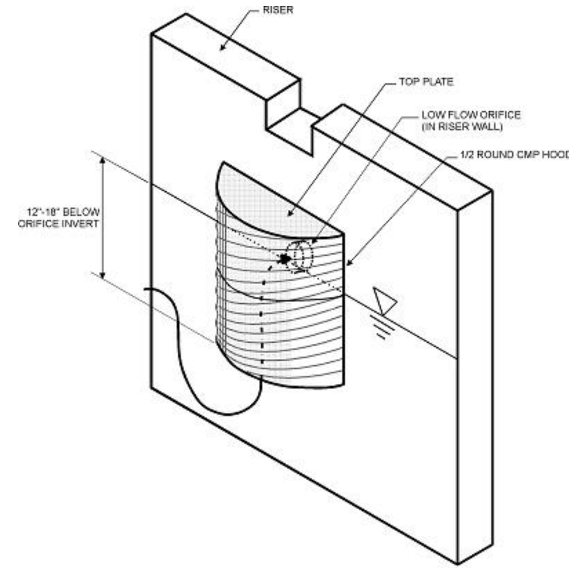



Figure 3.4.5-3  Half-Round CMP Orifice Hood

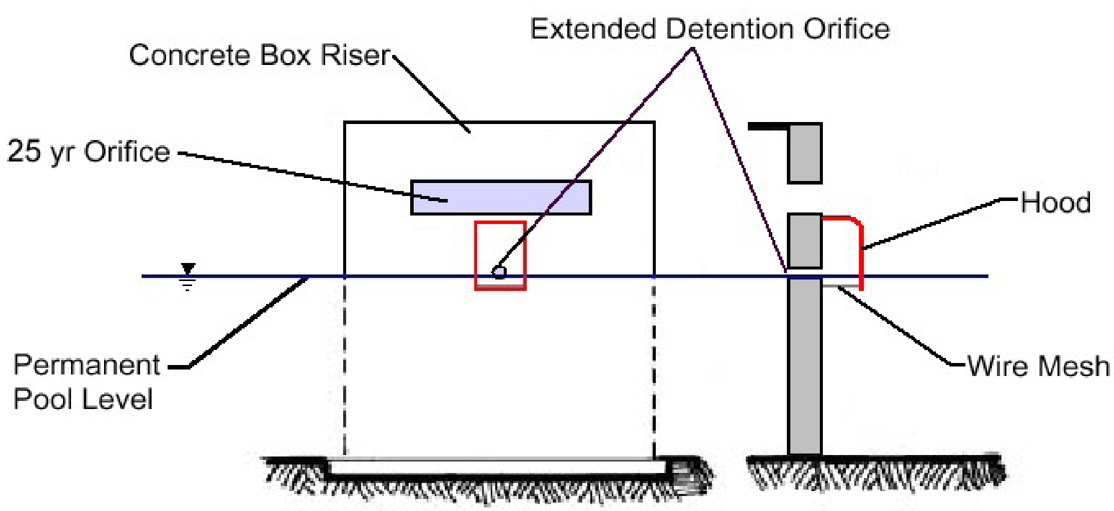



Figure 3.4.5-2  Hooded Outlet

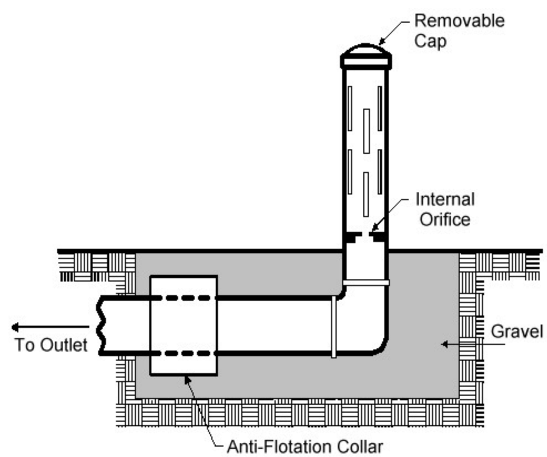



Figure 3.4.5-4  Internal Control for Orifice Protection

## 3.4.6 Trash Racks and Safety Grates

### 3.4.6.1 INTRODUCTION

The susceptibility of larger inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions:

- Keeping debris away from the entrance to the outlet works where they will not clog the critical portions of the structure
- Capturing debris in such a way that relatively easy removal is possible
- Ensuring that people and large animals are kept out of confined conveyance and outlet areas
- Providing a safety system that prevents anyone from being drawn into the outlet and allows them to climb to safety

When designed properly, trash racks serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985; Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors, including head losses through the rack, structural convenience, safety and size of outlet. Well-designed trash racks can also have an aesthetically pleasing appearance.

An example of trash racks used on a riser outlet structure is shown in **Figure 3.4.6-1**. Trash rack design should be based on the effective opening of the trash rack compared to the orifice, or outlet size. The outlet size should be the controlling or smaller open area, when compared to the effective open area of the trash rack.

The inclined vertical bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

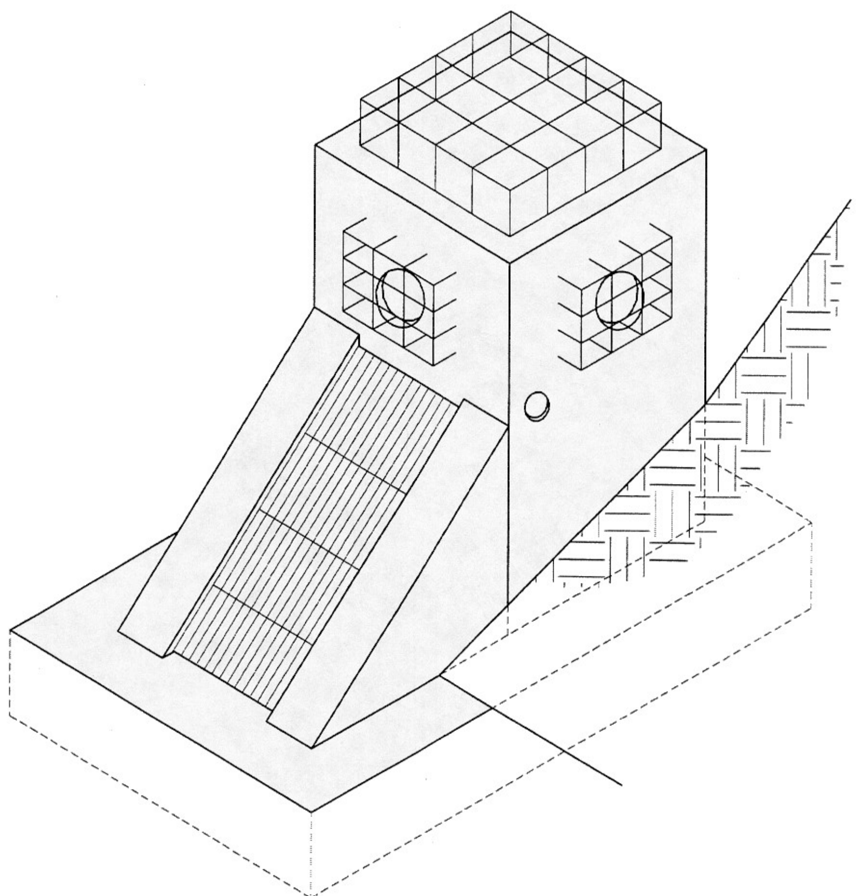


Figure 3.4.6-1 Example of Various Trash Racks Used on a Riser Outlet Structure
(Source: VDCR, 1999)

BACK TO TOC

#### 3.4.6.2 TRASH RACK DESIGN

Trash racks must be large enough such that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice.

The surface area of all trash racks should be maximized and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level—the slower the approach flow, the flatter the angle. Rack opening rules-of-thumb are found in literature. Figure 3.4.6-2 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack.

Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978; UDFCD, 1992). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore head losses through the grate should be calculated. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Metcalf & Eddy (1972) give the following equation (based on German experiments) for losses. Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50% is chosen as a working assumption.

$$H_g = K_{g1} (w/x)^{4/3} (V_u^2/2g) \sin \theta_g \tag{3.4.12}$$

*Where:*

**$H_g$** = head loss through grate (ft)

**$K_{g1}$** = bar shape factor:

- 2.42 - sharp edged rectangular
- 1.83 - rectangular bars with semicircular upstream
- 1.79 - circular bars
- 1.67 - rectangular bars with semi-circular up- and down¬stream faces

**w** = maximum cross-sectional bar width facing the flow (in)

**x** = minimum clear spacing between bars (in)

**$V_u$** = approach velocity (ft/s)

**$\theta_g$** = angle of the grate with respect to the horizontal (degrees)

BACK TO TOC

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

$$H_g = \frac{K_{g2} V_u^2}{2g} \qquad (3.4.13)$$

Where $K_{g2}$ is defined from a series of fit curves as:

- Sharp edged rectangular (length/thickness = 10)
  $K_{g2} = 0.00158 - 0.03217\ A_r + 7.1786\ A_r^2$
- Sharp edged rectangular (length/thickness = 5)$K_{g2}$
  $= -0.00731 + 0.69453\ A_r + 7.0856\ A_r^2$
- Round edged rectangular (length/thickness = 10.9)
  $K_{g2} = -0.00101 + 0.02520\ A_r + 6.0000\ A_r^2$
- Circular cross section
  $K_{g2} = 0.00866 + 0.13589\ A_r + 6.0357\ A_r^2$
- $A_r$ is the ratio of the area of the bars to the area of the grate section.

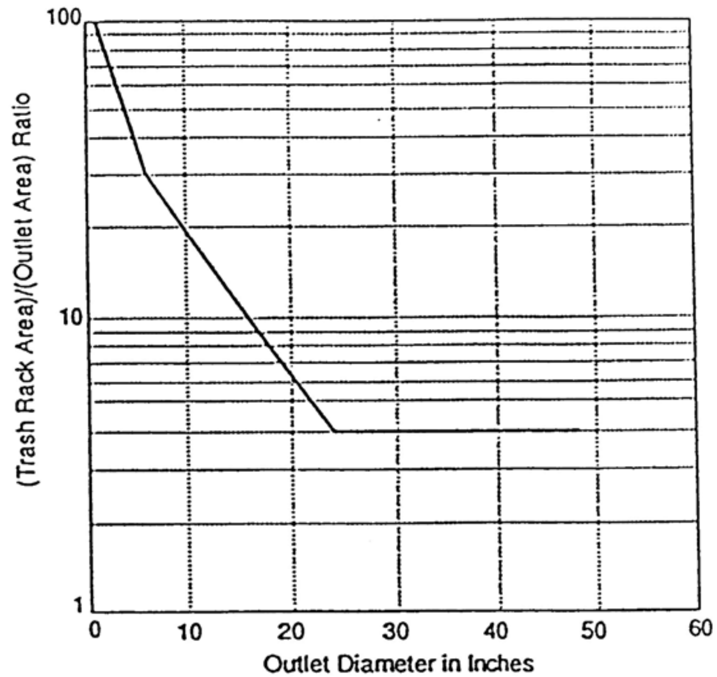



Figure 3.4.6-2 Minimum Rack Size vs. Outlet Diameter
(Source: UDCFD, 1992)

## 3.4.7 Secondary Outlets

### 3.4.7.1 INTRODUCTION

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. **Figure 3.4.7-1** shows an example of an emergency spillway.

In many cases, on-site stormwater storage facilities do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor.

### 3.4.7.2 EMERGENCY SPILLWAY DESIGN

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see **Figure 3.4.7-1**). The emergency spillway is proportioned to pass flows in excess of the design flood (typically the 100-year flood or greater) without allowing excessive velocities and without overtopping of the embankment. Flow in the emergency spillway is open channel flow (see Section 5.4, *Open Channel Design*, for more information). Normally, it is assumed that critical depth occurs at the control section.

NRCS (NRCS TR-55) manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both the inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper the 3:1 horizontal to vertical.

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated to the right.

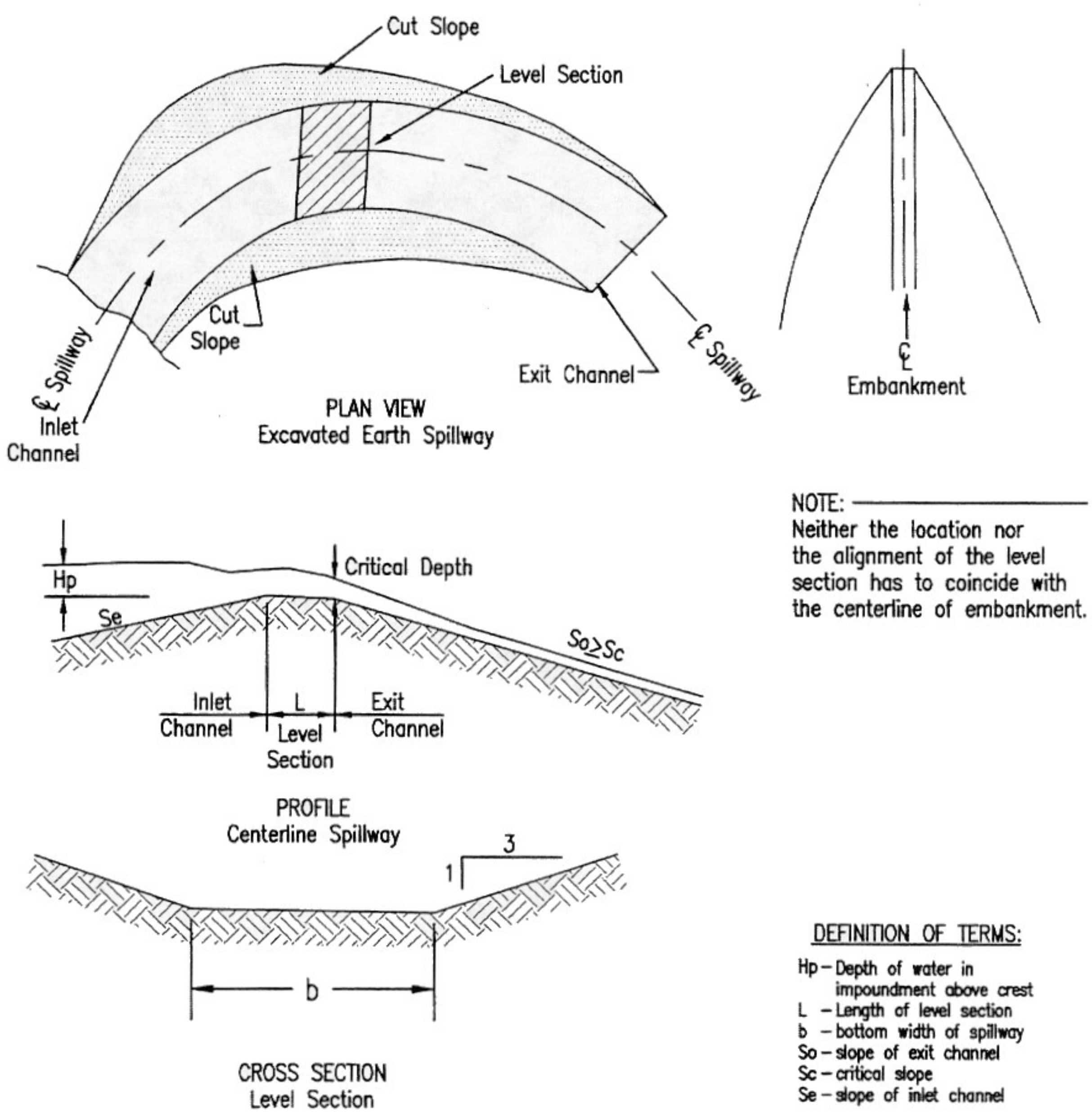



Figure 3.4.7-1 Emergency Spillway
(Source: VDCR, 1999)

BACK TO TOC

## References


Battiata, Joseph, Kelly Collins, David Hirschman, and Greg Hoffman. *The Runoff Reduction Method.* Center for Watershed Protection, Mechanicsville, VA.

Brater, Ernist F., Horace W. King. 1976. *Handbook of Hydraulics*. 6th ed. NY: McGraw Hill Book Co.

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

Debo, Thomas N. and Andrew J. Reese. 1995. *Municipal Stormwater Management*. Lewis Publishers: CRC Press, Inc., Boca Raton, Florida.

Ferguson, Bruce K. and Thomas Neil Debo. 1990. *On-site Stormwater Management.*

Hershfield, David M. 1961. *Rainfall Frequency Atlas of the United States, Technical Paper No. 40.* Engineering Division, Soil Conservation Service U.S. Department of Agriculture. Washington D.C.

Maryland Department of the Environment. 2000. *Maryland Stormwater Design Manual, Volumes I and II.* Center for Watershed Protection (CWP), Ellicott City, MD.

McEnroe, B.M., J.M. Steichen and R.M. Schweiger. 1988. *Hydraulics of Perforated Riser Inlets for Underground Outlet Terraces*. Trans ASAE, Vol. 31, No. 4, 1988.

NOAA. 1997. *Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States, NOAA Technical Memo NWS HYDRO-35.* National Oceanic and Atmospheric Administration (NOAA).

NOAA. 2013. *Atlas 14, Version 9.* National Oceanic and Atmospheric Administration (NOAA).

Pitt, Robert. 1994. *Small Storm Hydrology*. Unpublished report. Department of Civil Engineering, University of Alabama. Birmingham, Alabama.

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

Urban Drainage and Flood Control District. 1999. *Criteria Manual*. Denver, CO.

U.S. Department of Agriculture, Natural Resource Conservation Service, Engineering Division. 2004. *NRCS National Engineering Handbook 630.* Natural Resource Conservation Service (NRCS).

U.S. Department of Agriculture, Natural Resource Conservation Service, Engineering Division. 1986. *Urban Hydrology for Small Watersheds.* Technical Release 55. Natural Resource Conservation Service (NRCS).

U.S. Department of the Interior. 1997. *Water Measurement Manual.* 3rd ed. Bureau of Reclamation. Washington D.C.

U.S. Department of Transportation, Federal Highway Administration. 1984. *Hydrology*. Hydraulic Engineering Circular No. 19.

USGS. 2011. *Flood-frequency Relations for Urban Streams in Georgia*. Water Resources Investigation Report. U.S. Geological Survey.

USGS. 2003. *Techniques for Estimating Magnitude and Frequency of Floods in Rural Basins of Georgia*. Water Resources Investigation Report. U.S. Geological Survey.

Virginia Department of Conservation and Recreation. 1999. *Virginia Stormwater Management Handbook.*

Wycoff, Ronald L. and Udai P. Singh. 1976. *Preliminary Hydrologic Design of Small Flood Detention Reservoirs*. Water Resources Bulletin. Vol. 12, No. 2, pp 337-47.

# 4. Stormwater Best Management Practices

## 4.1 Stormwater Best Management Practices Overview

### 4.1.1 Best Management Practices

#### 4.1.1.1 INTRODUCTION

Stormwater Best Management Practices (BMPs) are engineered facilities designed to reduce and/or treat stormwater runoff, which mitigate the effects of increased stormwater runoff peak rate, volume, and velocity due to urbanization. This section provides an overview of BMPs that can be used to address the minimum stormwater management standards outlined in Section 2.2.

In terms of the Unified Stormwater Sizing Criteria, a BMP, or treatment train that includes two or more BMPs, should be designed to meet one or more of the following requirements:

- Reduce total runoff from the drainage area using the Runoff Reduction Volume, $RR_v$ (the runoff generated by a target rainfall event);
- Treat the Water Quality Volume, $WQ_v$ (the runoff generated by first 1.2 inches of rainfall);
- Control the Channel Protection Volume, $CP_v$ (24 hours of extended detention for the one-year, 24-hour rainfall event), where necessary or required;
- Control for Overbank Flood Protection, $Q_{p25}$ (detention of the post-development 25-year, 24-hour storm peak discharge rate to the pre-development rate), where required; and
- Provide for Extreme Flood Protection by either: (1) control of the peak discharge increase from the 100-year, 24-hour storm event, $Q_f$, through detention; or (2) safely pass $Q_f$ through the structural control and allow it to discharge into a receiving water whose protected floodplain is sufficiently sized to account for extreme flow increases without causing damage.

Localities choosing to adopt the GSMM as a guidance document may have more stringent local requirements for Flood Protection, such as detention of the post-development 50-year, 24-hour storm peak discharge rate to the pre-development rate.  These localities may wish to develop their own Technical Reference Sections (or Manual).

#### 4.1.1.2 TYPES OF BEST MANAGEMENT PRACTICES

**Table 4.1.1-1** lists many types of BMPs. These BMPs are recommended for use in a wide variety of applications. A detailed discussion of each of the BMPs, as well as design criteria and procedures for each, can be found in Sections 4.2 through 4.29.

BACK TO TOC

Table 4.1.1-1 Best Management Practices

| BMP | Description |
|---|---|
| Bioretention Areas | Bioretention areas are shallow stormwater basins or landscaped areas that utilize engineered soils and vegetation to capture and treat stormwater runoff. Bioretention areas may be designed with an underdrain that returns runoff to the conveyance system or designed without an underdrain to exfiltrate runoff into the soil. |
| Bioslopes | Bioslopes are linear, non-structural BMPs with a permeable media that allows stormwater runoff to infiltrate and filter through the practice before exiting through an underdrain. Generally, a pre-treatment device, such as filter strip, grass shoulder, or pea gravel diaphragm, is placed upstream of the bioslope to capture sediment and debris. |
| Downspout Disconnects | A downspout disconnect spreads rooftop runoff from individual downspouts across lawns, vegetated areas, and other pervious areas, where the runoff is slowed, filtered, and can infiltrate into the native soils. |
| Dry Detention / Dry Extended Detention Basins | Dry detention basins and dry extended detention (ED) basins are surface facilities intended to provide temporary storage of stormwater runoff to reduce downstream water quantity impacts. |
| Dry Wells | Dry wells are shallow excavations, typically filled with stone, that are designed to intercept and temporarily store post-construction stormwater runoff under the ground surface until it infiltrates into the underlying and surrounding soils. If properly designed, they can provide significant reductions in post-construction stormwater runoff rates, volumes, and pollutant loads on development sites. |
| Enhanced Dry or Wet Swales | Enhanced swales are vegetated open channels that are designed and constructed to capture and treat stormwater runoff within dry or wet cells formed by check dams or other structures. |
| Grass Channels | Grass channels are vegetated open channels that provide "biofiltering" of stormwater runoff as it flows across the grass surface. |
| Gravity Oil / Grit Separators | Gravity oil / grit separators are hydrodynamic controls that use the movement of stormwater runoff through a specially-designed structure to remove target pollutants. They are typically used on smaller, impervious, commercial sites and urban hotspots. |
| Green Roofs | Green roofs represent an alternative to traditional impervious roof surfaces and typically consist of underlying water proofing, drainage systems, and an engineered planting media. Stormwater runoff is captured and temporarily stored in the engineered planting media, where it is subjected to evaporation and transpiration before being conveyed back into the storm drain system. There are two different types of green roof systems. Intensive green roofs have a thick layer of soil, can support a diverse plant community, and may include trees. Extensive green roofs have a much thinner layer of soil that is comprised primarily of drought tolerant vegetation. |
| Infiltration Practices | An infiltration practice is a shallow excavation, typically filled with stone or an engineered soil mix, which is designed to temporarily hold stormwater runoff until it infiltrates into the surrounding soils. Infiltration practices are able to reduce stormwater quantity, recharge the groundwater, and reduce pollutant loads. |
| Multi-Purpose Detention Basins | Multi-purpose detention basins are on-site areas used for one or more specific activities, such as parking lots and rooftops, as well as for the temporary storage of runoff. |
| Organic Filters | Organic filters are surface media filters that use organic materials, such as leaf compost or a peat/sand mixture, as the filter media. Runoff is filtered through the media prior to discharging through an underdrain system. The organic media may be able to provide enhanced removal of some contaminants, such as heavy metals. |

Table 4.1.1-1 Best Management Practices (continued)

| BMP | Description |
|---|---|
| Permeable Paver Systems | A permeable paver system is a pavement surface composed of structural units with void areas that are filled with pervious materials such as gravel, sand, or grass turf. The system is installed over a gravel base course that provides structural support and stores stormwater runoff that infiltrates through the system into underlying permeable soils. |
| Pervious Concrete | Pervious concrete is the term for a mixture of coarse aggregate, portland cement, and water that allows for rapid infiltration of water. The concrete overlays a stone aggregate reservoir that provides temporary storage as runoff infiltrates into underlying permeable soils and/or out through an underdrain system. |
| Porous Asphalt | Porous asphalt is asphalt with large void spaces to allow water to drain through it. Porous asphalt allows water to infiltrate into the subsoil through the paved surface and a base, aggregate layer that acts as both a structural layer and container to temporarily hold water. Porous asphalt is generally used on sidewalks, bicycle paths, or roads with low traffic volumes. |
| Proprietary Systems | Proprietary controls are manufactured structural control systems available from commercial vendors that are designed to treat stormwater runoff and/or provide water quantity control. Proprietary systems often can be used on small sites and in space-limited areas. |
| Rainwater Harvesting | Rainwater harvesting is a common stormwater management practice used to catch rainfall and store it for later use. Typically, gutters and downspout systems are used to collect the water from roof tops and direct it to a storage tank. Rainwater harvesting systems can be either above or below the ground. Once captured in the storage tank, the water may be used for non-potable indoor (requires treatment) and outdoor uses. |
| Regenerative Stormwater Conveyance | A regenerative stormwater conveyance (RSC) is a practice that provides treatment, infiltration, and conveyance to stormwater runoff through a combination of pools, riffles (with either cobble rocks or boulders), native vegetation, an underlying sand layer, and wood chips. RSCs can also be used to repair areas with large amounts of erosion. |

| BMP | Description |
|---|---|
| Sand Filter:<br>» Surface Sand Filter<br>» Perimeter Sand Filter<br>» Underground Sand Filter | Sand filters are multi-chamber structures designed to treat stormwater runoff through filtration, using a sand bed as its primary filter media. Filtered runoff may be returned to the conveyance system through an underdrain system, or allowed to partially exfiltrate into the soil. A surface sand filter is a ground-level open air structure that consists of a pretreatment sediment forebay and a filter bed chamber. A perimeter sand filter is an enclosed system typically just below the ground in a vault along the edge of an impervious area such as a parking lot. Underground sand filters are sand filter systems located in an underground vault. |
| Site Reforestation / Revegetation | Site reforestation/revegetation is a process of planting trees, shrubs and other native vegetation in disturbed pervious areas to restore the area to pre-development or better conditions. The process can be used to establish mature native plant communities, such as forests, in pervious areas that have been disturbed by clearing, grading and other land disturbing activities. These plant communities intercept rainfall and slow and filter the stormwater runoff to improve infiltration in the ground. Areas that have been reforested or revegetated should be maintained in an undisturbed, natural state over time. These areas must be designated as conservation areas and protected in perpetuity through a legally enforceable conservation instrument (e.g., conservation easement, deed restriction). |
| Soil Restoration | Soil restoration is the process of tilling and adding compost and other amendments to soils to restore them to their pre-development conditions. This improves the soil's ability to reduce post-construction stormwater runoff rates, volumes and pollutant loads. This process is ideal for areas that have been disturbed by clearing, grading and other land disturbing activities. This process is generally used in conjunction with other practices including, but not limited to, vegetated filter strips, grass channels, and simple downspout disconnections. |

BACK TO TOC

Table 4.1.1-1 Best Management Practices (continued)

| BMP | Description |
|---|---|
| Stormwater Planters / Tree Boxes | Stormwater planters are similar to bioretention areas in their design purpose to detain, filter, and infiltrate stormwater. In addition, stormwater planters utilize native or non-invasive flowers, shrubs and trees to provide aesthetic qualities to the site. Planters and tree boxes receive stormwater from a variety of sources such as, rooftops, downspouts and runoff from streets. |
| Stormwater Ponds:<br>» Wet Pond<br>» Wet Extended Detention Pond<br>» Micropool Extended Detention Pond<br>» Multiple Pond Systems | Stormwater ponds are constructed stormwater retention basins that have a permanent pool (or micropool) of water. Some runoff reduction is achieved within a stormwater pond or detention system through evaporation and transpiration. Stormwater ponds provide water quality treatment through sediment precipitation in the permanent pool. |
| Stormwater Wetlands:<br>» Shallow Wetland<br>» Extended Detention Shallow Wetland<br>» Pond/Wetland Systems<br>» Pocket Wetland | Stormwater wetlands are constructed wetland systems used for stormwater management. Stormwater wetlands consist of a combination of shallow marsh areas, open water, and semi-wet areas above the permanent water surface. As stormwater runoff flows through a wetland, it is treated, primarily through gravitational settling and biological uptake. |
| Submerged Gravel Wetlands | Submerged gravel wetland systems use wetland plants in submerged gravel or crushed rock media to remove stormwater pollutants. An anaerobic zone is created within the gravel wetland that provides additional treatment for contaminants. |
| Underground Detention | Underground detention tanks and vaults provide temporary storage of stormwater runoff for space-limited areas where there is not adequate land for a dry detention basin or multi-purpose detention area. |
| Vegetated Filter Strip | Vegetated filter strips are uniformly graded and densely vegetated sections of land that provide “biofiltering” of stormwater runoff as it flows across the surface. |

#### 4.1.1.3 USING OTHER OR NEW BEST MANAGEMENT PRACTICES

Innovative technologies should be allowed and encouraged providing there is sufficient documentation as to their effectiveness and reliability. Communities can allow the use of controls not included in this Manual at their discretion, but should not do so without independently derived information concerning performance, maintenance, application requirements, and limitations.

More specifically, new BMP designs will not be accepted for inclusion in the Manual until independent pollutant removal performance monitoring data determine that the practice can meet the TSS and other selected pollutant concentration removal targets, and that the practice conforms with all necessary criteria for treatment, maintenance, and environmental impact.



BACK TO TOC

### 4.1.2 Best Management Practice Pollutant Removal Capabilities

Best Management Practices (BMPs) are intended to provide water quality treatment for stormwater runoff. Though each of these practices provides pollutant removal capabilities, the relative capabilities vary between BMPs and for different pollutant types.

Pollutant removal capabilities for a given BMP are based on a number of factors including the physical, chemical and/or biological processes that take place in the practice and the design and size of the facility. In addition, pollutant removal efficiencies for the same BMP type and facility design can vary widely depending on the tributary land use and area, incoming pollutant concentrations, rainfall pattern, time of year, maintenance frequency, and numerous other factors.

To assist designers in evaluating the relative pollutant removal performance of the various structural control options, **Table 4.1.3-1** provides design removal efficiencies for each of the above listed BMPs. It should be noted that these values are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. A BMP design may be capable of exceeding these performances; however, the values in the table are minimum reasonable values that can be assumed to be achieved when the practice is sized, designed, constructed, and maintained in accordance with recommended specifications in this Manual.

Where the pollutant removal capabilities of an individual BMP are not deemed sufficient for a given site application, additional practices may be used in series in a "treatment train" approach. More details on using BMPs in series are provided in Subsection 4.1.6.1.

For additional information and data on the range of pollutant removal capabilities for various BMPs, refer to the National Pollutant Removal Performance Database (3rd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.1.3 Best Management Practice Selection

#### 4.1.3.1 BMP SCREENING PROCESS

Outlined below is a screening process for BMPs. This process is intended to assist the site designers and engineers in selecting the most appropriate BMPs for a development site, and provides guidance on factors to consider in their location.

In general the following five criteria should be evaluated in order to select the appropriate structural control(s) or group of controls for a development:

- Stormwater Management and Treatment
- Runoff Reduction Capability
- Water Quality Performance
- Site Applicability
- Cost Considerations - both capital and maintenance

In addition, for a given site, the following factors should be considered and any specific design criteria or restrictions need to be evaluated:

- Physiographic Factors
- Soils
- Special Watershed or Stream Considerations

Finally, environmental regulations that may influence the location of a BMP on-site, or may require a permit, need to be considered. See Section 2.4 for additional information regarding permitting information.

#### 4.1.3.2 DESIGN PROCESS

Using **Table 4.1.3-1**, the site designer can evaluate and screen the overall applicability of BMPs, as well as the constraints of the site in question. Following are details of the various screening categories and individual characteristics used to evaluate structural controls.

**Stormwater Management and Treatment**

The first group of columns in **Table 4.1.3-1** examines the capability of each BMP option to provide runoff reduction. The second set of columns references water quality treatment, downstream channel protection, overbank flood protection, extreme flood protection, and provides an overview of the pollutant removal performance of each BMP option when designed, constructed, and maintained according to the criteria and specifications in this Manual. The presence of a check mark indicates that the structural control can be used to meet a unified stormwater sizing criterion. An 'X' entry in the table means that the practice cannot or is not typically used to meet a unified stormwater sizing criterion. This does not necessarily mean that it should be eliminated from consideration, but rather is a reminder that more than one BMP may be needed at a site (e.g., a bioretention area used in conjunction with dry detention storage). A 'Ť' entry indicates that a BMP may be able to meet the Unified Sizing Criterion depending upon size, configuration, and design constraints.

- **Runoff Reduction Volume ($RR_v$)** – Indicates whether a BMP reduces and/or removes a portion of the Runoff Reduction Volume ($RR_v$) through storage, infiltration, exfiltration, and root uptake. Runoff reduction percentages are listed in **Table 4.1.2.2** BMP Runoff Reduction Credits.
- **Water Quality Volume ($WQ_v$)** – Indicates whether a BMP provides treatment of the water quality volume ($WQ_v$).
- **Channel Protection ($CP_v$)** – Indicates whether the BMP can be used to provide extended detention of the channel protection volume ($CP_v$).
- **Overbank Flood Protection ($Q_{p25}$)** – Indicates whether a BMP can be used to meet the overbank flood protection criteria.
- **Extreme Flood Protection ($Q_f$)** – Indicates whether a BMP can be used to meet the extreme flood protection criteria.
- **Total Phosphorous Removal** – Indicates the capability of a BMP to remove phosphorus in runoff, which may be of particular concern with certain downstream receiving waters.
- **Total Nitrogen Removal** – IIndicates the capability of a BMP to remove nitrogen in runoff, which may also be of particular concern with certain downstream receiving waters.
- **Fecal Coliform Removal** – IIndicates the capability of a BMP to remove fecal coliform and associated bacteria in runoff. This capability may be of particular focus in areas with public beaches, shellfish beds, and/or water regulatory quality criteria under the Total Maximum Daily Load (TMDL) program.
- **Metals Removal** – Indicates the capability of a BMP to remove trace metals, which may be present in stormwater runoff from designated hotspots, or where subsurface soil and/or groundwater contamination may be present. Examples of hotspots include: gas stations, convenience stores, marinas, public works storage areas, vehicle service and maintenance areas, commercial nurseries, and auto recycling facilities.

Please see the specific design criteria of each BMP for more information on the capability of each practice to treat runoff from designated hotspots.

### Site Applicability

The third group of columns in **Table 4.1.3-1** provides an overview of specific site conditions or criteria that must be met for a particular BMP to be suitable. In some cases, these values are recommended values or limits that can be exceeded or reduced with proper design or depending on specific circumstances. Please see the specific criteria section of each practice for more details.

- **LID/GI** – Indicates that the BMP is a LID/GI structure and can be used in those applications.
- **Drainage Area** – Indicates the approximate minimum or maximum drainage area that is considered suitable for the BMP. If the drainage area present at a site is slightly greater than the maximum allowable drainage area for a practice, some leeway can be permitted if more than one practice is installed. The minimum drainage areas indicated for ponds and wetlands should not be considered inflexible limits, and may be increased or decreased depending on water availability (baseflow or groundwater), the mechanisms employed to prevent outlet clogging, and/or design variations used to maintain a permanent pool (e.g., liners).
- **Space Required (% of Impervious Area)** – Comparative index expresses how much space a BMP typically consumes at a site in terms of the approximate area required as a percentage of impervious area draining to the control. BMP design constraints affecting the site area consumed by the practice are also included in this column. For example, the flow path for a downspout disconnect must be 15 feet; the total area of the downspout disconnect can vary as long as the BMP meets this design constraint.
- **Maximum Site Slope** – Evaluates the effect of slope on the best management practice. Specifically, the slope restrictions refer to how flat the area where the facility is installed must be and/or how steep the contributing drainage area or flow length can be.
- **Minimum Head (Elevation Difference)** – Provides an estimate of the minimum elevation difference needed at a site (from inflow to outflow) to allow for gravity operation within the BMP.
- **Water Table** – Indicates the minimum depth to the seasonally high water table from the bottom, or floor, of a BMP.

### Cost Considerations

The last group of columns in **Table 4.1.3-1** provides cost considerations and an estimate of the maintenance burden for each BMP option.

- **Construction Cost** – Structural controls are ranked according to their relative construction cost per impervious acre treated, as estimated from cost surveys.
- **Maintenance** – Assesses the relative maintenance effort needed for a BMP in terms of three criteria: frequency of scheduled maintenance, chronic maintenance problems (such as clogging), and reported failure rates. It should be noted that best management practices require routine inspection and maintenance.

**Table 4.1.3-2** provides the runoff reduction values for each of the BMP Practices.

# Table 4.1.3-1 BMP Selection Guide

| BMP | Runoff Reduction | Stormwater Management & Treatment | | | | | | | Site Applicability | | | | | | Cost Considerations | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | RR *** | $WQ_v$ / TSS | $CP_v$ | $Q_{p25}$ / $Q_f$ | Total Phosphorus | Total Nitrogen | Fecal Coliform | Metals | LID/GI | Drainage Area (ac) | Space Req'd (% of Imperv. Drainage Area) | Max Site Slope | Minimum Head (Elevation Difference) | Depth to Water Table | Construction Cost | Maintenance Burden |
| Bioretention Basins [3, 5, 6] | Yes | 85% | † | † | 80% | 60% | 90% | 95% | Yes | 5 max | 3-6% | 20% | 3 ft | 2 ft | Med-High | Med |
| Bioslopes [7] | Yes | 85% | † | X | 60% | 25% | 60% | 75% | Yes | N/A | N/A | 5% | N/A | 2 ft | Med | Med |
| Downspout Disconnects [2] | Yes | 80% | X | X | 25% | 25% | N/A** | 40% | Yes | 2,500 ft² | Min. length of flow path 15' | 6% | N/A | No restrictions | Low | Low |
| Dry Detention Basins [6] | No | 60% | X | ✓ | 10% | 30% | N/A** | 50% | No | 75 max | N/A | 15% | 3 ft | 2 ft | Low | Low |
| Dry Extended Detention Basins [2] | No | 60% | ✓ | ✓ | 10% | 30% | N/A** | 50% | No | No restrictions | 1-3% | 15% | 4-8 ft | 2 ft | Low | Low |
| Dry Wells [2] | Yes | 100% | † | X | 100% | 100% | 100% | 100% | Yes | 2,500 ft² | 5-10% | 6% | 2 ft | 2 ft | Med | Med |
| Enhanced Dry Swales [1] | Yes | 80% | † | X | 50% | 50% | X | 40% | Yes | 5 max | 10-20% | 4% | 3-5 ft | 2 ft | Med | Low |
| Enhanced Wet Swales [1] | No | 80% | † | X | 25% | 40% | X | 20% | Yes | 5 max | 10-20% | 4% | 1 ft | Below | Med | Low |
| Grass Channels [1] | Minimal | 50% | † | X | 25% | 20% | X | 30% | Yes | 5 max | 10% | 4% | <1 ft | 2 ft | Low | Low |
| Gravity (oil-grit) Separators [2] | No | 40% | X | X | 5% | 5% | N/A | N/A | No | 5 | N/A | 6% | 4 ft | 2 ft | High | High |
| Green Roofs [2] | Yes | 80% | X | X | 50% | 50% | N/A** | N/A** | Yes | N/A | No restrictions | 25% | 6-12 in | N/A | High | Low |
| Infiltration Trenches [10] | Yes | 100% | † | † | 100% | 100% | 100% | 100% | Yes | 5 max | 2-3% | 6% | 1 ft | 2 ft | High | High |
| Multi-Purpose Detention Basins [2] | No | Varies | X | † | N/A** | N/A** | N/A** | N/A** | No | No restrictions | 1-3% | 15% | 4-8 ft | 2 ft | Low | Low |
| Organic Filters [2] | No | 80% | † | X | 60% | 40% | 50% | 75% | Yes | 10 | 3-5% | 6% | 5-8 ft | 2 ft | High | High |
| Permeable Paver Systems [2] | Yes | 80% | † | † | 50% | 50% | N/A** | 60% | Yes | N/A | No restrictions | 6% | 2-4 ft | 2 ft | High | High |
| Pervious Concrete [2] | Yes | 80% | † | † | 50% | 65% | N/A** | 60% | Yes | N/A | No restrictions | 6% | 2-4 ft | 2 ft | High | High |
| Porous Asphalt (excludes OGFC) [2] | Yes | 80% | † | † | 50% | 50% | X | 60% | Yes | N/A | 0% | N/A | N/A | 2 ft | Med | Med |
| Proprietary Systems [2] | Varies | Varies | Varies | Varies | Varies | Varies | Varies | Varies | No | Varies | Varies | Varies | Varies | Varies | Varies | Varies |
| Rainwater Harvesting [2] | Based on Demand | Varies | † | X | Varies | Varies | Varies | Varies | Yes | No restrictions | Varies | No restritions | N/A | N/A | Med | High |
| Regenerative Stormwater Conveyance [8] | No | 80% | X | X | 70% | 70% | N/A** | N/A** | Yes | 50 max | Varies | 10% | Varies | Above | High | Med |
| Sand Filters [1] | No | 80% | † | X | 50% | 25% | 40% | 50% | Yes | 2-10 max | 2-3% | 6% | 2-5 ft | 2 ft | High | High |
| Site Reforestation/Revegetation [2] | No** | N/A** | N/A** | N/A** | N/A** | N/A** | N/A** | N/A** | Yes | N/A | 10,000 ft² Min. | 25% | N/A | No restrictions | Med | Low |
| Soil Restoration [2] | No** | N/A** | N/A** | N/A** | N/A** | N/A** | N/A** | N/A** | Yes | N/A | No restrictions | 10% | N/A | 1.5 ft | Med | Low |
| Stormwater Planters / Tree Boxes [2] | Yes | 80% | X | X | 60% | 60% | 80% | N/A | Yes | 2,500 ft² | 5% | 6% | 2 ft | 2 ft | High | Med |
| Stormwater Ponds [2] | No | 80% | ✓ | ✓ | 50% | 30% | 70% | 50% | No | 10-25 min | 2-3% | 15% | 6-8 ft | 2 ft (if aquifer) | Low | Low |
| Stormwater Wetlands – Level 1 [1] | No | 80% | ✓ | ✓ | 40% | 30% | 70% | 50% | Yes | 25 min | 3-5% | 8% | 2-3 ft | 2 ft (if aquifer) | Med | Med |
| Stormwater Wetlands – Level 2 [4] | No | 85% | X | X | 75% | 55% | 85% | 60% | Yes | 5 min | 3-5% | Flat | 2-3 ft | 2 ft (if aquifer) | Med-High | Med |
| Submerged Gravel Wetlands [2] | No | 80% | X | X | 50% | 20% | 70% | 50% | No | 5 | 3-5% | 4% | 2-5 ft | No restrictions | High | High |
| Underground Detention [2] | No | 0% | ✓ | ✓ | 0% | 0% | 0% | 0% | No | 25 max | N/A | 15% | 4-8 ft | 2 ft | High | Med |
| Vegetated Filter Strips [1] | Yes | 60% | † | X | 20% | 20% | X | 40% | Yes | 5 max | 20% | 6% | <1 ft | 1-2 ft | Low | Low |

✓ - BMP can meet the stormwater management or treatment requirement
† - BMP may meet the stormwater management or treatment requirement depending on size, configuration, and site constraints
X - BMP may contribute but is not likely to fully meet the stormwater management or treatment requirement

* - Minimum drainage area of ten acres is required to maintain the permanent pool (unless groundwater is present).
**- Helps restore pre-development hydrology, which implicitly reduces post-construction stormwater runoff rates, volumes and pollutant loads
*** - Runoff reduction percentages are listed in **Table 4.1.3-2** (BMP Runoff Reduction Credits)

Pollutant Removal References:


1: Original Georgia Stormwater Management Manual, 2001
2: Coastal Stormwater Supplement to the Georgia Stormwater Management Manual, 2009
3: Bioretention - Watershed Benefits. Low Impact Development Urban Design Tools. 04 April 2014.
4: The Next Generation of Stormwater Wetlands. EPA Wetlands and Watersheds Article Series (2008) Center for Watershed Protection
5: Bioretention Performance, Design, Construction, and Maintenance. North Carolina Cooperative Extension Service. Hunt, William. 2006
6: North Carolina Department of Environment and Natural Resources Stormwater Best Management Practices Manual. 2007
7: Washington State Department of Transportation (WSDOT) Highway Runoff Manual, 2011.
8: West Virginia Stormwater Management Design Guidance Manual, 2012
9: Georgia Department of Transportation (GDOT) Drainage Manual, 2014
10: Pollutant removal rates based on 100% infiltration with no underdrain

Table 4.1.3-2 BMP Runoff Reduction Credits

| BMP | Runoff Reduction RR (%) * | BMP | Runoff Reduction RR (%) * |
|---|---|---|---|
| Bioretention Area (w/o underdrain) [1] | 100% | Permeable Paver System (w/underdrain) [1] | 50% |
| Bioretention Area (w/upturned underdrain) [1] | 75% | Pervious Concrete (w/o underdrain) [1] | 100% |
| Bioretention Area (w/underdrain) [1] | 50% | Pervious Concrete (w/upturned underdrain) [1] | 75% |
| Bioslopes (A & B hydrologic soils) [1] | 50% | Pervious Concrete (w/underdrain) [1] | 50% |
| Bioslopes (C & D hydrologic soils) [1] | 25% | Porous Asphalt (w/o underdrain) [1] | 100% |
| Downspout Disconnects (A & B hydrologic soils) [2] | 50% | Porous Asphalt (w/upturned underdrain) [1] | 75% |
| Downspout Disconnects (C & D hydrologic soils) [1] | 25% | Porous Asphalt (w/underdrain) [1] | 50% |
| Dry Detention Basins | 0% | Porous Asphalt (OGFC, PEM) [3] | 0% |
| Dry Extended Detention Basins | 0% | Proprietary Systems [3] | Varies |
| Dry Wells 1 | 100% | Rainwater Harvesting [3] | Based on Demand |
| Enhanced Dry Swales (w/underdrain) [1] | 50% | Regenerative Stormwater Conveyance | 0% |
| Enhanced Dry Swales (w/o underdrain) [1] | 100% | Sand Filters [2] | 0% |
| Enhanced Wet Swales [2] | 0% | Site Reforestation/Revegetation [1] | 0% |
| Grass Channels (A & B hydrologic soils) [2] | 25% | Soil Restoration (Can be used to remediate C & D Soils) [1] | 0% |
| Grass Channels (C & D hydrologic soils) [1] | 10% | Stormwater Planters / Tree Boxes [1] | 50% |
| Gravity (oil-grit) Separators [1] | 0% | Stormwater Ponds [2] | 0% |
| Green Roofs [1] | 60% | Stormwater Wetlands – Level 1 [2] | 0% |
| Infiltration Trenches [1] | 100% | Stormwater Wetlands – Level 2 [2] | 0% |
| Multi-Purpose Detention Basins | 0% | Submerged Gravel Wetlands [2] | 0% |
| Organic Filters [2] | 0% | Underground Detention (Not including infiltration) [2] | 0% |
| Permeable Paver System (w/o underdrain) [1] | 100% | Vegetated Filter Strips (A & B hydrologic soils) [2] | 50% |
| Permeable Paver System (w/ upturned underdrain) [1] | 75% | Vegetated Filter Strips (C & D hydrologic soils) [1] | 25% |

BMP pollutant removal rates and other BMP specific design criteria are listed in Table 4.1.3-1 (BMP Selection Guide)

* Runoff reduction percentages listed are maximum allowable credit values. These values, similar to other pollutant removal rates, are based on the performance and operating efficiency of the BMP.

[1]Runoff reduction percent removals are based on values from the former Georgia Stormwater Management Manual's Coastal Stormwater Supplement, 2009.

[2]Runoff reduction percent removals are based on values from the Center for Watershed Protection, 2008.

[3]Runoff reduction percent removals are not available

BACK TO TOC

Additional considerations should be taken into account when designing a BMP. They include physiographic factors, soils, special watershed or stream considerations, and temperature.

**Physiographic Factors**

Three key factors to consider are low-relief, high-relief, and karst terrain. In the state of Georgia, low-relief (very flat) areas are primarily located in the Coastal Plain and along the Atlantic coast. High-relief (steep and hilly) areas are found throughout the Piedmont and far northern parts of the state. Karst and major carbonaceous rock areas are generally found in the western portions of the state. Special geotechnical testing requirements may be needed in karst areas. The local reviewing authority should be consulted to determine if a project is subject to terrain constraints..

- 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. In other cases high-relief terrain may impact dam heights to the point that the use of a practice becomes infeasible.
- Karst terrain can limit the use of some BMPs as the infiltration of polluted waters directly into underground streams found in karst areas may be prohibited. In addition, ponding areas may not reliably hold water in karst areas.

**Soils**

Key evaluation factors are based on an initial investigation of the NRCS hydrologic soils groups at the site. Note that more detailed geotechnical tests are usually required for infiltration feasibility and during design to confirm permeability and other factors.

**Special Watershed or Stream Considerations**

The design of BMPs is fundamentally influenced by the nature of downstream receiving water bodies. Consequently, designers should determine the Use Classification of the watershed in which their project is located prior to design (see Georgia Department of Natural Resources Environmental Protection Division Water Quality Control Rules Chapter 391-3-6). In addition, designers should consult with appropriate review authorities to determine if their development project is subject to additional BMP criteria as a result of an adopted local watershed plan or special provision.

In some cases, higher pollutant removal or environmental performance is needed to fully protect aquatic resources and/or human health and safety within a particular watershed or receiving water. Therefore, special design criteria for a particular practice or the exclusion of one or more practices may need to be considered within these watersheds or protected areas. Examples of important watershed factors to consider include:

- **Primary and Secondary Trout Streams** – 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. Some BMPs can have adverse downstream impacts on cold-water streams, and their design may need to be modified or use restricted. BMPs that result in reduced thermal and sediment loading, such as shaded or underground stormwater detention and green roof technologies should be considered where stormwater runoff may impact trout streams.
- **High Quality Streams** (High quality streams with a watershed impervious cover less than approximately 15%) – These streams may also possess high quality cool water or warm water aquatic resources and/or endangered species. The design objectives are to maintain habitat quality through the same techniques used for cold-water streams, with the exception that stream warming is not as severe of a design constraint. These streams may also be specially designated by local authorities.
- **Wellhead Protection** –Areas that recharge existing public (or known private) water supply wells present a unique management challenge. The key design constraint is to prevent possible groundwater contamination by preventing infiltration of hotspot runoff. At the same time, recharge of unpolluted stormwater is encouraged to maintain flow in streams and wells during dry weather.

BACK TO TOC

- **Reservoir or Drinking Water Protection** – Watersheds that deliver surface runoff to a public water supply reservoir or impoundment are a special concern. Depending on the treatment available at the water intake, it may be necessary to achieve a greater removal rates for pollutants of concern, such as bacteria pathogens, nutrients, sediment, and/or metals. One particular management concern for reservoirs is ensuring that stormwater hotspots are adequately treated so that they do not contaminate drinking water.
- **Swimming/Shellfish** – Watersheds that drain to public swimming waters or shellfish harvesting areas require a higher level of stormwater treatment to prevent closings caused by bacterial contamination from stormwater runoff. In these watersheds, BMPs should be explicitly designed to maximize bacteria removal.
- **Temperature** – In many cases, the water running off the site may flow over an impervious surface that has absorbed the sun's heat, and can potentially cause thermal pollution to streams. Water temperatures are also increased due to shallow ponds and impoundments along a watercourse and fewer trees along streams to shade the water. This may have an adverse effect on aquatic life, particularly in trout streams as mentioned above. **Table 4.1.3-3** shows a list of the BMPs and if they can potentially reduce thermal pollution. BMPs that encourage infiltration appear to be the most effective in mitigating thermal impacts.

Additionally, Better Site Design principles (see Section 2.3 for more information), including conservation of natural buffers and stream canopy can maintain shade and reduce thermal pollution.

Table 4.1.3-3 – List of BMPs with Temperature Reduction Possibility

| BMP | *Temperature Reduced? | BMP | *Temperature Reduced? |
|---|---|---|---|
| Bioretention Areas | Yes | Pervious Concrete | No |
| Bioslopes | Yes | Porous Asphalt | No |
| Downspout Disconnects | No | Proprietary Systems | No |
| Dry Detention Basins | No | Rainwater Harvesting | No |
| Dry Enhanced Swales/Wet Enhanced Swales | No | Regenerative Stormwater Conveyance | Yes |
| Dry Extended Detention Basins | No | Sand Filter | Yes |
| Dry Wells | Yes | Site Reforestation/Revegetation | Yes |
| Grass Channel | Yes | Soil Restoration | Yes |
| Gravity (Oil-Grit) Separators | No | Stormwater Planters/Tree Boxes | Yes |
| Green Roofs | Yes | Stormwater Ponds | No |
| Infiltration Practices | Yes | Stormwater Wetlands | No |
| Multi-Purpose Detention Basin | No | Submerged Gravel Wetlands | Yes |
| Organic Filter | Yes | Underground Detention | Yes |
| Permeable Paver System | No | Vegetated Filter Strip | Yes |

*The effects of BMPs on temperature can be dependent on many factors, including the sizing of the BMP and the presence of an underdrain.

### 4.1.3.3 EXAMPLE APPLICATION

A 20-acre institutional area (e.g., church and associated buildings) is being constructed in a dense urban area within metropolitan Atlanta. The existing impervious coverage of the site is 40%. The site drains to an urban stream that is highly impacted from hydrologic alterations (accelerated channel erosion). The stream channel is deeply incised; consequently, flooding is not a problem. The channel drains to an urban river that is a tributary to a phosphorus limited drinking water reservoir. Low permeability soils limit infiltration practices.

- **Objective**: Avoid additional disruptions to receiving channels and reduce pollutant loads for sediment and phosphorus to receiving waters.
- **Target Removals**: Provide stormwater management to mitigate for accelerated channel incision and reduce loadings of key pollutants by:
  - » *Sediment*: 80%
  - » *Phosphorus*: 40%
- **Activity/Runoff Characteristics**: The proposed site will have large areas of impervious surface in the form of parking and structures. However, there will be a large contiguous portion of turf grass proposed for the front of the parcel that will have a relatively steep slope (approximately 10%) and will drain to the storm drain system associated with the entrance drive. Stormwater runoff from the site is expected to exhibit fairly high sediment levels and seasonally high phosphorus levels (due to turf grass management).

While there is a downstream reservoir to consider, there are no special watershed factors or physiographic factors that preclude the use of any of the practices from the BMP list. However, due to the size of the drainage area, most stormwater ponds and wetlands are removed from consideration. In addition, the site's impermeable soils remove an infiltration trench from being considered. Vegetated filter strips and downspout disconnects could be considered for pretreatment; however, the steep slope may reduce their effectiveness and preclude their use.

To provide additional pollutant removal capabilities in an attempt to better meet the target removals, bioretention, surface sand filters, and/or perimeter sand filters can be used to treat the parking lot and driveway runoff. Bioretention provides some removal of phosphorus while improving the aesthetics of the site. Surface sand filters provide higher phosphorus removal at a comparable unit cost to bioretention, but are not as aesthetically pleasing. The perimeter sand filter is a flexible, easy to access practice (but at higher cost) that provides good phosphorus removal and high oil and grease trapping ability. A bioslope may be an ideal BMP for use in higher slope situations and should be designed to complement the site's aesthetics and accommodate pedestrian infrastructure.

Designers may consider the feasibility of using green roofs, pervious concrete, porous asphalt, and/or permeable pavers wherever possible to reduce the overall imperviousness of the property. However, the compacted subsurface soils may reduce the effectiveness of installing porous surfaces and can require the use of an extensive underdrain system that may become expensive. Soil restoration and site revegetation may be used to mitigate the effects of the urbanized soils. Installation of a green roof can increase the building's construction costs and should be considered early in the design process.

To provide overbank flood control, as well as channel protection storage, a micropool ED pond, dry extended detention basin, or multi-purpose detention basin will likely be needed, unless some downstream regional storage is available to control the overbank flood. The site drainage system can be designed so that the bioretention and/or sand filters drain to the micropool ED pond or detention basin for redundant treatment. Vegetated dry swales could also be used to convey runoff to the pond, which would provide pretreatment, runoff reduction, TSS removal, and some water quality volume reduction. Pocket wetlands, wet swales, and submerged gravel wetlands were eliminated from consideration due to the potential for nuisance conditions. Underground sand filters could also be used at the site; however, cost and aesthetic considerations were significant enough to eliminate them from consideration.

BACK TO TOC

## 4.1.4 On-Line Versus Off-Line Best Management Practices

### 4.1.4.1 INTRODUCTION

Stormwater best management practices (BMPs) are designed to be either "on-line" or "off-line." On-line BMPs are designed to receive, but not necessarily control or treat, entire runoff volumes up to the $Q_{p25}$ or $Q_f$ event. On-line BMPs must be able to handle the entire range of storm flows. Off-line facilities are designed to receive only a specified flow rate through the use of a flow regulator (diversion structure, flow splitter, etc.). Flow regulators are typically used to divert the water quality volume ($WQ_v$) or runoff reduction volume ($RR_v$) to an off-line BMP sized and designed to treat and control the $WQ_v$. After the design runoff flow has been treated and/or controlled it is returned to the conveyance system. **Figure 4.1.4.1** shows an example of an off-line sand filter and an on-line enhanced dry swale.

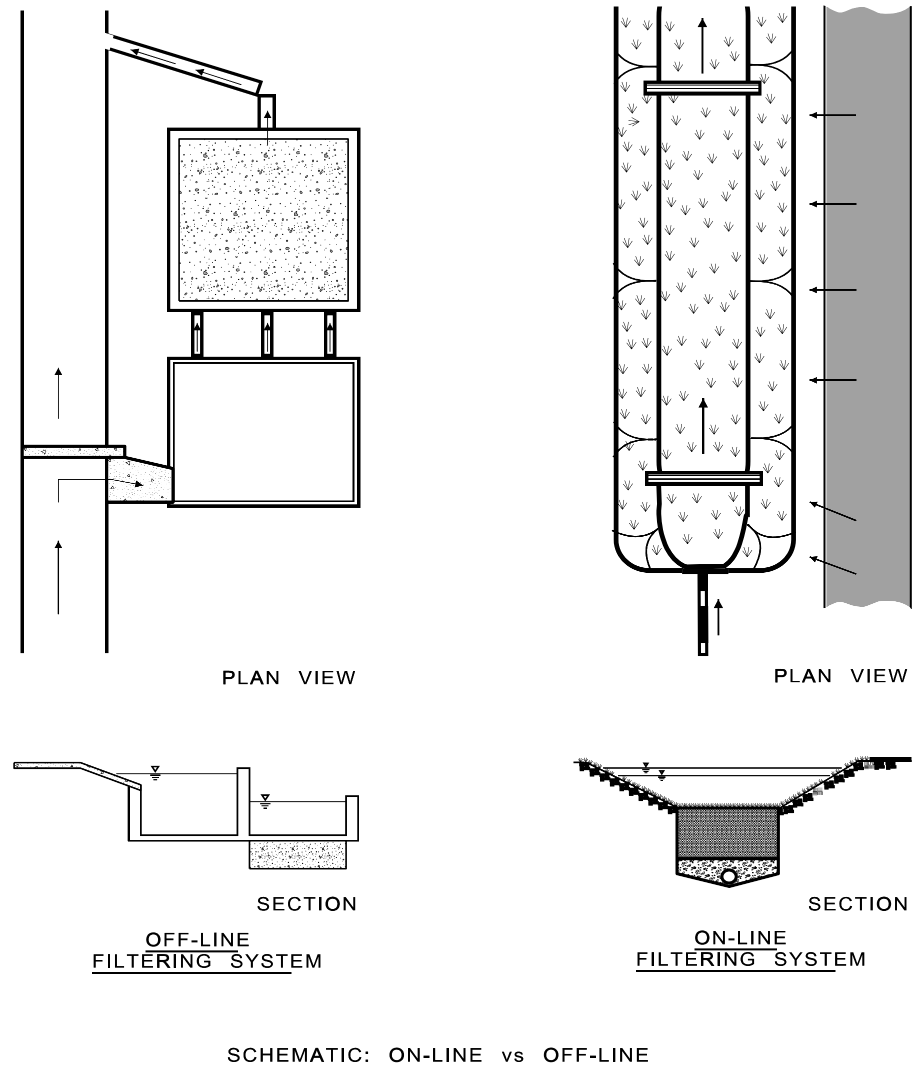



**Figure 4.1.4-1** Example of On-Line versus Off-Line BMPs
(Source: CWP, 1996)



BACK TO TOC

#### 4.1.4.2 FLOW REGULATORS

Flow regulation to off-line best management practices can be achieved by either:

- Diverting the water quality volume or other specific maximum flow rate to an off-line BMP, or
- Bypassing flows in excess of the design flow rate.

The peak water quality flow rate ($Q_{wq}$) can be calculated using the procedure found in Subsection 3.1.7.2.

Flow regulators can be flow splitter devices, diversion structures, or overflow structures. Several examples are shown on the right and on the following pages.

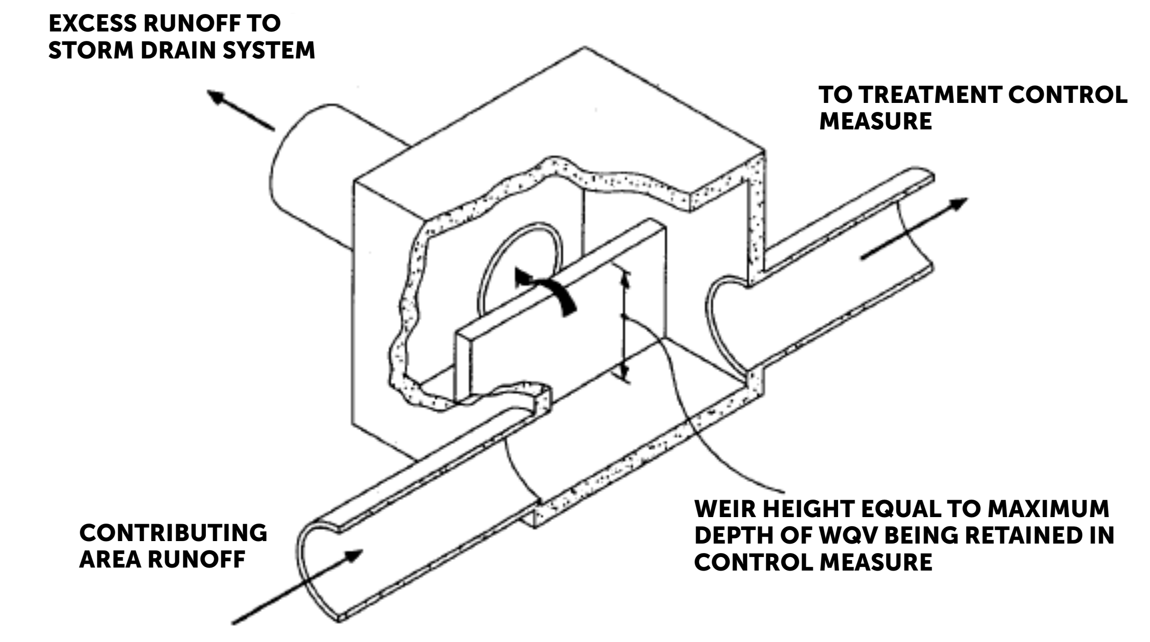



**Figure 4.1.4-2** Pipe Interceptor Diversion Structure
(Source: City of Sacramento, 2000



BACK TO TOC

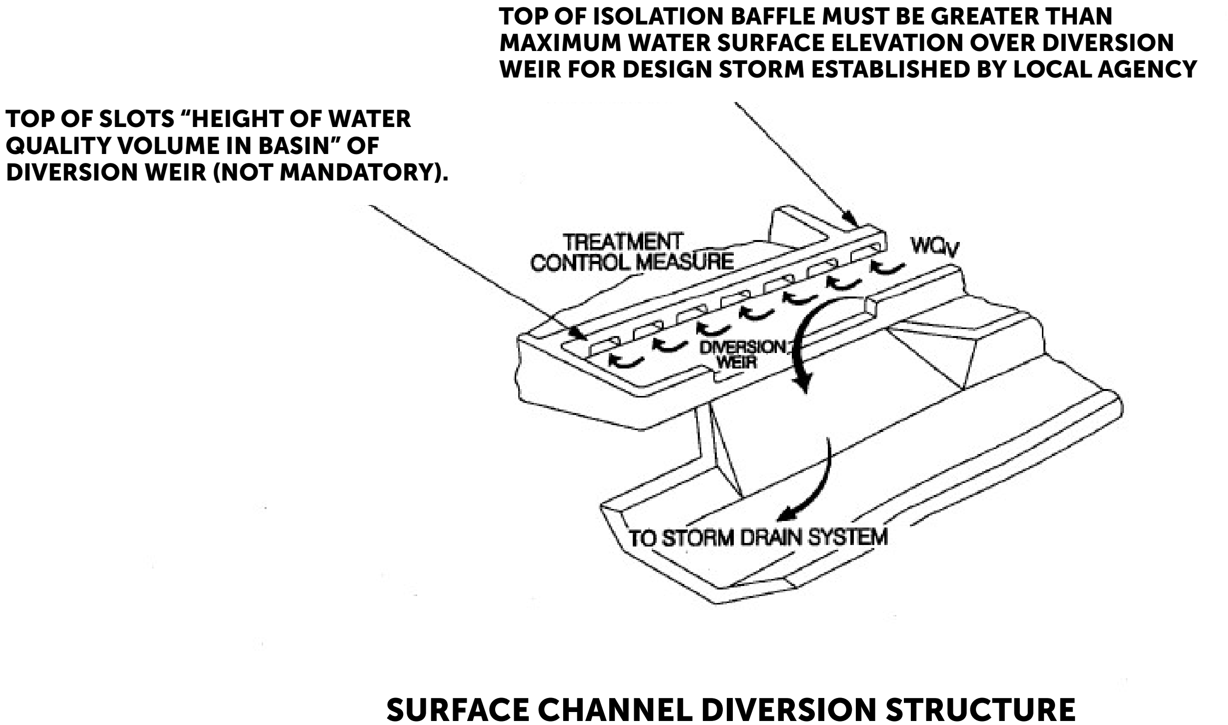



**Figure 4.1.4-3** Surface Channel Diversion Structure
(Source: City of Sacramento, 2000)

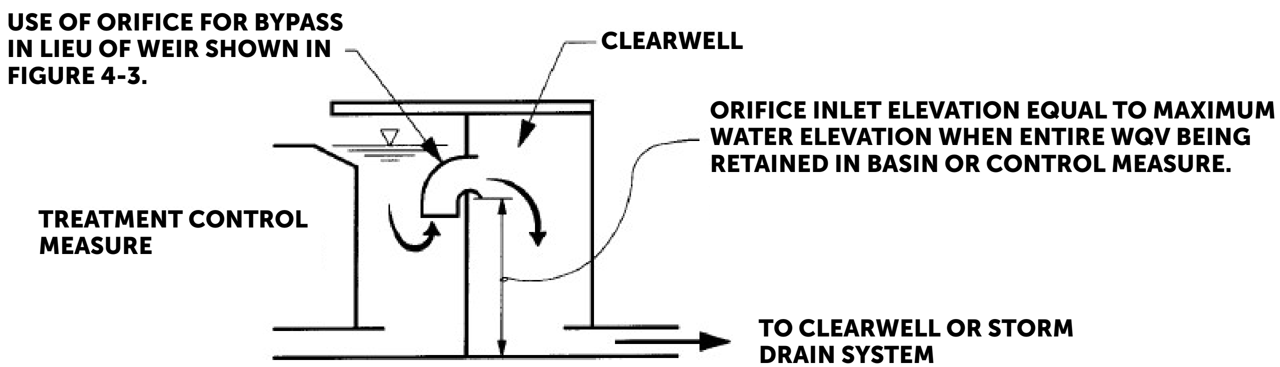



**Figure 4.1.4-4** Outlet Flow Regulator
(Source: City of Sacramento, 2000)

## 4.1.5 Regional vs. On-site Stormwater Management

### 4.1.5.1 INTRODUCTION

Using individual, on-site best management practices or a treatment train of one or more BMPs for each development is the typical approach for controlling stormwater quantity and quality. The developer finances the design and construction of these controls and, initially, is responsible for all operation and maintenance.

A potential alternative approach is for a community to install a few strategically located regional stormwater BMPs in a subwatershed rather than requiring on-site controls (see **Figure 4.1.5-1**). For this Manual, regional BMPs are defined as facilities designed to manage stormwater runoff from multiple projects and/or properties through a local jurisdiction-sponsored program, where the individual properties may assist in the financing of the facility, and the requirement for on-site practices is either eliminated or reduced

### 4.1.5.2 ADVANTAGES AND DISADVANTAGES OF REGIONAL BEST MANAGEMENT PRACTICES

Regional stormwater facilities can be more cost-effective because it is easier and less expensive to build, operate, and maintain one large facility than several small ones. Regional stormwater facilities can also be better maintained than individual site BMPs because they are large, highly visible, and typically the responsibility of local government entities.

There are also several disadvantages to regional stormwater BMPs. In many cases, a community must provide capital construction funds for a regional facility, including the costs of land acquisition. However, if a downstream developer is the first to build, that person could be required to construct the facility and later be compensated by upstream developers for the capital construction costs and annual maintenance expenditures.

Conversely, an upstream developer may have to establish temporary control structures if the regional facility is not in place before construction. Maintenance responsibilities generally shift from the homeowner or developer to the local government when a regional approach is selected. The local government would need to establish a stormwater utility or some other program to fund and implement stormwater management facilities. Finally, a large in-stream facility can pose a greater disruption to the natural flow network and is more likely to affect wetlands within the watershed.

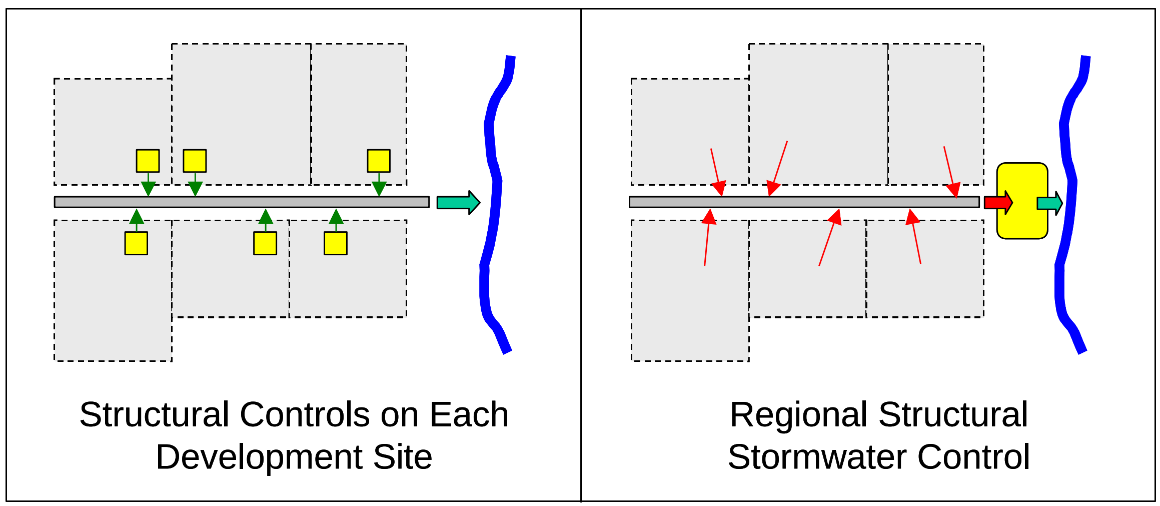



**Figure 4.1.5-1** On-site versus Regional Stormwater Management

Below are summarized some of the "pros" and "cons" of regional stormwater practices.

**Advantages of Regional Stormwater Management Facilities**

- Reduced Construction Costs – Design and construction of a single regional stormwater management facility can be more cost-effective than numerous individual on-site BMPs.
- Reduced Operation and Maintenance Costs – Rather than multiple owners and associations being responsible for the maintenance of several stormwater facilities on their developments, it is simpler and more cost effective to establish scheduled maintenance of a single regional facility.
- Higher Assurance of Maintenance – Regional stormwater facilities are far more likely to be adequately maintained as they are large and have a higher visibility, and are typically the responsibility of local government.
- Maximum Utilization of Developable Land – Developers would be able to potentially maximize the utilization of the proposed development for the purpose intended by minimizing the land normally set aside for the construction of stormwater BMPs.
- Retrofit Potential – Regional facilities can be used by a community to mitigate existing developed areas that have insufficient or no best management practices for water quality and/or quantity, as well as provide for future development.
- Other Benefits – Well-sited regional stormwater facilities can serve as a recreational and aesthetic amenity for a community.

**Disadvantages of Regional Stormwater Controls**

- Location and Siting – Regional stormwater facilities may be difficult to site, particularly for large facilities or in areas with existing development.
- Capital Costs – The community must typically provide capital construction funds for a regional facility, including the costs of land acquisition.
- Maintenance – The local government is typically responsible for the operation and maintenance of a regional stormwater facility.
- Need for Planning – The implementation of regional stormwater management facilities requires substantial planning, financing, and permitting. Land acquisition must be in place ahead of future projected growth.

**For in-stream regional facilities:**

- Water Quality and Channel Protection – Without on-site water quality and channel protection, regional controls do not protect smaller streams upstream from the facility from degradation, streambank erosion, and thermal impacts. Further, diverting stormwater runoff to a stormdrain system for discharge to a regional facility eliminates the opportunity to provide local groundwater recharge and runoff reduction.
- Ponding Impacts – Upstream inundation from a regional facility impoundment can eliminate floodplains, wetlands, and other habitat.
- Additional Planning – The implementation of in-stream regional facilities requires substantial planning, permitting and increased regulatory compliance.

BACK TO TOC

#### 4.1.5.3 IMPORTANT CONSIDERATIONS FOR THE USE OF REGIONAL STORMWATER MANAGEMENT FACILITIES

If a community decides to implement a regional stormwater facility, then it must ensure that the conveyances between the individual upstream developments and the regional facility can handle the design peak flows and volumes without causing adverse impact or property damage. Full-buildout conditions in the regional facility drainage area should be used in the analysis.

In addition, unless the system consists of completely non-erodable conveyances (storm drains, pipes, concrete channels, etc.), on-site best management practices for water quality and downstream channel protection should be required for all developments within the regional facility's drainage area. Federal water quality provisions do not allow the degradation of water bodies from untreated stormwater discharges, and it is U.S. EPA policy to not allow regional stormwater facilities that would degrade stream quality between the upstream development and the regional facility. Further, without adequate channel protection, aquatic habitats and water quality in the channel network upstream of a regional facility may be degraded by streambank erosion if they are not protected from bankfull flows and high velocities. Based on these concerns, both the EPA and the U.S. Army Corps of Engineers have expressed opposition to in-stream regional stormwater control facilities. In-stream facilities should be avoided if possible and will likely be permitted on a case-by-case basis only.

It is important to note that siting and designing regional facilities should ideally be done within the context of stormwater master planning or watershed planning to be effective.

### 4.1.6 Using Best Management Practices in Series

#### 4.1.6.1 STORMWATER TREATMENT TRAINS

TThe minimum stormwater management standards are an integrated planning and design approach, the components of which work together to limit adverse impacts of urban development on downstream waters and riparian areas. This approach is sometimes called a stormwater "treatment train". When considered comprehensively, a treatment train consists of all the design concepts, better site design practices, and BMPs that work to attain water quality and quantity goals. This is illustrated in **Figure 4.1.6-1.**

**Runoff and Load Generation** – The initial part of the "train" is located at the source of runoff and pollutant load generation, and consists of better site design and pollution prevention practices that reduce runoff and stormwater pollutants.

**Pretreatment** – The next step in the treatment train consists of pretreatment measures. These measures typically do not provide sufficient pollutant removal to meet the runoff reduction or 80% TSS reduction goal, but do provide calculable water quality benefits that may be applied towards meeting the $WQ_v$ treatment requirement. These measures include:

- The use of stormwater better site design practices and site design credits to reduce the water quality volume ($WQ_v$)
- Using best management practices such as downspout disconnects, vegetated filter strips, and/or soil restoration and site revegetation to provide pretreatment and runoff reduction
- Pretreatment facilities, such as sediment forebays, on best management practices that provide storage and/or filtration of runoff


**Figure 4.1.6-1** Generalized Stormwater Treatment Train

BACK TO TOC

Primary Treatment and/or Quantity Control – The last step is primary water quality treatment and/or quantity (channel protection, overbank flood protection, and/or extreme flood protection) control. This is achieved through the use of one or more of the best management practices shown in **Table 4.1.1-1** as a stand-alone stormwater management practice or in series.

### 4.1.6.2 USE OF MULTIPLE BEST MANAGEMENT PRACTICES IN SERIES

Many combinations of best management practices in series may exist for a site. The combinations of best management practices are limited only by the need to employ measures of proven effectiveness and meet local regulatory and physical site requirements. **Figures 4.1.6-3 through 4.1.6-5** illustrate the application of the treatment train concept for: a moderate density residential neighborhood, a small commercial site, and a large shopping mall site.

In **Figure 4.1.6-3** rooftop runoff drains over grassed yards to backyard grass channels. Runoff from front yards and driveways reaches roadside grass channels. Finally, all stormwater flows drain to a micropool ED stormwater pond.

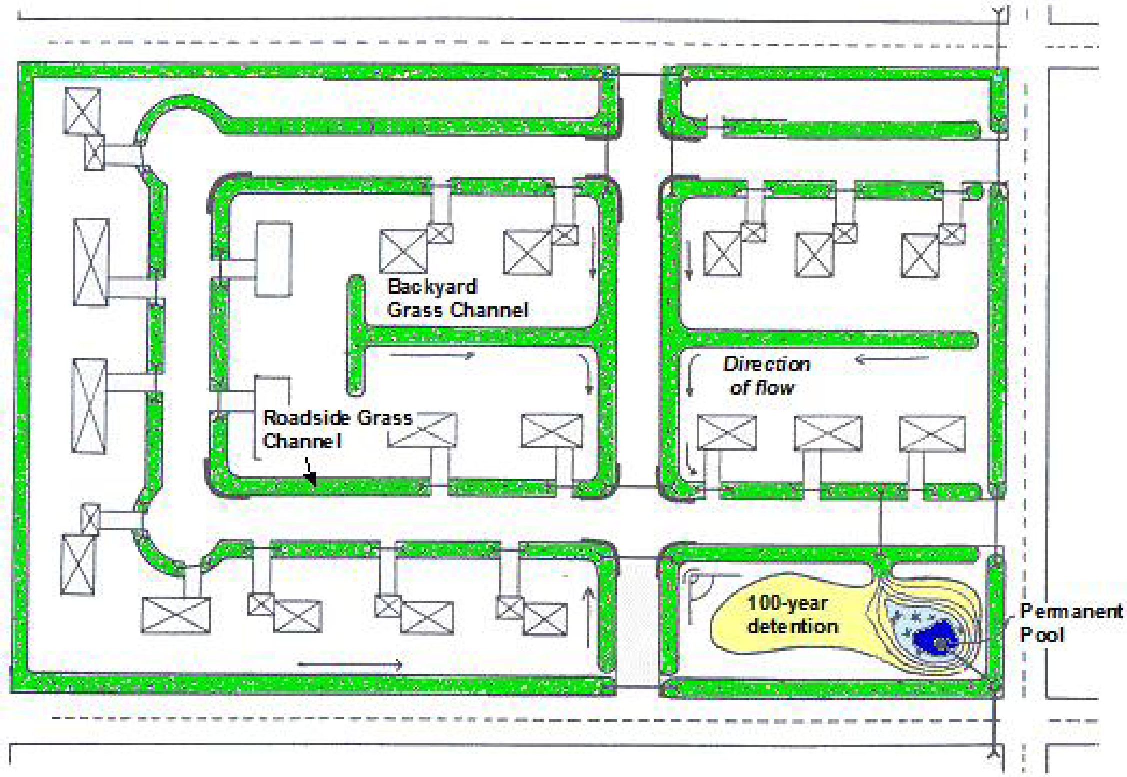



**Figure 4.1.6-3** Example Treatment Train – Residential Subdivision
(Adapted from: NIPC, 2000)

BACK TO TOC

A gas station and convenience store is depicted in **Figure 4.1.6-4**. In this case, the decision was made to intercept hydrocarbons and oils using a commercial gravity (oil-grit) separator located on the site prior to draining to a perimeter sand filter for removal of finer particles and TSS. No BMP for channel protection is required as the system drains to the municipal storm drain pipe system. Overbank and extreme flood protection is provided by a regional stormwater control downstream.

**Figure 4.1.6-5** shows an example treatment train for a commercial shopping center. In this case, runoff from rooftops and parking lots drains to a depressed parking lot, perimeter grass channels, and bioretention areas. Slotted curbs are used at the entrances to these swales to better distribute the flow and settle out the very coarse particles at the parking lot edge for sweepers to remove. Runoff is then conveyed to a wet ED pond for additional pollutant removal and channel protection. Overbank and extreme flood protection is provided through parking lot detention.

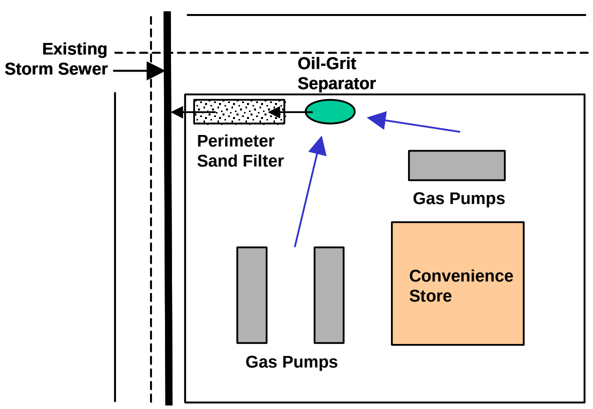



**Figure 4.1.6-4** Example Treatment Train - Commercial Development

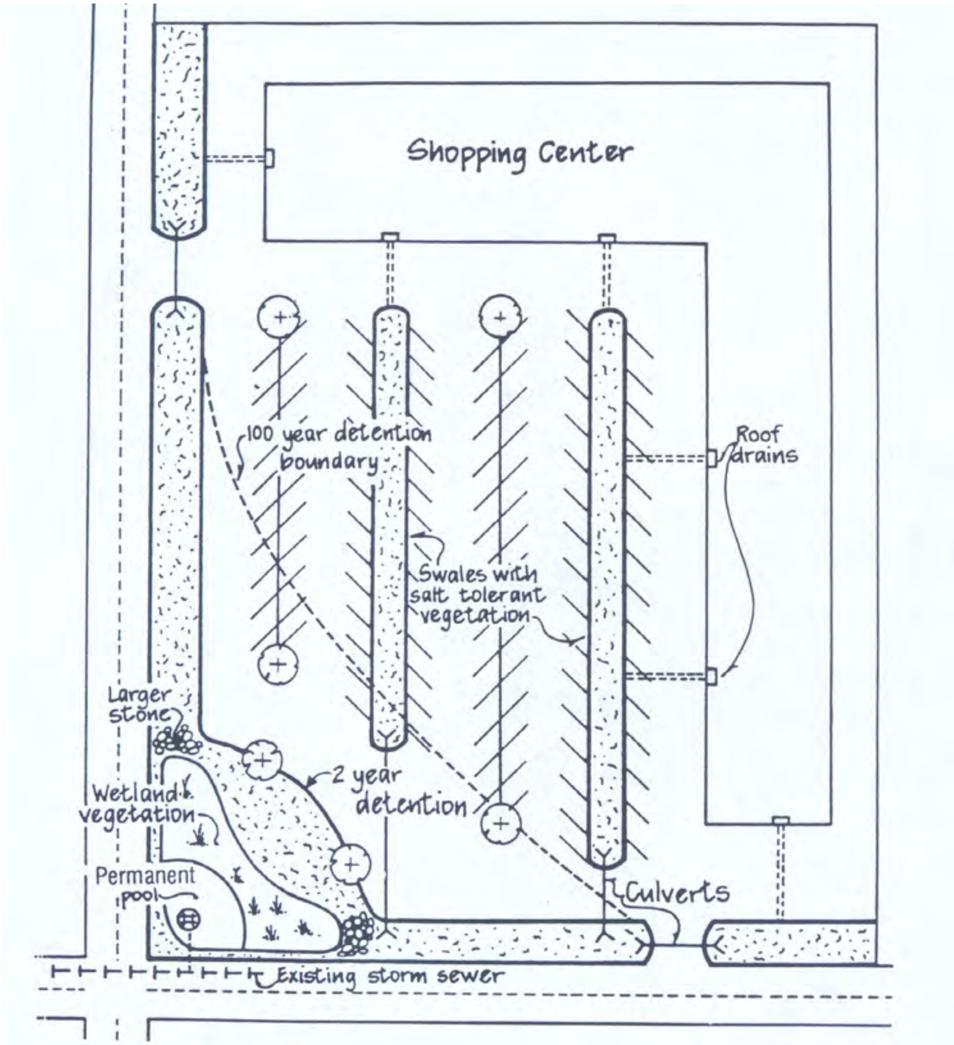



**Figure 4.1.6-5** Example Treatment Train – Commercial Development
(Source: NIPC, 2000)

### 4.1.6.3 CALCULATION OF POLLUTANT REMOVAL FOR WATER QUALITY BEST MANAGEMENT PRACTICES IN SERIES

To estimate the pollutant removal rate of water quality based best management practices in series, the following steps are used to determine the pollutant removal:

- For each drainage area, list the BMPs in order, upstream to downstream, along with their expected average pollutant removal rates from **Table 4.1.3-1** for the pollutants of concern.
- Apply the following equation for calculation of approximate total accumulated pollution removal for BMPs in series:

Final Pollutant Removal = BMP1 removal rate + (remaining pollutant load * BMP2 removal rate) + ... for other Controls in series

# 4.2 Bioretention Areas

**Description**: Shallow stormwater basin or landscaped area that utilizes engineered soils or native, well-draining soil and vegetation to capture and treat runoff.

**LID/GI Consideration**: Low land requirement, adaptable to many situations, and often a small BMP used to treat runoff close to the source.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Maximum contributing drainage area of 5 acres
- Treatment area consists of ponding area, organic/mulch layer, planting media, and vegetation
- Requires landscaping plan
- Standing water has a maximum drain time of 24 hours
- Pretreatment recommended to prevent clogging of underdrains or native soil
- Ponding depth should be a maximum of 12 inches, preferably 9 inches

### ADVANTAGES / BENEFITS

- Applicable to small drainage areas
- Effective pollutant removals
- Appropriate for small areas with high impervious cover, particularly parking lots
- Natural integration into landscaping for urban landscape enhancement
- Good retrofit capability
- Can be planned as an aesthetic feature and meet local planting requirements

### DISADVANTAGES / LIMITATIONS

- Requires landscaping
- Not recommended for areas with steep slopes
- Medium to high capital cost
- Medium cost maintenance burden
- Soils may clog over time (may require cleaning or replacing)

### MAINTENANCE REQUIREMENTS

- Inspect and repair or replace treatment area components such as mulch, plants, and scour protection, as needed
- Ensure bioretention area is draining properly so it does not become a breeding ground for mosquitos
- Remove trash and debris
- Ensure mulch is 3-4 inches thick in the practice
- Requires plant maintenance plan

### POLLUTANT REMOVAL

- 85% **Total Suspended Solids**
- 95% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 80/60% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- 90% **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- ✓ Runoff Reduction
- ✓ Water Quality
- ★ Channel Protection
- ★ Overbank Flood Protection
- ★ Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- L Land Requirement
- M/H Capital Cost
- M Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *Yes*

**Soils:** *Engineered soil media is composed of sand, fines, and organic matter*

**Other Considerations:** Use of native plants is recommended

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 100% of the runoff reduction volume provided (no underdrain)
- 75% of the runoff reduction volume provided (upturned underdrain system)
- 50% of the runoff reduction volume provided (underdrain)

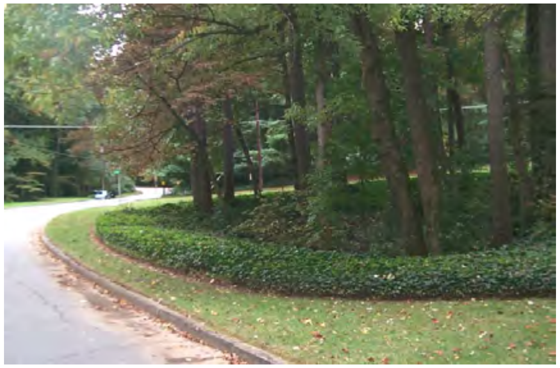
BACK TO TOC

## 4.2.1 General Description

Bioretention areas are structural stormwater controls that capture and infiltrate, or at least temporarily store the water quality volume ($WQ_v$) using soils and vegetation in shallow basins or landscaped areas.

Bioretention areas are engineered controls that convey runoff to the "treatment area," which consists of a ponding area, organic or mulch layer, planting soil, and vegetation. If the native soils are adequate, the captured stormwater runoff will infiltrate into the surrounding soils. If not, the filtered runoff is typically collected and returned to the conveyance system, through an underdrain system. Bioretention areas slightly differ from rain gardens in that they are an engineered structure that has a larger drainage area and may include an underdrain. For additional information of designing a Rain Garden in a residential lot, see the following website: https://www.atlantawatershed.org/greeninfrastructure/atlanta-residential-gi-nov-2012022013/

There are numerous design applications for bioretention areas including along highway and roadway drainage swales, within larger landscaped pervious areas, and as landscaped islands in impervious or high-density environments. A variety of bioretention areas are shown **Figure 4.2-1 through Figure 4.2-4.**



*Left*: **Figure 4.2-1** Landscaped Bioretention Area

*Middle Left*: **Figure 4.2-2** Landscaped Island

*Bottom*: **Figure 4.2-3** Bioretention Area near Parking Lot

*Middle Right*: **Figure 4.2-4** Bioretention Area after Storm

BACK TO TOC

### 4.2.2 Stormwater Management Suitability

Bioretention areas can be designed for water quantity and quality, i.e. the removal of stormwater pollutants, depending upon the native soils. Bioretention areas can provide runoff quantity control, particularly for smaller runoff volumes such as those generated by the water quality storm event (1.2 inches). These facilities may sometimes be used to partially or completely meet channel protection requirements on smaller sites. However, bioretention areas will typically need to be used in conjunction with another control to provide channel protection, as well as overbank flood protection. Bioretention areas need to be designed and maintained to safely bypass higher flows.

- Runoff Reduction
  Bioretention areas 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 with a higher infiltration rate of native soils. A bioretention area, with no underdrain, can be designed to provide 100% of the runoff reduction volume, if properly maintained. In order to provide runoff reduction for a bioretention area that is designed without an underdrain, a soils test or other reliable resource must indicate that the ponding area of the bioretention area will drain within 24 hours and entire bioretention area will drain within 72 hours. A bioretention area can be designed with an upturned underdrain to provide 75% of the runoff reduction volume, if properly maintained. An upturned underdrain is recommended in decent subsoils due to the clogging potential created during construction. Finally a bioretention area can be designed with an underdrain to provide 50% of the runoff reduction volume, if properly maintained.

- Water Quality
  A bioretention area is an excellent stormwater treatment practice due to its variety of pollutant removal mechanisms. Each of the components of the bioretention area is designed to perform a specific function. The grass filter strip (or grass channel) 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 planting soil in the bioretention area acts as a filtration system, and clay in the soil provides adsorption sites for hydrocarbons, heavy metals, nutrients and other pollutants. Both woody and herbaceous plants in the ponding area provide vegetative uptake of runoff and pollutants and also serve to stabilize surrounding soils.

- Channel Protection
  For smaller sites, a bioretention area may be designed to capture the entire channel protection volume ($CP_v$). Given that a bioretention area facility 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 bioretention area, another control must be used to provide $CP_v$ extended detention.

- Overbank Flood Protection
  Another control in conjunction with a bioretention area will likely be required to reduce the post-development peak flow of the 25-year storm ($Q_p$) to pre-development levels (detention).

- Extreme Flood Protection
  Bioretention areas must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the ponding area, mulch layer and vegetation.

*Credit for the volume of runoff reduced in the bioretention area may be taken in the overbank flood protection and extreme flood protection calculations. If the practice is designed to provide Runoff Reduction for Water Quality compliance, then the practice is given credit for Channel Protection and Flood Control requirements by allowing the designer to compute an Adjusted CN (see* *Subsection 3.1.7.5* *for more information).*

### 4.2.3 Pollutant Removal Capabilities

Bioretention areas are presumed to be able to remove 85% of the total suspended solids (TSS) load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Other pollutants bioretention areas can remove include Phosphorus, Nitrogen, metals (such as Cadmium, Copper, Lead, and Zinc), and Pathogens (such as Fecal Coliform). A bioretention area that is undersized, poorly designed, or maintained improperly can have reduced pollutant removal performance. Proper design of the bioretention area is critical to ensure that pollutants can be properly removed from stormwater that drains into the ground or storm sewer system. Proper designs improve the quality of our environment. One of the benefits of using an Upturned Underdrain (see **Figure 4.2-6**) is that this particular type of design will increase the removal of nitrogen in the soil.

For additional information and data on pollutant removal capabilities for bioretention areas, see the National Pollutant Removal Performance Database (3rd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.2.4 Application and Site Feasibility Criteria

Bioretention areas are suitable for many types of development, from single-family residential to high-density commercial projects. Because of its ability to be incorporated in landscaped areas, the use of bioretention areas is extremely flexible whether they are placed along roadways or in areas undergoing development or re-development. Bioretention areas are an ideal stormwater control for use as roadway median strips and parking lot islands and are also good candidates for the treatment of runoff from pervious areas, such as a golf course. Bioretention areas can also be used to retrofit existing development with stormwater quantity and quality treatment capacity. Curbs are not required for this type of practice.

Because of the many design constraints including limited ponding depths and inlet velocities, bioretention areas generally have a maximum drainage area of 5 acres or less. The size of the bioretention area is generally 3-6% of the contributing drainage area but varies significantly depending on each component of the bioretention area's ability to capture and infiltrate stormwater runoff and the percent imperviousness of the drainage area.

*The following criteria should be evaluated to ensure the suitability of a bioretention area for meeting stormwater management objectives on a site or development.*

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **YES**
- Regional Stormwater Control – **NO**

**Physical Feasibility -** Physical Constraints at Project Site

- Drainage Area – 5 acres or less. If the drainage area is greater than 5 areas, the drainage area can be divided up into multiple areas with each drainage area having a bioretention area.
- Space Required – Rough rule of thumb of 3-6% of the contributing drainage area
- Site Slope – Slopes should be a maximum of 20%, 5% preferred
- Minimum Depth to Water Table – A separation distance of 2 feet is recommended between the bottom of the bioretention area and the elevation of the seasonally high water table.
- Soils – Native soils if they have at least 0.5 inch/hr infiltration ability. Otherwise engineered media is needed including coarse sand, silt, clay, and other organic matter. The recommended standard media depth is 36 inches, with a minimum depth of 18 inches and a maximum depth of 48 inches. A qualified, licensed professional should test the soils to determine the best depth of soil for the practice.

**Other Constraints / Considerations**

- **Hot spots** – Do not use for hot spot runoff.
- **Damage to existing structures and facilities** – Consideration should be given to the impact of water exfiltrating the bioretention areas on nearby road bases.
- **Proximity** – The following is a list of specific setback requirements for the location of a bioretention area:
  - » 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** – Evaluate for stream warming when an underdrain system is used.

In addition, careful consideration should be given to the potential of perched or raised groundwater levels. Provide adequate distance from building foundations or use impermeable liner on side of excavated area nearest to structure.

**Challenges and Potential Solutions for Coastal Areas**

- **Poorly Drained Soils**—This condition minimizes the ability of bioretention areas to reduce stormwater runoff rates and volumes. One solution would be to include an underdrain system. An alternative would be to use a small stormwater wetlands or wet swales to intercept and treat stormwater runoff.
- **Flat Terrain**—May be difficult to provide adequate drainage so multiple smaller bioretention areas may be needed.
- **Shallow Water Table**—This can prevent the provision of 2 feet of clearance between the bottom of the bioretention area and the top of the water table which may cause stormwater runoff to pond in the bioretention area. Possible solutions are to minimize the depth of the planting media, or consider using stormwater ponds, wetlands, or wet swales to intercept and treat stormwater runoff.
- **Karst topography**—This condition usually warrants the use of an underdrain or impermeable liner to avoid infiltration into karst subsoils.

## 4.2.5 Planning and Design Criteria

Before designing the bioretention area, the following data is necessary:

- Existing and proposed site, topographic and location maps, and field reviews.
- Impervious and pervious areas. Other means may be used to determine the land use data.
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site.
- Design data from nearby storm sewer structures.
- Water surface elevation of nearby water systems as well as the depth to seasonally high groundwater.
- Infiltration testing of native soils at proposed elevation of bottom of bioretention area.

*The following criteria are to be considered minimum standards for the design of a bioretention area. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.2.5.1 LOCATION AND LAYOUT

Bioretention areas vary based on site constraints such as proposed and existing infrastructure, soils, existing vegetation, contributing drainage area, and utilities. Bioretention area systems are designed for intermittent flow and must be allowed to drain and reaerate between rainfall events. They should not be used on sites with a continuous flow from groundwater, sump pumps, or other sources. Bioretention area locations should be integrated into the site planning process and aesthetic and maintenance considerations should be taken into account in their siting and design. Elevations must be carefully worked out to ensure that the desired runoff flow enters the facility with no more than the maximum design depth and velocity.

BACK TO TOC

### 4.2.5.2 GENERAL DESIGN

- A bioretention area consists of the following:
  1. A pretreatment area, usually consisting of a grass filter strip between the contributing drainage area and the ponding area or a forebay to ease maintenance of the mulch, sand, or soil layers.
  2. Ponding area containing vegetation with an engineered planting media.
  3. Organic/mulch layer to protect planting media.
  4. Native soils to infiltrate the treated runoff, (see description of infiltration trenches, Section 4.12, for infiltration criteria).
  5. Where native soils have low infiltration rates include gravel and perforated pipe underdrain system to collect runoff that has filtered through the soil layers and pipe it to the storm sewer system. An upturned underdrain system can be used, however, the system should be 12-18" below the bottom of the planted area to reduce saturated conditions in root zone.
  6. Overflow, diversion or bypass structure to safely route larger storms through or around the bioretention area.
- A bioretention area design may include some of the following:
  - » Optional level spreader to spread and filter runoff.
  - » For curbed pavements use an inlet deflector to direct flow into the practice.
  - » A splash/erosion prevention pad at the inlet to the practice.

See **Figure 4.2-5** and **Figure 4.2-6** for an overview of the various components of a bioretention area.

### 4.2.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- Recommended minimum dimensions of a bioretention area are 3-6% of the total drainage area, though modeling is recommended to accurately size the area.
- The maximum recommended ponding depth of the bioretention areas is 12 inches.
- A grass filter strip or channel can be used for pretreatment. The length of the grass channel or width of the grass filter strip depends on the drainage area, land use, and channel slope. Design guidance on grass channels for pretreatment can be found in Section 4.9 (Grass Channel) and filter strips can be found in Section 4.29 (*Vegetated Filter Strip*). A pea gravel diaphragm flow spreader can also be used.
- The mulch layer should consist of 3 to 4 inches of triple-shredded hardwood mulch. This provides additional benefits such as removing sediment and metals and retaining soil moisture.
- If the native soils cannot suffice for the planting media used within the bioretention area planting beds, then an engineered soil mix should be provided that meets the following specifications:
  - » **Texture**: Sandy loam or loamy sand
  - » **Sand Content:** Soils should contain 35%-60% clean, washed sand
  - » **Topsoil Content**: Soils should contain 20%-30% topsoil
  - » **Organic Matter Content:** Soils should contain 10%-25% organic matter
  - » **Clay:** Soils should contain less than 15%
  - » **Infiltration Rate:** Soils should have an infiltration rate of at least 0.50 inches per hour (in/hr), although an infiltration rate of between 2 and 4 in/hr is preferred
  - » **Phosphorus Index (P-Index)**: Soils should have a P-Index of less than 30
  - » **Exchange Capacity (CEC)**: Soils should have a CEC that exceeds 10 milliequivalents (meq) per 100 grams of dry weight
  - » **pH**: Soils should have a pH of 6-8

For additional information on the soils for a Bio-retention Area, refer to Appendix D.

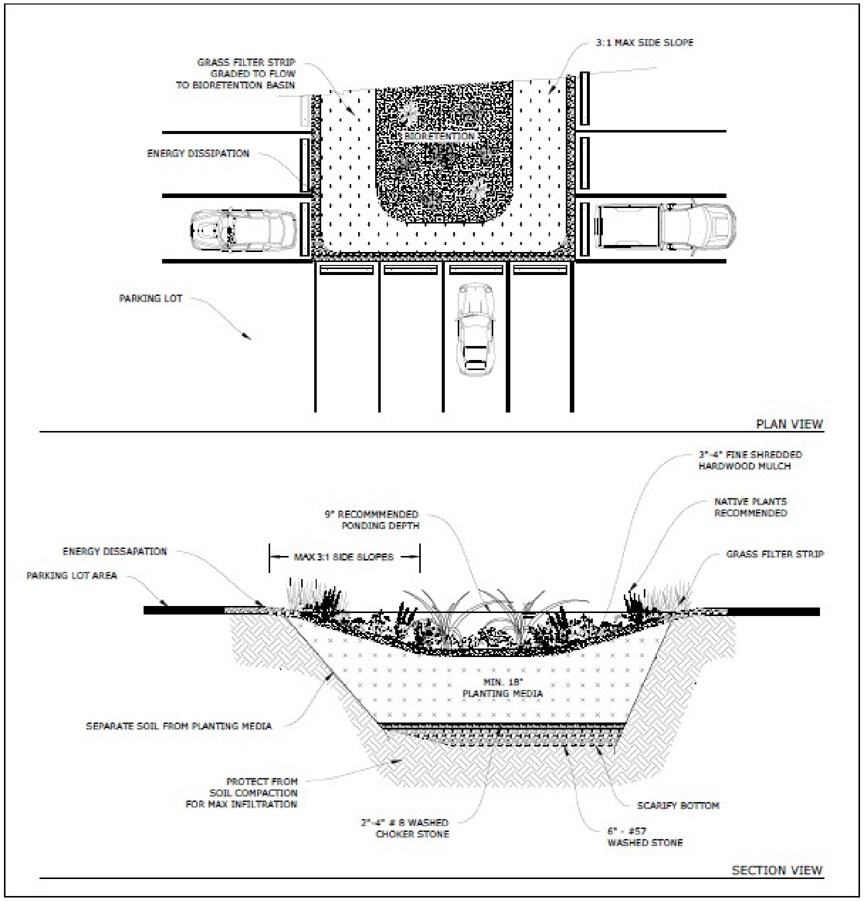



**Figure 4.2.-5** Schematic of Typical Bioretention Area Without an Underdrain
(Source: AECOM 2015)

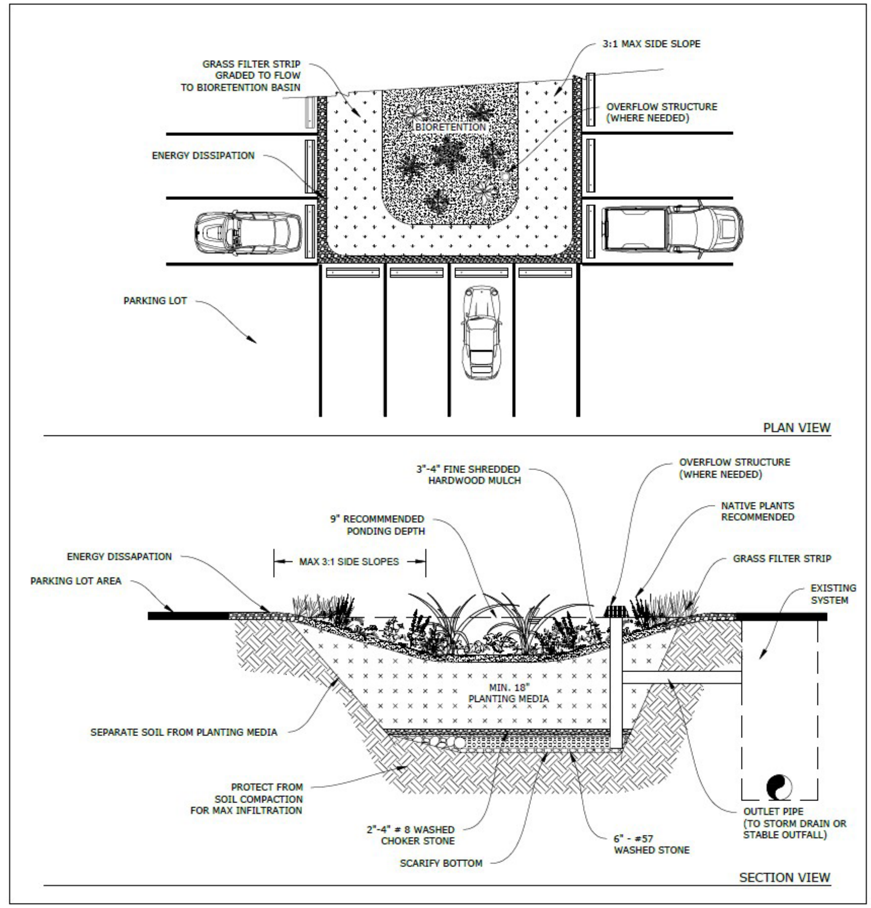



**Figure 4.2.-6** Schematic of a Typical Bioretention Area with an Upturned Underdrain
(Source: AECOM, 2015)

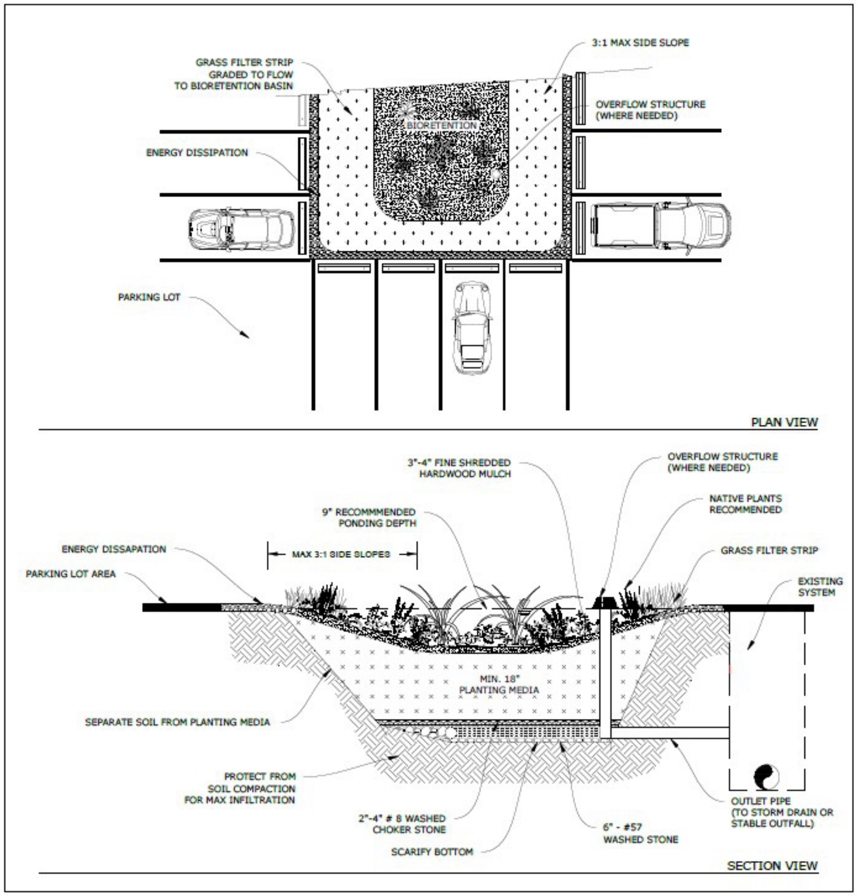



**Figure 4.2.-7** Schematic of a Typical Bioretention Area with Underdrain
(Source: AECOM, 2015)

It is recommended that the underdrain collection system, to monitor the internal water levels, should be equipped with a 4-6-inch perforated PVC pipe (AASHTO M 252) in an 8-inch gravel layer. The pipe should have 3/8-inch perforations, spaced at a minimum of 6-inch centers, with a minimum of 4 holes per row. The pipe is spaced at a maximum of 10 feet on center and a minimum grade of 0.5% must be maintained. Should the size of the pipe need to be bigger, a qualified licensed professional should model the area to ensure the size of the pipe is sufficient. It is recommended to separate the stone and the soil in the practice. This can be done by placing a permeable geotextile fabric between the gravel layer and the planting media. Alternatively, a stone choker layer 2-3 inches deep of #89 stone could be used to separate the practice from the soil. It is recommended that a qualified licensed professional be consulted to determine how this should be done based on the native soils and other physical properties around the practice.

### 4.2.5.4 PRETREATMENT/INLETS

- Adequate pretreatment is provided when a forebay, grass filter strip, or grass channel is provided.
- Inlet protection should be designed to reduce the velocity and energy of stormwater entering the practice and prevent scour of the mulch and plantings. Inlet protection may include splash blocks, a stone diaphragm, a level spreader or another similar device.

### 4.2.5.5 OUTLET STRUCTURES

- Outlet structures should be included in the design of a bioretention area configuration to ensure that larger storms can be bypassed without damaging the practice. Exceptions include small bioretention areas with flow bypass structures. Outlet configurations can include riser boxes and/or emergency spillway channels.

BACK TO TOC

#### 4.2.5.6 SAFETY FEATURES

- Bioretention areas generally do not require any special safety features, provided side slopes are maintained at 3:1 or flatter. Fencing of bioretention area facilities is not generally desired.

#### 4.2.5.7 LANDSCAPING

- Landscaping is critical to the performance and function of bioretention areas; the vegetation filters and transpires runoff and the root systems encourage infiltration.
- Vegetation should be selected to match the look and maintenance effort desired by locals and those responsible for maintaining the facility.
- The bioretention area should be vegetated to resemble a terrestrial forest ecosystem, with a mature tree canopy, subcanopy of understory trees, shrub layer, and herbaceous ground cover. Three species each of trees, shrubs, and grass/herbaceous species should be planted to avoid creating a monoculture. When determining what trees should be planted in the bioretention area, remember that tree leaves can clog the bioretention area. Consider using trees that only drop their leaves once in the fall.
- Woody vegetation should not be specified at inflow locations.
- Plants should be installed prior to mulch.
- Choose plants based on factors such as whether they are native or not, resistance to drought and inundation, cost, aesthetics, maintenance, etc. Planting recommendations for bioretention area facilities are as follows:
  - » Native plant species should be preferred over non-native species.
  - » Vegetation should be selected based on a specified zone of hydric tolerance.
  - » A selection of trees with an understory of shrubs and herbaceous materials should be provided.
- Additional information and guidance on the appropriate woody and herbaceous species appropriate for bioretention areas in Georgia, and their planting and establishment, can be found in Appendix D.

#### 4.2.5.8 CONSTRUCTION CONSIDERATIONS

- Construction equipment should be restricted from the bioretention area to prevent compaction of the native soils.
- A dense and vigorous vegetative cover should be established over the contributing pervious drainage areas before runoff can be accepted into the facility. Otherwise the sediment from the stormwater runoff will clog the pores in the planting media and native soils.

#### 4.2.5.9 CONSTRUCTION AND MAINTENANCE COSTS

- A budget level construction cost estimate is between \$3.00 and \$10.00 per square foot.
- A budget level maintenance cost estimate is between \$1.50 and \$3.50 per square foot annually.

## 4.2.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of a bioretention area.**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for an bioretention area. Create a rough layout of the bioretention area dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the bioretention area.**

Consider whether the bioretention area is intended to:

» Meet a runoff reduction* target or water quality (treatment) target. For information on the sizing of a BMP utilizing the runoff reduction approach, see Step 3A. For information on the sizing of the BMP utilizing the water quality treatment approach, see Step 4A. **Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*

» Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)

» Provide a possible solution to a drainage problem

» Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3A, 3B, and 3C for a runoff reduction approach, or skip Step 3 and complete Steps 4A and 4B for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) **Calculate the Stormwater Runoff Reduction Target Volume**

Calculate the Runoff Reduction Volume using the following formula:

$$RR_V = (P)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$RR_V$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the practice:

Using the $RR_V$ calculated above, determine the minimum Volume of the Practice (VP)

$$(VP_{MIN}) \geq RR_v \text{ (target)} / (RR\%)$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice
**$VP_{MIN}$** = Minimum storage volume required to provide Runoff Reduction Target Volume ($ft^3$)
**$RR_v$ (target)** = Runoff Reduction Target Volume ($ft^3$)

(Step 3B) Determine the storage volume of the practice and the Pretreatment Volume

To determine the actual volume provided in the bioretention area, use the following equation:

$$VP = (PV + VES\ (N))$$

*Where*:

**VP** = Volume provided (temporary storage)
**PV** = Ponding Volume
**VES** = Volume of Engineered Soils
**N** = Porosity

To determine the porosity, a qualified licensed professional should be consulted to determine the proper porosity based on the engineered soils used. Most soil media has a porosity of 0.25 and gravel a value of 0.40.

Provide pretreatment by using a grass filter strip or pea gravel diaphragm, 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. Pre-treatment may also be desired to reduce flow velocities or 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. Refer to Section 4.9 for design criteria for a grass channel and Section 4.29 for vegetated filter strips.

(Step 3C) Determine whether the minimum storage volume was met

When the $VP \geq VP_{MIN}$, then the Runoff Reduction requirements are met for this practice. Proceed to Step 5.

When the $VP < VP_{MIN}$, then the BMP must be sized according to the $WQ_V$ treatment method (See Step 4).

(Step 4A) Calculate the Target Water Quality Volume

Calculate the Water Quality Volume using the following formula:

$$WQ_V = (1.2)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$WQ_V$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)
**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4B) **If using the practice for Water Quality treatment, determine the footprint of the bioretention area practice and the pretreatment volume required**

The peak rate of discharge for the water quality design storm is needed for sizing of off-line diversion structures (see Subsection 3.1.7). If designing off-line, follow steps (a) through (d) below:

(a) Using $WQ_v$, compute CN
(b) Compute time of concentration using TR-55 method
(c) Determine appropriate unit peak discharge from time of concentration
(d) Compute $Q_{wq}$ from unit peak discharge, drainage area, and $WQ_v$.

To determine the minimum surface area of the bioretention area, use the following formula:

$$A_f = (WQ_v)\ (d_f)\ /\ [\ (k)\ (h_f + d_f)\ (t_f)]$$

*Where*:

**$A_f$** = surface area of ponding area ($ft^2$)
**$WQ_v$** = water quality volume ($ft^3$)
**$d_f$** = media depth (ft)
**k** = coefficient of permeability of planting media (ft/day) (use 1 ft/day for silt-loam if engineered soils is being used)
**$h_f$** = average height of water above bioretention area bed (ft)
**$t_f$** = design planting media drain time (days) (1 day is recommended maximum)

(Step 5) **Calculate the adjusted curve numbers for $CP_V$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm). See Subsection 3.1.7.5 or Appendix B-2 for a detailed bioretention area design example**

(Step 6) **Size flow diversion structure, if needed**

A flow regulator (or flow splitter diversion structure) should be supplied to divert the $WQ_v$ (or $RR_v$) to the bioretention area.

Size low flow orifice, weir, or other device to pass $Q_{wq}$.

(Step 7) **Size underdrain system**

See Subsection 4.2.5.3 (*Physical Specifications/Geometry*)

(Step 8) Design 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.

(Step 9) Prepare Vegetation and Landscaping Plan

A landscaping plan for the bioretention area should be prepared to indicate how it will be established with vegetation. See Subsection 4.2.5.7 (*Landscaping*) and Appendix D for more details.

See Appendix B-2 for a Bioretention Area Design Example

### 4.2.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.3 Bioslope

**Description:** Specialized media filtration BMP typically used in longitudinal applications to treat stormwater along an impervious area (road, park-ing lot, etc.).

**LID/GI Consideration:** Adaptable to many linear situations, and often a small BMP used to treat runoff close to the source.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Longitudinal slopes must be less than 5%
- Minimum 2 foot width
- Side slopes 3:1 or flatter; 4:1 recommended
- Length is usually the length of adjacent paved area being treated
- Sized to capture the water quality peak flow rate of discharge
- Pretreatment most commonly provided through a filter strip

### ADVANTAGES / BENEFITS

- Requires minimal land
- Reduces runoff volume and velocity

### DISADVANTAGES / LIMITATIONS

- Limited to sheet flow uses only
- Not suitable for embankment slopes steeper than 3:1
- Does not meet quantity control stormwater requirements

### MAINTENANCE REQUIREMENTS

- Avoid damaging or rutting the permeable soil layer when mowing grass
- Remove sediment and debris from filter strip and adjacent areas

### POLLUTANT REMOVAL

85% **Total Suspended Solids**

75% **Metals** - Cadmium, Copper, Lead, and Zinc removal

60/25% **Nutrients** - Total Phosphorus / Total Nitrogen removal

60% **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

✓ Runoff Reduction

✓ Water Quality

★ Channel Protection

● Overbank Flood Protection

● Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

L Land Requirement

M Capital Cost

M Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *No*
**Drainage Area:** *Contributing* upstream flow must be less than 150 feet

**Soils:** *No restrictions*

**Other Considerations:** Permeable soil layer

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- HSG A & B: 50% of the runoff reduction volume provided
- HSG C & D: 25% of the runoff reduction volume provided

### 4.3.1 General Description

Bioslopes (also referred to as ecology embankments) are water quality best management practices that use a permeable engineered soil media to capture and treat stormwater runoff from adjacent paved areas (see **Figure 4.3-1**). Bioslopes are typically installed along embankments or other slopes and designed to treat sheet flow stormwater runoff.

Bioslopes are designed with limited longitudinal slopes to force the flow to be slow and uniform, thus allowing for particulates to settle and limiting the effects of erosion. Once infiltrated into the highly permeable engineered soil layer, an underdrain is typically used to remove the treated stormwater from the embankment or slope. Larger flow rates, from less frequent storm events, in the form of sheet flow bypass the engineered soil media by overtopping and continuing down the embankment or slope.

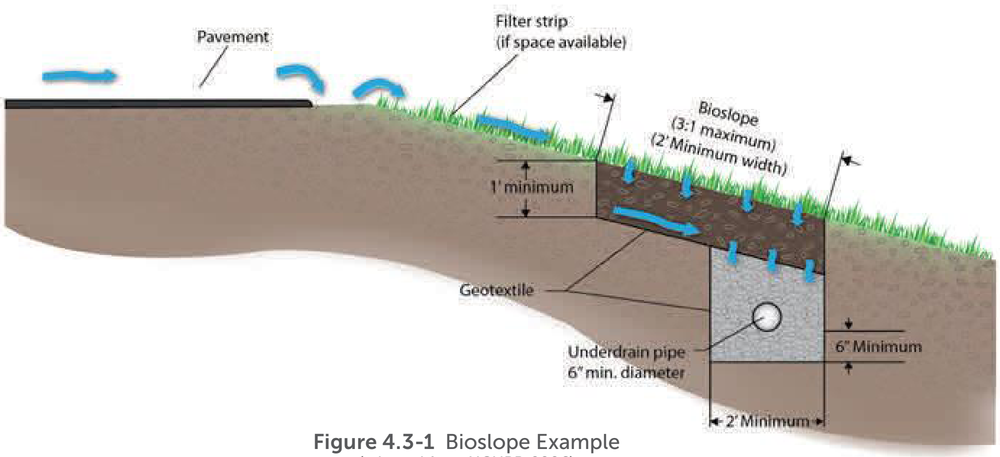



**Figure 4.3-1** Bioslope Example
(adapted from NCHRP, 2006)

### 4.3.2 Stormwater Management Suitability

Bioslopes are designed primarily for stormwater quality and have only a limited ability to provide a small amount of channel protection volume.

- Runoff Reduction
  Bioslopes are an effective low impact development (LID) practice 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 the higher the infiltration rate of the native soils. Bioslopes can be designed to provide 50% of the runoff reduction volume for type A and B hydrologic soils or 25% of the runoff reduction volume for type C and D hydrologic soils.

- 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. Subsection 4.3.3 provides median pollutant removal efficiencies that can be used for planning and design purposes.

- 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).

### 4.3.3 Pollutant Removal Capabilities

Bioslopes are presumed to be able to remove 85% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed bioslopes can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- *Total Suspended Solids* – 85%
- *Total Phosphorus* –60%
- *Total Nitrogen* –25%
- *Fecal Coliform* – 60%
- *Heavy Metals* – 75%

For additional information and data on pollutant removal capabilities for bioslopes, see research and testing results from the Washington State Department of Transportation and published removal rates from a synthesis performed by the National Cooperative Highway Research Program.

### 4.3.4 Application and Feasibility Criteria

Bioslopes can be used in a variety of development types; however, they are primarily applicable to linear roadway applications where a rural (no curb and gutter) cross section is utilized. The impervious cover in the contributing drainage area is relatively small, and consists of paved areas adjacent to the embankment or slope where the practice has been installed.

Since bioslopes require a relatively small amount of land, they are more commonly selected when right-of-way availability is limited. Other practices that would be required at the bottom of the embankment or slope, such as enhanced swales or grass channels, most always require a larger amount of land. Bioslopes may not be desirable in some more aesthetically landscaped or sodded areas, due to the need for keeping the media layer free and clear of plantings and other obstructions. Only when the same infiltration rate of the media could be preserved, can grass be planted directly on top of the engineered media. This may include frequently maintained "half-cut" sod or similar vegetation.

The topography and location of a site will determine the applicability of the use of bioslopes. Overall, the topography should allow for the design of a bioslope with sufficient slope and cross-sectional area to maintain non-erosive velocities.

*The following criteria should be evaluated to ensure the suitability of a bioslope for meeting stormwater management objectives on a site or development.*

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **NO**
- Regional Stormwater Control – **NO**

**Physical Feasibility -** Physical Constraints at Project Site

- **Drainage Area** – Contributing upstream flow must be less than 150 feet
- **Space Required** – N/A
- **Site Slope** – Typically no more than 5%
- **Minimum Head** – N/A
- **Minimum Depth to Water Table** – 2 feet required between the bottom of the media layer and the elevation of the seasonally high water table
- **Soils** – Engineered media

**Other Constraints / Considerations**

- **Aquifer Protection** – Exfiltration should not be allowed in hotspot areas

### 4.3.5 Planning and Design Criteria

*The following criteria are to be considered minimum standards for the design of a bioslope. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

#### 4.3.5.1 LOCATION AND LAYOUT

- A bioslope should be sited such that the topography allows for the design with sufficiently mild slope and cross-sectional area to maintain non-erosive velocities.
- Bioslopes should have a maximum contributing upstream flow length of 150 feet, or less.

#### 4.3.5.2 GENERAL DESIGN

- Bioslopes are designed to treat the $WQ_v$ through a flow rate-based design, and to safely pass larger storm flows by checking velocities. Flow enters the bioslope via sheet flow through a pretreatment filter strip area, or a minimum 2 foot wide grass strip.
- A bioslope consists of a permeable soil mixture that overlays an underdrain system. Flow enters the media layer where it is filtered through the soil bed. Runoff is collected and conveyed by a perforated pipe and gravel underdrain system to the outlet. **Figure 4.3-2** provides a schematic for typical components of a bioslope.

The following equation and variables are used for the sizing of a bioslope:

$$W = \frac{C\, Q_{wq}\, SF}{KL}$$

*Where*:

**W** = bioslope width (parallel with flow path) (feet)

**C** = conversion factor = 43,200 [(in/hr)/(ft/s)]

**$Q_{wq}$** = water quality volume peak flow ($ft^3/s$, refer to Subsection 3.1.7.2)

**SF** = safety factor equal to 1 (unitless, typical throughout Georgia)

**k** = infiltration rate, use long-term infiltration rate of 10 (inches/hour)

**L** = bioslope length (perpendicular with flow path) (feet)

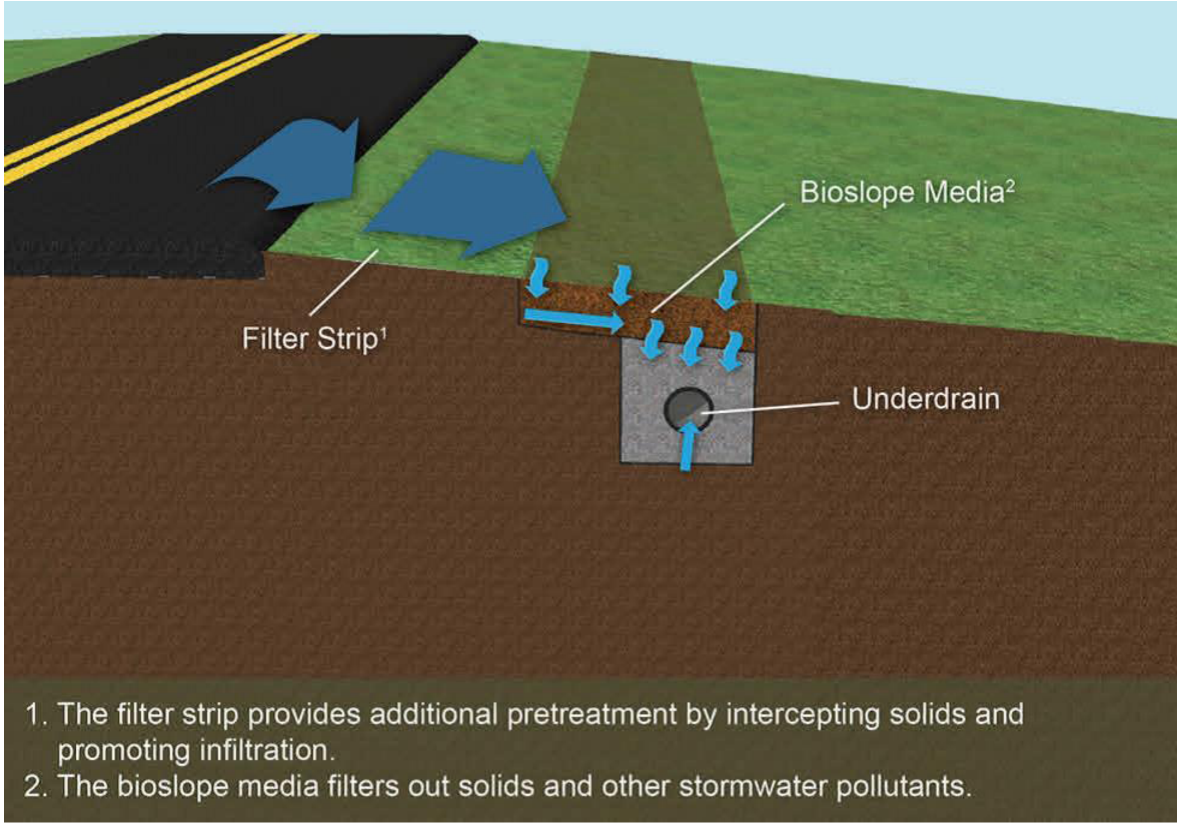



**Figure 4.3-2** Schematic of Bioslope
(Source: GDOT Drainage Manual, 2014)

#### 4.3.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- Embankment slopes should be 3:1 or flatter. When slopes steeper than 4:1 are used, additional measures should be taken to ensure stabilization of vegetation along the slope.

- Longitudinal slopes (parallel with the embankment) should be no more than 5%.

- The area between the impervious surface, or paved area, should preferably be no more than 30 feet to avoid reconcentration of stormwater runoff creating erosion and scour through the engineered media.

- The bioslope consists of a permeable soil layer of at least 12 inches in depth, above an underdrain. The soil media should contain a mixture of crushed rock, dolomite, gypsum, and perlite as shown in **Table 4.3-1**. This mixture has an initial infiltration rate of 50 inches per hour and an infiltration rate of 28 inches per hour long-term. For sizing, an infiltration rate of 10 inches per hour is used in calculations as a factor of safety. A permeable filter fabric is placed between the media and underlying soil. Where an underdrain collection system is utilized, it should be equipped with at least a 6-inch diameter perforated PVC pipe longitudinal underdrain in a gravel layer.

Table 4.3-1 Bioslope Media Mixture

(Source: adapted from WSDOT, 2011)

| Engineered Media Amendment | Quantity |
|---|---|
| Aggregate:<br>» #89 stone<br>» No recycled material<br>» Non-limestone material mineral aggregate | 3 cubic yards (CY)<br>(Note: 3 CY is used as a baseline for other mixture components; adjust total quantity based on bioslope dimensions) |
| Perlite:<br>» Horticultural grade, free of any toxic materials<br>» 0-30% passing US No. 18 Sieve<br>» 0-10% passing US No. 30 Sieve | 1 CY per 3 CY of mineral aggregate |
| Dolomite: $CaMg(CO_3)_2$ (calcium magnesium carbonate)<br>» Agricultural grade, free of any toxic materials<br>» 100% passing US No. 8 Sieve<br>» 0% passing US No. 16 Sieve | 10 pounds per CY of perlite |
| Gypsum: Non-calcined, agricultural gypsum $CaSO_4 \bullet 2H_2O$ (hydrated calcium sulfate)<br>» Agricultural grade, free of any toxic materials<br>» 100% passing US No. 8 Sieve<br>» 0% passing US No. 16 Sieve | 1.5 pounds per CY of perlite |

#### 4.3.5.4 PRETREATMENT/INLETS

- Where space allows, vegetated filter strips should be used as the pretreatment device for bioslopes. See Section 4.29 for more information on vegetated filter strips.
- Where space does not allow for a full width vegetated filter strip, a grassed area can still be used as pretreatment to the best extent possible.
- A pea gravel diaphragm can also be used for pretreatment where space constraints do not allow for enough of a grassed area between the bioslope and the paved surface.

#### 4.3.5.5 OUTLET STRUCTURES

- The underdrain system should discharge to a storm drainage structure or a stable outfall.

#### 4.3.5.6 EMERGENCY SPILLWAY

- Bioslopes must be adequately designed to safely pass flows that exceed the design storm flows.

#### 4.3.5.7 MAINTENANCE ACCESS

- Adequate access should be provided for all bioslopes for inspection and maintenance.

#### 4.3.5.8 LANDSCAPING

- Surrounding vegetation is typically a grassed or vegetated filter strip,  Vegetation should be well established and thriving to prevent erosion and sedimentation.

#### 4.3.5.9 ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

**Physiographic Factors -** Local terrain design constraints

- **Low Relief** – Ideal for embankment sheet flow conditions
- **High Relief** – Often infeasible if longitudinal slopes are greater than 5%
- **Karst** – No exfiltration of hotspot runoff from bioslopes; use impermeable liner

**Soils**

No additional criteria

**Special Downstream Watershed Considerations**

- **Aquifer Protection** – No exfiltration of hotspot runoff from bioslopes; use impermeable liner

#### 4.3.5.10 CONSTRUCTION CONSIDERATIONS

- Construction equipment should be restricted from the bioslope area to prevent compaction of the native soils.
- A dense and vigorous vegetative cover should be established over the contributing pervious drainage areas before runoff can be accepted into the bioslope. Otherwise the sediment from the stormwater runoff will clog the pores in the planting media and native soils.

## 4.3.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of an bioslope**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a bioslope. Create a rough layout of the bioslope dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the bioslope.**

Consider whether the bioslope is intended to:

» Meet a runoff reduction* target or water quality (treatment) target. For information on the sizing of a BMP utilizing the runoff reduction approach, see Step 3A. For information on the sizing of the BMP utilizing the water quality treatment approach, see Step 4A. **Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*

» Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)

» Provide a possible solution to a drainage problem

» Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3A, 3B, and 3C for a runoff reduction approach, or skip Step 3 and complete Step 4A and 4B for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) **Calculate the Stormwater Runoff Reduction Target Volume**

Calculate the Runoff Reduction Volume using the following formula:

$$RR_V = (P)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$RR_V$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the practice:

Using the $RR_V$ calculated above, determine the minimum Volume of the Practice (VP)

$$(VP_{MIN}) \geq RR_v \text{ (target)} / (RR\%)$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice
**$VP_{MIN}$** = Minimum storage volume required to provide Runoff Reduction Target Volume ($ft^3$)
**$RR_v$ (target)** = Runoff Reduction Target Volume ($ft^3$)

(Step 3B) Determine the storage volume of the practice and the Pre-treatment Volume

To determine the actual volume provided in the bioslope, use the following equation:

$$VP = (PV + VES\ (N))$$

*Where*:

**VP** = Volume provided (temporary storage)
**PV** = Ponding Volume
**VES** = Volume of Engineered Soils
**N** = Porosity

To determine the porosity, a qualified licensed professional should be consulted to determine the proper porosity based on the engineered soils used. Most soil media has a porosity of 0.25 and gravel a value of 0.40.

Provide pretreatment by using a grass filter strip or pea gravel diaphragm, 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. Pre-treatment may also be desired to reduce flow velocities or 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. Refer to Section 4.9 for design criteria for a grass channel and Section 4.29 for vegetated filter strips.

(Step 3C) Determine whether the minimum storage volume was met

When the $VP \geq VP_{MIN}$, then the Runoff Reduction requirements are met for this practice. Proceed to Step 5.

When the $VP < VP_{MIN}$, then the BMP must be sized according to the $WQ_v$ treatment method (See Step 4A).

(Step 4A) Calculate the Target Water Quality Volume

Calculate the Water Quality Volume using the following formula:

$$WQ_V = (1.2)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$WQ_V$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)
**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4B) If using the practice for Water Quality treatment, determine the length and width of the bioslope practice and the Pretreatment Volume required

Determine Bioslope Dimensions:

Refer to Subsection 3.1.7.2 and 3.1.7.5 to calculate the water quality volume peak flow rate using a reduced CN for any $RR_V$ provided, then find either length or width using the equation in Subsection 4.3.5. In most applications, site restrictions limit the available length.

» Longitudinal slope cannot exceed 5% (2 to 4% recommended)

» Width should be a minimum of 2 feet

» Ensure that side slopes are no steeper than 3:1 (4:1 recommended)

See Subsection 4.3.5.3 (*Physical Specifications / Geometry*) for more details

(Step 5) Calculate the adjusted curve numbers for $CP_V$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm). See Subsection 3.1.7.5 for more information

(Step 6) Size underdrain system

See Subsection 4.2.5.3 (*Physical Specifications/Geometry*)

### 4.3.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

*Regular inspection and maintenance is critical to the effective operation of a bioslope as designed. Maintenance responsibility for a bioslope should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.*

# 4.4 Downspout Disconnects

**Figure 4.4.1** Downspout Disconnect
Source: (Center for Watershed Protection)

**Description**: Where site characteristics permit, downspout disconnects can be used to spread rooftop runoff from individual downspouts across lawns and other pervious areas, where it is slowed, filtered and allowed to infiltrate into the native soils.

**LID/GI Considerations**: If properly designed, downspout disconnects can provide measurable reductions in post-construction stormwater run-off rates, volumes and pollutant loads on development sites.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Maximum length of flow path in contributing drainage areas is 75 feet
- Minimum length of flow path in pervious areas below downspout disconnects is 15 feet and equal to or greater than the length of the flow path in the contributing drainage area
- Maximum impervious rooftop drainage area to one disconnected downspout is 2,500 square feet
- Maximum slope of pervious area beneath the downspout is 6 percent
- Runoff must be conveyed as sheet flow from the downspout and across open areas to maintain proper disconnect
- Downspout disconnects should be designed to convey stormwater runoff away from buildings to prevent damage to building foundations

### ADVANTAGES / BENEFITS

- Helps restore pre-development hydrology on development sites
- Reduces post-construction stormwater runoff rates, volumes and pollutant loads
- Relatively low construction cost and long-term maintenance burden
- Encourages groundwater recharge

### DISADVANTAGES / LIMITATIONS

- Provides greater stormwater management benefits on sites with permeable soils (i.e., hydrologic soil group A and B soils)
- Level spreaders must be needed at the downspout to dissipate flow
- Clay soils or soils that have been compacted by construction equipment greatly reduce the effectiveness of this practice, and soil amendments may be needed

### ROUTINE MAINTENANCE REQUIREMENTS

- Maintenance of areas receiving disconnected runoff is generally the same as that required for other lawn or landscaped areas.
- Areas receiving runoff should be protected from future compaction (e.g., by planting trees or shrubs along the perimeter).
- Gutters and downspouts should be kept clear of dirt, debris, vegetation, and other buildup.
- Downspout disconnects are often used in conjunction with other BMPs. Ensure that upstream and/or downstream BMPs are maintained in accordance with this manual.

### POLLUTANT REMOVAL

- 60% **Total Suspended Solids**
- 40% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 25/25% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- N/A **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- ✓ Runoff Reduction
- ✓ Water Quality
- Channel Protection
- Overbank Flood Protection
- Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- L Land Requirement
- L Capital Cost
- L Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Not recommended*
**Roadway Projects:** *Not applicable*

**Soils:** *Disconnects should be directed over HSG A, B, or C (e.g., sands, sandy loams, loams).*

**Other Considerations:** Erosion and sediment control practices should not be located in vegetated areas receiving disconnected runoff. Construction vehicles and equipment should avoid areas receiving disconnected runoff to minimize disturbance and compaction.

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 50% of the $RR_v$ conveyed to the practice (A & B hydrologic soils)
- 25% of the $RR_v$ conveyed to the practice (C & D hydrologic soils)

BACK TO TOC

### 4.4.1 General Description

As the name implies, a downspout disconnect is the most basic of all low impact development practices that can be used to "receive" rooftop runoff. Where site characteristics permit, they can be used to spread rooftop runoff from individual downspouts across lawns and other pervious areas, where it is slowed, filtered, and allowed to infiltrate into the soil. If properly designed, downspout disconnects can provide measurable reductions in post-construction stormwater runoff rates, volumes, and pollutant loads on development sites.

In order to use downspout disconnects to receive post-construction stormwater runoff, downspouts must be designed to discharge to a lawn or other pervious area (**Figure 4.4.2**). The pervious area located below the downspout disconnect should slope away from buildings and other impervious surfaces to prevent damage to building foundations and discourage rooftop runoff from entering the storm drain system.

The primary concern associated with a downspout disconnect (**Figure 4.4.3**) is the length of the flow path over the lawn or other pervious area below the disconnection point. To provide adequate residence time for stormwater runoff, the length of the flow path in the pervious area below a downspout disconnect should be equal to or greater than the length of the flow path of the contributing drainage area. If this cannot be accomplished, due to site characteristics or constraints, site planning and design teams should consider using other low impact development practices. A typical schematic for a downspout disconnect is shown in **Figure 4.4-3**.

**Figure 4.4.2** Downspout Disconnects to Pervious Areas
(Source: Center for Watershed Protection)

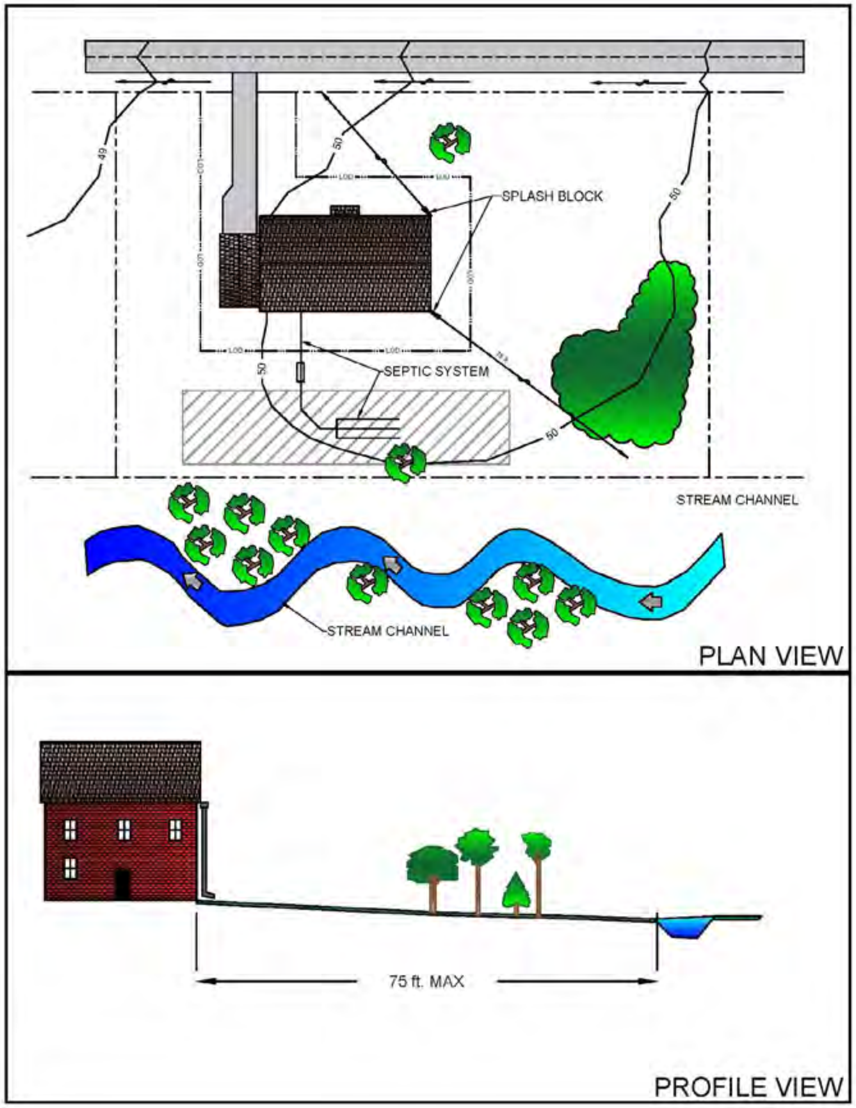



**Figure 4.4-3** Schematic of Downspout Disconnect

### 4.4.2 Stormwater Management Suitability

- Runoff Reduction
  Runoff reduction credit can be applied to the stormwater runoff generated from the rooftop drainage area that drains to a properly designed, installed, and maintained downspout disconnect. As shown in **Table 4.1.3-2**, the runoff reduction volume ($RR_v$) conveyed through a downspout disconnect located on A/B or amended soils is reduced by 50%. Reduce the $RR_v$ conveyed through a downspout disconnect located on C/D soils by 25%.

- Water Quality
  If installed as per the recommended design criteria and properly maintained, 80% total suspended solids removal will be applied to the water quality volume ($WQ_v$) flowing to the disconnected downspout.

- Channel Protection
  No channel protection volume ($CP_v$) storage is provided by a disconnected downspout. Stormwater runoff generated by the impervious rooftop area and pervious receiving area should be routed to a downstream regional BMP that provides storage and treatment of the $CP_v$. Proportionally adjust the post-development runoff curve number (CN) to account for the runoff reduction provided by a downspout disconnect for the contributing drainage area when calculating the channel protection volume for the regional BMP.

- Overbank Flood Protection
  Proportionally adjust the post-development runoff CN to account for the runoff reduction provided by a downspout disconnect for the contributing drainage area when calculating the overbank peak discharge ($Q_{p25}$) on a development site.

- Extreme Flood Protection
  No Extreme Flood Protection volume is provided by a downspout disconnect. Proportionally adjust the post-development runoff curve number (CN) to account for the runoff reduction provided by a downspout disconnect for the contributing drainage area when calculating the extreme peak discharge ($Q_f$) on a development site.



BACK TO TOC

### 4.4.3 Pollutant Removal Capabilities

Downspout disconnects are presumed to remove 80% of the total suspended solids (TSS) in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Other pollutants that can be removed by downspout disconnects include Phosphorus, Nitrogen, and metals (such as Cadmium, Copper, Lead, and Zinc).

In order to provide the most efficient pollutant removal, downspout disconnects should be sized with the appropriate length, width, and slope. The maximum drainage area should not be exceeded, and rooftop runoff should enter the disconnected area as sheet flow to ensure proper pollutant removal.

For additional information and data on pollutant removal capabilities for downspout disconnects, see the National Pollutant Removal Performance Database (Version 3) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.4.4 Application and Site Feasibility Criteria

Downspout disconnects are ideal for residential, commercial, industrial, and municipal development projects where buildings are surrounded by fairly level lawn and grass areas. Planners, designers, and developers should ensure that the pervious area beneath the downspout is at least the same length as the flow path of the contributing runoff. Additionally, disconnected stormwater runoff should not be allowed to "reconnect", or flow across impervious areas, before reaching a downstream regional BMP or being discharged off-site.

**General Feasibility**

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – NO
- Regional Stormwater Control – NO

**Physical Feasibility -** Physical Constraints at Project Site

- **Drainage Area** – The contributing impervious rooftop drainage area should be 2,500 square feet or less per downspout. If the total rooftop area is greater than 2,500 square feet, the rooftop can be divided up into multiple drainage areas with each drainage area flowing to a separate, properly sized, downspout disconnect.
- **Space Required** – The length of the disconnect should be a minimum of 15 feet and equal to or greater than the contributing flow path length. The length of the flow path within the contributing drainage area should be 75 feet or less.
- **Site Slope** – Slope of the pervious area below the downspout should be a maximum of 6% and a minimum of 0.5% (1% to 5% is recommended).
- **Minimum Depth to Water Table** – Separation from the water table is not required for a downspout disconnect.
- **Soils** – Disconnects should be directed over HSG A, B, or C (e.g., sands, sandy loams, loams). Clayey soils or soils that have been compacted by construction equipment greatly reduce the effectiveness of this practice, such that soil amendments may be needed.

**Other Constraints / Considerations**

- **Hot spots** – May be used for hot spot runoff
- **Damage to existing structures and facilities** – Downspout disconnects should be designed to convey stormwater runoff away from buildings to prevent damage to building foundations
- **Proximity** – Downspout disconnects may be used without restriction near:
  - » Private water supply wells
  - » Open water
  - » Public water supply reservoirs
  - » Public water supply wells
- **Property Lines** – Downspout disconnects may be used near property lines; however, ensure that stormwater runoff is not redirected to cause additional flooding to another homeowner.
- **Trout Stream** – Downspout disconnects help treat stormwater for pollutants and reduce the volume and velocity of runoff. Therefore, downspout disconnects are an effective BMP for use where trout streams or other protected waters may receive stormwater runoff.

**Coastal Areas**

- **Poorly Drained Soils** – Downspout disconnects may be used with poorly drained soils; however, less runoff reduction will be provided in these cases.
- **Flat Terrain** – Downspout disconnects require minimal slope to treat and convey stormwater runoff; therefore, they may be used in areas of flatter terrain.
- **Shallow Water Table** – Downspout disconnects may be used with a shallow water table.

## 4.4.5 Planning and Design Criteria

Before designing the downspout disconnect, the following data is necessary:

- Existing and proposed site, topographic, and location maps, as well as field reviews
- The proposed site design including, buildings, parking lots, sidewalks, stairs, handicapped ramps, and landscaped areas
- Architectural roof plan for rooftop pitches and downspout locations
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site
- Information about downstream BMPs and receiving waters
- Design data from nearby storm sewer structures

*The following criteria are to be considered minimum standards for the design of a downspout disconnect. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.4.5.1 LOCATION AND LAYOUT

- Downspout disconnects should be located where there is at least 15 linear feet of managed turf, grass, lawn, or landscaped area at the receiving end of the downspout.
- Disconnects should not be upstream of an impervious area, which will allow rooftop runoff to "reconnect". Locate disconnects upstream of vegetated or forested areas, regional BMPs, and/or stormwater drainage systems.
- The slope of the pervious receiving area should be between 0.5% and 6%, with 1% to 5% slopes being ideal.
- Ensure that the length of each disconnected area is equal to or greater than the contributing flow path of the rooftop runoff.
- The length of the contributing flow path should no greater than 75 feet.
- Rooftop runoff should enter the disconnected pervious area as sheetflow. Level spreaders should be used at the discharge end of the downspout to dissipate energy and spread out the flow.

### 4.4.5.2 GENERAL DESIGN

A level (0.5 % to 6% slope) managed turf, grass, lawn, or landscaped area at the receiving end of the downspout, between 15 and 75 feet long. Downspout disconnects can be constructed on HSG A, B, C or D soils.

### 4.4.5.3 LANDSCAPING

- Vegetation commonly planted in the pervious areas located below downspout disconnects includes turf, shrubs, trees, and other herbaceous vegetation. Although managed turf is most commonly used, site planning and design teams are encouraged to use trees, shrubs and/or other native vegetation to help establish mature native plant communities in the pervious areas located below downspout disconnects.
- Methods used to establish vegetative cover within the pervious area below a downspout disconnect should achieve at least 75 percent vegetative cover within one year of installation.
- To help prevent soil erosion and sediment loss, landscaping should be provided immediately after the downspout disconnect has been completed. Temporary irrigation may be needed to quickly establish vegetative cover within the pervious areas below downspout disconnects.

### 4.4.5.4 CONSTRUCTION CONSIDERATIONS

To help ensure that downspout disconnects are properly installed on a development site, planning and design teams should consider the following recommendations:

- Simple erosion and sediment control measures, such as temporary seeding and erosion control mats, should be used within the pervious areas located below downspout disconnects.
- To help prevent soil compaction, heavy vehicular and foot traffic should be kept out of the pervious areas located below downspout disconnects during and after construction.
- Construction contracts should contain a replacement warranty that covers at least three growing seasons to help ensure adequate growth and survival of the vegetation planted within the pervious area located below a downspout disconnect.

### 4.4.5.5 CONSTRUCTION AND MAINTENANCE COSTS

Construction and maintenance of areas receiving disconnected runoff is generally no different than that required for other lawn or landscaped areas. Soil amendments may be required for downspout disconnects constructed on HSG C or D soils or on previously compacted areas, such as gravel driveways or grassed parking areas.

### 4.4.6 Design Procedures

(Step 1) **Determine the goals and primary function of the downspout disconnect.**

Consider whether the downspout disconnect is intended to:

» Meet a runoff reduction* target or water quality (treatment) target.
**Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*

» Provide a possible solution to a drainage problem.

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the downspout disconnect has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the downspout disconnect should be centered on the restrictions/requirements, goals, targets, and primary function(s) of a downspout disconnect. By considering the primary function, as well as, topographic and soil conditions, the design elements of the downspout disconnect can be determined (i.e. planting media, outlet, etc.).

(Step 2) **Determine if the development site and conditions are appropriate for the use of a downspout disconnect.**

Consider the application and site feasibility criteria in this chapter to determine if site conditions are suitable for a downspout disconnect. Ensure that the drainage area and ground surface slope do not exceed the maximum criteria. Calculate the drainage area flow path and determine if the disconnected pervious area is of equal or greater length. Finally, review the NRCS data and perform site and geotechnical investigations, if necessary, to determine soil type and condition.

*Complete Steps 3A and 3B for a runoff reduction approach, or skip Step 3 and complete Step 4 for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) **Calculate the Stormwater Runoff Reduction Target Volume.**

Calculate the Runoff Reduction Volume using the following formula:

$$RR_V = (P)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$RR_V$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 3B) **If using the practice for Runoff Reduction, apply the BMP Runoff Reduction Credit to the Runoff Reduction Volume calculated in Step 3A.**

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the downspout disconnect – either 50% for HSG A/B soils, or 25% for HSG C/D soils. If the practice meets the 2500 square foot rooftop restriction, the minimum and maximum slope requirement, and the 15 – 75 foot flow path requirement then a credit of 50% or 25% (depending upon soil type) of the runoff volume to this practice is awarded. This volume reduction can be used to calculate a reduced CN for $CP_v$, $Q_{p25}$, and $Q_f$ calculations.

Multiply the Runoff Reduction Credit by the Runoff Reduction Volume ($RR_v$)

$$RR_v \text{ x Runoff Reduction Credit (\%)} = RR_v \text{ (Credited)}$$

*Where*:

**$RR_v$** = Runoff Reduction Volume conveyed to the practice

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice

**$RR_v$ (Credited)** = Runoff Reduction Volume provided ($ft^3$)

(Step 4) **Calculate the Target Water Quality Volume.**

Calculate the Water Quality Volume using the following formula:

$$WQ_v = (1.2)\ (R_v)\ (A)\ /\ 12$$

*Where*:

**$WQ_v$** = Water Quality Volume ($ft^3$)

**1.2** = Target rainfall amount to be treated (inches)

**$R_v$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)

**12** = Unit conversion factor (in/ft)

(Step 5) **Calculate the adjusted curve numbers for $CP_v$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm). See Subsection 3.1.7.5 for more information.**

(Step 6) **Prepare a site vegetation and planting plan.**

A planting plan for the downspout disconnect area should be prepared to indicate how it will be established with vegetation.

See Subsection 4.4.5.3 (*Landscaping*) and Appendix D for more details.

### 4.4.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.5 Dry Detention Basins

**Description**: A surface storage basin or facility designed to provide water quality treatment and water quantity control through detention of stormwater runoff. Dry detention basins differ from dry extended detention (ED) basins (Section 4.6) in that they do not provide 24-hour detention of the channel protection volume ($CP_v$).

**LID/GI Considerations**: While dry detention basins are primarily used for storage and control of larger storm events, their use as a BMP contributes to a site's overall perviousness and aesthetics. The open grassed area of a dry detention basin can be used for multiple purposes, such as landscaped or recreational areas. Additionally, dry detention basins dissipate energy in the stormwater runoff it receives and provide opportunities for some sedimentation of suspended solids.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Embankments should be less than 20 feet in height and should have side slopes no steeper than 2:1 (horizontal to vertical) although 3:1 is preferred.
- The depth of the basin should not exceed 10 feet.
- Geotechnical slope stability analysis is recommended for embankments greater than 10 feet in height.
- All embankments must be designed to State of Georgia guidelines for dam safety.
- Storage volumes greater than 100 acre-feet are subject to the requirements of the Georgia Safe Dams Act (Georgia Annotated Code 12-5-370) unless the facility is excavated to this depth.
- The dry detention basin bottom should be graded toward the outlet to prevent standing water conditions.
- The outfall(s) of dry detention basins should always be stabilized to prevent scour.
- An emergency spillway should be provided to safely convey large flood events.

### ADVANTAGES / BENEFITS

- Moderate removal rate of urban pollutants
- High community acceptance
- Useful for water quality treatment and flood control
- Dry detention basins can serve multiple use purposes on a development site.

### DISADVANTAGES / LIMITATIONS

- Potential for thermal impacts/downstream warming
- Dam height restrictions for high relief areas
- Detention basin drainage can be problematic for low relief terrain.

### ROUTINE MAINTENANCE REQUIREMENTS

- Remove debris from inlet and outlet structures
- Maintain side slopes and outlet structure
- Remove invasive vegetation
- Monitor sediment accumulation and remove periodically

### POLLUTANT REMOVAL

- 60% **Total Suspended Solids**
- 50% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 10/30% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- N/A **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- Runoff Reduction
- ★ Water Quality
- Channel Protection
- ✓ Overbank Flood Protection
- ✓ Extreme Flood Protection

✓ suitable for this practice

★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- M Land Requirement
- L Capital Cost
- L Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Not recommended*
**Roadway Projects:** *Yes*

**Soils:** *Dry detention basins can be used with almost all soils and geology, with minor design adjustments for regions of karst topography or in rapidly percolating soils, such as sand.*

**Other Considerations:** In order to convey low flows through dry detention basins, designers should provide a pilot channel. Plantings on the bank of the detention pond to provide shade around the pilot channel and the basin outlet or designing the pilot channel as a grass channel can reduce thermal impacts.

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 0% of the runoff reduction volume provided

### 4.5.1 General Description

Dry detention basins are surface facilities intended to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts. These facilities temporarily detain stormwater runoff, releasing the flow over a period of time. They are designed to completely drain following a storm event and are normally dry between rain events. See **Figure 4.5-1** for a typical dry detention basin schematic.

Dry detention basins provide limited pollutant removal benefits and are not intended for runoff reduction. Detention-only facilities should be used in a treatment train approach with other BMPs that provide runoff reduction, additional water quality treatment, and channel protection (see Subsection 4.1.6).

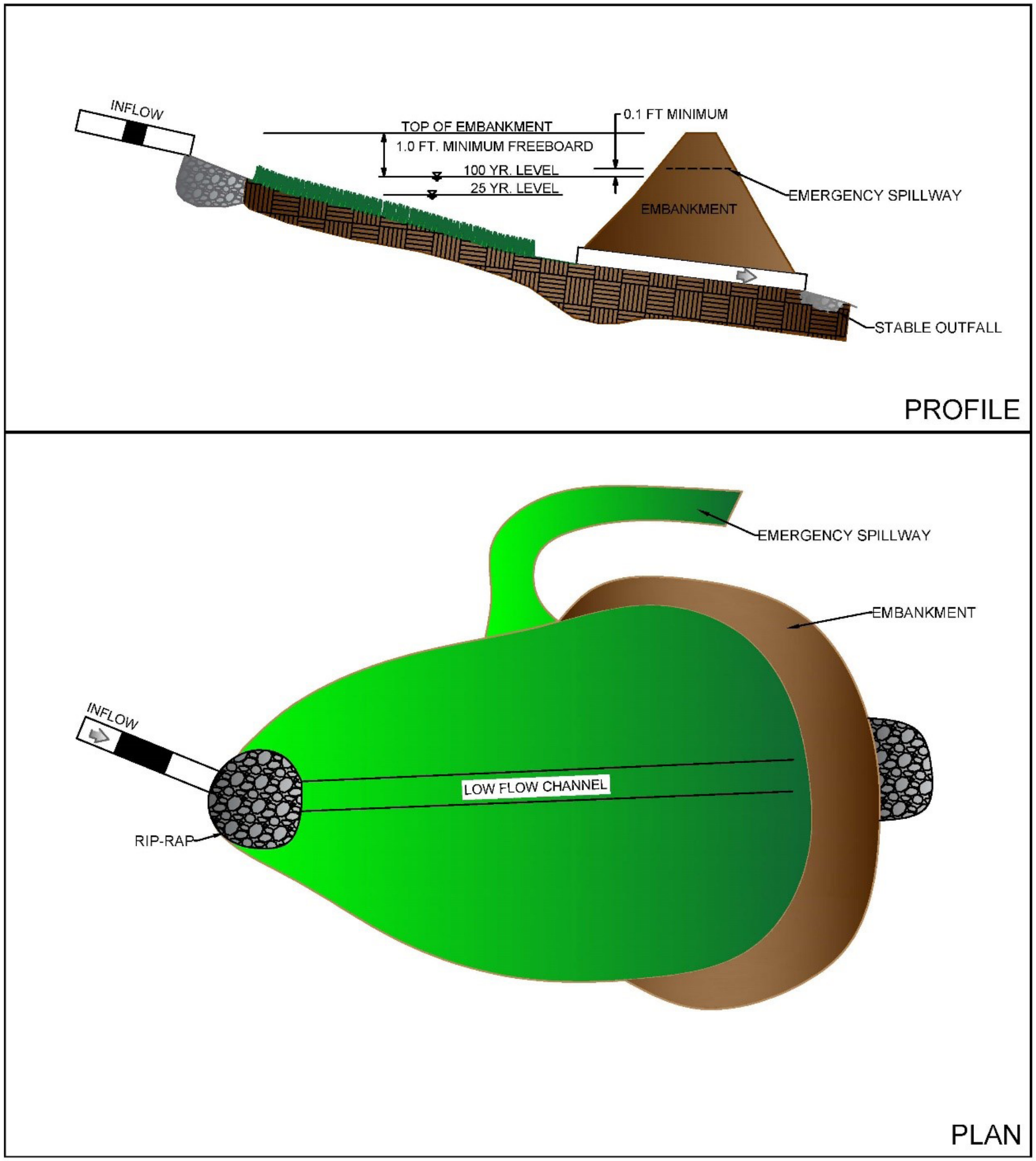



**Figure 4.5-1** Schematic of Dry Detention Basin

BACK TO TOC

### 4.5.2 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 as 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 to provide the additional required water quality treatment.

- Channel Protection
  Dry detention basins are not generally used to store and treat the channel protection volume ($CP_v$). A dry extended detention basin (Section 4.6) should be used if the $CP_v$ must be treated within the facility.

- Overbank Flood Protection
  Dry detention basins are intended to provide overbank flood protection (peak flow reduction of the 25-year storm, $Q_{p25}$).

- Extreme Flood Protection
  Dry detention basins can be designed to control the extreme flood (100-year, $Q_f$) storm event.

### 4.5.3 Pollutant Removal Capabilities

Dry detention basins are presumed to remove 60% of the total suspended solids (TSS) load in typical urban post development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Although they can be effective at removing some pollutants through settling, they are less effective at removing soluble pollutants because of the absence of a permanent pool. Other pollutants that dry detention basins can remove include Phosphorus, Nitrogen, and metals (such as Cadmium, Copper, Lead, and Zinc).

For additional information and data on pollutant removal capabilities for dry detention basins, see the National Pollutant Removal Performance Database (3rd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.5.4 Application and Site Feasibility Criteria

Dry detention basins have traditionally been one of the most widely used stormwater BMPs. Dry detention basins can easily be designed for flood control, and this is actually the primary purpose of most detention basins in the ground today. However, if pollutant removal efficiency is an important consideration, then dry detention basins may not be the most appropriate choice.

*The following criteria should be evaluated to ensure the suitability of dry detention basins for meeting stormwater management objectives on a site.*

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **NO**
- Regional Stormwater Control – **YES**

**Physical Feasibility –** Physical Constraints at Project Site

- **Drainage Area** – In general, dry detention basins should be used on sites with a minimum drainage area of 10 acres.
- **Space Required** – Approximately of 2-3% of the contributing drainage area.
- **Site Slope** – Dry detention basins can be used on sites with slopes up to about 15%.
- Vegetated and rip rap embankments should be less than 20 feet in height and have side slopes no steeper than 2:1 (horizontal to vertical), although 3:1 is preferred.
- **Minimum Depth to Water Table** – The base of the detention facility should not intersect the groundwater table

- **Soils** – Dry detention basins can be used in almost all soils and geology, with minor design adjustments for regions of karst (i.e., limestone) topography or in rapidly percolating soils, such as sand. In these areas, detention basins should be designed with an impermeable liner to prevent groundwater contamination or sinkhole formation.

**Other Constraints / Considerations**

- **Hot spots** – Dry detention basins can accept runoff from stormwater hotspots, but need significant separation from groundwater when used for this purpose.
- **Damage to existing structures and facilities** – Dry Detention basins should be designed to safely store and/or bypass the overbank flood ($Q_{p25}$) and extreme flood ($Q_f$) storms to prevent overflow or failure, which may cause damage to site structures and facilities.
- **Proximity** – The following is a list of specific setback requirements for the location of a dry detention basin:
  - » 10 feet from building foundations
  - » 10 feet from property lines
  - » 100 feet from private water supply wells
  - » 100 feet from open water (measured from edge of water)
  - » 200 feet from public water supply reservoirs (measured from edge of water)
  - » 1,200 feet from public water supply wells

- **Trout Stream** – In cold water streams, dry detention basins should be designed to detain stormwater for a relatively short time (i.e., less than twelve hours) to minimize the potential amount of stream warming that occurs in the practice. In addition, careful consideration should be given to the potential of perched or raised groundwater levels.

**Coastal Areas**

- **Poorly Drained Soils** – Poorly draining soils do not generally inhibit a dry detention basin's ability to temporarily store and treat stormwater runoff and completely drain between rain events, as the bottom of the BMP is sloped to provide for flow.
- **Flat Terrain** – The local slope should be relatively flat in order to maintain reasonably flat side slopes
- **Shallow Water Table** – Except for the case of hot spot runoff, the only consideration regarding ground water is that the base of the detention facility should not intersect the ground water table.

## 4.5.5 Planning and Design Criteria

Before designing dry detention basins, the following data is necessary:

- Existing and proposed site, topographic, and location maps, as well as field reviews
- Impervious and pervious areas. Other means may be used to determine the land use data.
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site.
- Design data from nearby storm sewer structures.
- Water surface elevation of nearby water systems as well as depth to the seasonally high groundwater level.

*The following criteria are to be considered minimum standards for the design of a dry detention basin. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

### 4.5.5.1 LOCATION AND LAYOUT

Dry detention basins should be located downstream of other BMPs providing runoff reduction and/or additional treatment of the water quality volume ($WQ_v$). See Subsection 4.1.6 for more information on the use of multiple BMPs in a treatment train.

BACK TO TOC

#### 4.5.5.2 GENERAL DESIGN

- Dry detention basins are sized to provide storage and control for multiple rain events, including:
  - » Storage of the water quality volume ($WQ_v$);
  - » Temporary storage of the volume of runoff required to provide overbank flood ($Q_{p25}$) protection (i.e., reduce the post-development peak flow of the 25-year storm event to the pre-development rate);
  - » Control of the 100-year storm ($Q_f$), if required.
- Routing calculations must be used to demonstrate that the storage volume is adequate. See Section 3.3 (*Storage Design*) for procedures on the design of detention storage.
- Storage volumes greater than 100 acre-feet are subject to the requirements of the Georgia Safe Dams Act (Georgia Annotated Code 12-5-370) unless the facility is excavated to this depth.

#### 4.5.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- Vegetated embankments should be less than 20 feet in height and should have side slopes no steeper than 2:1 (horizontal to vertical) although 3:1 is preferred. Riprap-protected embankments should be no steeper than 2:1. Geotechnical slope stability analysis is recommended for embankments greater than 10 feet in height and is mandatory for embankment slopes steeper than those given above. All embankments must be designed to State of Georgia guidelines for dam safety.
- The depth of the basin should not exceed 10 feet.
- Areas above the normal high water elevations of the detention facility should be sloped toward the basin to allow drainage. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions.
- Designing dry detention basins with a high length to width ratio (i.e., at least 1.5:1) and incorporating other design features to maximize the flow path effectively increases detention time by eliminating the potential of flow to short circuit the basin. Designing basins with relatively flat side slopes can also help to lengthen the effective flow path.
- While there is no minimum slope requirement, enough elevation drop is needed from the basin inlet to the basin outlet to ensure that flow can move through the system.
- A low flow or pilot channel across the facility bottom from the inlet to the outlet is recommended to convey low flows and prevent standing water conditions. To prevent stream warming, designers should place landscaping to provide shade around the pilot channel and the basin outlet. Designing the pilot channel as a grass channel also reduces thermal impacts. A minimum slope of 1% is recommended for grass swales and 0.5% for armored pilot channels.
- Adequate maintenance access must be provided for all dry detention basins.

#### 4.5.5.4 PRETREATMENT/INLETS

- Inflow channels are to be stabilized with flared aprons, or the equivalent.
- Pretreatment for a dry detention basin is usually provided by a sediment forebay. The sediment forebay should be sized to 0.1 inches of runoff per impervious acre of contributing drainage area for dry detention basins
- Pretreatment may also be provided by using a grass filter strip, pea gravel diaphragm, or grass channel. Where filter strips are used, 100% of the contributing runoff should flow across the filter strip. Refer to Section 4.9 for design criteria for a grass channel and Section 4.29 for vegetated filter strips.

BACK TO TOC

#### 4.5.5.5 OUTLET STRUCTURES

- For a dry detention basin, the outlet structure is sized for Qp25 control (based upon hydrologic routing calculations) and can consist of a weir, orifice, outlet pipe, combination outlet, or other acceptable control structure. Small outlets that will be prone to clogging or difficult to maintain are not acceptable.
- The water quality orifice should have a minimum diameter of 3 inches and be adequately protected from clogging by an external trash rack. The orifice diameter may be reduced to 1 inch if internal orifice protection is used (e.g., a perforated vertical stand pipe with 0.5-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.
- Seepage control or anti-seep collars should be provided for all outlet pipes.
- Riprap, plunge pools or pads, or other energy dissipators are to be placed at the end of the outlet to prevent scouring and erosion. If the basin discharges to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance.

#### 4.5.5.6 SAFETY FEATURES

- An emergency spillway should be included in the dry detention basin design to safely pass the extreme flood flow. The spillway prevents basin water levels from overtopping the embankment and causing structural damage. The emergency spillway must be designed to State of Georgia guidelines for dam safety and located so that downstream structures will not be impacted by spillway discharges.
- A minimum of 1 foot of freeboard must be provided, measured from the top of the water surface elevation for the extreme flood, to the lowest point of the dam embankment, not counting the emergency spillway. A safety bench shall be provided for embankments greater than 10 feet in height and having a side slope steeper than 3:1. For large basins, the safety bench shall extend no less than 15 feet outward from the normal water edge to the toe of the basin side slope. The slope of the safety bench shall not exceed 6%.
- Stormwater should be conveyed to and from dry detention basins safely and to minimize erosion potential.

#### 4.5.5.7 LANDSCAPING

- Designers should maintain a vegetated buffer around the dry detention basin and select plants within the detention zone (i.e., the portion of the basin up to the elevation where stormwater is detained) that can withstand both wet and dry periods. The side slopes of dry basins should be relatively flat to reduce safety risks.
- Trees planted on or near the side slopes of the dry detention basin can intercept and slow rainfall, reducing its erosive force before hitting the ground. The trees also transpire soil moisture within the basin. There is an added benefit of rainfall storage that is held within the tree canopy and does not reach the ground.
- Plantings should be designed not to conflict with the current drainage of the basin.
- All trees should be kept away from any drainage structures to allow for maintenance access and repairs as needed.

#### 4.5.5.8 CONSTRUCTION CONSIDERATIONS

Construction equipment should be restricted from the dry detention basin to prevent compaction of soils.

#### 4.5.5.9 CONSTRUCTION AND MAINTENANCE COSTS

Construction costs associated with dry extended detention basins range considerably and are based on a cost per unit area treated. One study evaluated the cost of all basin systems (Brown and Schueler, 1997). Adjusting for inflation using a RSMeans Construction Cost Index, the cost of dry ED basins can be estimated with the equation:

$$C = 22.7V^{0.760}$$

*Where*:

**C** = Total construction cost including design and permitting cost
**V** = Total volume required to control the 10-year storm (cubic ft)

Using this equation, typical construction costs are:

**$ 76,200** for a 1 acre-foot basin
**$ 438,000** for a 10 acre-foot basin
**$ 2,530,000** for a 100 acre-foot basin

Dry ED basins are typically less costly than stormwater (wet) basins, for example, that would provide equivalent flood storage, as less excavation is required.

### 4.5.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of a dry detention basins.**
Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a dry detention basin. Create a rough layout of the dry detention basin dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the dry detention basin.**
**Consider whether the dry detention basin is intended to:**

» Meet a water quality (treatment) target. See Step 3 to size the BMP utilizing the water quality treatment approach.

» Provide a possible solution to a drainage problem

» Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3 for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3) **Calculate the Target Water Quality Volume**

Calculate the Water Quality Volume using the following formula:

$$WQ_V = (1.2)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$WQ_V$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4) **Size flow diversion structure, if needed**

A flow regulator (or flow splitter diversion structure) should be supplied to divert the $WQ_v$ to the dry detention basins facility. Size low flow orifice, weir, or other device to pass $Q_{wq}$.

(Step 5) **Determine 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.

(Step 6) **Calculate the adjusted curve numbers for $CP_V$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm).**

(Step 7) **Compute $WQ_v$ orifice release rate and size $WQ_v$ orifice.**

» Size a water quality orifice to release the calculated $WQ_v$.

» The $WQ_v$ elevation is then determined from the stage-storage relationship. The invert of the $WQ_v$ orifice is located at the water quality detention elevation, and the orifice is sized to allow for temporary storage of the water quality storage volume

» The water quality orifice should be adequately protected from clogging by an acceptable external trash rack.

(Step 8) **Design embankment(s) and spillway(s).**

Size emergency spillway, calculate the 100-year, 24-hour storm water surface elevation, set the top of the embankment elevation, and analyze safe passage of the $Q_f$. Set the invert elevation of the emergency spillway 0.1 foot above the 100-year, 24-hour storm water surface elevation.

(Step 9) **Investigate potential basin hazard classification**

Design and construction of the detention facility may be required to meet the Georgia Dam Safety standards.

(Step 10) **Prepare a site vegetation and landscaping plan.**

A vegetation scheme for the dry detention basin should be prepared to indicate how the basin bottom, side slopes, and embankment will be stabilized and established with vegetation.

### 4.5.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

BACK TO TOC

# 4.6 Dry Extended Detention Basins

**Description**: A surface storage basin or facility designed to provide water quality treatment and water quantity control through extended detention (ED) of stormwater runoff. Dry ED basins differ from dry detention basins in that they provide 24-hour detention of the channel protection volume ($CP_v$).

**LID/GI Considerations:** Similar to dry detention, dry ED basins contribute to a site's overall perviousness and aesthetics. The open grassed area of a dry ED basin can also be used for multiple purposes, such as landscaped or recreational areas.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Embankments should be less than 20 feet in height and should have side slopes no steeper than 2:1 (horizontal to vertical) although 3:1 is preferred.
- The depth of the basin should not exceed 10 feet.
- Geotechnical slope stability analysis is recommended for embankments greater than 10 feet in height.
- All embankments must be designed to State of Georgia guidelines for dam safety.
- Storage volumes greater than 100 acre-feet are subject to the requirements of the Georgia Safe Dams Act (Georgia Annotated Code 12-5-370) unless the facility is excavated to this depth.
- The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions.
- The outfall of dry ED basins should always be stabilized to prevent scour.
- An emergency spillway should be provided to safely convey large flood events.

### ADVANTAGES / BENEFITS

- Moderate removal rate of urban pollutants
- High community acceptance
- Useful for water quality treatment, channel protection, and flood control
- Dry ED basins can serve multiple purposes on a development site.
- Settling pools within the dry ED basin mitigate potential thermal impacts.

### DISADVANTAGES / LIMITATIONS

- Dam height restrictions for high relief areas
- Drainage from the ED basin can be problematic for low relief terrain.

### ROUTINE MAINTENANCE REQUIREMENTS

- Remove debris from inlet and outlet structures
- Maintain side slopes and outlet structure
- Remove invasive vegetation
- Monitor sediment accumulation and remove periodically

### POLLUTANT REMOVAL

- 60% **Total Suspended Solids**
- 50% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 10/30% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- N/A **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- Runoff Reduction
- ★ Water Quality
- ✓ Channel Protection
- ✓ Overbank Flood Protection
- ✓ Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- M Land Requirement
- L Capital Cost
- L Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Not recommended*
**Roadway Projects:** *Yes*

**Soils:** *Dry ED basins can be used with nearly all soils and geology, with minor design adjustments for regions of karst topography or in rapidly percolating soils, such as sand.*

**Other Considerations:** In order to convey low flows through dry ED basins, designers should provide a pilot channel. Designers should provide shade around the pilot channel and the basin outlet to prevent stream warming.

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 0% of the runoff reduction volume provided

BACK TO TOC

## 4.6.1 General Description

Dry ED basins are surface facilities intended to provide for the temporary storage and treatment of stormwater runoff to reduce downstream water quality and water quantity impacts. These facilities temporarily detain stormwater runoff, releasing the flow over a period of time. They are designed to completely drain within 24 – 72 hours after a storm event and are normally dry between rain events. A typical schematic for a dry ED basin is shown in **Figure 4.6-1**. Dry ED basins provide downstream channel protection through extended detention of the channel protection volume ($CP_v$). Dry ED basins are intended to provide overbank flood protection (peak flow reduction of the 25-year, 24-hour storm, $Q_{p25}$) and can be designed to control the extreme flood (100-year, 24-hour, $Q_f$) storm event.

Dry ED basins provide limited pollutant removal benefits and are not intended for runoff reduction. Dry ED-basins should be used in a treatment train approach with other BMPs that provide runoff reduction and/or additional water quality treatment (see Subsection 4.1.6).

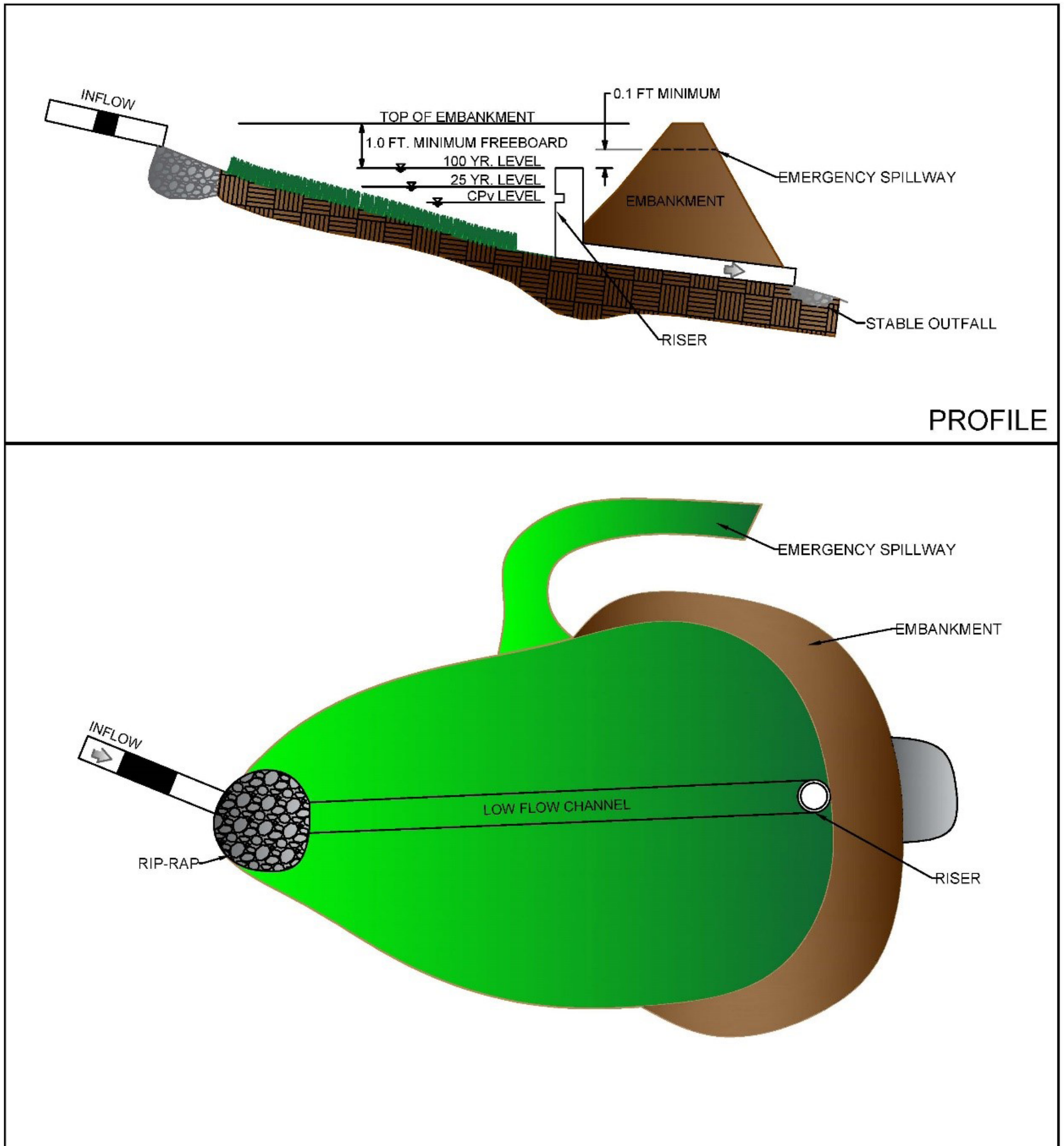



**Figure 4.6-1** Schematic of dry ED Basin

### 4.6.2 Stormwater Management Suitability

- Runoff Reduction
  Another BMP should be used in a treatment train with dry ED basins to provide runoff reduction as they are not designed to provide $RR_v$ as a stand-alone BMP.

- Water Quality
  If installed as 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 ED basin. Another BMP should be used in a treatment train with dry ED basins to provide the additional required water quality treatment.

- Channel Protection
  Dry ED 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 ED basins are intended to provide overbank flood protection (peak flow reduction of the 25-year, 24-hour storm, $Q_{p25}$).

- Extreme Flood Protection
  Dry ED basins can be designed to control the extreme flood (100-year, 24-hour storm, $Q_f$) rainfall event.

### 4.6.3 Pollutant Removal Capabilities

Dry ED basins are presumed to remove 60% of the total suspended solids (TSS) load in typical urban post development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Although they can be effective at removing some pollutants through settling, they are less effective at removing soluble pollutants because of the absence of a permanent pool. Other pollutants that dry ED basins can remove include Phosphorus, Nitrogen, and metals (such as Cadmium, Copper, Lead, and Zinc).

For additional information and data on pollutant removal capabilities for dry ED basins, see the National Pollutant Removal Performance Database (Version 3) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.6.4 Application and Site Feasibility Criteria

A dry ED basin temporarily stores runoff and releases that runoff at a controlled rate over a specified period of time.  By definition, dry ED basins are dry structures during non-precipitation periods. Dry ED basins are capable of providing water quality improvement, downstream flood control, channel erosion control, and mitigation of post-development runoff to pre-development levels. A dry ED facility improves runoff quality primarily through the gravitational settling of pollutants.

*The following criteria should be evaluated to gage the suitability of dry ED basins for meeting stormwater management objectives on a site or development.*

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **NO**
- Regional Stormwater Control – **YES**

**Physical Feasibility –** Physical Constraints at Project Site

- **Drainage Area** – In general, dry ED basins should be used on sites with a minimum drainage area of 10 acres.
- **Space Required** – Roughly 2-3% of the contributing drainage area.
- **Site Slope** – Dry ED basins can be used on sites with slopes up to about 15%.
- Vegetated and rip rap embankments should be less than 20 feet in height and should have side slopes no steeper than 2:1 (horizontal to vertical), although 3:1 is preferred.
- **Minimum Depth to Water Table** – Except for the case of hotspot runoff, the only consideration regarding groundwater is that the base of the dry ED basin should not intersect the groundwater table.

- **Soils** – Dry ED basins can be used in nearly all soils and geology, with minor design adjustments for regions of karst (i.e., limestone) topography or in rapidly percolating soils, such as sand. In these areas, dry ED basins should be designed with an impermeable liner to prevent groundwater contamination or sinkhole formation.

**Other Constraints / Considerations**

- **Hot spots** – Dry ED basins can accept runoff from stormwater hotspots, but need significant separation from groundwater when used for this purpose.
- **Damage to existing structures and facilities** – Dry ED basins should be designed to safely store and/or bypass the overbank flood ($Q_{p25}$) and extreme flood ($Q_f$) event to prevent overflow or failure, which may cause damage to site structures and facilities.
- **Proximity** – The following is a list of specific setback requirements for the location of dry ED basins:
  - » 10 feet from building foundations
  - » 10 feet from property lines
  - » 100 feet from private water supply wells
  - » 100 feet from open water (measured from edge of water)
  - » 200 feet from public water supply reservoirs (measured from edge of water)
  - » 1,200 feet from public water supply wells
- **Trout Stream** – Dry ED basins should not be used where receiving water temperature is a concern. In addition, careful consideration should be given to the potential for perched or raised groundwater levels.

**Coastal Areas**

- **Poorly Drained Soils** – Poorly draining soils do not inhibit a dry ED basin's ability to temporarily store and treat stormwater runoff and completely drain within 24-72 hours.
- **Flat Terrain** – The local slope needs to be relatively flat in order to maintain reasonably flat side slopes. While there is no minimum slope requirement, enough elevation drop is needed from the basin inlet to the basin outlet to ensure that flow can move through the facility.
- **Shallow Water Table** – Except for the case of hot spot runoff, the only consideration regarding ground water is that the base of the dry ED basin should not intersect the ground water table.

## 4.6.5 Planning and Design Criteria

Before designing dry ED basins, the following data is necessary:

- Existing and proposed site, topographic, and location maps, as well as field reviews.
- Impervious and pervious areas. Other means may be used to determine land use data.
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site.
- Design data from nearby storm sewer structures.
- Water surface elevation of nearby water systems, as well as depth to the seasonally high groundwater table.

*The following criteria are to be considered minimum standards for the design of a dry ED basin. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.6.5.1 LOCATION AND LAYOUT

Dry ED basins are to be located downstream of other BMPs providing runoff reduction and/or treatment of the water quality volume ($WQ_v$). See Subsection 4.1.6 for more information on the use of multiple BMPs in a treatment train.

BACK TO TOC

#### 4.6.5.2 GENERAL DESIGN

- Dry ED basins are sized to provide storage and control for multiple rain events, including:
  - » Storage of the water quality volume ($WQ_v$)
  - » 24-hour storage of the channel protection volume ($CP_v$)
  - » Temporary storage of the volume of runoff required to provide overbank flood ($Q_{p25}$) protection (i.e., reduce the post-development peak flow of the 25-year storm event to the pre-development rate), and
  - » Control of the 100-year storm ($Q_f$), if required.
- Routing calculations must be used to demonstrate that the storage volume is adequate. See Section 3.3 (Storage Design) for procedures on the design of detention storage.
- Storage volumes greater than 100 acre-feet are subject to the requirements of the Georgia Safe Dams Act (Georgia Annotated Code 12-5-370) unless the facility is excavated to this depth.

#### 4.6.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- Vegetated embankments should be less than 20 feet in height and should have side slopes no steeper than 2:1 (horizontal to vertical), although 3:1 is preferred. Riprap-protected embankments should be no steeper than 2:1. Geotechnical slope stability analysis is recommended for embankments greater than 10 feet in height and is mandatory for embankment slopes steeper than those given above. All embankments must be designed to State of Georgia guidelines for dam safety.
- The depth of the basin should not exceed 10 feet.
- Areas above the normal high water elevations of the detention facility should be sloped toward the basin to allow drainage and prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions.
- Designing dry ED basins with a high length to width ratio (i.e., at least 1.5:1) and incorporating other design features to maximize the flow path effectively increases the detention time in the system by eliminating the potential of flow to short circuit the basin. Designing basins with relatively flat side slopes can also help to lengthen the effective flow path.
- While there is no minimum slope requirement, enough elevation drop is needed from the basin inlet to the basin outlet to ensure that flow can move through the facility.
- A low-flow or pilot channel across the facility bottom from the inlet to the outlet is recommended to convey low flows and prevent standing water conditions. In order to prevent stream warming, designers should place landscaping to provide shade around the pilot channel and the basin outlet. A minimum slope of 1% is recommended for grass swales and 0.5% for armored pilot channels.
- Adequate maintenance access must be provided for all dry ED basins.

#### 4.6.5.4 PRETREATMENT/INLETS

- Inflow channels are to be stabilized with flared aprons, or the equivalent.
- Pretreatment for a dry ED basin is usually provided by a sediment forebay. The sediment forebay should be sized for 0.1 inches per impervious acre of contributing drainage area.
- Pretreatment may also be provided by using a grass filter strip, pea gravel diaphragm, or grass channel. Where filter strips are used, 100% of the contributing runoff should flow across the filter strip. Refer to Section 4.9 for design criteria for a grass channel and Section 4.29 for vegetated filter strips.

#### 4.6.5.5 OUTLET STRUCTURES

- For a dry ED basin, the outlet structure is sized for $Q_{p25}$ control (based upon hydrologic routing calculations) and can consist of a weir, orifice, outlet pipe, combination outlet, or other acceptable control structure. Small outlets that will be subject to clogging or are difficult to maintain are not acceptable.
- A dry ED basin has a channel protection orifice with a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. The orifice diameter may be reduced to 1 inch if internal orifice protection is used (e.g., an overperforated vertical stand pipe with 0.5-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.
- Seepage control or anti-seep collars should be provided for all outlet pipes.
- Riprap, plunge pools or pads, or other energy dissipaters are to be placed at the end of the outlet to prevent scouring and erosion. If the basin discharges to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance from the dry ED basin.

#### 4.6.5.6 SAFETY FEATURES

- An emergency spillway should be included in the dry ED basin design to safely pass the extreme flood flow. The spillway prevents water levels from overtopping the embankment and causing structural damage. The emergency spillway must be designed to State of Georgia guidelines for dam safety and must be located so that downstream structures will not be impacted by spillway discharges.
- A minimum of 1 foot of freeboard must be provided, measured from the top of the water surface elevation for the extreme flood, to the lowest point of the dam embankment, not counting the emergency spillway.
- Stormwater should be conveyed to and from dry ED basins safely and to minimize erosion potential.

#### 4.6.5.7 LANDSCAPING

- Designers should maintain a vegetated buffer around dry ED basins, selecting plants within the detention zone (i.e., the portion of the basin up to the elevation where stormwater is detained) that can withstand both wet and dry periods. The side slopes of dry ED basins should be relatively flat to reduce safety risks.
- Trees planted on or near the side slopes of the dry ED basin can intercept and slow rainfall, reducing its erosive force before hitting the ground. The trees also transpire soil moisture within the basin. There is an added benefit of rainfall storage that is held within the canopy of the trees and does not reach the ground.
- Plantings should be designed not to conflict with the current drainage of the basin.
- All trees should be kept away from any drainage structures to allow for maintenance access and repairs as needed.

#### 4.6.5.8 CONSTRUCTION CONSIDERATIONS

Construction equipment should be restricted from the dry ED basin to prevent compaction of native soils.

#### 4.6.5.9 CONSTRUCTION AND MAINTENANCE COSTS

Construction costs associated with dry extended detention basins range considerably and are based on a cost per unit area treated. One study evaluated the cost of all basin systems (Brown and Schueler, 1997). Adjusting for inflation using a RSMeans Construction Cost Index, the cost of dry ED basins can be estimated with the equation:

$$C = 22.7V^{0.760}$$

*Where*:

**C** = Total construction cost including design and permitting cost
**V** = Total volume required to control the 10-year storm (cubic feet)

Using this equation, typical construction costs are:
**$ 76,200** for a 1 acre-foot basin
**$ 438,000** for a 10 acre-foot basin
**$ 2,530,000** for a 100 acre-foot basin

Dry ED basins are typically less costly than stormwater (wet) basins, for example, that would provide equivalent flood storage, as less excavation is required.

BACK TO TOC

### 4.6.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of dry extended detention basins.**
Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a dry extended detention basin. Create a rough layout of the dry extended detention basin dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the dry extended detention basin.**
Consider whether the dry extended detention basin is intended to:

- Meet a water quality (treatment) target. See Step 3 to size the BMP utilizing the water quality treatment approach.
- Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)
- Provide a possible solution to a drainage problem
- Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3 for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3) **Calculate the Target Water Quality Volume**
Calculate the Water Quality Volume using the following formula:

$$WQ_V = (1.2)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$WQ_V$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

BACK TO TOC

(Step 4) **Size flow diversion structure, if needed**

A flow regulator (or flow splitter diversion structure) should be supplied to divert the $WQ_v$ to the dry extended detention basins facility.

Size low flow orifice, weir, or other device to pass $Q_{wq}$.

(Step 5) **Determine 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.

(Step 6) **Calculate the $CP_V$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm) flow rates and volumes.**

(Step 7) **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 $Q_f$. Set the invert elevation of the emergency spillway 0.1 foot above the 100-year water surface elevation.

(Step 8) **Investigate potential basin hazard classification.**

The design and construction of the dry ED basin may be required to meet the Georgia Dam Safety standards.

(Step 9) **Prepare a site Vegetation and Landscaping Plan.**

A vegetation scheme for the dry ED basin should be prepared to indicate how the basin bottom, side slopes and embankment will be stabilized and established with vegetation. The use of native vegetation is highly recommended for these facilities.

### 4.6.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

## 4.7 Dry Wells

(Source: City of Portland, OR, 2008)

**Description**: Dry wells are low impact development practices that are located below the surface of development sites. They consist of shallow excavations, typically filled with stone, that are designed to intercept and temporarily store post-construction stormwater runoff until it infiltrates into the underlying and surrounding soils.

**LID/GI Considerations**: Use of a dry well decreases the post construction runoff volume from a site, decreasing the pollutant load as well as thermal and erosive impacts to receiving waters. Dry wells are installed underground, which allows for multiple uses of development space.

### KEY CONSIDERATIONS

#### DESIGN CRITERIA

- Dry wells should be designed to completely drain within 24 hours of the end of a rainfall event.
- There should be at least 2 feet of separation distance between the bottom of a dry well and the top of the water table.
- Dry wells should be designed with slopes that are as close to flat as possible to help ensure that stormwater runoff is evenly distributed throughout the stone reservoir.

#### ADVANTAGES / BENEFITS

- Helps restore pre-development hydrology on development sites
- Reduces post-construction stormwater runoff rates, volumes, and pollutant loads
- Well-suited for use on urban development sites

#### DISADVANTAGES / LIMITATIONS

- Can only be used to “receive” runoff from small drainage areas of 2,500 square feet or less
- Should not be used on development sites that have soils with infiltration rates of less than 0.5 inches per hour

#### ROUTINE MAINTENANCE REQUIREMENTS

- A dry well and its components shown in Figure 4.7-1 must be inspected and maintained at least annually.
- Replace gravel when more than 6 inches of sediment has accumulated.
- Inspect after major storm events to ensure water does not pond for more than 48 hours. If extended ponding occurs, the gravel may need to be replaced.

#### POLLUTANT REMOVAL

- 100% **Total Suspended Solids**
- 100% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 100/100% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- 100% **Pathogens** – Fecal Coliform

#### STORMWATER MANAGEMENT SUITABILITY

- ✓ Runoff Reduction
- ✓ Water Quality
- ★ Channel Protection
- Overbank Flood Protection
- Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

#### IMPLEMENTATION CONSIDERATIONS

- L Land Requirement
- M Capital Cost
- M Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *Not recommended*

**Soils:** *Dry wells should be considered for use on development sites where fine sediment (e.g., clay, silt) loads will be relatively low, as high sediment loads will cause them to clog and fail. Permeable soils with a water table low enough to provide for the infiltration of stormwater runoff are recommended.*

**Other Considerations:** *Dry wells should not be located beneath a driveway, parking lot or other impervious surface.*

L=Low M=Moderate H=High

#### RUNOFF REDUCTION CREDIT

- 100% runoff reduction volume

BACK TO TOC

## 4.7.1 General Description

Dry wells (also known as seepage pits and French drains) are low impact development practices that are located below the surface of development sites. They consist of shallow excavations, typically filled with stone, that are designed to intercept and temporarily store post-construction stormwater runoff until it infiltrates into the underlying and surrounding soils (**Figure 4.7-1**). If properly designed, they can provide significant reductions in post-construction stormwater runoff rates, volumes, and pollutant loads on development sites.

As infiltration-based low impact development practices, dry wells are limited to use in areas where the soils are permeable enough and the water table is low enough to provide for the infiltration of stormwater runoff. They should only be considered for use on development sites where fine sediment (e.g., clay, silt) loads will be relatively low, as high sediment loads will cause them to clog and fail. In addition, dry wells should be carefully sited to avoid potential contamination of water supply aquifers.

The primary concern associated with the design of a dry well is its storage capacity, which directly influences its ability to reduce stormwater runoff rates, volumes, and pollutant loads. Site planning and design teams should strive to design dry wells that can accommodate the stormwater runoff volume generated by the target runoff reduction rainfall event (e.g., 85th percentile rainfall event).

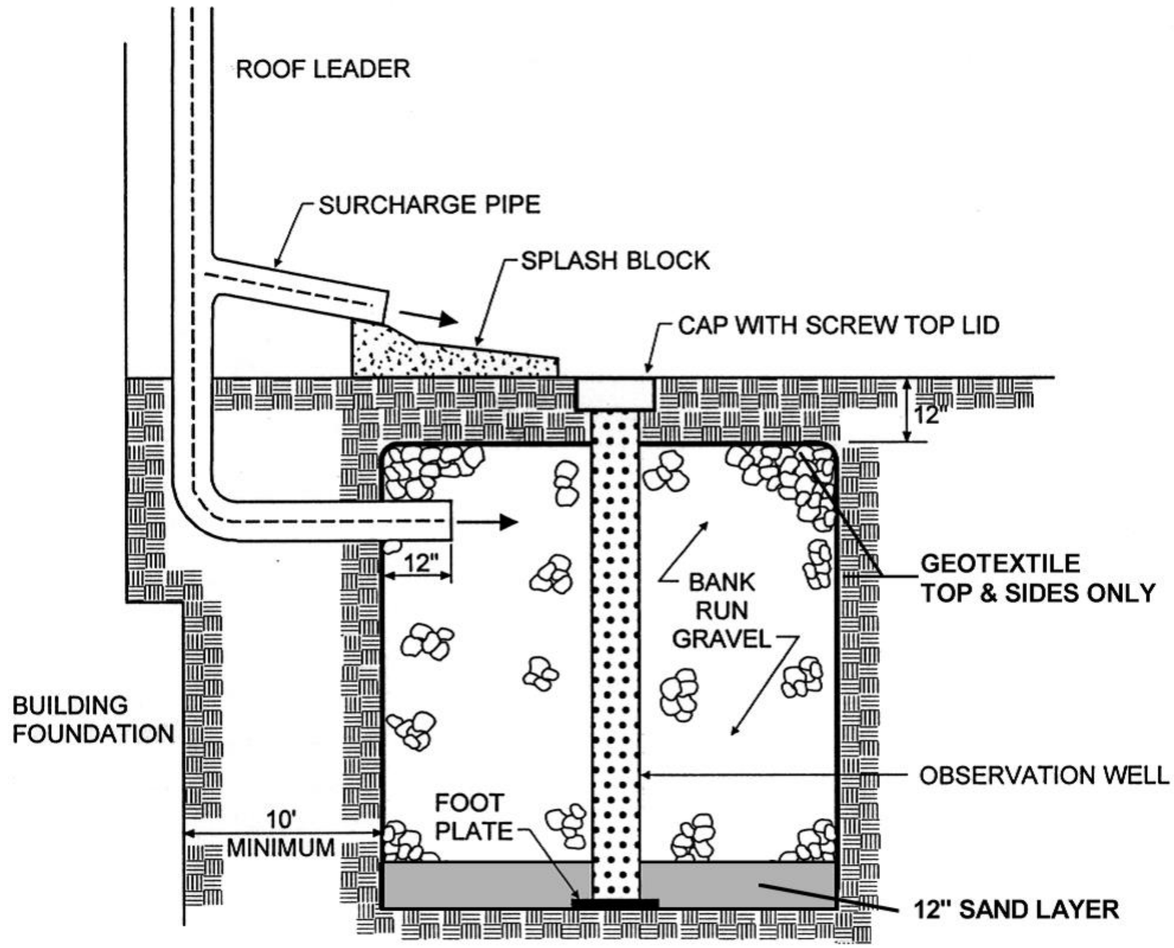



**Figure 4.7-1: Dry Well**
(Source: Maryland Department of the Environment, 2000)

If this cannot be accomplished due to site characteristics or constraints, site planning and design teams should consider using dry wells in combination with other runoff reducing low impact development practices, such as bioretention areas (Section 4.2) and rainwater harvesting (Section 4.19), to supplement the stormwater management benefits provided by the dry wells.

### 4.7.2 Stormwater Management Suitability

The Center for Watershed Protection (Hirschman et al., 2008) recently documented the ability of dry wells to reduce annual stormwater runoff volumes and pollutant loads on development sites, as follows:

- Stormwater Runoff Reduction
  Subtract 100% of the storage volume provided by a dry well from the runoff reduction volume ($RR_v$) conveyed through the dry well.
- Water Quality Protection
  If installed as per the recommended design criteria and properly maintained, 100% total suspended solids removal will be applied to the water quality volume ($WQ_v$) flowing to the dry well.
- Channel Protection
  Proportionally adjust the post-development runoff curve number (CN) to account for the runoff reduction provided by a dry well when calculating the channel protection volume ($CP_v$) on a development site (See Subsection 3.1.7.5).
- Overbank Flood Protection
  Proportionally adjust the post-development runoff CN to account for the runoff reduction provided by a dry well when calculating the overbank peak discharge ($Q_{p25}$) on a development site (See Subsection 3.1.7.5).
- Extreme Flood Protection
  Proportionally adjust the post-development runoff CN to account for the runoff reduction provided by a dry well when calculating the extreme peak discharge ($Q_f$) on a development site (See Subsection 3.1.7.5).

### 4.7.3 Pollutant Removal Capabilities

Dry wells are presumed to remove 100% of the total suspended solids (TSS) load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Dry wells also remove 100% of the Phosphorus, Nitrogen, metals (such as Cadmium, Copper, Lead, and Zinc), and fecal coliform in contributing runoff.

In order to provide the most efficient pollutant removal, dry wells should be constructed in permeable soils of hydrologic soil group A or B. The maximum drainage area and site surface slope should not be exceeded to ensure proper pollutant removal.

For additional information and data on pollutant removal capabilities for dry wells, see the National Pollutant Removal Performance Database (Version 3) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

BACK TO TOC

### 4.7.4 Application and Site Feasibility Criteria

Dry wells can be used to treat stormwater runoff on a wide variety of development sites, including residential, commercial, and institutional development sites in rural, suburban, and urban areas. Although they are particularly well-suited to receive rooftop runoff, they can also be used to receive stormwater runoff from other small drainage areas, such as local streets and roadways, driveways, small parking areas, and disturbed pervious areas (e.g., lawns, parks, community open spaces). When compared with other low impact development practices, dry wells have a moderate construction cost, a moderate maintenance burden, and require only a small amount of surface area.

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **YES**
- Regional Stormwater Control – **NO**

**Physical Feasibility –** Physical Constraints at Project Site

- **Drainage Area** – The size of the contributing drainage area should be 2,500 square feet or less.
- **Space Required** – Dry well surface area requirements vary according to the size of the contributing drainage area and the infiltration rate of the soils in which the dry well is located. In general, dry wells require about 5-10% of the size of their contributing drainage areas.
- **Flow Path** – The length of flow path in contributing drainage areas should be 150 feet or less in pervious drainage areas and 75 feet or less in impervious drainage areas.
- **Site Slope** – Although dry wells may be used on development sites with slopes of up to 6%, they should be designed with slopes that are as close to flat as possible to help ensure that stormwater runoff is evenly distributed throughout the stone reservoir.
- **Minimum Depth to Water Table** – 2 feet
- **Minimum Head** – 2 feet
- **Soils** – Dry wells should be designed to completely drain within 24 hours of the end of a rainfall event. Consequently, dry wells generally should not be used on development sites that have soils with infiltration rates of less than 0.50 inches per hour (i.e., hydrologic soil group C and D soils).

**Other Constraints / Considerations**

- **Hot spots** – May be used for hot spot runoff
- Damage to existing structures and facilities – Dry wells should not be used in areas where their operation may create a risk for basement flooding, interfere with subsurface sewage disposal systems, or affect other underground structures.
- Dry wells should be designed so that overflow drains away from buildings to prevent damage to building foundations.
- **Proximity** – Dry wells may be used without restriction, except within:
  - » 10 feet from building foundations
  - » 10 feet from property lines
  - » 100 feet from private water supply wells
  - » 1,200 feet from public water supply wells
  - » 100 feet from septic systems
  - » 100 feet from surface waters
  - » 400 feet from public water supply surface waters
- **Trout Stream** – Use of a dry well reduces a site's runoff pollutant load, as well as the volume and velocity of stormwater runoff. Therefore, dry wells are an effective BMP for use where trout streams or other protected waters may receive stormwater runoff.

**Coastal Areas**

- **Poorly drained soils, such as hydrologic soil groups C and D** – Limits the ability of dry wells to reduce stormwater runoff rates, volumes, and pollutant loads. Dry wells should not be used on development sites that have soils with infiltration rates of less than 0.5 inches per hour (i.e., hydrologic soil group C and D soils). Use other low impact development practices, such as rainwater harvesting (Section 4.19), to "receive" stormwater runoff in these areas.

- **Well drained soils, such as hydrologic soil groups A and B** – Enhance the ability of dry wells to reduce stormwater runoff rates, volumes, and pollutant loads, but may allow stormwater pollutants to reach groundwater aquifers with greater ease. Rooftop runoff is relatively clean, so this should not prevent the use of dry wells, even at stormwater hotspots and in areas known to provide groundwater recharge to water supply aquifers. However, rooftop runoff should not be allowed to comingle with runoff from other impervious surfaces in these areas if it will be sent to a dry well.
- **Flat Terrain** – Does not influence the use of dry wells. In fact, dry wells should be designed with slopes that are as close to flat as possible.
- **Shallow Water Table** – In coastal areas it may be difficult to provide 2 feet of clearance between the bottom of the dry well and the top of the water table. This may occasionally cause stormwater runoff to pond in the bottom of the dry well. Ensure that the distance from the bottom of the dry well to the top of the water table is at least 2 feet. Reduce the depth of the stone reservoir in dry wells to 18 inches, if necessary.
- **Tidally-influenced drainage system** – Does not influence the use of dry wells.

## 4.7.5 Planning and Design Criteria

Before designing the dry well, the following data is needed:

- Existing and proposed site, topographic, and location maps, as well as field reviews
- The proposed site design, including buildings, parking lots, sidewalks, stairs, handicapped ramps, and landscaped areas
- Architectural roof plan for rooftop pitches and downspout locations
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site
- Information about downstream BMPs and receiving waters
- Design data from nearby storm sewer structures
- Water surface elevation of nearby water systems as well as the depth to the seasonally high groundwater table

*The following criteria are to be considered minimum standards for the design of a dry well. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

### 4.7.5.1 LOCATION AND LAYOUT

- Dry wells should be located in a lawn or other disturbed pervious area and should be designed so that the top of the dry well is located as close to the surface as possible. Dry wells should not be located beneath a driveway, parking lot, or other impervious surface.
- Although dry wells may be installed on development sites with slopes of up to 6%, they should be designed with slopes that are as close to flat as possible to help ensure that stormwater runoff is evenly distributed throughout the stone reservoir.
- Dry wells should be used on development sites that have underlying soils with an infiltration rate of 0.50 inches per hour or greater, as determined by NRCS soil survey data and subsequent field testing.
- Although the number of infiltration tests needed on a development site will ultimately be determined by the local development review authority, at least one infiltration test is recommended for each dry well that will be used on the development site.
- Since clay lenses or any other restrictive layers located below the bottom of a dry well will reduce soil infiltration rates, infiltration testing should be conducted within any confining layers that are found within 4 feet of the bottom of a proposed dry well.

- The depth from the bottom of a dry well to the top of the water table should be at least 2 feet to prevent nuisance ponding and ensure proper operation of the dry well.

- If used to receive rooftop runoff, dry wells should be preceded by a leaf screen installed in the gutter or downspout. This will prevent leaves and other large debris from clogging the dry well.

- If used to receive non-rooftop runoff, dry wells should be preceded by a pea gravel diaphragm or equivalent level spreader device (e.g., concrete sills, curb stops, curbs with sawteeth cut into them) and a vegetated filter strip that is designed according to the planning and design criteria provided in Section 4.29.

- Consideration should be given to the stormwater runoff rates and volumes generated by larger storm events (e.g., 25-year, 24-hour storm event) to help ensure that these larger storm events are able to safely bypass the dry well. An overflow, such as a vegetated filter strip (Section 4.29) or grass channel (Section 4.9), should be designed to convey the stormwater runoff generated by these larger storm events safely out of the dry well.

#### 4.7.5.2 GENERAL DESIGN

- Dry wells should be used to receive stormwater runoff from small drainage areas of 2,500 square feet or less. The stormwater runoff rates and volumes from larger contributing drainage areas typically become too large to be properly treated by a dry well.

- The length of the flow path within the contributing drainage area should be 150 feet or less for pervious drainage areas and 75 feet or less for impervious drainage areas. In contributing drainage areas with longer flow paths, stormwater runoff tends to become shallow, concentrated flow (Claytor and Schueler, 1996), which can significantly reduce the stormwater management benefits that dry wells can provide. In these situations, bioretention areas (Section 4.2) and infiltration practices (Section 4.12) should be used to "receive" post-construction stormwater runoff.

- Dry wells should be designed to provide enough storage for the stormwater runoff volume generated by the target runoff reduction rainfall event (e.g., 85th percentile rainfall event).

- Dry wells should be designed to completely drain within 24 hours of the end of a rainfall event. Where site characteristics allow, it is preferable to design dry wells to drain within 12 hours of the end of a rainfall event to help prevent the formation of nuisance ponding conditions.

- Broader, shallower dry wells perform more effectively by distributing stormwater runoff over a larger surface area. However, a minimum depth of 18 inches is recommended for all dry well designs to prevent them from consuming a large amount of surface area on development sites. Whenever practical, the depth of dry wells should be kept to 36 inches or less.

- Dry wells should be filled with clean, washed stone. The stone used in the dry well should be 1.5 to 2.5 inches in diameter, with a void space of approximately 40% (e.g., GA DOT No. 3 Stone). Unwashed aggregate contaminated with soil or other fine particulates may not be used in the dry well. Underlying native soils should be separated from the dry well stone by a thin, 2 to 4 inch layer of choker stone (i.e., ASTM D 448 size No. 8, 3/8" to 1/8" or ASTM D 448 size No. 89, 3/8" to 1/16"). The choker stone should be placed between the dry well stone and the underlying native soils.

- The top and sides of the dry well should be lined with a layer of appropriate permeable filter fabric. The filter fabric should be a non-woven geotextile with a permeability that is greater than or equal to the infiltration rate of the surrounding native soils. The top layer of the filter fabric should be located 6 inches from the top of the excavation, with the remaining space filled with appropriate landscaping. This top layer serves as a sediment barrier and, consequently, will need to be replaced over time. Site planning and design teams should ensure that the top layer of filter fabric can be readily separated from the filter fabric used to line the sides of the dry well.

- An observation well should be installed in every dry well. An observation well consists of a 4 to 6 inch perforated PVC (AASHTO M 252) pipe that extends to the bottom of the dry well. The observation well can be used to observe the rate of drawdown within the dry well following a storm event. It should be installed along the centerline of the dry well, flush with the elevation of the surface of the dry well. A visible floating marker should be provided within the observation well and the top of the well should be capped and locked to prevent tampering and vandalism.

#### 4.7.5.3 PRETREATMENT/INLETS

Pretreatment and inlet protection need to be designed to reduce the velocity and energy of stormwater entering the practice and prevent scour of the mulch and plantings. Pretreatment and inlet protection may include splash blocks, a stone diaphragm, a level spreader, or similar device.

#### 4.7.5.4 OUTLET STRUCTURES

Outlet structures should be included in the design of a dry well to ensure that larger storms can be bypassed without damaging the practice. See Section 3.4 (*Outlet Structures*) for more guidance regarding the proper design and installation of an outlet structure.

#### 4.7.5.5 CONSTRUCTION AND MAINTENANCE COSTS

- The cost of a dry well varies based on the size. Typically, a small residential dry well costs between $1,350-$1,700.
- Larger dry wells can cost between $11,250-$16,900.
- Costs for downstream infiltration trenches or regional BMPs can be found in their respective sections in this manual.

#### 4.7.7.6 SAFETY FEATURES

Dry wells generally do not require any special safety features, provided side slopes are maintained at 3:1 or flatter. Fencing of dry wells is not generally desirable.

#### 4.7.5.7 LANDSCAPING

- The landscaped area above the surface of a dry well may be covered with pea gravel (i.e., ASTM D 448 size No. 8, 3/8″ to 1/8″). This pea gravel layer provides sediment removal and additional pretreatment upstream of the dry well and can be easily removed and replaced when it becomes clogged.
- Alternatively, a dry well may be covered with an engineered soil mix, such as that prescribed in Appendix D, and planted with managed turf or other herbaceous vegetation. This may be an attractive option when dry wells are placed in disturbed pervious areas (e.g., lawns, parks, and community open spaces).

#### 4.7.5.8 CONSTRUCTION CONSIDERATIONS

To help ensure that dry wells are successfully installed on a development site, site planning and design teams should consider the following recommendations:

- If dry wells will be used to receive non-rooftop runoff, they should only be installed after their contributing drainage areas have been completely stabilized. To help prevent dry well failure, stormwater runoff may be diverted around the dry well until the contributing drainage area has been stabilized.
- To help prevent soil compaction, heavy vehicular and foot traffic should be kept out of dry wells before, during, and immediately after construction. This can typically be accomplished by clearly delineating dry wells on all development plans and, if necessary, protecting them with temporary construction fencing.
- Excavation for dry wells should be limited to the width and depth specified in the development plans. Excavated material should be placed away from the excavation so as not to jeopardize the stability of the side walls.
- The native soils along the bottom of the dry well should be scarified or tilled to a depth of 3 to 4 inches prior to the placement of the choker stone and dry well stone.
- The sides of all excavations should be trimmed of large roots that will hamper the installation of the permeable filter fabric used to line the sides and top of the dry well.

### 4.7.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of a dry well.**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a dry well. Create a rough layout of the dry wells dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the dry well.**

Consider whether the dry well is intended to:

- Meet a runoff reduction* target or water quality (treatment) target. For information on the sizing of a BMP utilizing the runoff reduction approach, see Step 3A. For information on the sizing of the BMP utilizing the water quality treatment approach, see Step 4A. **Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*
- Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)
- Provide a possible solution to a drainage problem
- Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3A, 3B, and 3C for a runoff reduction approach, or skip Step 3 and complete Steps 4A and 4B for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) **Calculate the Stormwater Runoff Reduction Target Volume.**

Calculate the Runoff Reduction Volume using the following formula:

$$RR_V = (P)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$RR_V$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the practice:

Using the $RR_V$ calculated above, determine the minimum Volume of the Practice (VP)

$$(VP_{MIN}) \geq RR_v \text{ (target) / (RR\%)}$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice
**$VP_{MIN}$** = Minimum storage volume required to provide Runoff Reduction Target Volume ($ft^3$)
**$RR_v$ (target)** = Runoff Reduction Target Volume ($ft^3$)

**(Step 3B) Determine the storage volume of the practice and the Pretreatment Volume**

To determine the actual volume provided in the dry well, use the following equation:

$$VP = (PV + VES\ (N))$$

*Where*:

**VP** = Volume provided (temporary storage)
**PV** = Ponding Volume
**VES** = Volume of Engineered Soils
**N** = Porosity

To determine the porosity, a qualified licensed professional should be consulted to determine the proper porosity based on the engineered soils used. Most soil media has a porosity of 0.25 and gravel a value of 0.40.

Splash blocks or level spreaders should be considered to dissipate concentrated stormwater runoff at the inlet and prevent scour.

**(Step 3C) Determine whether the minimum storage volume was met.**

When the VP ≥ $VP_{MIN}$, then the Runoff Reduction requirements are met for this practice. Proceed to Step 5.

When the VP < $VP_{MIN}$, then the BMP must be sized according to the $WQ_V$ treatment method (See Step 4).

**(Step 4A) Calculate the Target Water Quality Volume.**

Calculate the Water Quality Volume using the following formula:

$$WQ_V = (1.2)\ (R_V)\ (A) / 12$$

*Where*:

**$WQ_V$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)
**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4B) **If using the practice for Water Quality treatment, determine the footprint of the dry well practice.**

The peak rate of discharge for the water quality design storm is needed for sizing of off-line diversion structures (see Subsection 3.1.7). If designing off-line, follow steps (a) through (d) below:

(a) Using $WQ_v$, compute CN.
(b) Compute time of concentration using the TR-55 method.
(c) Determine appropriate unit peak discharge from time of concentration.
(d) Compute $Q_{wq}$ from unit peak discharge, drainage area, and $WQ_v$.

To determine the minimum surface area of the dry well, use the following formula:

$$A_f = (WQ_v)\ (d_f)\ /\ [\ (k)\ (h_f + d_f)\ (t_f)]$$

*Where*:

**$A_f$** = surface area of ponding area ($ft^2$)
**$WQ_v$** = water quality volume ($ft^3$)
**$d_f$** = rock depth (ft)
**k** = coefficient of permeability of rock (ft/day)
**$h_f$** = average height of water above dry well bed (ft)
**$t_f$** = design rock drain time (days) (1 day is the recommended maximum)

(Step 5) **Calculate the adjusted curve numbers for $CP_V$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm). See Subsection 3.1.7.5 for more information.**

(Step 6) **Size the flow diversion structure, if needed.**

If the contributing drainage area to dry well exceeds the water quality treatment and/or storage capacity, flow regulator (or flow splitter diversion structure) should be supplied to divert the $WQ_v$ (or $RR_v$) to the dry well.

Size low-flow orifice, weir, or other device to pass $Q_{wq}$.

(Step 7) **Design emergency overflow facilities.**

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, 24-hour storm event.

(Step 8) **Prepare a site Vegetation and Landscaping Plan.**

A landscaping plan for the dry well should be prepared to indicate how it will be established with vegetation.

See Subsection 4.7.5.7 (*Landscaping*) and Appendix D for more details.

### 4.7.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.8 Dry Enhanced Swales/Wet Enhanced Swales

**Description**: Vegetated open channels that are explicitly designed and constructed to capture and treat stormwater runoff within dry or wet cells formed by check dams or other means.

**LID/GI Consideration**: Adaptable to many linear situations, and often a small BMP used to treat runoff close to the source.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Longitudinal slopes must be less than 4%
- Bottom width of 2 to 8 feet
- Side slopes 2:1 or flatter; 4:1 recommended
- Convey the 25-year storm event with a minimum of 6 inches of freeboard

### ADVANTAGES / BENEFITS

- Combines stormwater treatment with runoff conveyance system
- Less expensive than curb and gutter
- Reduces runoff velocity

### DISADVANTAGES / LIMITATIONS

- Higher maintenance than curb and gutter systems
- Cannot be used on steep slopes
- Possible resuspension of sediment
- Potential for odor / mosquitoes (wet swale)

### MAINTENANCE REQUIREMENTS

- Maintain grass heights of approximately 4 to 6 inches (dry swale)
- Remove sediment from forebay and channel

### POLLUTANT REMOVAL (DRY SWALE)

- 80% **Total Suspended Solids**
- 40% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 50/50% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- N/A **Pathogens** – Fecal Coliform

### POLLUTANT REMOVAL (WET SWALE)

- 80% **Total Suspended Solids**
- 20% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 25/40% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- N/A **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- ✓ Runoff Reduction
- ✓ Water Quality
- ★ Channel Protection
- Overbank Flood Protection
- Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- M Land Requirement
- M Capital Cost
- L Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *No*
**Drainage Area:** *5 acres max*

**Soils:** *No restrictions*

**Other Considerations:**

- Permeable soil layer (dry swale)
- Wetland plants (wet swale)

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- Dry Swale: 100% of the runoff reduction volume provided (no underdrain)
- Dry Swale: 50% of the runoff reduction volume provided (underdrain)
- Wet Swale: 0% of the runoff reduction volume provided

BACK TO TOC

### 4.8.1 General Description

Enhanced swales (also referred to as bioswales, *vegetated open channels*, or *water quality swales*) are conveyance channels engineered to capture and treat the water quality volume ($WQ_v$) for a drainage area. They differ from a normal drainage channel or swale through the incorporation of specific features that enhance stormwater pollutant removal effectiveness.

Enhanced swales are designed with limited longitudinal slopes to force the flow to be slow and shallow, thus allowing for particulates to settle and limiting the effects of erosion. Berms and/or check dams installed perpendicular to the flow path promote settling and infiltration.

There are two primary enhanced swale designs, the *dry swale* and the *wet swale* (or *wetland channel*). Below are descriptions of these two designs:

- Dry Swale – The dry swale is a vegetated conveyance channel designed to include a filter bed of prepared soil that may overlay an underdrain system. Dry swales are sized to allow the entire WQv to be filtered or infiltrated through the bottom of the swale. Because they are dry most of the time, they are often the preferred option in residential settings.
- Wet Swale – The wet swale is a vegetated channel designed to retain water or marshy conditions that support wetland vegetation. A high water table or poorly drained soils are necessary to retain water. The wet swale essentially acts as a linear shallow wetland treatment system, where the $WQ_v$ is retained.

Dry and wet swales are not to be confused with a filter strip or grass channel, which are not considered acceptable for meeting the TSS removal performance goal by themselves. Ordinary grass channels are not engineered to provide the same treatment capability as a well-designed dry swale with filter media. Filter strips are designed to accommodate overland flow rather than channelized flow and can be used as stormwater credits to help reduce the total water quality treatment volume for a site. Both of these practices may be used for pretreatment or included in a "treatment train" approach where redundant treatment is provided. Please see a further discussion of these types of BMPs in Sections 4.28 and 4.9, respectively.

Enhanced Dry Swale

Enhanced Wet Swale

**Figure 4.8-1** Enhanced Swale Examples

## 4.8.2 Stormwater Management Suitability

Enhanced swale systems are designed primarily for stormwater quality and have only a limited ability to provide channel protection or to convey higher flows to other controls.

- Runoff Reduction
  Dry swales, with no underdrain, can be designed to provide 100% of the runoff reduction volume, if properly maintained. In order to provide runoff reduction for a dry enhanced swale that is designed without an underdrain, a soils test or other reliable resource must indicate that the ponding area of the dry swale will drain within 24 – 48 hours. A dry swale area can also be designed with an underdrain to provide 50% of the runoff reduction volume, if properly maintained. Wet enhanced swales do not provide runoff reduction volume.

- Water Quality
  Dry swale systems rely primarily on filtration through an engineered media to provide removal of stormwater contaminants. Wet swales achieve pollutant removal both from sediment removal and biological removal. Subsection 4.8.3 provides median pollutant removal efficiencies that can be used for planning and design purposes.

- 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. 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. 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.

## 4.8.3 Pollutant Removal Capabilities

Both the dry and wet enhanced swale are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed swales can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- *Total Suspended Solids* – 80%
- *Total Phosphorus* – Dry Swale 50% / Wet Swale 25%
- *Total Nitrogen* – Dry Swale 50% / Wet Swale 40%
- *Fecal Coliform* – insufficient data
- *Heavy Metals* – Dry Swale 40% / Wet Swale 20%

For additional information and data on pollutant removal capabilities for enhanced dry and wet swales, see the National Pollutant Removal Performance Database (Version 3) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.8.4 Application and Feasibility Criteria

Enhanced swales can be used in a variety of development types; however, they are primarily applicable to residential and institutional areas of low to moderate density where the impervious cover in the contributing drainage area is relatively small, and along roads and highways. Dry swales are mainly used in moderate to large lot residential developments, small impervious areas (parking lots and rooftops), and along rural highways. Wet swales tend to be used for highway runoff applications, small parking areas, and in commercial developments as part of a landscaped area.

Because of their relatively large land requirement, enhanced swales are generally not used in higher density areas. In addition, wet swales may not be desirable for some residential applications, due to the presence of standing and stagnant water, which may create nuisance odor or mosquito problems.

The topography and soils of a site will determine the applicability of the use of one of the two enhanced swale designs. Overall, the topography should allow for the design of a swale with sufficient slope and cross-sectional area to maintain non-erosive velocities. The following criteria should be evaluated to ensure the suitability of an enhanced swale for meeting stormwater management objectives on a site or development.

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **NO**
- Regional Stormwater Control – **NO**

**Physical Feasibility -** Physical Constraints at Project Site

- **Drainage Area** – 5 acres maximum
- **Space Required** – Approximately 10 to 20% of the tributary impervious area
- **Site Slope** – Typically no more than 4% channel slope
- **Minimum Head** – Elevation difference needed at a site from the inflow to the outflow: 3 to 5 feet for dry swale; 1 foot for wet swale
- **Minimum Depth to Water Table** – 2 feet required between the bottom of a dry swale and the elevation of the seasonally high water table, if an aquifer or treating a hotspot; wet swale is below water table or placed in poorly drained soils
- **Soils** – Engineered media for dry swale

**Other Constraints / Considerations**

- **Aquifer Protection** – Exfiltration should not be allowed in hotspot areas

### 4.8.5 Planning and Design Criteria

*The following criteria are to be considered minimum standards for the design of an enhanced swale system. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

#### 4.8.5.1 LOCATION AND LAYOUT

- A dry or wet swale should be sited such that the topography allows for the design of a channel with sufficiently mild slope (unless small drop structures are used) and cross-sectional area to maintain non-erosive velocities.
- Enhanced swale systems should have a contributing drainage area of 5 acres or less.
- Swale siting should also take into account the location and use of other site features, such as buffers and undisturbed natural areas, and should attempt to aesthetically "fit" the facility into the landscape.
- 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.

#### 4.8.5.2 GENERAL DESIGN

Both types of enhanced swales are designed to treat the WQv through a volume-based design, and to safely pass larger storm flows. Flow enters the channel through a pretreatment forebay. Runoff can also enter along the sides of the channel as sheet flow through a flow spreader, such as a pea gravel trench along the top of the bank.

- Dry Swale
  A dry swale system consists of an open conveyance channel with a filter bed of permeable soils that may overlay an underdrain system. Flow passes into and is detained in the main portion of the channel where it is filtered through the soil bed. Runoff may be collected and conveyed by a perforated pipe and gravel underdrain system to the outlet. **Figure 4.8-2** provides a plan view and profile schematic for the design of a dry swale system.

- Wet Swale
  A wet swale or wetland channel consists of an open conveyance channel which has been excavated to the water table or to poorly drained soils. Check dams are used to create multiple wetland "cells," which act as miniature shallow marshes. **Figure 4.8-3** provides a plan view and profile schematic for the design of a wet swale system.

#### 4.8.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- Channel slopes between 1% and 2% are recommended unless topography necessitates a steeper slope, in which case 6- to 12-inch drop structures can be placed to limit the energy slope to within the recommended 1 to 2% range. Energy dissipation will be required below the drops. Spacing between the drops should not be closer than 50 feet. Depth of the storage volume at the downstream end should not exceed 18 inches.

- Dry and wet swales should have a bottom width of 2 to 8 feet to ensure adequate filtration. Wider channels can be designed, but should contain berms, walls, or a multi-level cross section to prevent channel braiding or uncontrolled sub-channel formation.

- Dry and wet swales are parabolic or trapezoidal in cross-section and are typically designed with moderate side slopes no greater than 2:1 for ease of maintenance and side inflow by sheet flow (4:1 or flatter recommended).

- Dry and wet swales should maintain a maximum $WQ_v$ ponding depth of 18 inches at the end point of the channel. A 12-inch average depth should be maintained.

- The peak velocity for the 2-year storm must be non-erosive for the soil and vegetative cover provided.

- If the system is on-line, channels should be sized to convey runoff from the overbank flood event ($Q_{p25}$) safely with a minimum of 6 inches of freeboard and without damage to adjacent property.

Dry Swale

- Dry swale channels are sized to store and infiltrate the entire water quality volume ($WQ_v$) with less than 18 inches of ponding and allow for full filtering through the permeable soil layer. The maximum ponding time is 48 hours, though a 24-hour ponding time is more desirable.

- The dry swale consists of a permeable soil layer of at least 30 inches in depth, above an underdrain. The soil media should have an infiltration rate of at least 1 foot per day (1.5 feet per day maximum) and contain a high level of organic material to facilitate pollutant removal. A permeable filter fabric is placed between the gravel layer and the overlying soil. Where an underdrain collection system is utilized, it should be equipped with at least a 4-inch diameter perforated PVC pipe (AASHTO M 252) longitudinal underdrain in a gravel layer.

- The channel and underdrain excavation should be limited to the width and depth specified in the design. The bottom of the excavated trench shall not be loaded in a way that causes soil compaction, and shall be scarified prior to placement of gravel and permeable soil. The sides of the channel shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling.

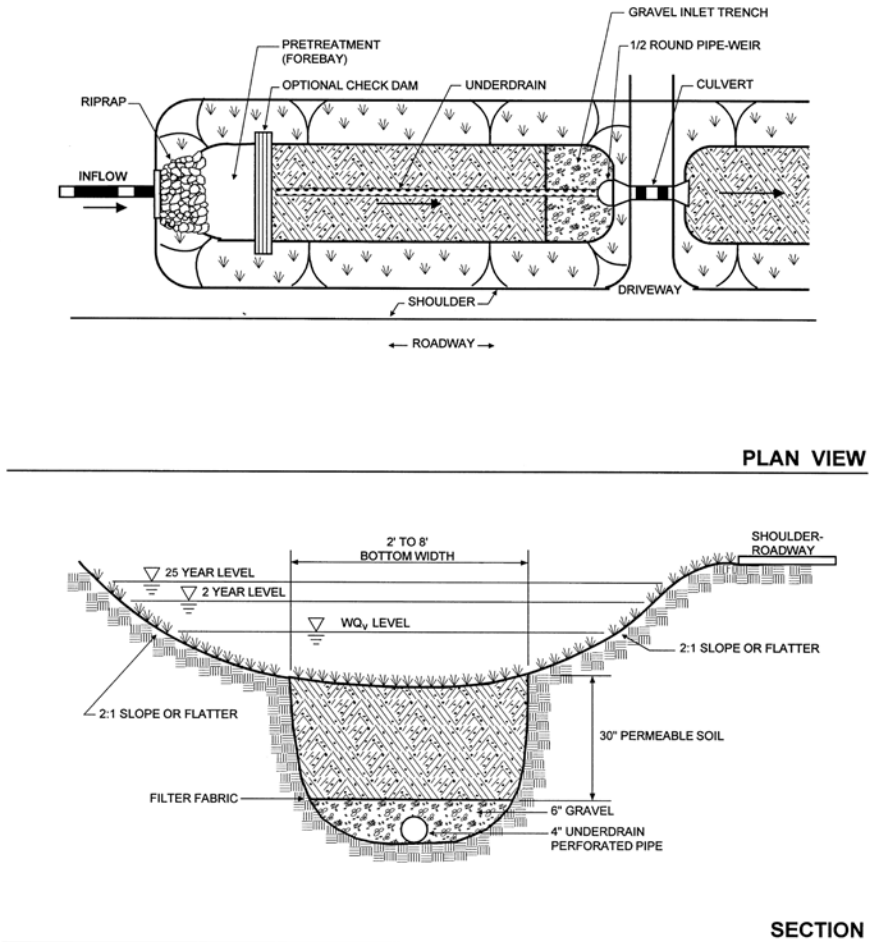



Figure 4.8-2 Schematic of Dry Swale
(Source: Center for Watershed Protection)

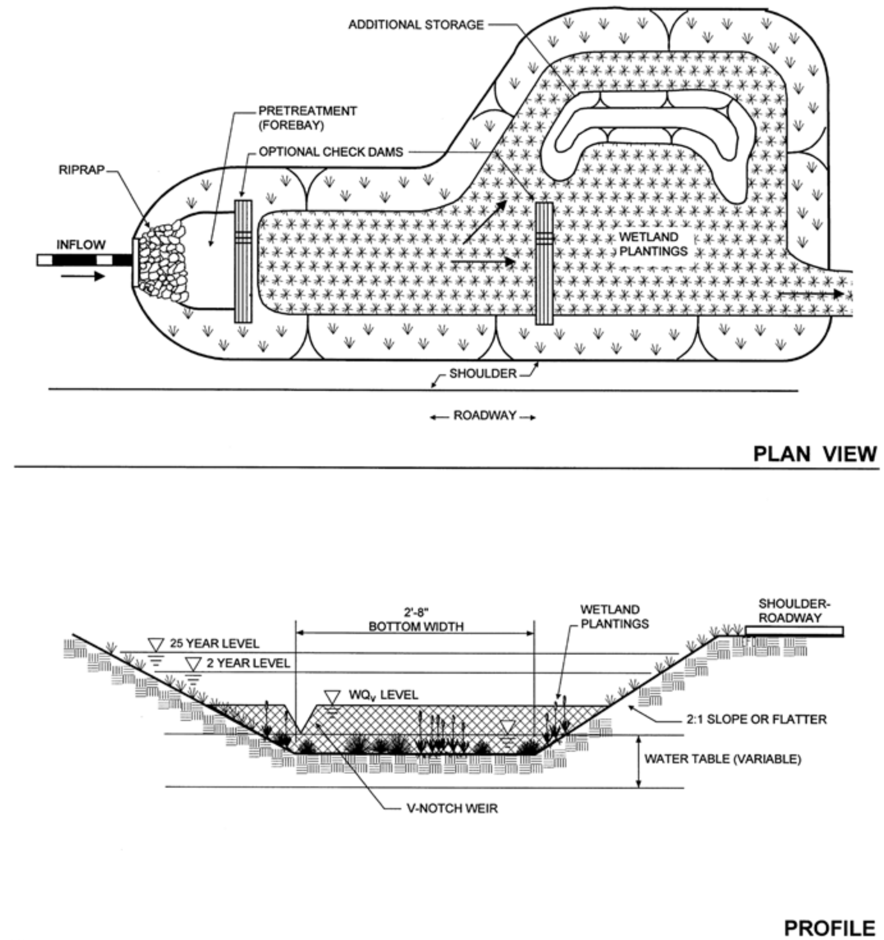



Figure 4.8-3 Schematic of Wet Swale
(Source: Center for Watershed Protection)

BACK TO TOC

#### Wet Swale

- Wet swale channels are sized to retain the entire water quality volume ($WQ_v$) with less than 18 inches of ponding at the maximum depth point.
- Check dams can be used to achieve multiple wetland cells. V-notch weirs in the check dams can be utilized to direct low flow volumes.

### 4.8.5.4 PRETREATMENT/INLETS

- Inlets to enhanced swales must be provided with energy dissipators such as riprap.
- Pretreatment of runoff in both a dry and wet swale system is typically provided by a sediment forebay located at the inlet. The pretreatment volume should be equal to 0.1 inches per impervious acre. This storage is usually obtained by providing check dams at pipe inlets and/or driveway crossings.
- Enhanced swale systems that receive direct concentrated runoff may have a 6-inch drop to a pea gravel diaphragm flow spreader at the upstream end of the control.
- Vegetated filter strips and gentle side slopes should be provided along the top of channels to provide pretreatment for lateral sheet flows.

### 4.8.5.5 OUTLET STRUCTURES

#### Dry Swale

Outlet protection must be used at any discharge point from a dry swale to prevent scour and downstream erosion. The underdrain system should discharge to the storm drainage infrastructure or a stable outfall.

#### Wet Swale

Outlet protection must be used at any discharge point from a wet swale to prevent scour and downstream erosion.

### 4.8.5.6 EMERGENCY SPILLWAY

Enhanced swales must be adequately designed to safely pass flows that exceed the design storm flows.

### 4.8.5.7 MAINTENANCE ACCESS

Adequate access should be provided for all dry and wet swale systems for inspection and maintenance.

### 4.8.5.8 SAFETY FEATURES

Ponding depths should be limited to a maximum of 18 inches.

### 4.8.5.9 LANDSCAPING

Landscape design should specify proper grass species and wetland plants based on specific site, soils and hydric conditions present along the channel. **Table 4.8-1** below provides a number of grass species that perform well in the stressful environment of an open channel BMP. In addition, wet swales may include other wetland species (see plant list in Section 5 of Appendix D). Select plant material capable of salt tolerance in areas that may include high salt levels.

Table 4.8-1: Common Grass Species for Dry and Wet Enhanced Swales

| Common Name | Scientific Name | Notes |
|---|---|---|
| Bermuda Grass | Cynodon dactylon | |
| Big Bluestem | Andropogon gerardii | Not for wet swales |
| Creeping Bentgrass | Agrostis palustris | |
| Red Fescue | Festuca rubra | Not for wet swales |
| Reed Canary Grass | Phalaris arundinacea | Wet swales |
| Redtop | Agrostis alba | |
| Smooth Brome | Bromus inermis | Not for wet swales |
| Switch Grass | Panicum virgatum | |

Note 1: These grasses are sod-forming and can withstand frequent inundation, and are thus ideal for swale or channel environments. Most are salt-tolerant as well.
Note 2: Where possible, one or more of these grasses should be in the seed mixes.

BACK TO TOC

The following information is specific guidance for wet swales:

**Wet Swale**

- Emergent vegetation should be planted, or wetland soils may be spread on the swale bottom for seed stock.
- Where wet swales do not intercept the groundwater table, a water balance calculation should be performed to ensure an adequate water budget to support the specified wetland species. See Subsection 3.1.8 for guidance on water balance calculations.

### 4.8.5.10 ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

**Physiographic Factors -** Local terrain design constraints

- **Low Relief** – Reduced need for use of check dams
- **High Relief** – Often infeasible if slopes are greater than 4%
- **Karst** – No exfiltration of hotspot runoff from dry swales; use impermeable liner

**Soils**

No additional criteria

**Special Downstream Watershed Considerations**

- **Aquifer Protection** – No exfiltration of hotspot runoff from dry swales; use impermeable liner.

### 4.8.5.11 CONSTRUCTION CONSIDERATIONS

- Construction equipment should be restricted from the enhanced swale area to prevent compaction of the native soils.
- A dense and vigorous vegetative cover should be established over the contributing pervious drainage areas before runoff can be accepted into the facility. Otherwise the sediment from the stormwater runoff will clog the pores in the planting media and native soils.

## 4.8.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of an enhanced swale.**
Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for an enhanced swale. Create a rough layout of the enhanced swale dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the enhanced swale and if a dry or wet swale is desired.**
Consider whether the enhanced swale is intended to:

- Meet a runoff reduction* target or water quality (treatment) target. For information on the sizing of a BMP utilizing the runoff reduction approach, see Step 3A. For information on the sizing of the BMP utilizing the water quality treatment approach, see Step 4.
**Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*
- Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)
- Provide a possible solution to a drainage problem
- Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.).

*Complete Step 3A, 3B, and 3C for a runoff reduction approach, or skip Step 3 and complete Step 4 for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) **Calculate the Stormwater Runoff Reduction Target Volume**
Calculate the Runoff Reduction Volume using the following formula:

$$RR_V = (P)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$RR_V$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the practice:

Using the $RR_V$ calculated above, determine the minimum Volume of the Practice (VP)

$$(VP_{MIN}) \geq RR_v \text{ (target)} / (RR\%)$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice
**$VP_{MIN}$** = Minimum storage volume required to provide Runoff Reduction Target Volume ($ft^3$)
**$RR_v$ (target)** = Runoff Reduction Target Volume ($ft^3$)

(Step 3B) Determine the storage volume of the practice and the Pretreatment Volume

To determine the actual volume provided in the enhanced swale, use the following equation:

$$VP = (PV + VES (N))$$

*Where*:

**VP** = Volume provided (temporary storage)
**PV** = Ponding Volume
**VES** = Volume of Engineered Soils
**N** = Porosity

To determine the porosity, a qualified licensed professional should be consulted to determine the proper porosity based on the engineered soils used. Most soil media has a porosity of 0.25 and gravel a value of 0.40.

Provide pretreatment by using a grass filter strip or pea gravel diaphragm, 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. Pre-treatment may also be desired to reduce flow velocities or 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. Refer to Section 4.9 for design criteria for a grass channel and Section 4.29 for vegetated filter strips.

(Step 3C) Determine whether the minimum storage volume was met.

When the $VP \geq VP_{MIN}$, then the Runoff Reduction requirements are met for this practice. Proceed to Step 5.

When the $VP < VP_{MIN}$, then the BMP must be sized according to the $WQ_V$ treatment method (See Step 4).

(Step 4) **Calculate the Target Water Quality Volume**

Calculate the Water Quality Volume using the following formula:

$$WQ_V = (1.2)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$WQ_V$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Site area (total drainage area) ($ft^2$)
**12** = Unit conversion factor (in/ft)

Using the $WQ_V$ calculated above, determine the actual size and Volume of the Practice (VP) as shown in Step 3B. Note that VP, calculated using the formula shown in Step 3B, should be greater than or equal to $WQ_V$.

(Step 5) **If applicable, calculate the adjusted curve numbers for $CP_V$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm). See Subsection 3.1.7.5 for more information.**

(Step 6) **Determine the length and channel base width of the enhanced swale practice and the Pretreatment Volume required.**

*Determine Swale Dimensions*:

Size bottom width, depth, length, and slope necessary to store VP, with less than 18 inches of ponding at the down-stream end.

» Slope cannot exceed 4% (1 to 2% recommended)

» Bottom width should range from 2 to 8 feet

» Ensure that side slopes are no greater than 2:1 (4:1 recommended)

See Subsection 4.8.5.3 (*Physical Specifications / Geometry*) for more details

(Step 7) **Compute number of check dams (or similar structures) required to detain the $RR_V$ or $WQ_V$, as applicable**

(Step 8) **Calculate draw-down time**

*Dry swale*: Planting soil should pass a maximum rate of 1.5 feet in 24 hours and must completely filter the ponded volume within 48 hours (24 hours preferred).

*Wet swale*: Must hold the $WQ_V$.

(Step 9) **Check 2-year and 25-year velocity erosion potential and freeboard**

Check for erosive velocities and modify design as appropriate. Provide 6 inches of freeboard.

(Step 10) **Design overflow weir or orifice at downstream berm, head-wall, or checkdam.**

(Step 11) Size flow diversion structure, if needed

A flow regulator (or flow splitter diversion structure) should be supplied to divert the $WQ_v$ (or $RR_v$) to the best management practice.

Size low flow orifice, weir, or other device to pass $Q_{wq}$.

(Step 12) Size underdrain system

See Subsection 4.2.5.3 (*Physical Specifications/Geometry*)

(Step 13) Design 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.

(Step 14) Design inlets, sediment forebay(s), and underdrain system (dry swale)

See Subsection 4.8.5.4 through 4.8.5.8 for more details.

(Step 15) Prepare Vegetation and Landscaping Plan

A landscaping plan for a dry or wet swale should be prepared to indicate how the enhanced swale system will be stabilized and established with vegetation.

See Subsection 4.8.5.9 (*Landscaping*) and Appendix D for more details.

See Appendix B-5 for an Enhanced Swale Design Example

## 4.8.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

*Regular inspection and maintenance is critical to the effective operation of an enhanced swale as designed. Maintenance responsibility for an enhanced swale should be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.*

# 4.9 Grass Channel

**Description**: Grass channels designed to enhance water quality through the settling of suspended solids.

**LID/GI Considerations**: If properly incorporated into overall site design, grass channels can help to reduce the impacts of impervious cover and partially infiltrate runoff with pervious soils. Grass channels can complement the natural landscape while providing aesthetic benefits.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Can be used as part of the runoff conveyance system to provide pretreatment
- Can act to partially infiltrate runoff from small storm events if underlying soils are pervious
- Less expensive than curb and gutter systems
- Should not be used on slopes greater than 4% (1-2% slopes recommended)
- Potential for bottom erosion and sediment resuspension
- Standing water may not be acceptable in some areas
- Contributing drainage area less than 5 acres
- Minimum residence time of 5 minutes
- Water quality rainfall event flow velocity less than 1.0 ft/s and flow depth less than 4 inches
- Side slopes are 3:1 or flatter
- Minimum soil infiltration rate of 0.25 in/hr
- Minimum 2-foot clearance from groundwater table

### ADVANTAGES / BENEFITS

- Lower cost
- Reduces runoff from impervious areas
- Ideal for linear environments (along roadways)
- Stormwater collection and conveyance
- Aesthetic benefits
- Well suited for linear environments, interchanges, and facilities
- May be contained within the roadway right-of-way

### DISADVANTAGES / LIMITATIONS

- Cannot achieve the 80% TSS removal target alone; must be used in series with other BMPs for removal credit
- Limitations for drainage area, flow, velocity, and flow depth
- Design dependent on existing site conditions and topography

### ROUTINE MAINTENANCE REQUIREMENTS

- Provides access to BMP and appropriate components
- Sediment cleanout, trash and debris removal, revegetation, and repair of erosion must be completed as necessary to maintain functionality

### POLLUTANT REMOVAL

- 50% **Total Suspended Solids**
- 30% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 25/20% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- N/A **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- ✓ Runoff Reduction
- ★ Water Quality
- ★ Channel Protection
- ● Overbank Flood Protection
- ● Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- L Land Requirement
- L Capital Cost
- L Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *Yes*
**Other Considerations:** Curb and gutter replacement

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 25% of the $RR_V$ conveyed to the practice (A & B hydrologic soils)
- 10% of the $RR_V$ conveyed to the practice (C & D hydrologic soils)

### 4.9.1 General Description

Grass channels, also termed "biofilters," settle suspended solids and other pollutants through filtration, infiltration, and biofiltration. Grass channels are also able to help meet runoff velocity targets for the water-quality design storm of small drainage areas. Vegetative practices offer a method to manage pollution while conveying stormwater runoff. Grass channels are well-suited to a number of applications and land uses, including treating runoff from roads and other impervious surfaces. They can also be used in a variety of ways including, but not limited to, a single BMP, a pretreatment to another BMP, or in a "treatment train".

Grass channels differ from enhanced dry swales in that they do not have an engineered filter media to enhance pollutant removal capabilities and therefore have a lower pollutant removal rate than dry enhanced swales. Grass channels are different from conventional roadside ditches in that they are designed for water quality purposes and help increase residence time while decreasing velocities. When properly incorporated into an overall site design, grass channels can reduce impervious cover, accent the natural landscape, and provide aesthetic benefits while removing pollutants.

When designing a grass channel, the two primary considerations are channel capacity and minimizing erosion. Runoff velocity should not exceed 1.0 foot per second during the peak discharge associated with the water quality design rainfall event, and the total length of a grass channel should provide at least 5 minutes of residence time. To enhance water quality treatment, grass channels must have broader bottoms, lower slopes and denser vegetation than most drainage channels. Additional treatment can be provided by placing check-dams across the channel below pipe inflows, and at various other points along the channel.

### 4.9.2 Stormwater Management Suitability

Grass channels, a form of 'biofilter,' are trapezoidal or parabolic shaped vegetated channels that work to enhance water quality through the settling of suspended solids through filtration, infiltration, and biofiltration.

- Runoff Reduction
  Grass channels are an effective low impact development (LID) practices that can be used in Georgia to reduce post-construction stormwater runoff and improve water quality. Like other LID practices, they become more effective with higher infiltration rates of 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 channel 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.

- 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 facility is typically designed to completely drain within 48-72 hours, the 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 grass channel, another practice must be used to provide $CP_v$ extended detention.

- Overbank Flood Protection
  Another practice used in conjunction with a grass channel will likely be required to reduce the post-development peak flow of the 25-year, 24-hour storm ($Q_p$) 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.

Credit for the volume of runoff reduced in the grass channel may be taken in the overbank flood protection and extreme flood protection calculations. If the practice is designed to provide Runoff Reduction for Water Quality compliance, then the practice is given credit for Channel Protection and Flood Control requirements by allowing the designer to compute an Adjusted CN (see Subsection 3.1.7.5 for more information).

### 4.9.3 Pollutant Removal Capabilities

Pollutant removal from grass channels is highly variable and depends primarily on the density of vegetation and contact time for filtration and infiltration. These, in turn, depend on soil and vegetation type, slope, and contact time. Research on fecal coliform removal has been inconclusive, but suggests that grass channels are generally not considered to be effective BMPs for treating bacterial loads.

For additional information and data on pollutant removal capabilities for grass channels, see the National Pollutant Removal Performance Database (3rd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.9.4 Application and Site Feasibility Criteria

Grass channels are best suited to treating smaller drainage areas of 5 acres or less. Flow must enter the practice as sheet flow spread out over the width (long dimension normal to flow) of the channel. For longer flow paths, special provision must be made to ensure design flows spread evenly across the filter strip.

Grass channels should have a side slope of 3:1 or flatter. The maximum flow depth through the channel should be no more than 4 inches. The base width of the channel should be 2-6 feet to ensure a sufficient filtering surface for water quality treatment, and the maximum width prevents braiding, which is the formation of small channels within the channel bottom. The bottom width is a dependent variable in the calculation of velocity based on Manning's equation. If a larger channel is needed, the use of a compound cross section is recommended. The depth from the bottom of the channel to the groundwater should be at least 2 feet to prevent a moist channel bottom, or contamination of the groundwater.

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **YES**
- Regional Stormwater Control – **NO**

**Physical Feasibility -** Physical Constraints at Project Site

- **Drainage Area** – 5 acres or less. 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% recommended.
- **Base Width** - Between 2-6 feet.
- **Minimum Depth to Water Table** – A separation distance of 1 foot is recommended between the bottom of the vegetated filter strip and the elevation of the seasonally high water table.
- **Runoff Velocities** – Most not be erosive. Maximum flow velocity of 1.0 ft/s or less is recommended.

**Other Constraints / Considerations**

**Location Requirements** – The following is a list of specific setback requirements for the location of a vegetated filter strip:

- Grass channels should not be used on soils that cannot sustain a dense grass cover with high retardance. Designers should choose a grass that can withstand relatively high velocity flows at the entrances of the BMP and both wet and dry periods. See Appendix D for a list of appropriate grasses for use in Georgia.
- Grass channels can be used on most soils with some restrictions on the most impermeable soils. Grass channels should not be used on soils with infiltration rates less than 0.25 inches per hour if infiltration of small runoff flows is intended.
- A grass channel should accommodate the peak flow for the water quality design storm $Q_{wq}$ (see Subsection 3.1.7).
- Incorporation of check dams within the channel will increase retention time.

- A 5-minute residence time is recommended for the water quality peak flow. Residence time may be increased by reducing the slope of the channel, increasing the wetted perimeter, or planting a denser grass (raising the Manning's n).

See Section 5.4 (*Open Channel Design*) for more information and specifications on the design of grass channels.

**Grass Channels for Pretreatment**

A number of other BMPs, including bioretention areas and infiltration trenches, may utilize a grass channel as a pretreatment measure. The length of the grass channel depends on the drainage area, land use, and channel slope. **Table 4.9.1** provides sizing guidance for grass channels for a 1-acre drainage area. The minimum grassed channel length should be 20 feet.

## 4.9.5 Planning and Design Criteria

Before designing the grass channel, the following data is necessary:

- Existing and proposed site, topographic and location maps, and field reviews.
- Impervious and pervious areas. Other means may be used to determine the land use data.
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site.
- Design data from nearby storm sewer structure(s).

Table 4.9-1: Grass Channel Sizing Guidance

Source: Claytor and Schueler, 1996

| Parameter | ≤ 33% Impervious | | Between 34% and 66% Impervious | | ≥ 67% Impervious | |
|---|---|---|---|---|---|---|
| Slope (max =4%) | < 2% | > 2% | < 2% | > 2% | < 2% | > 2% |
| Grass channel minimum length* (feet) *assumes 2-foot wide bottom width | 25 | 40 | 30 | 45 | 35 | 50 |

- Water surface elevation of nearby water systems as well as the depth to seasonally high groundwater.
- Infiltration testing of native soils at the proposed elevation of bottom of grass channel.
- Water surface elevation of nearby water systems as well as the depth to seasonally high groundwater.
- Infiltration testing of native soils at the proposed elevation of bottom of grass channel.

*The following criteria are to be considered minimum standards for the design of a grass channel. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.9.5.1 LOCATION AND LAYOUT

Grass channels vary based on site constraints such as proposed and existing infrastructure, soils, existing vegetation, contributing drainage area, and utilities. Grass channels are designed for intermittent flow and must be allowed to drain between rain fall events and should not be used on sites with a continuous flow from groundwater, sump pumps, or other sources. Grass channel locations should be integrated into the site planning process, with aesthetic and maintenance considerations taken into account in their siting and design.

## 4.9.6 Design Procedures

(Step 1) **Determine the goals and primary function of the grass channel**

Consider whether the grass channel is intended to:

- Meet water quality (treatment) target.
- Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$))
- Provide a possible solution to a drainage problem

Check with local officials and other agencies to determine if there are any additional watershed restrictions that may apply. In addition, consider if the grass channel has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the grass channel should be centered on the restrictions/requirements, goals, targets, and primary function(s) of a grass channel. By considering the primary function, as well as, topographic and soil conditions, the design elements of the grass channel can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

(Step 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 chapter. In addition, determine if site conditions are suitable for a grass channel. Create a rough layout of the grass channel dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 3) **Calculate the Target Water Quality Volume.**

Calculate the Water Quality Volume using the following formula:

$$WQ_v = (1.2)\ (R_v)\ (A)\ /\ 12$$

*Where*:

**$WQ_v$** = Water Quality Volume ($ft^3$)

**1.2** = Target rainfall amount to be treated (inches)

**$R_v$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)

**12** = Unit conversion factor (in/ft)

(Step 4) **Calculate the channel width based on an assumed channel flow depth:**

$$W = (n*Q)/(1.49*D^{5/3}*S^{1/2})$$

*Where*:

**W** = minimum bottom channel width (ft)

**n** = Manning's roughness coefficient

**Q** = peak runoff from the $WQ_v$ rain event (cfs). See Subsection 3.1.7.2.

**D** = flow depth (ft)

**S** = slope (%)

(Step 5) Calculate the minimum length (feet) of the grass channel using a 5-minute (300 seconds) residence time:

$$V = Q/(W*D)$$

$$L = V \times (300 \text{ seconds})$$

*Where*:

**L** = minimum length of channel (ft)
**V** = velocity through the channel (ft/s)

(Step 6) 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/sec.

(Step 7) Confirm the channel can pass the local design requirements with required freeboard.

(Step 8) Calculate the Stormwater Runoff Reduction Volume conveyed to the practice
Calculate the Runoff Reduction Volume using the following formula:

$$RR_v = (P)\,(R_v)\,(A) / 12$$

*Where*:

**$RR_v$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_v$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 9) Calculate $RR_v$ credited
Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the practice:

$$RR_v \text{ (credited)} = RR_v \text{ (RR\%)}$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice
**$RR_v$ (credited)** = Runoff Reduction Volume credited by this practice ($ft^3$)
**$RR_v$** = $RR_v$ conveyed to the practice

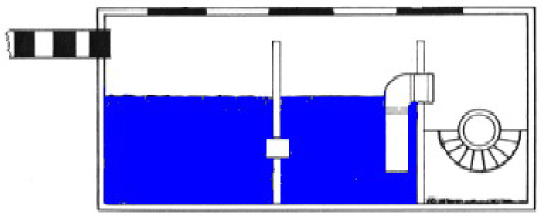
(Step 10) If Steps 5 and 6 are met, then downstream calculations can be performed with adjusted CN using the runoff reduction volume credited. Refer to Subsection 3.1.7.5.

(Step 11) Prepare Vegetation and Landscaping Plan
A landscaping plan for the grass channel should be prepared to indicate how it will be established with vegetation.
See Appendix D for more details.

### 4.9.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

## 4.10 Gravity (Oil-Grit) Separator



**Description**: An oil-grit separator is a device designed to remove suspended solids, oil, grease, debris, and floatables from stormwater runoff through gravitational settling, hydrodynamic separation, and trapping of pollutants. Oil-grit separators are also called gravity separators or oil-water separators.

**LID/GI Considerations**: Gravity oil-grit separators are not considered low impact development or green infrastructure. However, for ultra-urban development projects or stormwater retrofit designs, oil-grit separators may be one of few design options for removal of total suspended solids and/or pollutants from stormwater runoff.

### KEY CONSIDERATIONS

**DESIGN CRITERIA**

- Gravity separators are typically used for drainage areas of less than 5 acres.
- Limit the contributing drainage area to any individual gravity separator to have 1 acre or less of impervious cover.
- The total wet storage of the gravity separator unit should be at least 400 cubic feet per contributing impervious acre.
- The minimum depth of permanent pools should be 4 feet.
- Horizontal velocity through the separation chamber should be 1-3 ft/min or less.
- No velocities in the device should exceed the entrance velocity.
- A trash rack should be included in the design to capture floating debris, preferably near the inlet chamber.

**ADVANTAGES / BENEFITS**

- Well-suited for use on urban development sites, where larger or above-ground BMPs are not an option, or for stormwater retrofit projects
- Can be used as pretreatment for other BMPs
- Can replace a conventional junction or inlet structure
- Multiple inlets can connect to a single unit.
- Some designs require minimal drop between inlet and outlet.

**DISADVANTAGES / LIMITATIONS**

- Dissolved pollutants are not effectively removed by oil-grit separators.
- Oil-grit separators alone cannot achieve the 80% TSS removal target.
- Frequent maintenance is required.
- Performance is dependent on design and frequency of inspection and cleanout of unit.
- Some designs may require a confined space entry for inspection, maintenance, and repairs.

**ROUTINE MAINTENANCE REQUIREMENTS**

- Maintenance requirements for a proprietary system should be obtained from the manufacturer.
- Frequency of inspection and maintenance is dependent on land use, climate, and design of the gravity separator.
- Failure to provide adequate inspection and maintenance can result in the resuspension of accumulated solids.
- Proper disposal of oil, solids, and floatables removed from the gravity separator must be ensured.

**POLLUTANT REMOVAL**

- 40% **Total Suspended Solids**
- 5/5% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- N/A **Metals** - Cadmium, Copper, Lead, and Zinc removal
- N/A **Pathogens** – Fecal Coliform

**STORMWATER MANAGEMENT SUITABILITY**

- Runoff Reduction
- ★ Water Quality
- Channel Protection
- Overbank Flood Protection
- Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

**IMPLEMENTATION CONSIDERATIONS**

- L Land Requirement
- H Capital Cost
- H Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *Not recommended*

**Soils:** Gravity oil-grit separator systems can be installed in almost any soil or terrain.

**Other Considerations:** Install as an off-line device unless the separator can be sized to handle a small drainage area. Also, consider installing a manhole on the downstream side to provide easy access for sampling of effluent.

L=Low M=Moderate H=High

**RUNOFF REDUCTION CREDIT**

- 0% Runoff Reduction Credit is provided by this practice.

## 4.10.1 General Description

Gravity oil-grit separators are hydrodynamic separation devices that are designed to remove grit, heavy sediments, oil, grease, debris, and floatable matter from stormwater runoff through gravitational settling and trapping. Gravity separator units contain a permanent pool of water and typically consist of an inlet chamber, separation and storage chamber, a bypass chamber, and an access port for maintenance purposes (see Figure 4.10-1). Runoff enters the inlet chamber where heavy sediments and solids drop to the bottom. The flow moves into the main gravity separation chamber, where further settling of suspended solids takes place. Oil and grease are skimmed and stored in a waste oil storage compartment for future removal. After moving into the outlet chamber, the clarified runoff is then discharged.

The performance of these systems is based primarily on the relatively low solubility of petroleum products in water and the difference between their specific gravities. Gravity separators are not designed to separate other products, such as solvents, detergents, or dissolved pollutants. The typical gravity separator unit may be enhanced with a pretreatment swirl concentrator chamber, oil draw-off devices that continuously remove the accumulated light liquids, and flow control valves regulating the flow rate through the facility.

Gravity separators are often used in commercial, industrial, and transportation land uses and are intended primarily as a pretreatment measure for high-density or ultra-urban sites, or for use in hydrocarbon hotspots, such as gas stations and areas with high vehicular traffic. However, gravity separators cannot be used for the removal of dissolved or emulsified oils and pollutants such as coolants, soluble lubricants, glycols and alcohols.

Since resuspension of accumulated sediments is possible during heavy storm events, gravity separator units are typically installed off-line. Gravity separators are available as prefabricated proprietary systems from a number of different commercial vendors.

## 4.10.2 Stormwater Management Suitability

- Runoff Reduction
  Gravity oil-grit separators do not provide stormwater volume runoff reduction. Another BMP should be used in a treatment train with gravity oil-grit separators to provide runoff reduction. See Subsection 4.1.6 for more information about using BMPs in series.

- Water Quality
  If installed as per the recommended design criteria and properly maintained, 40% total suspended solids removal will be applied to the water quality volume ($WQ_v$) flowing to the gravity oil-grit separator. Another BMP should be used in a treatment train with gravity oil-grit separators to provide the additional required water quality treatment. (See Subsection 4.1.6.)

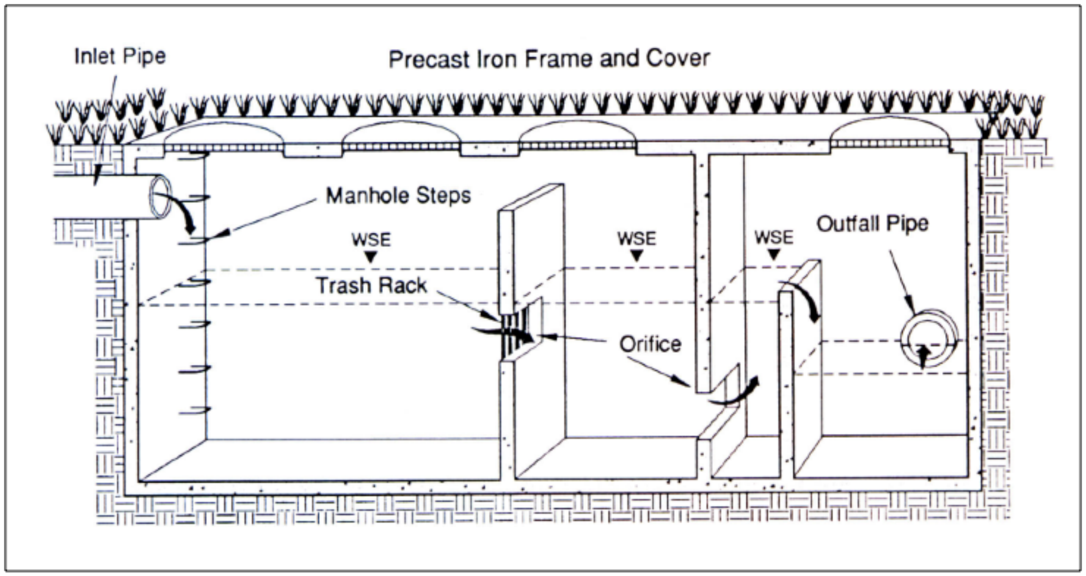



**Figure 4.10-1** Schematic of an Example Gravity (Oil-Grit) Separator
(Source: NVRC, 1992[1])



BACK TO TOC

- Channel Protection
  Gravity oil-grit separators do not provide channel protection. Another BMP should be used in a treatment train with gravity oil-grit separators to provide runoff reduction. (See Subsection 4.1.6.) Additionally, the gravity oil-grit separator should be designed off-line or with a bypass for higher flows.

- Overbank Flood Protection
  Gravity oil-grit separators do not provide overbank flood protection. Another BMP should be used in a treatment train with gravity oil-grit separators to provide runoff reduction. Additionally, the gravity oil-grit separator should be designed off-line or with a bypass for higher flows. (See Subsection 3.1.5.)

- Extreme Flood Protection
  Gravity oil-grit separators do not provide extreme flood protection. Another BMP should be used in a treatment train with gravity oil-grit separators to provide runoff reduction. Additionally, the gravity oil-grit separator should be designed off-line or with a bypass for higher flows. (See Subsection 3.1.5.)

## 4.10.3 Pollutant Removal Capabilities

Testing of gravity oil-grit separators has shown that they can remove between 40-50% of the TSS loading when used in an off-line configuration (Curran, 1996 and Henry, 1999). Gravity oil-grit separators also provide removal of debris, hydrocarbons, trash, and other floatables. They provide only minimal removal of nutrients and organic matter.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- *Total Suspended Solids* – 40%
- *Total Phosphorus* – 5%
- *Total Nitrogen* – 5%
- *Fecal Coliform* – insufficient data
- *Heavy Metals* – insufficient data

Actual field testing data and pollutant removal rates from an independent source should be obtained before using a proprietary gravity separator system.

## 4.10.4 Application and Site Feasibility Criteria

Conventional oil-grit separators contain a permanent pool of water and typically consist of an inlet chamber, separation and storage chamber, a bypass chamber, and an access port for maintenance purposes. Runoff enters the inlet chamber where heavy sediments and solids drop to the bottom. Then the flow moves into the main separation chamber, where further settling of suspended solids takes place. Oil and grease are skimmed and stored in a waste oil storage compartment for future removal. After moving into the outlet chamber, the clarified runoff is then discharged to the site's stormwater conveyance system.

A wide variety of separator systems are commercially-available in a variety of layouts, for which vendors have design data and procedures. Oil-grit separators are sized based on a design flow rate, the Water Quality Peak Flow Rate ($Q_{wq}$). This differs from how other stormwater BMPs are sized.

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **YES**
- Regional Stormwater Control – **NO**

**Physical Feasibility –** Physical Constraints at Project Site

- **Drainage Area** – Gravity oil-grit separators are typically used for drainage areas less than 5 acres. It is recommended that the contributing area to any individual gravity separator be limited to 1 acre or less of impervious cover.
- The total wet storage of the gravity separator unit should be at least 400 cubic feet per contributing impervious acre.
- **Space Required** – Gravity oil-grit separators are installed underground; therefore, minimal surface area is required for the device.
- Adequate maintenance access to each chamber must be provided for inspection and cleanout of a gravity separator unit.
- **Site Slope** – Gravity oil-grit separators may be installed on sites with slopes up to 6%.
- A minimum 20 foot wide maintenance right-of-way or drainage easement shall be provided for the oil-grit separator from any public or private road or driveway. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles. The right-of-way shall be located such that maintenance vehicles and equipment can access the oil-grit separator.
- **Minimum Depth to Water Table** – 2 feet
- **Minimum Head** – 4 feet (The minimum depth of the permanent pools should be 4 feet.)
- **Soils** – Gravity separator systems can be installed in almost any soil or terrain.

Check with manufacturer recommendations for additional site design constraints

**Other Constraints/Considerations**

- **Hot spots** – Gravity oil-grit separators are well-suited for hot spot runoff
- Damage to existing structures and facilities:
  - » Gravity oil-grit separators may increase the risk of flooding within a basement, affecting underground sewage pipes, or cause adverse effect to other underground structures.
  - » Gravity oil-grit separators should be designed so that overflow drains away from buildings to avoid causing damage to building foundations.
- **Trout Stream** – Gravity oil-grit separators will not reduce thermal impacts of stormwater runoff, nor are they effective at removing soluble pollutants. Therefore, they are not considered an effective means of protecting trout streams. However, in urban and highly developed areas or with a stormwater retrofit, gravity oil-grit separators may be an effective BMP for total suspended solids removal and hydrocarbons.

**Coastal Areas**

- **Poorly Draining Soils** – Poorly draining soils do not inhibit a gravity oil-grit separator's ability to temporarily store and treat stormwater runoff.
- **Flat Terrain** – Flat terrain and low site slopes do not interfere with the operation of a gravity oil-grit separator.
- **Shallow Water Table** – Review manufacturer's instructions regarding groundwater elevation. Anti-flotation calculations may be required when large open chambers are installed at or below the water table.

### 4.10.5 Planning and Design Criteria

Before designing the gravity oil-grit separator, the following data is necessary:

- Existing and proposed site, topographic, and location maps, as well as field reviews
- Impervious and pervious areas. Other means may be used to determine the land use data
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site
- Design data from nearby storm sewer structures
- Water surface elevation of nearby water systems and depth to seasonally high groundwater

*The following criteria are to be considered minimum standards for the design of a gravity oil-grit separator. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

BACK TO TOC

#### 4.10.5.1 LOCATION AND LAYOUT

Gravity oil-grit separators should be located upstream or downstream of other BMPs providing runoff reduction, additional treatment of the water quality volume ($WQ_v$), channel protection volume ($CP_v$), overbank flood protection ($Q_{P25}$), and extreme flood protection ($Q_f$). See Subsection 4.1.6 for more information on the use of multiple BMPs in a treatment train.

#### 4.10.5.2 GENERAL DESIGN

- The use of gravity (oil-grit) separators should be limited to the following applications:
  - » Pretreatment for other BMPs
  - » High-density, ultra-urban, or other space-limited development sites
  - » Hotspot areas where the control of grit, floatables, oil, and/or grease are required
- Gravity separators are rate-based devices. This contrasts with most other stormwater BMPs, which are sized based on capturing and treating a specific volume.
- Horizontal velocity through the separation chamber should be 1-3 ft/min or less. No velocities in the device should exceed the entrance velocity.

#### 4.10.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- The design criteria and specifications of a proprietary gravity separator unit should be obtained from the manufacturer.
- The separation chamber should provide for three separate storage volumes:
  - » A volume for separated oil storage at the top of the chamber
  - » A volume for settled solids accumulation at the bottom of the chamber
  - » A volume required to give adequate flow-through detention time for separation of oil and sediment from the stormwater flow
- Gravity separator units are typically designed to bypass runoff flows in excess of the design flow rate. Some designs have built-in high flow bypass mechanisms. Other designs require a diversion structure or flow splitter ahead of the device in the drainage system. An adequate outfall must be provided.
- A trash rack should be included in the design to capture floating debris, preferably near the inlet chamber to prevent debris from becoming oil impregnated.
- Ideally, a gravity separator design will provide an oil draw-off mechanism that empties to a separate chamber or storage area.
- Gravity separator units should be watertight to prevent possible groundwater contamination.

#### 4.10.5.4 PRETREATMENT/INLETS

- Gravity oil-grit separators are typically used for pretreatment in a hotspot area or where floatable debris and pollutants should be removed prior to additional treatment.
- Inlets sizing, slope, and invert placement should be sized based on manufacturer's recommendations for flow rate, volume, and structure size.

#### 4.10.5.5 OUTLET STRUCTURES

An important consideration when designing an oil-grit separator system for a site is how to bypass large storm events that exceed the design flow capacity around the separator without damaging the unit, exceeding the design flow capacity, or resuspending collected pollutants. Since resuspension of accumulated sediments and oil droplets is possible during heavy storm events, oil-grit separator units are typically installed off-line with a bypass to minimize pollutant wash-out or resuspension in these instances.

The outlet from the gravity separator needs to be able to convey stormwater leaving the gravity separator as well as the bypassed discharge without eroding the surrounding area. Typically the high flow outlet will discharge at a higher elevation than the low flow outlet.

#### 4.10.5.6 SAFETY FEATURES

- The deep inverts, open void sections, and sometimes larger pipe diameters into and out of oil-grit separators may present a fall or entrapment hazard. It is recommended that gravity oil-grit separators be constructed with manhole covers and/or grate lids with locking mechanisms. Structural loading calculations, such as H-20 loading for traffic areas, should be performed when sizing and installing gravity oil-grit separators.
- Some oil-grit separators are considered confined spaces. Additional training may be required to perform work inside the units.

#### 4.10.5.7 CONSTRUCTION CONSIDERATIONS

- Contributing drainage areas to the gravity oil-grit separator should be stabilized with appropriate erosion and sediment control devices, such as with temporary or permanent seeding before runoff can enter a newly installed-device.
- Newly installed gravity oil-grit separators should be inspected prior to being placed in service. Remove sediment and debris that may have been collected during delivery and installation.
- A minimum 20-foot wide maintenance right-of-way or drainage easement should be provided for the oil-grit separator from any public or private road or driveway.

#### 4.10.5.8 CONSTRUCTION AND MAINTENANCE COSTS

- Material and installation costs for gravity oil-grit separators can vary based on the size, location, treatment requirements, and manufacturer.
- Typically, gravity oil-grit separator systems range from approximately $5,000-$6,000 for a small catch basin or manhole insert type design to approximately $40,000 for a multi-chamber, high-volume, high-flow device.

BACK TO TOC

### 4.10.6 Design Procedures

In general, site designers should perform the following design procedures when designing a gravity oil-grit separator.

(Step 1) **Determine the goals and primary functions of the gravity oil-grit separator.**

- » A gravity oil-grit separator can be designed to provide pre-treatment of settled solids, oil, grease, debris, and floatables.
- » An oil-grit separator may also be used to provide some treatment of the water quality volume ($WQ_v$).
- » Check with local officials and other agencies to determine if there are any additional watershed restrictions that may apply. In addition, consider if the oil-grit separator has any site-specific design conditions or criteria. List any other requirements that may apply to or affect the design.

(Step 2) **Determine if the development site and conditions are appropriate for the use of a gravity oil-grit separator.**
Consider the application and site feasibility criteria in this chapter to determine if site conditions are suitable for a gravity oil-grit separator. Create a rough layout of the device dimensions taking into consideration existing trees, utility lines, power and telephone poles, roadways, sidewalks, curbs, and other obstructions.

(Step 3) **Compute runoff control volumes and rates.**
Oil-grit separators are typically sized based on the Water Quality peak flow rate ($Q_{wq}$). See Subsection 3.1.7.2 for more information on calculating the $Q_{wq}$. This differs from how other stormwater BMPs are sized. Refer to manufacturer instructions and tools to design the gravity oil-grit separator based on the appropriate design flow rate and volumes.

(Step 4) **Compute outlet release rate and size outlet.**

- » Use manufacturer-recommend design methods to determine the size, slope, and invert of the device outlet.
- » Ensure downstream receiving BMPs and/or storm drain systems can receive the volume and rate of stormwater flow from the gravity oil-grit separator and/or bypass system.
- » A hydraulic grade analysis should be performed to ensure that the receiving stormwater system can accept the flow, and that the oil-grit separator does not create a hydraulic head jump that exceeds the elevation of the upstream system.

### 4.10.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.11 Green Roof

**Description:** Green roofs represent an alternative to traditional impervious roof surfaces. They typically consist of underlying waterproofing and drainage materials and an overlying engineered growing media that is designed to support plant growth. Stormwater runoff is captured and temporarily stored in the engineered growing media, where it is subjected to the hydrologic processes of evaporation and transpiration with any remaining stormwater conveyed back into the storm drain system. This allows green roofs to provide measurable reductions in post-construction stormwater runoff rates, volumes, and pollutant loads on development sites.

**LID/GI Considerations:** Green roofs are an excellent method for reducing site impervious area, stormwater runoff volumes, pollutant loads, and thermal impacts of development. Green roofs provide outdoor areas and species habitat in developed and highly urbanized areas. The use of green roofs reduces the amount of ground surface area required to treat stormwater runoff, maximizing development space.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Engineered growing media should be a light-weight mix containing about 15% organic material.
- Waterproofing materials should be protected from root penetration by an impermeable root barrier.
- Green roofs may be installed on rooftops with slopes of up to 25%, but are not generally recommended for use on rooftops with slopes greater than 10%.
- The use of extensive green roof systems (2"-6" deep growing media) should be considered prior to the use of more complex and expensive intensive green roof systems.
- A landscaping plan should be prepared for all green roofs. The landscaping plan should be reviewed and approved by the local development review authority prior to construction.

### ADVANTAGES / BENEFITS

- Helps reduce post-construction stormwater runoff rates, volumes, and pollutant loads without consuming valuable land
- Particularly well-suited for use on urban development and redevelopment sites
- Use of green roofs allows for more development space on a project site.

### DISADVANTAGES / LIMITATIONS

- Can be difficult to establish vegetation in the harsh growing conditions found on rooftops in coastal Georgia.
- The roof structure must be capable of supporting the additional weight (live and dead load) of a green roof. Additional support, such as trusses, may be necessary for redevelopment projects to existing structures.

### ROUTINE MAINTENANCE REQUIREMENTS

- Inspect green roof for dead or dying vegetation. Dead vegetation should be removed along with any woody vegetation. Plant replacement vegetation as needed.
- Inspect waterproof membrane for leaks. Repair as needed.
- Remove invasive vegetation.
- Monitor sediment accumulation and remove periodically.

### POLLUTANT REMOVAL

- 80% **Total Suspended Solids**
- 50/50% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- N/A **Metals** - Cadmium, Copper, Lead, and Zinc removal
- N/A **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- ✓ Runoff Reduction
- ✓ Water Quality
- Channel Protection
- Overbank Flood Protection
- Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- L Land Requirement
- H Capital Cost
- L Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *No*

**Soils:** *Planting media should meet design recommendations.*

**Other Considerations:** *Drainage systems for the roof (gutters, downspouts, etc.) must be capable of handling large rainfall events.*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 60% of the runoff reduction volume provided

BACK TO TOC

### 4.11.1 General Description

Green roofs (also known as *vegetated roofs* or *eco-roofs*) represent an alternative to traditional impervious roof surfaces. They typically consist of underlying waterproofing and drainage materials and an overlying engineered growing media that is designed to support plant growth (**Figure 4.11-1**). Stormwater runoff is captured and temporarily stored in the engineered growing media, where it is subjected to the hydrologic processes of evaporation and transpiration, with any excess runoff conveyed back into the storm drain system. This allows green roofs to provide measurable reductions in post-construction stormwater runoff rates, volumes, and pollutant loads on development sites.

There are two different types of green roof systems: intensive green roof systems and extensive green roof systems. Intensive green roof systems (also known as *rooftop gardens*) have a thick layer of engineered growing media (i.e., 12-24 inches) that supports a diverse plant community that may include trees (**Figure 4.11-2**). Extensive green roof systems typically have a much thinner layer of engineered growing media (i.e., 2-6 inches) that supports a plant community that is comprised primarily of drought tolerant vegetation (e.g., sedums, succulent plants) (**Figure 4.11-3**).

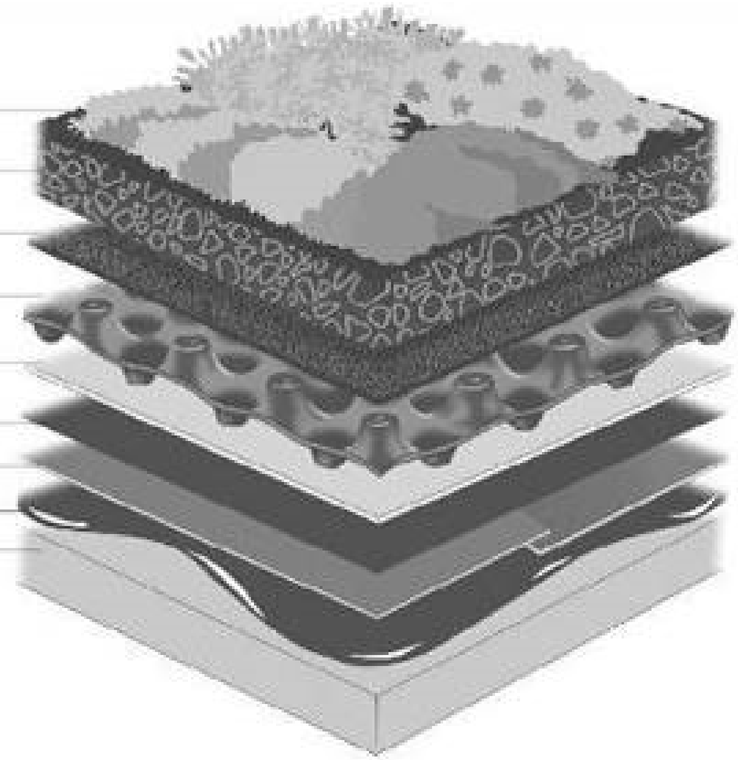



**Figure 4.11-1:** Components of a Green Roof System
(Source: Carter et al., 2007)

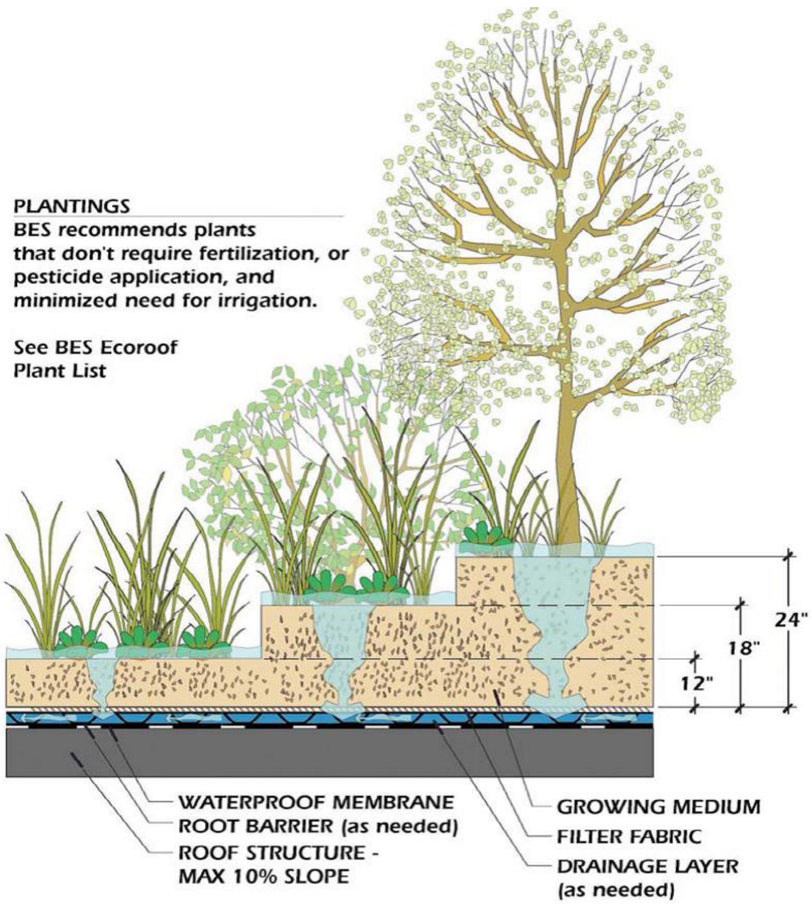



**Figure 4.11-2**: Intensive Green Roof System
(Source: City of Portland, OR, 2004)

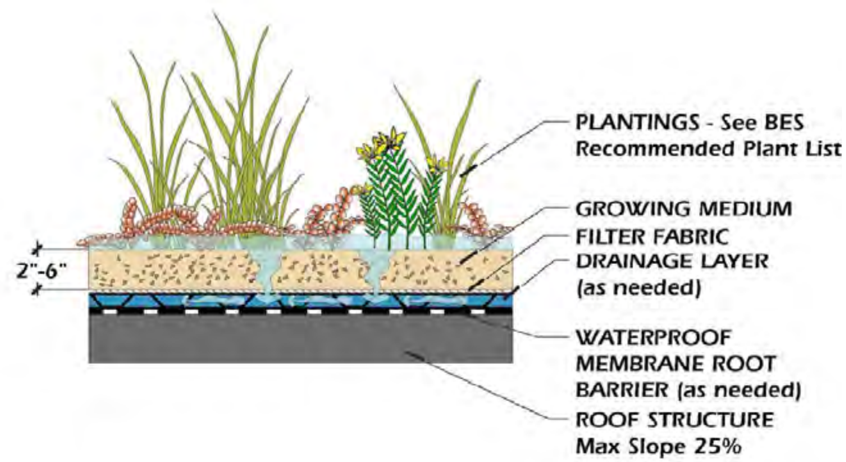



**Figure 4.11-3:** Extensive Green Roof System
(Source: City of Portland, OR, 2004)

Extensive green roof systems, which can cost up to twice as much as traditional impervious roof surfaces, are much lighter and less expensive than intensive green roof systems. Consequently, it is recommended that the use of extensive green roof systems be considered prior to the use of intensive green roof systems.

Extensive green roof systems typically contain multiple layers of roofing materials (**Figure 4.11-1**), and are designed to support plant growth while preventing stormwater runoff from ponding on the roof surface. Green roof systems are designed to drain stormwater runoff vertically through the engineered growing media and then horizontally through a drainage layer that is sloped towards an outlet. They are designed to require minimal long-term maintenance and, if the right plants are selected to populate the green roof, should not need supplemental irrigation or fertilization after an initial vegetation establishment period.

### 4.11.2 Stormwater Management Suitability

The Center for Watershed Protection (Hirschman et al., 2008) recently documented the ability of green roofs to reduce annual stormwater runoff volumes and pollutant loads on development sites.

- Runoff Reduction
  Runoff reduction credit can be applied to the green roof contributing drainage area if properly designed, installed, and maintained. As shown in **Table 4.1.3-2**, the runoff reduction volume (RRv) conveyed to a green roof may be reduced by 60%
- Water Quality
  If installed as per the recommended design criteria and properly maintained, 80% total suspended solids removal will be applied to the water quality volume ($WQ_v$) flowing to the green roof.
- Channel Protection
  No channel protection volume ($CP_v$) storage is provided by a green roof. Stormwater runoff generated by the contributing impervious rooftop area and pervious green roof should be routed to a downstream regional BMP that provides storage and treatment of the $CP_v$. Proportionally adjust the post-development runoff curve number (CN) to account for the runoff reduction provided by a green roof when calculating the $CP_v$ for the regional BMP. See Subsection 3.1.7.5 for more information about curve number reduction.
- Overbank Flood Protection
  No overbank flood protection volume storage is provided by a green roof. Proportionally adjust the post-development runoff CN to account for the runoff reduction provided by a green roof when calculating the overbank peak discharge ($Q_{p25}$) on a development site. See Subsection 3.1.7.5 for more information about curve number reduction.
- Extreme Flood Protection
  No extreme flood protection volume storage is provided by a green roof. Proportionally adjust the post-development runoff CN to account for the runoff reduction provided by a simple green roof for the contributing drainage area when calculating the extreme peak discharge ($Q_f$) on a development site. See Subsection 3.1.7.5 for more information about curve number reduction.

### 4.11.3 Pollutant Removal Capabilities

Green roofs are presumed to remove 80% of the total suspended solids (TSS) load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Green roofs remove 50% total Phosphorus and 50% total Nitrogen from stormwater. Green roofs are not presumed to remove fecal coliform or metals such as Cadmium, Copper, Lead, and Zinc.

In order to provide the most efficient pollutant removal, green roofs should be designed according to the criteria provided in this section. For additional information and data on pollutant removal capabilities for green roofs, see the National Pollutant Removal Performance Database (Version 3) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

## 4.11.4 Application and Site Feasibility Criteria

Green roofs can be used on a wide variety of development sites in rural, suburban, and urban areas. They are especially well-suited for use on commercial, institutional, municipal, and multi-family residential buildings on urban and suburban development and redevelopment sites. When compared with other low impact development practices, green roofs have a relatively high construction cost, a relatively low maintenance burden, and no additional surface area requirements beyond that which will be covered by the green roof. Although they can be expensive to install, green roofs are often a component of "green buildings," such as those that achieve certification in the Leadership in Energy and Environmental Design (LEED) Green Building Rating System.

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **YES**
- Regional Stormwater Control – **NO**

**Physical Feasibility –** Physical Constraints at Project Site

- **Drainage Area –** Green roofs should only be used to replace traditional impervious roof surfaces. They should not be used to treat any stormwater runoff generated elsewhere on the development site.
- **Space Required** – Green roofs require 100% of their contributing drainage areas.
- **Site Slope** – Although green roofs may be installed on rooftops with slopes of up to 25%, they are not recommended for use on rooftops with slopes greater than 10%.
- **Minimum Depth to Water Table** – Separation from the water table is not applicable to a green roof.
- **Minimum Head** – 6-12 inches
- **Soils** – An appropriate engineered growing media, consisting of approximately 80% lightweight inorganic material, 15% organic material, and 5% sand, should be used in green roof systems.

**Other Constraints / Considerations**

- **Hot spots –** May not be used for hot spot runoff
- **Damage to existing structures and facilities –** When designing a green roof, site planning and design teams must not only consider the stormwater storage capacity of the green roof, but also the structural capacity of the rooftop itself. To support a green roof, a rooftop must be designed to support an additional 15 to 30 pounds per square foot (psf) of load. Consequently, a structural engineer or other qualified professional should be involved with the design of a green roof to ensure that the rooftop itself has enough structural capacity to safely support the green roof system.
- **Proximity** – Green roofs may be used without restriction near:
  - » Private water supply wells
  - » Open water
  - » Public water supply reservoirs
  - » Public water supply wells
  - » Property Lines – Green roofs may be used near property lines; however, ensure that stormwater runoff is not redirected onto an adjacent owner's property. Stormwater runoff should remain on the property where it was generated until it is discharged to downstream receiving waters, a municipal stormwater system, or to the pre-construction point of discharge.
- **Trout Stream** – Green roofs help to treat stormwater for pollutants and reduce the volume and velocity of runoff. Therefore, green roofs are an effective BMP for use where trout streams or other protected waters may receive stormwater runoff.

**Coastal Areas**

Green roofs can be used without restriction in Coastal Georgia, where there is flat terrain, low site slopes, and shallow water tables.

BACK TO TOC

## 4.11.5 Planning and Design Criteria

Before designing the green roof, the following data is necessary:

- Architectural roof plan with rooftop pitches and downspout locations
- The proposed site design, including, buildings, parking lots, sidewalks, stairs and handicapped ramps, and landscaped areas for downspout discharge locations and bypass outfalls
- Information about downstream BMPs and receiving waters

*The following criteria are to be considered minimum standards for the design of a green roof. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

#### 4.11.5.1 LOCATION AND LAYOUT

- Green roof systems should be designed to provide enough storage for the stormwater runoff volume generated by the target runoff reduction rainfall event (e.g., 85th percentile rainfall event). The required dimensions of a green roof system are governed by several factors, including the hydraulic conductivity and moisture retention capacity of the engineered growing media and the porosity of the underlying drainage layer. Site planning and design teams are encouraged to consult with green roof manufacturers and/or materials suppliers to design green roof systems that provide enough storage for the stormwater runoff volume generated by the target runoff reduction rainfall event (e.g., 85th percentile rainfall event).
- During the design of a green roof system, site planning and design teams should consider not only the storage capacity of the green roof, but also the structural capacity of the rooftop itself. A structural engineer or other qualified professional should be involved with the design of a green roof to ensure that the rooftop itself has enough structural capacity to support the green roof system.
- Green roofs should not be used to treat any stormwater runoff generated elsewhere on the development site.

#### 4.11.5.2 GENERAL DESIGN

- All green roofs should be designed in accordance with the ASTM International Green Roof Standards (ASTM, 2005a, ASTM, 2005b, ASTM, 2005c, ASTM, 2005d, ASTM, 2006).
- Supplemental measures, such as battens, may be needed to ensure stability against sliding on rooftops with slopes of greater than 10%.All green roof systems should include a waterproofing layer that will prevent stormwater runoff from damaging the underlying rooftop. Waterproofing materials typically used in green roof installations include reinforced thermoplastic and synthetic rubber membranes.
- The waterproofing layer should be protected from root penetration by an impermeable, physical root barrier. Chemical root barriers or physical root barriers that have been impregnated with pesticides, metals, or chemicals that may leach into post-construction stormwater runoff should not be used.
- A drainage layer should be placed between the root barrier and the engineered growing media. The drainage layer should consist of synthetic or inorganic materials (e.g., gravel, recycled polyethylene) that are capable of both retaining water and providing efficient drainage when the layer becomes saturated. The required depth of the drainage layer will be governed by the required storage capacity of the green roof system and by the structural capacity of the rooftop itself. An appropriate engineered growing media, consisting of approximately 80% lightweight inorganic materials, 15% organic matter (e.g., well-aged compost), and 5% sand, should be installed above the drainage layer. The engineered growing media should have a maximum water retention capacity of approximately 30%.
- To prevent clogging within the drainage layer, the engineered growing media should be separated from the drainage layer by a layer of permeable filter fabric. The filter fabric should be a non-woven geotextile with a permeability that is greater than or equal to the hydraulic conductivity of the overlying engineered growing media.

BACK TO TOC

- The engineered growing media should be between 4-6 inches deep, unless synthetic moisture retention materials (e.g., drainage mat with moisture storage "cups") are placed directly beneath the engineered growing media layer. When synthetic moisture retention materials are used, a 2 inch deep engineered growing media layer may be used.
- To assist in conveying runoff to the building drainage system, a semi-rigid, plastic geo-composite drain or mat layer should be included in the design of a green roof. If the roof is flat, a perforated network may be necessary to help rainfall drain properly.
- Extend the roof flashing 6 inches above engineered growing media and protect by counter flashing.

#### 4.11.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

The drainage area of a green roof is comprised of the green roof itself. No additional stormwater runoff should be allowed to "run on" to the green roof with the exception of walking paths or vegetation access ways incorporated into the green roof design.

#### 4.11.5.4 PRETREATMENT/INLETS

Green roofs are designed to directly receive rainfall. Pretreatment and inlets are not required.

#### 4.11.5.5 OUTLET STRUCTURES

An outlet (e.g., scupper and downspout) should be provided to convey stormwater runoff out of the drainage layer and off of the rooftop when the drainage layer becomes saturated.

#### 4.11.5.6 SAFETY FEATURES

- Consideration should be given to the stormwater runoff rates and volumes generated by larger storm events (e.g., 25-year, 24-hour storm event) to design green roofs that are able to safely convey or bypass these flows. An overflow system, such as a traditional rooftop drainage system with inlets set slightly above the elevation of the surface of the green roof, should be designed to convey the stormwater runoff generated by these larger storm events safely off of the rooftop.
- Runoff shall flow through and exit green roof systems in a safe and non-erosive manner. Overflow structures should be capable of passing the 2-year, 24-hour design storm without inundating the roof.

#### 4.11.5.7 LANDSCAPING

- A landscaping plan should be prepared for all green roofs. The landscaping plan should be reviewed and approved by the local development review authority prior to construction.
- When developing a landscaping plan, site planning and design teams are encouraged to consult with a botanist, landscape architect, or other qualified professional to identify plants that will tolerate the harsh growing conditions found on rooftops in Georgia. Planting recommendations for green roofs include:
  » Drought- and full sun-tolerant vegetation that requires minimal irrigation after establishment
  » Low maintenance vegetation that is self-sustaining and does not require mowing, trimming or the use of fertilizers, pesticides, or herbicides
  » Vegetation that is fire resistant and able to withstand heat, cold, and high winds
- Since sedum and succulent plants possess many of the characteristics listed above, they are recommended for use on green roof systems installed in Georgia. Herbs, forbs, grasses, and other groundcovers may be used, but these plants typically have higher watering and maintenance requirements.
- Methods used to establish vegetative cover on a green roof should achieve at least 75 percent vegetative cover one year after installation.



BACK TO TOC

#### 4.11.5.8 CONSTRUCTION CONSIDERATIONS

To help ensure that green roofs are properly installed on a development site, site planning and design teams should consider the following recommendations:

- Care should be given to avoid damage to the waterproofing membrane during installation of the green roof. If the integrity of the membrane is compromised in a manner that may cause leaks or roof damage, the area should be identified and repaired. Visually inspect for damage and test the membrane for water tightness prior to installation of the engineered growing media.
- If the roof is sloped, stabilization measures may be required before installing the green roof to prevent soil from sliding down the roof. Some situations may allow the stabilization measures to be incorporated into the roof structure.
- Install the green roof according to the manufacturer's instructions. Usually the root barrier layer, walkway, and irrigation system are installed first.
- To help prevent compaction of the engineered growing media, heavy foot traffic should be kept off of green roof surfaces during and after construction.
- Construction contracts should contain a replacement warranty that covers at least three growing seasons to help ensure adequate growth and survival of the vegetation planted on a green roof.

#### 4.11.5.9 CONSTRUCTION AND MAINTENANCE COSTS

- Extensive green roofs can range from roughly $5-$20 per square foot.
- Intensive green roofs can range from roughly $20-$80 per square foot.
- Although the cost per square foot of a green roof is notably higher than a regular roof, green roofs have been reported to save costs associated with energy consumption and increasing the lifespan of the roof.

BACK TO TOC

### 4.11.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of a green roof.**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a green roof. Create a rough layout of the green roof dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the green roof.**

Consider whether the green roof is intended to:

- Meet a runoff reduction* target or water quality (treatment) target. For information on the sizing of a BMP utilizing the runoff reduction approach, see Step 3A. For information on the sizing of the BMP utilizing the water quality treatment approach, see Step 4. **Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*
- Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)
- Provide a possible solution to a drainage problem
- Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3A, 3B, and 3C for a runoff reduction approach, or skip Step 3 and complete Step 4 for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) **Calculate the Stormwater Runoff Reduction Target Volume**

Calculate the Runoff Reduction Volume using the following formula:

$$RR_V = (P)\,(R_V)\,(A)\,/\,12$$

*Where*:

**$RR_V$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the practice:

Using the $R_v$ calculated, determine the minimum Volume of the Practice (VP)

$$(VP_{MIN}) \geq RR_v \text{ (target) } / \text{ (RR\%)}$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice
**$VP_{MIN}$** = Minimum storage volume required to provide Runoff Reduction Target Volume ($ft^3$)
**$RR_v$ (target)** = Runoff Reduction Target Volume ($ft^3$)

(Step 3B) Determine the storage volume of the practice and the Pretreatment Volume

To determine the actual volume provided in the green roof, use the following equation:

$$VP = (PV + VES\ (N))$$

*Where*:

**VP** = Volume provided (temporary storage)
**PV** = Ponding Volume
**VES** = Volume of Engineered Soils
**N** = Porosity

To determine the porosity, a qualified licensed professional should be consulted to determine the proper porosity based on the engineered soils used. Most soil media has a porosity of 0.25 and gravel a value of 0.40.

(Step 3C) Determine whether the minimum storage volume was met.

When the VP $\geq VP_{MIN}$, then the Runoff Reduction requirements are met for this practice. Proceed to Step 5.

When the VP < $VP_{MIN}$, then the BMP must be sized according to the $WQ_v$ treatment method (See Step 4).

(Step 4) Calculate the Target Water Quality Volume

Calculate the Water Quality Volume using the following formula:

$$WQ_v = (1.2)\ (R_v)\ (A) / 12$$

*Where*:

**$WQ_v$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_v$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Site area (total drainage area) ($ft^2$)
**12** = Unit conversion factor (in/ft)

Using the $WQ_v$ calculated above, determine the actual size and Volume of the Practice (VP) as shown in Step 3B. Note that VP, calculated in Step 3B, should be greater than or equal to $WQ_v$.

(Step 5) Calculate the adjusted curve numbers for $CP_V$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm). See Subsection 3.1.7.5 for more information.

(Step 6) Prepare Site Vegetation and Landscaping Plan.
Vegetation is critical to the function and appearance of any green roof. Therefore landscaping plans should be provided according to the guidance in Subsection 4.11.5.7 (*Landscaping*) and Appendix D.

### 4.11.7 Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.12 Infiltration Practices

**Description:** Infiltration practices, which may also be classified as a runoff reducing low impact development practices, are shallow excavations, typically filled with stone or an engineered soil mix, that are designed to intercept and temporarily store post-construction stormwater runoff until it infiltrates into the underlying and surrounding soils. If properly designed, they can provide significant reductions in post-construction stormwater runoff rates, volumes and pollutant loads.

**LID/GI Considerations**: Because infiltration practices allow more infiltration to the soil surrounding the practice than other BMPs, they are considered a LID/GI Control. This helps restore a sites natural hydrology.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Pretreatment should be provided upstream of all infiltration practices
- Infiltration practices should be designed to completely drain within 72 hours of the end of a rainfall event
- Underlying native soils should have an infiltration rate of 0.5 in/hr or more
- The distance from the bottom of an infiltration practice to the top of the water table should be 2 feet or more
- Facilities include an excavated trench (2-10 foot depth) filled with stone media (1.5-2.5 inch diameter), as well as pea gravel and sand filter layers
- A pretreatment device is recommended upstream from the practice
- Observation wells are used to monitor percolation and performance of the practice
- Infiltration practices must not be placed under pavement or concrete

### ADVANTAGES / BENEFITS

- Considered a LID-GI control
- Provides for groundwater recharge
- Good for small sites with porous soils
- Helps restore pre-development hydrology on development sites and reduces post-construction stormwater runoff rates, volumes and pollutant loads
- Can be integrated into development plans as attractive landscaping features

### DISADVANTAGES / LIMITATIONS

- Can only be used to manage runoff from relatively small drainage areas of 5 acres or less
- Should not be used to "receive" stormwater runoff that contains high sediment loads
- Potential for groundwater contamination
- High clogging potential; should not be used on sites with fine-particle soils (clays or silts) in drainage areas
- Significant setback requirements
- Restrictions in karst areas
- Geotechnical testing required, two borings per practice

### • ROUTINE MAINTENANCE REQUIREMENTS

- Keep practice free of trash, debris, and dirt
- Inspect area for ponding water
- If structures become clogged, remove aggregate, wash and replace
- Can be susceptible to clogging, so locate in stabilized areas (i.e. not in tree canopy)
- Keep observation well easily and safely accessible
- Remove sediment from forebay or other pretreatment practice
- Replace pea gravel layer as needed

### STORMWATER MANAGEMENT SUITABILITY

✓ Runoff Reduction
✓ Water Quality
★ Channel Protection
★ Overbank Flood Protection
★ Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

L Land Requirement
H Capital Cost
H Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*

**Other Considerations:** *Highly applicable for roadway projects*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 100%- Runoff Reduction Volume

### POLLUTANT REMOVAL

100% **Total Suspended Solids**

100/100% **Nutrients** - Total Phosphorus / Total Nitrogen removal

100% **Metals** - Cadmium, Copper, Lead, and Zinc removal

100% **Pathogens** – Fecal Coliform

## 4.12.1 General Discussion

Infiltration practices are excavations typically filled with stone to create an underground reservoir for stormwater runoff (see **Figure 4.12-1**). This runoff volume gradually exfiltrates through the bottom and sides of the trench into the subsoil over a three-day period and eventually reaches the water table. By diverting runoff into the soil, an infiltration practice not only treats the water quality volume, but also helps to preserve the natural water balance on a site through groundwater recharge to preserve baseflow. Due to this fact, infiltration systems are limited to areas with highly porous soils where the water table and/or bedrock are located well below the bottom of the trench. In addition, infiltration practices must be carefully sited to avoid the potential for groundwater contamination.

Infiltration practices are not intended to trap sediment and must always be designed with a sediment forebay and grass channel for concentrated flow or filter strip for sheet flow, or other appropriate pretreatment measures to prevent clogging and failure. Due to their high potential for failure, these facilities must only be considered for sites where upstream sediment control can be ensured.

Although infiltration practices can provide significant reductions in post-construction stormwater runoff rates, volumes and pollutant loads, they have historically experienced high rates of failure due to clogging caused by poor design, poor construction and neglected maintenance. If infiltration practices are to be used on a development site, great care should be taken to ensure that they are adequately designed, carefully installed and properly maintained over time. They should only be applied on development sites that have permeable soils (i.e., hydrologic soil group A and B soils) and that have a water table and confining layers (e.g., bedrock, clay lenses) that are located at least two feet below the bottom of the trench or basin.

There are two major variations of infiltration practices, namely infiltration trenches and infiltration basins. A brief description of each of these design variants is provided below:

- Infiltration Trenches: Infiltration trenches are excavated trenches filled with stone. Stormwater runoff is captured and temporarily stored in the stone reservoir, where it exfiltrates into the surrounding and underlying native soils. Infiltration trenches can be used to manage post-construction stormwater runoff from contributing drainage areas of up to 2 acres and should only be used on development sites where sediment loads can be kept relatively low.

- Infiltration basins are shallow, landscaped excavations filled with an engineered soil mix. They are designed to capture and temporarily store stormwater runoff in the engineered soil mix, where it is subjected to the hydrologic processes of evaporation and transpiration while infiltrating into the surrounding soils. They are essentially non-underdrained bioretention areas (Section 4.2), and should also only be used on development sites where sediment loads can be kept relatively low. It should be noted that this example is only one method.

## 4.12.2 Stormwater Management Suitability

Infiltration practices can be designed for water quantity, but they are mostly used for water quality, i.e. the removal of stormwater pollutants, depending upon the native soils. Infiltration practices can provide runoff quantity control, particularly for smaller runoff volumes such as the runoff volume generated by the water quality storm event (1.2 inches). These facilities may sometimes be used to partially or completely meet channel protection requirements on smaller sites. However, infiltration practices will typically need to be used in conjunction with another control to provide channel protection, as well as overbank flood protection. Infiltration practices need to be designed and maintained to safely bypass higher flows.

BACK TO TOC

- Runoff Reduction
  Like other LID practices, infiltration practices become more effective with higher infiltration rates of native soils. An infiltration practice can be designed to provide 100% of the runoff reduction volume, if properly maintained. In order to provide runoff reduction with an infiltration practice that is designed without an underdrain, the infiltration practice should drain within 72 hours.

- Water Quality
  The infiltration practice is an excellent stormwater treatment practice due to the variety of pollutant removal mechanisms. Each of the components of the infiltration practice 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.

- Channel Protection
  For smaller sites, an infiltration practice may be designed to capture the entire channel protection volume ($CP_v$). Given that an infiltration practice is typically designed to completely drain over 48-72 hours, the 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 infiltration practice, another practice must be used to provide $CP_v$ extended detention.

- Overbank Flood Protection
  Although relatively rare, on some development sites, an infiltration practice can be designed to attenuate the overbank peak discharge ($Q_{p25}$).

- Extreme Flood Protection
  Although relatively rare, on some development sites, an infiltration practice can be designed to attenuate the extreme peak discharge ($Q_f$).

*Credit for the volume of runoff reduced in the infiltration practice may be taken in the overbank flood protection and extreme flood protection calculations. If the practice is designed to provide Runoff Reduction for Water Quality compliance, then the practice is given credit for Channel Protection and Flood Control requirements by allowing the designer to compute an Adjusted CN (see Subsection 3.1.7.5 for more information).*

### 4.12.3 Pollutant Removal Capabilities

Infiltration practices are presumed to be able to remove 80% of the total suspended solids (TSS) load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Infiltration practices can also remove Phosphorus, Nitrogen, metals, and pathogens. An undersized, poorly designed and/or neglected infiltration practice may have reduced pollutant removal performance. Proper design of infiltration practices is critical to ensure that pollutants are properly removed from stormwater runoff. Due to clogging issues, Infiltration Practices should not be used for removing sediment or other coarse material.

For additional information and data on pollutant removal capabilities for infiltration practices, see the National Pollutant Removal Performance Database (3rd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.12.4 Application and Site Feasibility Criteria

Infiltration practices can be used to manage post-construction stormwater runoff on development sites in rural, suburban and urban areas where the soils have adequate permeability and the water table is low enough to provide for the infiltration of stormwater runoff. They are generally suited for medium-to-high density residential, commercial and institutional developments. Infiltration practices should be considered for use on development sites where the

- Subsoil is sufficiently permeable to provide a reasonable infiltration rate.

- Fine sediment (e.g., clay, silt) loads will be relatively low, as high sediment loads will cause them to clog and fail.

- The water table is low enough to prevent groundwater contamination that could potentially contaminate water supply aquifers.
- Impervious areas where there are not high levels of fine particles (clay/silt soils) in the runoff.

Infiltration practices can either be used to capture sheet flow or concentrated flow from a drainage area. Due to their relatively narrow shape, infiltration practices can be adapted to many different types of sites, such as in retrofit situations. Unlike some other structural stormwater practices, infiltration practices can easily fit into the margin, perimeter, or other unused areas of developed sites.

To protect groundwater from potential contamination, runoff from designated hotspot land uses or activities must not be infiltrated. Infiltration practices should not be used for manufacturing or industrial sites, where there is a potential for high concentrations of soluble pollutants and heavy metals. In addition, infiltration should not be considered for areas with a high pesticide concentration. Infiltration practices are also not suitable in areas with karst geology, unless adequate geotechnical testing by qualified individuals in accordance with local requirements suggests otherwise.

*The following criteria should be evaluated to ensure the suitability of an infiltration practice for meeting stormwater management objectives on a site.*

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **YES**
- Regional Stormwater Control – **NO**

**Physical Feasibility -** Physical Constraints at Project Site

- **Drainage Area –** Maximum of 5 acres. Although infiltration practices can be used to manage stormwater runoff from contributing drainage areas as large as 5 acres in size, contributing drainage areas of between 2,500 square feet and 2 acres are preferred.
- **Space Required –** Will vary depending on the depth of the practice. In general, infiltration practices require about 5% of the size of their contributing drainage areas.
- **Site Slope –** No more than 6% slope (for pre-construction facility footprint). However, they should be designed with slopes that are as close to flat as possible.
- **Minimum Head –** Unless a shallow water table is found on the development site, all infiltration practices should be designed to be at least 36 inches deep. Infiltration basins may be designed with a maximum ponding depth of 12 inches, although a ponding depth of 9 inches is recommended to help prevent nuisance ponding conditions. Unless a shallow water table is found on the development site, all infiltration basin planting beds should be at least 36 inches deep.
- **Minimum Depth to Water Table –** Two feet is recommended between the bottom of the infiltration practice and the elevation of the seasonally high water table
- **Soils –** Infiltration practices should be designed to completely drain within 72 hours of the end of a rainfall event. Consequently, infiltration practices generally should not be used on development sites that have soils with infiltration rates of less than 0.5 inches per hour (i.e., hydrologic soil group C and D soils).

**Other Constraints/Considerations**

- **Aquifer Protection** – No hotspot runoff allowed; meet setback requirements in design criteria.

**Coastal Areas**

- **Poorly Draining Soils**— this condition minimizes the ability of an infiltration practice to reduce stormwater runoff rates, volumes, and pollutant loads. Infiltration practices should not be used on development sites that have soils with infiltration rates of less than 0.5 inches per hour (i.e., hydrologic soil group C and D soils). Another consideration would be to use other low impact development and stormwater management practices, such as rainwater harvesting (Section 4.19) and underdrained bioretention areas (Section 4.2), to manage post-construction stormwater runoff in these areas.

- **Well-Draining Soils**—this condition enhances the ability of infiltration practices to reduce stormwater runoff rates, volumes and pollutant loads, but may allow stormwater pollutants to reach groundwater aquifers with greater ease. A potential solution is to avoid the use of infiltration-based stormwater management practices, including infiltration practices, at stormwater hotspots and in areas known to provide groundwater recharge to water supply aquifers, unless adequate pretreatment is provided upstream. Another potential solution is to use bioretention areas (Section 4.2) or dry enhanced swales (Section 4.6) with liners and underdrains at stormwater hotspots and in areas known to provide groundwater recharge to water supply aquifers.
- **Flat Terrain**—does not negatively influence the infiltration practice. In fact, infiltration practices should be designed with slopes that are as close to flat as possible.
- **Shallow Water Table**—it may be difficult to provide two feet of clearance between the bottom of the infiltration practice and the top of the water table, which may occasionally cause stormwater runoff to pond in the bottom of the infiltration practice. There are several potential solutions to this problem:
  - » Ensure that the distance from the bottom of the infiltration practice and the top of the water table is at least 2 feet.
  - » Reduce the depth of the stone reservoir in infiltration practices to no less than18 inches.
  - » Reduce the depth of the planting bed in infiltration basins to no less than 18 inches.
  - » Use stormwater ponds (Section 4.25), stormwater wetlands (Section 4.26), or grass channel (Section 4.9), instead of infiltration practices to intercept and treat stormwater runoff in these areas.
- **Tidally-influenced drainage system**—does not influence the infiltration practice.

## 4.12.5 Planning and Design Criteria

*The following criteria are to be considered minimum standards for the design of an infiltration practice. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.12.5.1 LOCATION AND LAYOUT

- Infiltration practices should be used on development sites that have underlying soils with an infiltration rate of 0.5 inches per hour (in/hr) or greater, as determined by NRCS soil survey data and subsequent field testing. See Appendix D for additional information on the field infiltration test protocol. Note that soil testing should be approved by the local development review authority prior to testing.
- Infiltration practices should have a contributing drainage area of 5 acres or less, with 2 acres or less being preferred.
- There should be at least 24 inches between the bottom of the infiltration practice and the elevation of the seasonally high water table.
- Clay lenses, bedrock or other restrictive layers below the bottom of the trench will reduce infiltration rates unless excavated.
- Minimum setback requirements for infiltration practice facilities (when not specified by local ordinance or criteria):
  - » From a property line – 10 feet
  - » From a building foundation – 25 feet
  - » From a private well – 100 feet
  - » From a public water supply well – 1,200 feet
  - » From a septic system tank/leach field – 100 feet
  - » From surface waters – 100 feet
  - » From surface drinking water sources – 400 feet (100 feet for a tributary)
- To reduce the potential for costly maintenance and/or system reconstruction, it is strongly recommended that the trench be located in an open space, with the top of the structure as close to the ground surface as possible. Infiltration practices should not be located beneath paved surfaces, such as parking lots.
- Infiltration practices are designed for intermittent flow and must be allowed to drain for reaeration of the surrounding soil between rainfall events. They must not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

BACK TO TOC

#### 4.12.5.2 GENERAL DESIGN

A well-designed infiltration practice consists of:

1. Excavated a shallow trench or basin backfilled with sand, coarse stone, and pea gravel;
2. Appropriate pretreatment measures; and
3. One or more observation well to show how quickly the trench basin dewaters or to determine if the practice is clogged.

#### 4.12.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- Infiltration practices should be designed to completely drain within 72 hours of the end of a rainfall event. Where site characteristics allow, it is preferable to design infiltration practices to drain within 48 hours of the end of a rainfall event to help prevent the formation of nuisance ponding conditions.
- Infiltration practice depths should be between 3-8 feet, to provide for easier maintenance. The width of a trench should be less than 25 feet.
- The surface area required is calculated based on the practice depth, soil infiltration rate, aggregate void space, and fill time (this will need to be modeled).
- The bottom slope of an infiltration practice should be flat across its length and width to evenly distribute flows, encourage uniform infiltration through the bottom, and reduce the risk of clogging.
- Stone aggregate used in the trench should be washed, bank-run gravel, 1.5-2.5 inches in diameter with a void space of about 40% (GADOT No.3 Stone). Aggregate contaminated with soil shall not be used. A porosity value (void space/total volume) of 0.32 should be used in calculations, unless aggregate specific data exist.
- An observation well should be installed in every infiltration practice. An observation well consists of a 4 to 6 inch perforated PVC (AASHTO M 252) pipe that extends to the bottom of the infiltration practice. The observation well can be used to observe the rate of drawdown within the infiltration practice following a storm event. It should be installed along the centerline of the infiltration practice, flush with the elevation of the surface of the infiltration practice. A visible floating marker should be provided within the observation well and the top of the well should be capped and locked to prevent tampering and vandalism.
- Since clay lenses or any other restrictive layers located below the bottom of an infiltration practice will reduce soil infiltration rates, infiltration testing should be conducted within any confining layers that are found within 4 feet of the bottom of a proposed infiltration practice.
- Infiltration practices should be located in an open pervious area and should be designed so that the top of the practice is located as close to the surface as possible. Infiltration practices should not be located beneath a driveway, parking lot or other impervious surface.
- Broader, shallower infiltration practices perform more effectively by distributing stormwater runoff over a larger surface area. However, a minimum depth of 36 inches is recommended for all infiltration practices to prevent them from consuming a large amount of surface area on development sites. Whenever practical, the depth of infiltration practices should be kept to 60 inches or less.
- Underlying native soils should be separated from the stone reservoir by a thin, 2-4 inch layer of choker stone (i.e., ASTM D 448 size No. 8, 3/8" to 1/8" or ASTM D 448 size No. 89, 3/8" to 1/16") or a 6-inch layer of clean, washed sand. The choker stone or sand should be placed between the stone reservoir and the underlying native soils.



BACK TO TOC

- Consideration should be given to the stormwater runoff rates and volumes generated by larger storm events (e.g., 25-year, 24-hour storm event) to help ensure that these larger storm events are able to safely bypass the infiltration practice. An overflow system should be designed to convey the stormwater runoff generated by these larger storm events safely out of the infiltration practice. Methods that can be used to accommodate the stormwater runoff rates and volumes generated by these larger storm events include:
  - Using storm drain inlets set slightly above the elevation of the surface of an infiltration practice to collect excess stormwater runoff. This will create some ponding on the surface of the infiltration practice, but can be used to safely convey excess stormwater runoff over the surface of the practice.
  - Using yard drains or storm drain inlets set at the maximum ponding depth of an infiltration basin to collect excess stormwater runoff.
  - Using a spillway with an invert set slightly above the elevation of maximum ponding depth to convey stormwater runoff generated by larger storm events safely out of an infiltration basin.
  - Placing a perforated pipe (e.g., underdrain) near the top of the stone reservoir or planting bed to provide additional conveyance of stormwater runoff after the infiltration practice has been filled.

#### 4.12.5.4 PRETREATMENT/INLETS

- Pretreatment practices are recommended to be used in conjunction with an infiltration practice to prevent clogging and failure.
- For an infiltration practice receiving sheet flow from an adjacent drainage area, the pretreatment system should consist of a vegetated filter strip (refer to Section 4.29).
- For concentrated flow, pretreatment should consist of a sediment forebay, vault, plunge pool, or similar sedimentation chamber (with energy dissipaters) sized to a minimum of 10% of the water quality volume ($WQ_v$). Exit velocities from the pretreatment chamber must be non-erosive for the 2-year, 24-hour design storm.

#### 4.12.5.5 OUTLET STRUCTURES

Outlet structures are not required for infiltration practices.

#### 4.12.5.6 EMERGENCY SPILLWAY

A non-erosive overflow channel or pipe should be provided to safely pass flows that exceed the storage capacity of the infiltration practice to a stabilized downstream area or watercourse.

#### 4.12.5.7 MAINTENANCE ACCESS

- Adequate access should be provided to an infiltration practice for inspection and maintenance.
- Include access roads and ramps for appropriate equipment to all applicable components of the infiltration practice (observation well, forebay, etc.).
- Provide space to safely exit and enter public roads (if necessary).

#### 4.12.5.8 SAFETY FEATURES

In general, infiltration practices are not likely to pose a physical threat to the public and do not need to be fenced.

#### 4.12.5.9 LANDSCAPING

- Infiltration practices should fit into and blend with the surrounding area. Native grasses are preferable, if compatible. A basin may be covered with permeable topsoil and planted with grass in a landscaped area.
- The landscaped area above the surface of an infiltration practice may also be covered with pea gravel (i.e., ASTM D 448 size No. 8, 3/8″ to 1/8″). This pea gravel layer provides sediment removal and additional pretreatment upstream of the infiltration practice and can be easily removed and replaced when it becomes clogged.

- Alternatively, an infiltration practice may be covered with an engineered soil mix, such as that prescribed for an infiltration basin, and planted with managed turf or other herbaceous vegetation. This may be an attractive option when infiltration practices are placed in disturbed pervious areas (e.g., lawns, parks and community open spaces).
- A landscaping plan should be prepared for all infiltration basins. The landscaping plan should be reviewed and approved by the local development review authority prior to construction.
- Vegetation commonly planted in infiltration basins includes native trees, shrubs and other herbaceous vegetation. When developing a landscaping plan, site planning and design teams should choose vegetation that will be able to stabilize soils and tolerate the stormwater runoff rates and volumes that will pass through the infiltration basin. Vegetation used in infiltration basins should also be able to tolerate both wet and dry conditions. See Appendix D for a list of grasses and other plants that are appropriate for use in infiltration practices installed in the state of Georgia.
- A mulch layer, consisting of 2-4 inches of fine shredded hardwood mulch or shredded hardwood chips, should be included on the surface of an infiltration basin.
- Methods used to establish vegetative cover within an infiltration basin should achieve at least 75% vegetative cover one year after installation.
- To help prevent soil erosion and sediment loss, landscaping should be provided immediately after an infiltration practice has been installed. Temporary irrigation may be needed to quickly establish vegetative cover within an infiltration basin.
- The soils used within infiltration basin planting beds should be an engineered soil mix that meets the following specifications:
  - **Texture:** Sandy loam or loamy sand should be used.
  - **Sand Content:** Soils should contain 85%-88% clean, washed sand.
  - **Topsoil Content:** Soils should contain 8%-12% topsoil.
  - **Organic Matter Content:** Soils should contain 3%-5% organic matter.
  - **Infiltration Rate:** Soils should have an infiltration rate of at least 0.25 inches per hour (in/hr), although an infiltration rate of between 1 and 2 in/hr is preferred.
  - **Phosphorus Index (P-Index):** Soils should have a P-Index of less than 30.
  - **Exchange Capacity (CEC):** Soils should have a CEC that exceeds 10 milliequivalents (meq) per 100 grams of dry weight.
  - **pH:** Soils should have a pH of 6-8.
- The organic matter used within an infiltration basin planting bed should be a well-aged compost that meets the specifications outlined in Appendix D.

#### 4.12.5.10 ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

**Physiographic Factors -** Local terrain design constraints

- **Low Relief** – No additional criteria.
- **High Relief** – Maximum site slope of 6%.
- **Karst** – Not suitable without adequate geotechnical testing.

**Special Downstream Watershed Considerations-** No additional criteria

### 4.12.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of an infiltration practice.**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for an infiltration practice. Create a rough layout of the infiltration practice dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the infiltration practice.**

Consider whether the infiltration practice is intended to:

- Meet a runoff reduction* target or water quality (treatment) target. For information on the sizing of a BMP utilizing the runoff reduction approach, see Step 3A. For information on the sizing of the BMP utilizing the water quality treatment approach, see Step 4A. **Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*
- Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)
- Provide a possible solution to a drainage problem
- Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3A, 3B, and 3C for a runoff reduction approach, or skip Step 3 and complete Steps 4A and 4B for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) **Calculate the Stormwater Runoff Reduction Target Volume**

Calculate the Runoff Reduction Volume using the following formula:

$$RR_v = (P)\ (R_v)\ (A)\ /\ 12$$

*Where*:

**$RR_v$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_v$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)
**12** = Unit conversion factor (in/ft)



BACK TO TOC

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the practice:

Using the $RR_V$ calculated, determine the minimum Volume of the Practice (VP)

$$(VP_{MIN}) \geq RR_v \text{ (target) / (RR\%)}$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice

**$VP_{MIN}$** = Minimum storage volume required to provide Runoff Reduction Target Volume ($ft^3$)

**$RR_v$ (target)** = Runoff Reduction Target Volume ($ft^3$)

**(Step 3B) Determine the storage volume of the practice and the Pretreatment Volume**

To determine the actual volume provided in the infiltration practice, use the following equation:

$$VP = (PV + VES (N))$$

*Where*:

**VP** = Volume provided (temporary storage)

**PV** = Ponding Volume

**VES** = Volume of Engineered Soils

**N** = Porosity

To determine the porosity, a qualified licensed professional should be consulted to determine the proper porosity based on the engineered soils used. Most soil media has a porosity of 0.25 and gravel a value of 0.40.

Provide pretreatment by using a grass filter strip or pea gravel diaphragm, 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. Pre-treatment may also be desired to reduce flow velocities or 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. Refer to Section 4.9 for design criteria for a grass channel and Section 4.29 for vegetated filter strips.

**(Step 3C) Determine whether the minimum storage volume was met.**

When the $VP \geq VP_{MIN}$, then the Runoff Reduction requirements are met for this practice. Proceed to Step 5.

When the $VP < VP_{MIN}$, then the BMP must be sized according to the $WQ_V$ treatment method (See Step 4).

(Step 4A) Calculate the Target Water Quality Volume

Calculate the Water Quality Volume using the following formula:

$$WQ_v = (1.2)\ (R_v)\ (A) / 12$$

*Where*:

**$WQ_v$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_v$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)
**A** = Site area (total drainage area) ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4B) If using the practice for Water Quality treatment, determine the footprint of the infiltration practice and the Pretreatment Volume required.

The peak rate of discharge for the water quality design storm is needed for sizing of off-line diversion structures (see Subsection 3.1.7). If designing off-line, follow steps (a) through (d):

(a) Using $WQ_v$, compute CN
(b) Compute time of concentration using TR-55 method
(c) Determine appropriate unit peak discharge from time of concentration
(d) Compute $Q_{wq}$ from unit peak discharge, drainage area, and $WQ_v$.

The required dimensions of an infiltration practice that will be filled with a planting media can be determined using the following equation, which is based on Darcy's Law:

$$A_{in} = (WQ_v)(d_{in}) / [(k_{in})(h_{in} + d_{in})(t_{drain})]$$

*Where*:

**$A_{in}$** = surface area of infiltration basin ($ft^2$)
**$WQ_v$** = Water Quality Volume, calculated in Step 4 ($ft^3$)
**$d_{in}$** = depth of infiltration basin planting bed (ft) (use 3 feet or more, unless a shallow water table is found on the development site)
**$k_{in}$** = coefficient of permeability of infiltration basin planting bed (ft/day) (use kin = 0.5 ft/day for engineered soil mix specified)
**$h_{in}$** = average height of ponded water above infiltration basin (ft) (use 50% of maximum ponding depth)
**$t_{drain}$** = design infiltration basin drain time (days) (use 72 hours or less)

(Step 5) Calculate the adjusted curve numbers for $CP_v$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm). See Subsection 3.1.7.5 for more information.

(Step 6) Size flow diversion structure, if needed.

A flow regulator (or flow splitter diversion structure) should be supplied to divert the $WQ_v$ (or $RR_v$) to the infiltration practice.

(Step 7) Size the underdrain system.

See Subsection 4.12.5.3 (*Physical Specifications/Geometry*)

(Step 8) Design the emergency overflow system.

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, 24-hour storm event.

(Step 9) Prepare Vegetation and Landscaping Plan.

A landscaping plan for an infiltration practice should be prepared to indicate how it will be established with vegetation. See Subsection 4.12.5.9 (*Landscaping*) and Appendix D for more details.

See Appendix B-4 for an Infiltration Trench Design Example

### 4.12.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.13 Multi-Purpose Detention Areas

**Description**: A facility designed primarily for another purpose, such as parking lots and rooftops that can provide water quantity control through detention of stormwater runoff.

**LID/GI Consideration**: Low land requirement, adaptable to many situations, and often a BMP used to treat runoff close to the source.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Must be designed to minimize potential safety risks, potential property damage, and inconvenience to the facility's primary purposes
- Adequate grading and drainage must be provided to allow full use of facility's primary purposes following a storm event
- Maximum depth of detention ponding in a parking lot should be 6 inches
- Must have a minimum slope of 0.5% towards the outlet, 1% or greater is recommended

### ADVANTAGES / BENEFITS

- Allows for multiple uses of site areas and reduces the need for downstream detention facilities
- Used in conjunction with water quality BMPs
- Adequate grading and drainage must be provided to allow full use of facility's primary purposes following a storm event

### DISADVANTAGES / LIMITATIONS

- Controls for stormwater quantity only – not intended to provide water quality treatment
- Restrictions in ponding depths

### ROUTINE MAINTENANCE REQUIREMENTS

- Keep practice free of trash, debris, and dirt
- Inspect area for excessive ponding of water
- Remove sediment as needed

### STORMWATER MANAGEMENT SUITABILITY

- Runoff Reduction
- ★ Water Quality
- Channel Protection
- ★ Overbank Flood Protection
- ★ Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- L Land Requirement
- L Capital Cost
- L Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *No*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 0% Runoff Reduction Credit is provided by this practice

BACK TO TOC

### 4.13.1 General Description

Multi-purpose detention facilities are site areas primarily used for one or more specific activities that are also designed to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts. Example of multi-purpose detention areas include:

- Parking Lots
- Rooftops
- Sports Fields
- Recessed Plazas

Multi-purpose detention areas are normally dry between rain events, and by their very nature must be useable for their primary function the majority of the time. As such, multi-purpose detention areas should not be used for extended detention.

Multi-purpose detention areas are not intended for water quality treatment and must be used in a treatment train approach with other BMPs that provide treatment of the $WQ_v$ (see Section 4.1).

### 4.13.2 Design Criteria and Specifications

#### Location

- Multi-purpose detention areas can be located upstream or downstream of other BMPs providing treatment of the water quality volume ($WQ_v$). See Section 4.1 for more information on the use of multiple BMPs in a treatment train.

#### ‘General Design

- Multi-purpose detention areas may be sized to temporarily store a portion or all of the volume of runoff required to provide overbank flood ($Q_{p25}$) protection (i.e., reduce the post-development peak flow of the 25-year storm event to the pre-development rate) and control the 100-year storm ($Q_f$) if required. Routing calculations must be used to demonstrate that the storage volume is adequate. See Section 3.3 (*Storage Design*) for procedures on the design of detention storage.
- All multi-purpose detention facilities must be designed to minimize potential safety risks, potential property damage, and inconvenience to the facility’s primary purposes. Emergency overflows are to be provided for storm events larger than the design storm. The overflow must not create an adverse impact to downstream properties or the conveyance system.

#### Parking Lot Storage

- Parking lot detention can be implemented in areas where portions of large, paved lots can be temporarily used for runoff storage without significantly interfering with normal vehicle and pedestrian traffic. Parking lot detention can be created in two ways: by using ponding areas along sections of raised curbing, or through depressed areas of pavement at drop inlet locations.
- The maximum depth of detention ponding in a parking lot, except at a flow control structure, should be 6 inches for a 10-year, 24-hour storm, and 9 inches for a 100-year, 24-hour storm. The maximum depth of ponding at a flow control structure is 12 inches for a 100-year, 24-hour storm.
- The storage area (portion of the parking lot subject to ponding) must have a minimum slope of 0.5% towards the outlet to ensure complete drainage following a storm. A slope of 1% or greater is recommended.
- Fire lanes used for emergency equipment must be free of ponding water for runoff events up to the extreme storm (100-year) event.
- Flows are typically backed up in the parking lot using a raised inlet.

#### Rooftop Storage

- Rooftops can be used for detention storage as long as the roof support structure is designed to address the weight of ponded water and is sufficiently waterproofed to achieve a minimum service life of 30 years. All rooftop detention designs must meet Georgia State Building Code and local building code requirements.
- The minimum pitch of the roof area subject to ponding is 0.25 inches per foot.
- The rooftop storage system must include another mechanism for draining the ponding area in the event that the primary outlet is clogged.
- See Section 4.11 for information and guidance on Green Roof practices.

#### Sports Fields

- Athletic facilities such as football fields, soccer fields, and tracks can be used to provide stormwater detention. This is accomplished by constructing berms around the facilities, which in essence creates very large and shallow detention basins. Outflow can be controlled through the use of an overflow weir or other appropriate control structure. Proper grading must be performed to ensure complete drainage of the facility.

#### Public Plazas

- In high-density areas, recessed public common areas such as plazas and pavilions can be utilized for stormwater detention. These areas can be designed to flood no more than once or twice annually, and provide important open recreation space during the rest of the year.
- Consult the design criteria for the dry detention basins (see Subsection 4.5.6, *Dry Detention Basins*) for the Multi-purpose Detention Basins sizing and design steps.

### 4.13.3 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.14 Organic Filter

**Description**: Design variant of the surface sand filter using organic materials in the filter media.

**LID/GI Consideration**: BMP used to treat runoff close to the source.

## KEY CONSIDERATIONS

### DESIGN CRITERIA
- Minimum head requirement of 5 to 8 feet
- Maximum contributing drainage area of 10 acres

### ADVANTAGES / BENEFITS
- Applicable to small drainage areas
- Good for highly impervious areas
- High pollutant removal capability
- Removal of dissolved pollutants is greater than sand filters due to cation exchange capacity

### DISADVANTAGES / LIMITATIONS
- Intended for hotspot, space-limited applications, or for areas requiring enhanced pollutant removal capability
- High maintenance burden
- Filter may require more frequent maintenance than most of the other stormwater BMPs
- Severe clogging potential if exposed soil surfaces exist upstream

### ROUTINE MAINTENANCE REQUIREMENTS
- Inspect for clogging – rake first inch of sand
- Remove sediment from forebay and chamber
- Replace sand filter media as needed

### POLLUTANT REMOVAL
- 80% Total Suspended Solids
- 75% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 60/40% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- 50% **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY
- Runoff Reduction
- ✓ Water Quality
- ★ Channel Protection
- Overbank Flood Protection
- Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS
- L Land Requirement
- H Capital Cost
- H Maintenance Burden

**Residential Subdivision Use:** *No*
**High Density/Ultra-Urban:** *Yes*
**Special Considerations:** *Hotspot areas*

**L**=Low **M**=Moderate **H**=High

### RUNOFF REDUCTION CREDIT
- 0% of the runoff reduction volume provided



BACK TO TOC

### 4.14.1 General Description

The organic filter is a design variant of the surface sand filter, which uses organic materials such as leaf compost or a peat/sand mixture as the filter media. The organic material enhances pollutant removal by providing adsorption of contaminants such as soluble metals, hydrocarbons, and other organic chemicals.

As with the surface sand filter, an organic filter consists of a pretreatment chamber, and one or more filter cells. Each filter bed contains a layer of leaf compost or the peat/sand mixture, followed by filter fabric and a gravel/perforated pipe underdrain system. The filter bed and subsoils can be separated by an impermeable polyliner or concrete structure to prevent movement into groundwater.

Organic filters are typically used in high-density applications, or for areas requiring an enhanced pollutant removal ability. Maintenance is typically higher than the surface sand filter facility due to the potential for clogging. In addition, organic filter systems have a higher head requirement than sand filters.

### 4.14.2 Pollutant Removal Capabilities

Peat/sand filter systems provide good removal of bacteria and organic waste metals.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- *Total Suspended Solids* – 80%
- *Total Phosphorus* – 60%
- *Total Nitrogen* – 40%
- *Fecal Coliform* – 50%
- *Heavy Metals* – 75%

### 4.14.3 Design Criteria and Specifications

- Organic filters are typically used on relatively small sites (up to 10 acres), to minimize potential clogging.
- The minimum head requirement (elevation difference needed at a site from the inflow to the outflow) for an organic filter is 5 to 8 feet.
- Organic filters can utilize a variety of organic materials as the filtering media. Two typical media bed configurations are the peat/sand filter and compost filter (see **Figure 4.14-1**). The peat filter includes an 18-inch 50/50 peat/sand mix over a 6-inch sand layer and can be optionally covered by 3 inches of topsoil and vegetation. The compost filter has an 18-inch compost layer. Both variants utilize a gravel underdrain system.
- The type of peat used in a peat/sand filter is critically important. Fibric peat in which undecomposed fibrous organic material is readily identifiable is the preferred type. Hemic peat containing more decomposed material may also be used. Sapric peat made up of largely decomposed matter should not be used in an organic filter.
- Typically, organic filters are designed as "off-line" systems, meaning that the water quality volume ($WQ_v$) is diverted to the filter facility through the use of a flow diversion structure and flow splitter. Stormwater flows greater than the $WQ_v$ are diverted to other BMPs or downstream using a diversion structure or flow splitter.
- Consult the design criteria for the surface sand filter (see Subsection 4.21.4, *Sand Filters*) for the organic filter sizing and design steps.

## 4.14.4 Inspection and Maintenance equirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

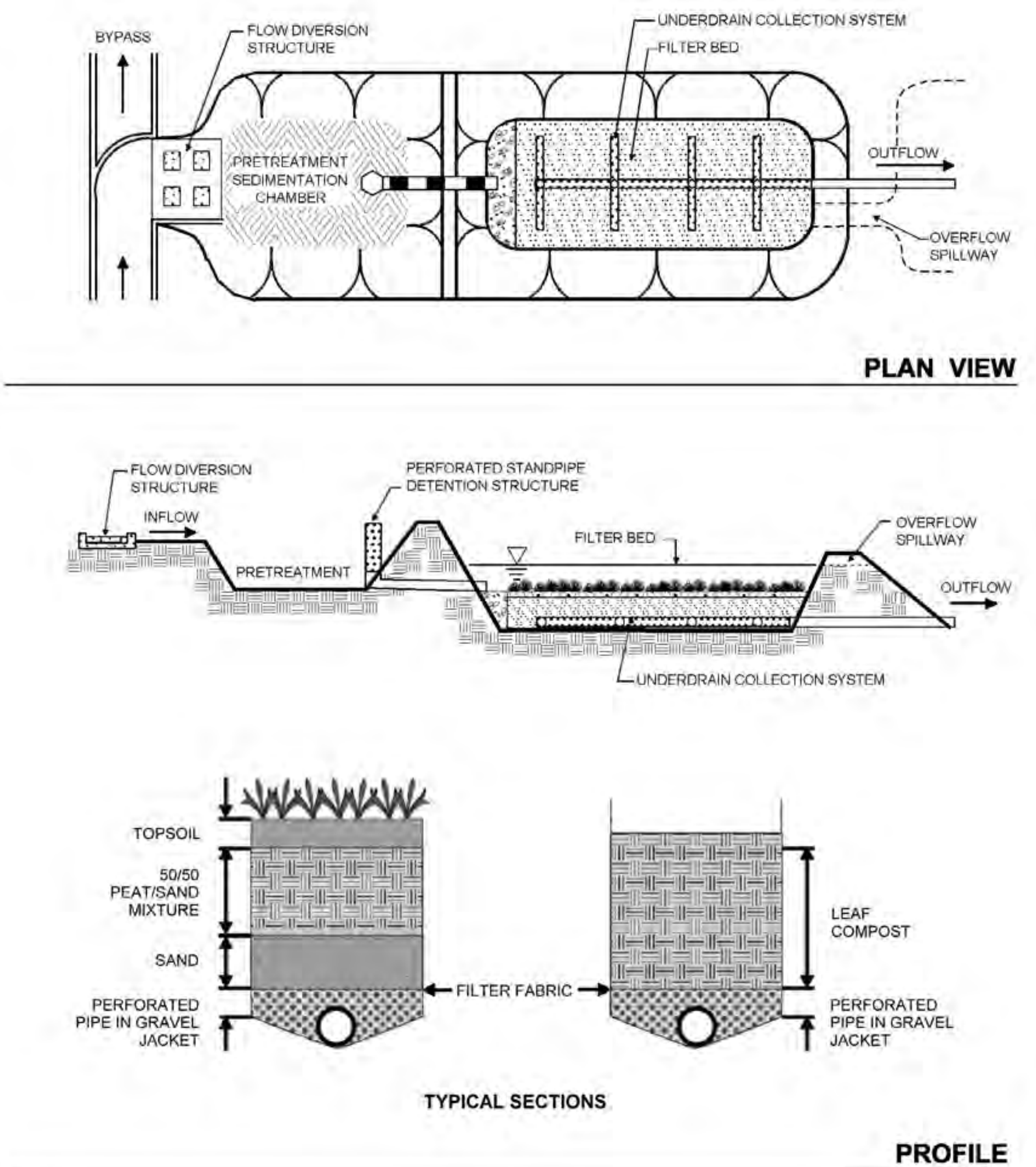
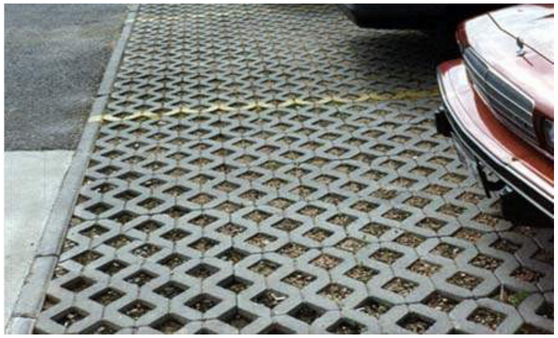



**Figure 4.14-1** Schematic of Organic Filter
(Source: Center for Watershed Protection)

## 4.15 Permeable Paver Systems



**Description**: A pavement surface composed of structural units with void areas that are filled with pervious materials such as gravel, sand, or grass turf. Permeable paver systems are installed over a gravel base course that provides structural support and stores stormwater runoff that infiltrates through the system into underlying permeable soils.

**LID/GI Consideration:** A permeable paver system provides water quality benefits in addition to groundwater recharge and a reduction in stormwater volume. The use of permeable paver systems should result in a reduction of impervious area on a site.

### KEY CONSIDERATIONS

**DESIGN CRITERIA**

- Intended for low traffic areas, or for residential or overflow parking applications, not ideal for areas with a tree canopy
- Aesthetically pleasing
- Americans with Disabilities Act (ADA) compliant
- Should be a minimum of two feet above the natural water table
- Should be a minimum of 15 feet away from buildings

**ADVANTAGES / BENEFITS**

- Surface flow reduction of peak flows, volume, and stormwater runoff
- High level of pollutant removal
- Aesthetic pleasing options
- Reusable product
- Longer life than traditional pavement
- Decreases impermeable area

**DISADVANTAGES / LIMITATIONS**

- High cost compared to conventional pavements
- Potential for high failure rate if not adequately maintained or used in unstabilized areas
- Geotechnical analysis of soils required
- Ineffective under tree canopy, due to clogging
- Requires specialized knowledge for proper installation

**ROUTINE MAINTENANCE REQUIREMENTS**

- High maintenance requirements
- Weed and remove grass out of bricks/blocks as necessary (unless concrete grid pavers are used)
- Sweep or vacuum the pavers as necessary

**POLLUTANT REMOVAL**

- 80% **Total Suspended Solids**
- 60% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 50/50% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- N/A **Pathogens** – Fecal Coliform

**STORMWATER MANAGEMENT SUITABILITY**

- ✓ Runoff Reduction
- ✓ Water Quality
- ★ Channel Protection
- ★ Overbank Flood Protection
- ★ Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

**IMPLEMENTATION CONSIDERATIONS**

- L Land Requirement
- H Capital Cost
- H Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *Yes*

**Other Considerations:** *Overflow parking, driveways, & related uses*

L=Low M=Moderate H=High

**RUNOFF REDUCTION CREDIT**

- 100% of the runoff reduction credit if an underdrain is not used
- 75% of the runoff reduction credit if an upturned underdrain is used
- 50% of the runoff reduction credit if an underdrain is used

BACK TO TOC

### 4.15.1 General Description

Modular permeable paver systems are structural units with regularly inter dispersed void areas used to create a load-bearing pavement surface. The void areas are filled with pervious materials (gravel) to create a system that can infiltrate stormwater runoff. Permeable paver systems provide water quality benefits in addition to groundwater recharge and a reduction in stormwater volume. The use of permeable paver systems should result in a reduction of impervious area on a site.

There are many different types of modular permeable paver systems available from different manufacturers, including both pre-cast and mold in-place concrete blocks, concrete grids, interlocking bricks, and plastic mats with hollow rings or hexagonal cells (see **Figure Page 1**).

Modular permeable paver systems are typically placed on a gravel (stone aggregate) base course. Runoff infiltrates through the permeable paver surface into the gravel base course, which acts as a storage reservoir as it exfiltrates runoff to the underlying soil. The infiltration rate of the soils in the subgrade must be adequate to support drawdown of the entire runoff capture volume within 48-72 hours. Special care must be taken during construction to avoid undue compaction of the underlying soils, which could affect the soils' infiltration capability.

Modular permeable paver systems are typically used in low-traffic areas with low to no tree coverage such as:

- Parking pads in parking lots
- Overflow parking areas
- Residential driveways
- Residential street parking lanes
- Recreational trails
- Golf cart and pedestrian paths
- Emergency vehicle and fire access lanes

A major drawback is the cost and complexity of modular permeable paver systems compared to conventional pavements. Permeable paver systems require a very high level of construction workmanship to ensure that they function as designed. In addition, there is the difficulty and cost of rehabilitating the surfaces when they become clogged. Therefore, consideration of permeable paver systems should include the construction and maintenance requirements and costs.

### 4.12.2 Stormwater Management Suitability

Permeable paver systems can be designed for water quantity, but they are mostly used for water quality, i.e. the removal of stormwater pollutants, depending upon the native soils. Permeable paver systems can provide runoff quantity control, particularly for smaller runoff volumes such as the runoff volume generated by the water quality storm event (1.2 inches). These facilities may sometimes be used to partially or completely meet channel protection requirements on smaller sites. However, permeable paver systems will typically need to be used in conjunction with another control to provide channel protection, as well as overbank flood protection. Permeable paver systems need to be designed and maintained to safely bypass higher flows.

- Runoff Reduction
  Like other LID practices, permeable paver systems become more effective with higher infiltration rates of native soils. A permeable paver system can be designed to provide 100% of the runoff reduction volume, if properly maintained.

- Water Quality
  The permeable paver system is an excellent stormwater treatment practice due to the variety of pollutant removal mechanisms. Each of the components of the permeable paver system 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 permeable paver system acts as a filtration system, and clay in the soil provides adsorption sites for hydrocarbons, heavy metals, nutrients and other pollutants.

- Channel Protection
  For smaller sites, a permeable paver system may be designed to capture the entire channel protection volume ($CP_v$). Given that a permeable paver system is typically designed to completely drain over 48-72 hours, the 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 permeable paver system, another practice must be used to provide $CP_v$ extended detention.

- Overbank Flood Protection
  Although relatively rare, on some development sites, a permeable paver system can be designed to attenuate the overbank peak discharge ($Q_{p25}$).

- Extreme Flood Protection
  Although relatively rare, on some development sites, a permeable paver system can be designed to attenuate the extreme peak discharge ($Q_f$).

*Credit for the volume of runoff reduced in the permeable paver system may be taken in the overbank flood protection and extreme flood protection calculations. If the practice is designed to provide runoff reduction for water quality compliance, then the practice is given credit for channel protection and flood control requirements by allowing the designer to compute an Adjusted CN (see Section 3.1.7.5 for more information).*

### 4.15.3 Pollutant Removal Capabilities

As they provide for the infiltration of stormwater runoff, permeable paver systems have a high removal of both soluble and particulate pollutants, since they become trapped, absorbed or broken down in the underlying soil layers. Due to the potential for clogging, permeable paver surfaces should not be used for the removal of sediment or other coarse particulate pollutants.

For additional information and data on pollutant removal capabilities for permeable pavers, see the National Pollutant Removal Performance Database (3rd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.15.4 Design Criteria and Specifications

- Permeable paver systems can be used where the underlying in-situ subsoils have an infiltration rate of at least 0.5 inches per hour. Therefore, permeable paver systems are not suitable on sites with hydrologic group D or most group C soils, or soils with a high (>30%) clay content. During construction and preparation of the subgrade, special care must be taken to avoid compaction of soils.

- Permeable paver systems should typically be used in applications where the pavement receives tributary runoff only from impervious areas. The ratio of the contributing impervious area to the permeable paver surface area should be no greater than 3:1.

- If runoff is coming from adjacent pervious areas, it is important that those areas be fully stabilized to reduce sediment loads and prevent clogging of the permeable paver surface.

- It is recommended that the subsoil of the permeable paver systems have a slope of 0% and the surface have a slope of 0.5% if possible.

- A minimum of 2 feet of clearance is required between the bottom of the gravel base course and underlying bedrock or the seasonally high groundwater table.

- Permeable paver systems should be sited at least 15 feet down gradient from buildings and

- 100 feet away from drinking water wells.

- An appropriate modular permeable paver should be selected for the intended application. A minimum of 40% of the surface area should consist of open void space. If it is a load bearing surface, then the pavers should be able to support the maximum load.

- The permeable paver infill is selected based upon the intended application and required infiltration rate. Masonry sand (such as ASTM C-33 concrete sand or GADOT Fine Aggregate Size No. 10) has a high infiltration rate (8 in/hr) and should be used in applications where no vegetation is desired. A sandy loam soil has a substantially lower infiltration rate (1 in/hr), but will provide for growth of a grass ground cover.

- A 1-inch top course (filter layer) of sand (ASTM C-33 concrete sand or GADOT Fine Aggregate Size No. 10) underlain by filter fabric is placed under the permeable pavers and above the gravel base course.
- The gravel base course should be designed to store at a minimum the water quality volume ($WQ_v$). The stone aggregate used should be washed, bank-run gravel, 1.5 to 2.5 inches in diameter with a void space of about 40% (GADOT No.3 Stone). Aggregate contaminated with soil should not be used. A porosity value (void space/total volume) of 0.32 should be used in design calculations.
- The gravel base course must have a minimum depth of 12 inches. The following equation can be used to determine if the depth of the storage layer (gravel base course) needs to be greater than the minimum depth:

$$RR_V = A\ [(p1)(d1)]$$

*Where*:

**$RR_V$** - Runoff reduction volume ($ft^3$)
**A** - area of permeable paver system ($ft^2$)
**p1** - porosity of base layer (% void)
**d1** - depth of base layer (ft)
*Note that this formula works for surfaces with a 0% slope.*

- The upper surface of the subgrade should be lined with filter fabric or an 8-inch layer of sand (ASTM C-33 concrete sand or GADOT Fine Aggregate Size No. 10) and be completely flat to promote infiltration across the entire surface.
- Permeable paver system designs must use some method to convey larger storm rainfall event flows to the conveyance system. One option is to use storm drain inlets set slightly above the elevation of the pavement. This would allow for some ponding above the surface, but also accepting bypass flows that are too large to be infiltrated by the permeable paver system. This also helps to address concerns about handling flows when the paver system is clogged.
- For the purpose of sizing downstream conveyance and structural control system, permeable paver surface areas can be assumed to be 40% impervious. In addition, credit can be taken for the runoff volume infiltrated from other impervious areas using the methodology described in Section 3.1.

### 4.15.5 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of a permeable paver system.**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a permeable paver systems. Create a rough layout of the permeable paver systems dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the permeable paver system.**

Consider whether the permeable paver system is intended to:

- Meet a runoff reduction* target or water quality (treatment) target. For information on the sizing of a BMP utilizing the runoff reduction approach, see Step 3A. For information on the sizing of the BMP utilizing the water quality treatment approach, see Step 4A. **Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*
- Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)
- Provide a possible solution to a drainage problem
- Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.).

*Complete Step 3A, 3B, and 3C for a runoff reduction approach, or skip Step 3 and complete Steps 4A and 4B for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) **Calculate the Stormwater Runoff Reduction Target Volume**

Calculate the Runoff Reduction Volume using the following formula:

$$RR_V = (P)\,(R_V)\,(A) / 12$$

*Where*:

**$RR_V$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the practice:

Using the $RR_V$ calculated, determine the minimum Volume of the Practice (VP)

$$(VP_{MIN}) \geq RR_v \text{ (target) / (RR\%)}$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice
**$VP_{MIN}$** = Minimum storage volume required to provide Runoff Reduction Target Volume ($ft^3$)
**$RR_v$ (target)** = Runoff Reduction Target Volume ($ft^3$)

**(Step 3B) Determine the storage volume of the practice and the Pretreatment Volume**

To determine the actual volume provided in the permeable paver system, use the following equation:

$$VP = (PV + VES\ (N))$$

*Where*:

VP = Volume provided (temporary storage)
**PV** = Ponding Volume
**VES** = Volume of Engineered Soils
**N** = Porosity

To determine the porosity, a qualified licensed professional should be consulted to determine the proper porosity based on the engineered soils used. Most soil media has a porosity of 0.25 and gravel a value of 0.40.

Provide pretreatment by using a grass filter strip or pea gravel diaphragm, 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 may also be desired to reduce flow velocities or 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. Refer to Section 4.9 for design criteria for a grass channel and Section 4.29 for vegetated filter strips.

**(Step 3C) Determine whether the minimum storage volume was met.**

When the $VP \geq VP_{MIN}$, then the Runoff Reduction requirements are met for this practice. Proceed to Step 5.

When the $VP < VP_{MIN}$, then the BMP must be sized according to the $WQ_V$ treatment method (See Step 4).

(Step 4A) Calculate the Target Water Quality Volume

Calculate the Water Quality Volume using the following formula:

$$WQ_V = (1.2)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$WQ_V$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Site area (total drainage area) ($ft^2$)
**12** = Unit conversion factor (in/ft)

**(Step 4B) If using the practice for Water Quality treatment, determine the footprint of the infiltration practice and the Pretreatment Volume required.**

The peak rate of discharge for the water quality design storm is needed for sizing of off-line diversion structures (see Subsection 3.1.7). If designing off-line, follow steps (a) through (d) below:

.

(a) Using $WQ_V$, compute CN
(b) Compute time of concentration using TR-55 method
(c) Determine appropriate unit peak discharge from time of concentration
(d) Compute $Q_{wq}$ from unit peak discharge, drainage area, and $WQ_V$.

To determine the minimum surface area of the permeable paver system, use the following formula:

$$A_f = (WQ_v)(d_g + d_{es})\ /\ [(k_g * d_g * t_g) + (k_{es} * d_{es} * t_{es})]$$

*Where*:

**$A_f$** = surface area of permeable paver system ($ft^2$)
**$WQ_v$** = Water Quality Volume ($ft^3$)
**$d_g$** = gravel depth
**$d_{es}$** = engineered soil layer depth (ft)
**$k_g$** = coefficient of permeability for gravel (ft/day)
**$k_{es}$** = coefficient of permeability for engineered soil (ft/day)
**$t_g$** = drain time of gravel (days)
**$t_{es}$** = drain time of engineered soil (days)

**(Step 5) Calculate the adjusted curve numbers for $CP_V$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm). See Subsection 3.1.7.5 for more information.**

**(Step 6) Design system outlets**

Determine which type of outlet design will be used for the permeable paver system. There are two types of outlet design that are generally used if infiltration is not possible without additional assistance. They are an underdrain system and overflow system. The underdrain system should include a series of perforated pipes that run longitudinal with the pavers to remove additional stormwater runoff that could not otherwise infiltrate into the surrounding soil. An overflow system directs water that cannot be infiltrated into the subsoil and moves it to another location, for instance another BMP or storm sewer system.

(Step 7) Erosion and Sediment Control/Base Protection

Determine the stormwater discharges to the construction site that could potentially erode and clog the system. Take the proper steps to stabilize the site and prevent erosion when construction begins.

(Step 8) Select Permeable Paver System and finalize design

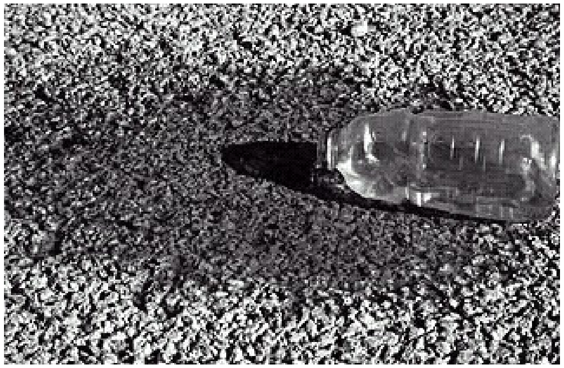
Select the most appropriate paver system based on the specific site conditions. Finalize the design the of the practice. Make sure that the soil is stabilized by using filter fabric or other method as determined by the designer.

### 4.15.6 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs should include considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

BACK TO TOC

# 4.16 Pervious Concrete



**Description**: Pervious concrete is a mixture of coarse aggregate, Portland cement and water that allows for rapid infiltration of water and overlays a stone aggregate reservoir. This reservoir provides temporary storage as runoff infiltrates into underlying permeable soils and/or out through an underdrain system.

**LID/GI Consideration:** Since pervious concrete is often designed primarily for stormwater quantity, but can also provide runoff quality control when used as an integral part of the stormwater management system plan, it can be considered an LID/GI control.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Refer to ACI 522 Report on Pervious Concrete for design considerations
- Construction recommended by National Ready Mixed Concrete Association (NRMCA) certified personnel as specified in ACI 522.1
- Typically used for low volume auto traffic areas, or for overflow parking applications
- Not recommended on sites with low permeability soils, wellhead protection zones, or water supply aquifer recharge areas
- Although not typical, pervious concrete can be designed to accommodate heavier vehicles under certain circumstances

### ADVANTAGES / BENEFITS

- Provides reductions in runoff volume, stormwater runoff, and impervious area
- Helps minimize size of detention ponds
- Particularly well suited in capturing "first flush" water quality volume
- Reduces standing water on pavement
- May help to reduce stormwater management costs

### DISADVANTAGES / LIMITATIONS

- Somewhat higher installation cost than for conventional pavement
- Infiltration testing of existing soils may be required
- Not typically recommended for areas with heavy traffic or trucks.
- Not recommended under tree canopy

### ROUTINE MAINTENANCE REQUIREMENTS

- Keep concrete free of trash, debris, and dirt
- Use street sweepers or vacuum trucks to clean pervious concrete as needed
- Keep grass surrounding the area trimmed and remove grass clippings from area
- Occasional pressure washing may be necessary

### POLLUTANT REMOVAL

80% **Total Suspended Solids**

60% **Metals** - Cadmium, Copper, Lead, and Zinc removal

50/65% **Nutrients** - Total Phosphorus / Total Nitrogen removal

N/A **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

✓ Runoff Reduction
✓ Water Quality
★ Channel Protection
★ Overbank Flood Protection
★ Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

L Land Requirement
H Capital Cost
H Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*

**Other Considerations:** *Overflow parking, driveways, & related uses*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 100% of the runoff reduction credit if an underdrain is not used
- 75% of the runoff reduction credit if an upturned underdrain is used
- 50% of the runoff reduction credit if an underdrain is used

BACK TO TOC

## 4.16.1 General Description

Pervious concrete (also referred to as *enhanced porosity concrete, porous concrete, Portland cement pervious pavement* and *pervious pavement*) is a subset of a broader family of pervious pavements including porous asphalt, and various kinds of grids and paver systems. Pervious concrete is generally regarded to have a greater ability than permeable asphalt to maintain its porosity in hot weather and thus is provided as a limited application control. Please see **Figure 4.16-1** for typical applications.

Past failures have been attributed to poor design, inadequate construction techniques, soils with low permeability, heavy vehicular traffic and poor maintenance. However, pervious concrete has experienced increasing success in Georgia due to advancements in concrete mix designs, installation techniques and maintenance acceptance. In particular, wider acceptance of the NRMCA Pervious Concrete Contractor certification program (as required by ACI 522) has led to improved contractor performance

Pervious concrete consists of a specially formulated mixture of Portland cement, uniform, open graded course aggregate, and water. The concrete layer has a high permeability, such that the underlying permeable soil layer allows rapid percolation of rainwater through the surface and into the layers beneath. The void space in pervious concrete is in the 15-22% range, as opposed to 3-5% for conventional pavements. The permeable surface is placed over a layer of open-graded gravel and crushed stone. The void spaces in the stone act as a storage reservoir for runoff.

Pervious concrete is often designed primarily for stormwater quantity (i.e. the removal of stormwater volume). However, it can provide runoff quality control, when used as an integral part of the stormwater management system plan. Pervious concrete systems can be designed to capture and infiltrate the water quality volume ($WQ_v$) and the channel protection volume ($CP_v$) as well as providing an infiltration option for overbank protection and extreme flood protection.

Modifications or additions to the standard design have been used to pass flows and volumes in excess of the water quality volume, or to increase storage capacity and treatment. These include but are not limited to:

- Placing a perforated pipe near the top of the crushed stone reservoir to pass excess flows after the reservoir is filled
- Providing surface detention storage in a parking lot, adjacent swale, or detention pond with suitable overflow conveyance
- Connecting the stone reservoir layer to a stone filled trench
- Adding a sand layer and perforated pipe beneath the stone layer for filtration of the water quality volume
- Placing an underground detention tank or vault system beneath the layers

**Figure 4.16-1** Typical Pervious Concrete System Applications
(Photos by Bruce Ferguson, Don Wade)

BACK TO TOC

In order to meet water quality standards (WQv) and accommodate extended detention requirements for the $CP_v$, the minimum drawdown time should be 24-48 hours. Longer drawdown is acceptable as necessary to infiltrate, bypass or detain and release larger storm events with a maximum drawdown time of 5 days. Undue compaction, which could affect the soils' infiltration capability, should be avoided.

Pervious concrete systems are typically used in low-traffic areas such as:

- Parking pads in parking lots
- Overflow parking areas
- Residential street parking lanes
- Recreational trails
- Golf cart and pedestrian paths
- Emergency vehicle and fire access lanes

It is recommended that subsoil slopes should be equal to or less than 0.5% slope. Surface slopes should be 6% or less, with 2% maximum slope preferable. The seasonally high water table or bedrock should be a minimum of two feet below the bottom of the pervious concrete, if infiltration is relied on to drain the stored volume.

Pervious concrete has the positive characteristics of volume reduction due to infiltration, groundwater recharge, and an ability to blend into the normal urban landscape relatively unnoticed. It also allows a reduction in the cost of other stormwater infrastructure, which may offset the greater upfront capital cost of pervious (versus impervious) concrete.

Like other infiltration controls, pervious concrete should not be used in areas that experience high rates of wind erosion or in drinking water aquifer recharge areas.

### 4.16.2 Stormwater Management Suitability

Pervious concrete can provide runoff quantity control, particularly for smaller runoff volumes such as the runoff volume generated by the water quality storm event (1.2 inches). These facilities may sometimes be used to partially or completely meet channel protection requirements on smaller sites. However, pervious concrete will typically need to be used in conjunction with another control to provide channel protection, as well as overbank flood protection.

- Runoff Reduction
  Like other LID practices, pervious concrete becomes more effective with higher infiltration rates of native soils. Pervious concrete can be designed to provide 100% of the runoff reduction volume, if properly maintained.

- Water Quality
  Pervious concrete provides some amount of pollutant removal, but is not well suited for removing sediment due to the sediment potentially clogging the pores within the pervious concrete.

- Channel Protection
  For smaller sites, the pervious concrete may be designed to capture the entire channel protection volume ($CP_v$). Given that the storage volume under the pervious concrete is typically designed to completely drain over 48-72 hours, the 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 pervious concrete, another practice must be used to provide $CP_v$ extended detention.

- Overbank Flood Protection
  Although relatively rare, on some development sites, pervious concrete can be designed to attenuate the overbank peak discharge ($Q_{p25}$).

- Extreme Flood Protection
  Although relatively rare, on some development sites, pervious concrete can be designed to attenuate the extreme peak discharge ($Q_f$).

*Credit for the volume of runoff reduced in pervious concrete may be taken in the overbank flood protection and extreme flood protection calculations. If the practice is designed to provide runoff reduction for water quality compliance, then the practice is given credit for channel protection and flood control requirements by allowing the designer to compute an Adjusted CN (see Subsection 3.1.7.5 for more information).*

BACK TO TOC

### 4.16.3 Pollutant Removal Capabilities

As they provide for the infiltration of stormwater runoff, pervious concrete systems have a high removal rate for both soluble and particulate pollutants, which become trapped, absorbed or broken down in the underlying soil layers. Due to the potential for clogging, pervious concrete surfaces should not be used for the removal of sediment or other coarse particulate pollutants.

For additional information and data on pollutant removal capabilities for pervious concrete, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the national Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

Pollutant removal can be improved through routine vacuuming, sweeping, and high pressure washing of pervious concrete systems, maintaining a drainage time of at least 24 hours, pretreating the runoff, having organic material in the subsoil, and using clean washed aggregate (EPA, 1999).

### 4.16.4 Application and Site Feasibility Criteria

Pervious concrete is suitable for many types of development, from single-family residential to high-density commercial projects. Though not often used for heavily traveled roads, it is well suited for alleys, driveways, and parking lots.

*The following criteria should be evaluated to ensure the suitability of pervious concrete for meeting stormwater management objectives on a site or development.*

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **YES**
- Regional Stormwater Control – **NO**

**Physical Feasibility -** Physical Constraints at Project Site

- **Site Slope** – Slopes should be a maximum of 6%
- **Minimum Depth to Water Table** – A separation distance of 2 feet is recommended between the bottom of the rock storage area of the pervious concrete and the elevation of the seasonally high water table.
- **Soils** – Native soils if they have at least 0.5 inch/hr infiltration ability.

Other Constraints/Considerations

- **Hot spots** – Do not use for hot spot runoff.
- **Damage to existing structures and facilities** – Consideration should be given to the impact of water exfiltrating the pervious concrete.
- **Proximity** – The following is a list of specific setback requirements for the location of pervious concrete:
  - » 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** – Evaluate for stream warming when an underdrain system is used.

In addition, careful consideration should be given to the potential of perched or raised groundwater levels. Provide adequate distance from building foundations or use impermeable liner on side of excavated area nearest to structure.

**Challenges and Potential Solutions for Coastal Areas**

- » **Poorly Drained Soils**—This condition minimizes the ability of pervious concrete to reduce stormwater runoff rates and volumes. One solution would be to include an underdrain system.
- » **Shallow Water Table**—*This can prevent the provision of 2 feet of clearance between the bottom of the pervious concrete and the top of the water table which may cause stormwater runoff to pond in the storage layer of the pervious concrete.*

### 4.16.5 Planning and Design Criteria

- Pervious concrete systems can be used where the underlying in-situ subsoils have an infiltration rate greater than 0.5 inches per hour. Infiltration rates for in-situ soils should be determined through geotechnical investigation prior to design of the system. During construction and preparation of the subgrade, over compaction of soils should be avoided. Refer to ACI 522 for compaction recommendations.

- Pervious concrete systems should typically be used in applications where the pavement receives tributary runoff only from impervious areas. It is recommended that the ratio of tributary impervious area to the area of the pervious concrete system should be a maximum of 1:1. Pervious concrete systems should be sized for a minimum drawdown time of 24 hours and a maximum drawdown time of 5 days.

- Tributary runoff from adjacent pervious areas is not recommended. However, if it is necessary for runoff to come from adjacent pervious areas, it is important that those areas be fully stabilized to reduce sediment loads and prevent clogging of the pervious surface. Pretreatment using dry or wet enhanced swales or vegetated filter strips for removal of course sediments is recommended and continued maintenance of these areas will be required. (see Sections 4.6 and 4.29)

- A minimum of four feet of clearance is recommended (may be reduced to two feet in coastal areas) between the bottom of the gravel base course and underlying bedrock or the seasonally high groundwater table.

- Pervious concrete systems should be sited at least 10 feet down-gradient from buildings and 100 feet away from drinking water wells.

- To protect groundwater from potential contamination, runoff from designated hotspot land uses or activities must not be infiltrated. Pervious concrete should not be used on manufacturing or industrial sites, where there is potential for high concentrations of soluble pollutants and heavy metals. In addition, pervious concrete should not be considered for areas with high pesticide concentrations. Pervious concrete is also not suitable in areas with karst geology without adequate geotechnical testing and approval by qualified individuals and in accordance with local requirements.

- Pervious concrete systems can be used independently or in conjunction with other stormwater management system components to effectively infiltrate, bypass, or detain and time-release all required storm events..

- For the purpose of sizing downstream conveyance and structural control system, pervious concrete surface areas can be estimated as 35% impervious.

- For treatment control, the design volume should be, at a minimum, equal to the water quality volume. The water quality storage volume is contained in the surface layer and the aggregate reservoir. The storm duration (fill time) is normally short compared to the infiltration rate of the sub-grade, so storage duration of two hours can be used for design purposes. The total storage volume in a layer is equal to the percent of void space times the volume of the layer.

- For the purpose of sizing downstream conveyance and structural control system, pervious concrete surface areas can be estimated as 35% impervious.

- For treatment control, the design volume should be, at a minimum, equal to the water quality volume. The water quality storage volume is contained in the surface layer and the aggregate reservoir. The storm duration (fill time) is normally short compared to the infiltration rate of the sub-grade, so storage duration of two hours can be used for design purposes. The total storage volume in a layer is equal to the percent of void space times the volume of the layer.

- The cross-section typically consists of two layers, as shown in **Figure 4.16-2**. The aggregate reservoir can sometimes be avoided or minimized if the sub-grade is sandy and there is adequate time to infiltrate the necessary runoff volume into the sandy soil without by-passing the water quality volume. A description for each of the layers is presented below:
  - » **Pervious Concrete Layer** – The pervious concrete layer consists of an open-graded concrete mixture usually ranging from depths of 6 – 12 inches depending on required bearing strength and pavement design requirements. Pervious concrete can be assumed to contain 18 percent voids (porosity = 0.18) for design purposes. Thus, for example, a 6 inch thick pervious concrete layer would hold 1.08 inches of rainfall. Refer to ACI 522R (most current version) for recommendations on mix proportioning for pervious concrete. See the GCPA specifications (referenced) as well.
  - » **Reservoir Layer** – The reservoir gravel base course consists of washed, bank-run gravel, 1.5 2.5 inches in diameter with a void space of about 40% (GADOT No.3 Stone). The depth of this layer depends on the desired storage volume, which is typically the water quality volume (WQv) at a minimum. Typical depths for the reservoir layer range from 2-4 feet. Aggregate contaminated with soil shall not be used. A porosity value (void space/ total volume) of 0.40 should be used in calculations unless aggregate-specific data exist.

**Figure 4.16-2** Pervious Concrete System Section
(Modified From: LAC 2000

  - » **Filter Fabric** – Filter fabric can be used is certain applications, as site conditions warrant. General guidance for the use of filter fabrics is below. Actual use should be under the guidance of a Georgia licensed engineer.
  - » **Geotextiles** consisting of permeable materials should line the sides of the aggregate base to prevent migration of adjacent soils into it and subsequent permeability and storage capacity reduction. Geotextiles are not recommended under the aggregate base in an infiltration design because they can accumulate fine particulates that inhibit infiltration.

» Geomembranes consisting of impermeable materials should be used to accomplish the following:

- Provide a barrier on the side and bottom of the aggregate base in a detention design to prevent infiltration into the subgrade typically due to soil instability, the presence of stormwater hotspots, or potential for groundwater contamination. Geomembrane barriers reduce the credit for TSS removal from 85% to 70%.
- Line the sides of the aggregate base whenever structure foundations of conventional pavement are 20 feet or less from the permeable pavement (to avoid the risk of structural damage due to seepage). The use of geomembranes for this purpose will not reduce credit for TSS removal in the system.

» Geogrids may be used on top of subgrade soils for additional structural support, especially in very weak, saturated soils. All manufacturer requirements must be followed in design and installation of geogrids.

» **Underlying Soil** – Pervious concrete systems cannot be used in fill soils. The underlying soil should have an infiltration capacity of at least 0.50 in/hr. as initially determined from NRCS soil textural classification, and subsequently confirmed by field geotechnical tests. The minimum geotechnical testing is one test hole per 5000 square feet, with a minimum of two borings per practice (taken within the proposed limits of the facility). Test borings are recommended to determine the soil classification, seasonal high ground water table elevation, impervious substrata, and an initial estimate of permeability. Often a double-ring infiltrometer test is done at subgrade elevation to determine the impermeable layer, and, for safety, one-half the measured value is allowed for infiltration calculations.

- The pit excavation should be limited to the width and depth specified in the design. Excavated material should be placed away from the open trench to avoid jeopardizing the stability of the trench sidewalls. The bottom of the excavated trench should not be loaded so as to cause compaction, and should be scarified prior to placement of reservoir base material. The sides of the trench should be trimmed of all large roots. The sidewalls should be uniform with no voids and scarified prior to backfilling. All pervious concrete systems should be protected during construction and constructed after upstream areas are stabilized.

- An observation well consisting of perforated PVC pipe 4-6 inches in diameter should be placed at the downstream end of the facility and protected during site construction. The well should be used to determine actual infiltration rates for use in final design of the pervious concrete system.

- A warning sign should be placed at the facility that states, "Pervious Paving used on this site to reduce pollution. Do not resurface with non-pervious material. Call (XXX) XXX-XXXX for more information."

- Details of construction of the concrete layer are beyond the scope of this manual. However, construction by NRMCA certified personnel, following the guidelines of ACI 522R and ACI 522.1, is recommended.



BACK TO TOC

### 4.16.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of the pervious concrete.**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for pervious concrete. Create a rough layout of the pervious concrete dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the pervious concrete.**

Consider whether the pervious concrete is intended to:

» Meet a runoff reduction* target or water quality (treatment) target. For information on the sizing of a BMP utilizing the runoff reduction approach, see Step 3A. For information on the sizing of the BMP utilizing the water quality treatment approach, see Step 4A. **Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*

» Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)

» Provide a possible solution to a drainage problem

» Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3A, 3B, and 3C for a runoff reduction approach, or skip Step 3 and complete Steps 4A and 4B for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) **Calculate the Stormwater Runoff Reduction Target Volume**

Calculate the Runoff Reduction Volume using the following formula:

$$RR_V = (P)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$RR_V$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

BACK TO TOC

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the practice:

Using the $RR_V$ calculated, determine the minimum Volume of the Practice (VP)

$$(VP_{MIN}) \geq RR_v \text{ (target)} / (RR\%)$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice
**$VP_{MIN}$** = Minimum storage volume required to provide Runoff Reduction Target Volume ($ft^3$)
**$RR_v$ (target)** = Runoff Reduction Target Volume ($ft^3$)

**(Step 3B) Determine the storage volume of the practice and the Pretreatment Volume**

To determine the actual volume provided in the pervious concrete, use the following equation:

$$VP = (VBL) * (N)$$

*Where*:

**VP** = Volume provided (temporary storage)
**VBL** = Volume of Base Layer
**N** = Porosity

To determine the porosity, a qualified licensed professional should be consulted to determine the proper porosity. Most gravel has a value of 0.40.

Provide pretreatment by using a grass filter strip or pea gravel diaphragm, 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. Pre-treatment may also be desired to reduce flow velocities or 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. Refer to Section 4.9 for design criteria for a grass channel and Section 4.29 for vegetated filter strips.

**(Step 3C) Determine whether the minimum storage volume was met.**

When the $VP \geq VP_{MIN}$, then the Runoff Reduction requirements are met for this practice. Proceed to Step 5.

When the $VP < VP_{MIN}$, then the BMP must be sized according to the $WQ_V$ treatment method (See Step 4).

**(Step 4A) Calculate the Target Water Quality Volume**

Calculate the Water Quality Volume using the following formula:

$$WQ_V = (1.2)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$WQ_V$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Site area (total drainage area) ($ft^2$)
**12** = Unit conversion factor (in/ft)

**(Step 4B) If using the practice for Water Quality treatment, determine the footprint of the pervious concrete and the Pretreatment Volume required**

The peak rate of discharge for the water quality design storm is needed for sizing of off-line diversion structures (see Subsection 3.1.7). If designing off-line, follow steps (a) through (d) below:

(a) Using $WQ_v$, compute CN
(b) Compute time of concentration using TR-55 method
(c) Determine appropriate unit peak discharge from time of concentration
(d) Compute $Q_{wq}$ from unit peak discharge, drainage area, and $WQ_v$.

To determine the minimum surface area of the pervious concrete, use the following formula:

$$A_f = (WQ_v)(d_{pc} + d_{rl})/[(k_{pc} * d_{pc} * t_{pc})+(k_{rl} * d_{rl} * t_{rl})]$$

*Where*:

**$A_f$** = surface area of pervious ($ft^2$)
**$WQ_v$** = Water Quality Volume ($ft^3$)
**$d_{pc}$** = pervious concrete depth (ft)
**$d_{rl}$** = reservoir layer depth (ft)
**$k_{pc}$** = coefficient of permeability for pervious concrete (ft/day)
**$k_{rl}$** = coefficient of permeability for reservoir layer (ft/day)
**$t_{pc}$** = drain time of pervious concrete (days)
**$t_{rl}$** = drain time of reservior layer (days)

**(Step 5) Size underdrain system (if applicable)**

See Subsection 4.2.5.3 (*Physical Specifications/Geometry*)

### 4.16.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.17 Porous Asphalt

**Description**: Porous asphalt is asphalt with reduced sands or fines and larger void spaces, which allow water to drain through it. Porous asphalt allows water to infiltrate into the subsoil below through the paved surface and base layer. This aggregate base layer acts as a structural layer and container to temporarily hold stormwater.

**LID/GI Consideration:** Porous asphalt can be used to reduce the effective impervious area on a site, therefore reducing the design volumes and peak discharges that must be controlled. Porous asphalt can also eliminate problems with standing water, provide groundwater recharge, control erosion of streambeds and riverbanks, and facilitate pollutant removal.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Intended for low-traffic areas, or for residential or overflow parking applications
- Not ideal for areas with a tree canopy or high-traffic flow
- Soil infiltration rate of 0.5 in/hr or greater is required if no underdrain is present
- Aesthetically pleasing
- Americans with Disabilities Act (ADA) compliant
- Not appropriate as water quality treatment BMP for drainage discharged from other areas

### ADVANTAGES / BENEFITS

- Surface flow reduction of peak flows, volume, and stormwater runoff
- Can be used as a pretreatment for other BMPs for pollutants other than TSS
- High level of pollutant removal other than TSS
- Decreases impermeable area

### DISADVANTAGES / LIMITATIONS

- Potential for high failure rate if not adequately maintained or if used in unstabilized areas
- Not recommended for areas with sediment-laden runoff that can clog porous pavement
- Subgrade cannot be over-compacted
- Construction must be sequenced to avoid compaction and clogging of the pavement.

### ROUTINE MAINTENANCE REQUIREMENTS

- Sweep or vacuum the asphalt to increase pavement life and avoid clogging
- Keep contributing drainage area free of debris and address any areas of erosion

### POLLUTANT REMOVAL

80% **Total Suspended Solids**

60% **Metals** - Cadmium, Copper, Lead, and Zinc removal

50/50% **Nutrients** - Total Phosphorus / Total Nitrogen removal

N/A **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

✓ Runoff Reduction

✓ Water Quality

★ Channel Protection

★ Overbank Flood Protection

★ Extreme Flood Protection

✓ suitable for this practice

★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

L Land Requirement

M Capital Cost

M Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *Yes*

**Soils:** *Not recommended for use with hydrologic soils group 'D' and 'C' without underdrain.*

**Other Considerations:** *Overflow parking, driveways, & related uses*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 100% of the runoff reduction credit if an underdrain is not used
- 75% of the runoff reduction credit if an upturned underdrain is used
- 50% of the runoff reduction credit if an underdrain is used

### 4.17.1 General Description

Porous asphalt is an asphalt driving surface that permits the infiltration of water through the pavement and into underlying soils. Porous asphalt can be used to reduce the effective impervious area on a site, therefore reducing the design volumes and peak discharges that must be controlled. This will allow a reduction in the cost of other stormwater infrastructure, which may offset the greater placement cost. Porous asphalt can also eliminate problems with standing water, provide for groundwater recharge, control erosion of streambeds and riverbanks, facilitate pollutant removal, reduce thermal pollution of receiving waters, and provide for a more aesthetically pleasing site.

Porous asphalt consists of open-graded coarse aggregate, bonded together by asphalt cement, with sufficiently interconnected voids to make it highly permeable to water (see **Figure 4.17-1**). Porous asphalt is best applied in areas that experience low amounts of vehicle traffic and have little to no tree coverage, including:

- Parking pads in parking lots
- Overflow parking areas
- Residential driveways
- Residential street parking lanes
- Recreational trails
- Golf cart and pedestrian paths
- Emergency vehicle and fire access lanes
- Plazas

BACK TO TOC

Porous asphalt is not recommended and may not be approved for use in areas that experience high amounts of traffic volume, heavy loads, or areas with high amounts of sediment runoff (i.e. construction areas).

A major drawback of this BMP is the cost and complexity of porous asphalt compared to conventional pavements. Porous asphalt requires a very high level of construction workmanship to ensure that it functions as designed. In addition, there is the difficulty and cost of rehabilitating porous asphalt surfaces should they become clogged. Therefore, consideration of porous asphalt should include the construction and maintenance requirements and costs.

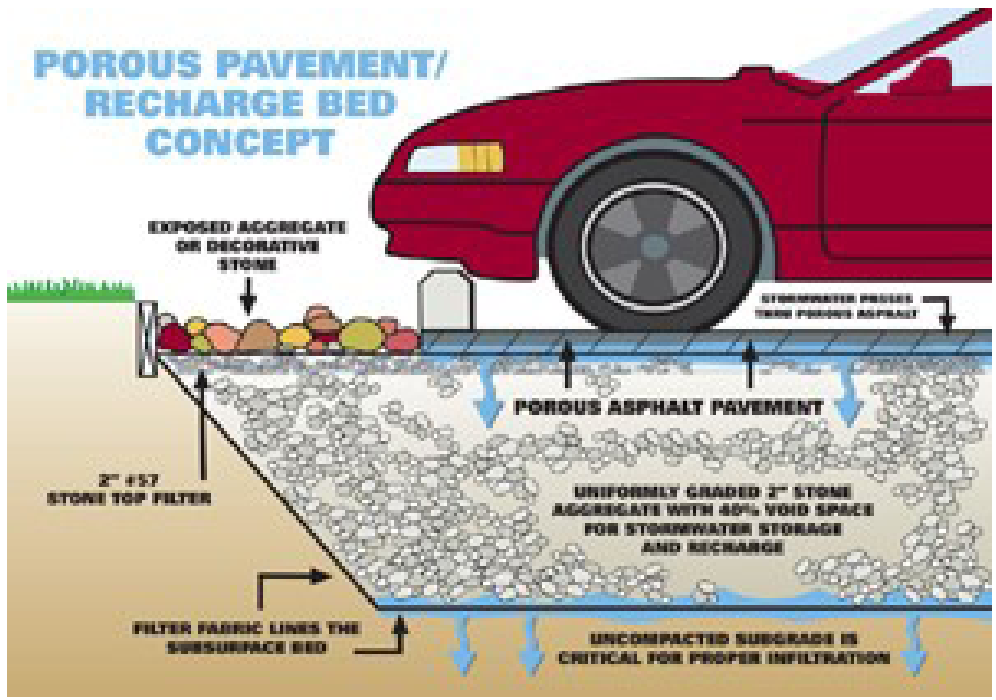



**Figure 4.17-1**

Porous asphalt is designed primarily for impervious area reduction and the subsequent reduction in stormwater treatment volumes and peak discharges, particularly for smaller storm events. These include:

- Placing a perforated pipe near the top of the crushed stone reservoir to pass the excess flows after the reservoir is filled
- Connecting the stone reservoir layer to a stone-filled trench
- Adding a sand layer and perforated pipe beneath the stone layer for filtration of the water quality volume

Porous asphalt is typically placed on a gravel (stone aggregate) base course. Runoff infiltrates through the porous asphalt into the gravel base course, which acts as a storage reservoir as it exfiltrates to the underlying soil. The infiltration rate of the soils in the subgrade must be adequate to support drawdown of the entire runoff capture volume within 24-48 hours. Special care must be taken during construction to avoid undue compaction of the underlying soils, which could affect the soils' infiltration capability.

Another type of porous asphalt is called open-graded friction course (OGFC). OGFC is a thin permeable layer of asphalt that encompasses a support structure consisting of uniform, coarse aggregate with minimal fines, and serves as an overlay for conventional asphalt pavements. OGFC has a high void content that creates permeability, allowing for the infiltration of stormwater runoff.

### 4.17.2 Stormwater Management Suitability

Porous asphalt can provide runoff quantity control, particularly for smaller runoff volumes such as the runoff volume generated by the water quality storm event (1.2 inches). These facilities may sometimes be used to partially or completely meet channel protection requirements on smaller sites. However, porous asphalt will typically need to be used in conjunction with another control to provide channel protection, as well as overbank flood protection.

- Runoff Reduction
  Like other LID practices, porous asphalt becomes more effective with higher infiltration rates of native soils. Porous asphalt can be designed to provide 100% of the runoff reduction volume, if properly maintained.

- Water Quality
  Porous asphalt provides some amount of pollutant removal, but is not well suited for removing sediment due to the sediment potentially clogging the pores within the porous asphalt.

- Channel Protection
  For smaller sites, the porous asphalt may be designed to capture the entire channel protection volume ($CP_v$). Given that the storage volume under the porous asphalt is typically designed to completely drain over 48-72 hours, the 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 porous asphalt, another practice must be used to provide $CP_v$ extended detention.is diverted to the porous asphalt, another practice must be used to provide $CP_v$ extended detention.

- Overbank Flood Protection
  Although relatively rare, on some development sites, porous asphalt can be designed to attenuate the overbank peak discharge ($Q_{p25}$).

- Extreme Flood Protection
  Although relatively rare, on some development sites, porous asphalt can be designed to attenuate the extreme peak discharge ($Q_f$).

*Credit for the volume of runoff reduced in porous asphalt may be taken in the overbank flood protection and extreme flood protection calculations. If the practice is designed to provide runoff reduction for water quality compliance, then the practice is given credit for channel protection and flood control requirements by allowing the designer to compute an Adjusted CN (see* *Subsection 3.1.7.5* *for more information).*

### 4.17.3 Pollutant Removal Capabilities

As it provides for the infiltration of stormwater As porous asphalt provides infiltration of stormwater runoff, it has a high removal rate for both soluble and fine particulate pollutants, which can be trapped, absorbed, or broken down in the underlying soil layers. Due to the potential for clogging, porous asphalt should not be used for the removal of sediment or other coarse particulate pollutants. Maintenance efforts and frequency is directly related to the amount of accumulated or trapped sediment. OGFC has a TSS removal of 50%. There is not sufficient data for nutrients, fecal coliform, or metal removal rates to be determined.

For additional information and data on pollutant removal capabilities for porous asphalt and OGFC, see the National Pollutant Removal Performance Database (3rd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.17.4 Application and Site Feasibility Criteria

Porous asphalt is suitable for many types of development, from single-family residential to high-density commercial projects. Though not often used for heavily traveled roads, it is well suited for alleys, driveways, and parking lots.

BACK TO TOC

*The following criteria should be evaluated to ensure the suitability of porous asphalt for meeting stormwater management objectives on a site or development.*

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **YES**
- Regional Stormwater Control – **NO**

**Physical Feasibility -** Physical Constraints at Project Site

- **Site Slope** – Slopes should be a maximum of 20%, 5% preferred
- **Minimum Depth to Water Table** – A separation distance of 2 feet is recommended between the bottom of the rock storage area of the porous asphalt and the elevation of the seasonally high water table.
- **Soils** – Native soils if they have at least 0.5 inch/hr infiltration ability.

**Other Constraints / Considerations**

- **Hot spots** – Do not use for hot spot runoff.
- **Damage to existing structures and facilities** – Consideration should be given to the impact of water exfiltrating the porous asphalt.
- **Proximity** – The following is a list of specific setback requirements for the location of porous asphalt:
    - » 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** –Evaluate for stream warming when an underdrain system is used.

In addition, careful consideration should be given to the potential of perched or raised groundwater levels. Provide adequate distance from building foundations or use impermeable liner on side of excavated area nearest to structure.

**Challenges and Potential Solutions for Coastal Areas**

- **Poorly Drained Soils**— This condition minimizes the ability of porous asphalt to reduce stormwater runoff rates and volumes. One solution would be to include an underdrain system.
- **Shallow Water Table**— This can prevent the provision of 2 feet of clearance between the bottom of the porous asphalt and the top of the water table which may cause stormwater runoff to pond in the storage layer of the porous asphalt.

## 4.17.5 Planning and Design Criteria

- The detailed design of porous asphalt is beyond the scope of this Manual. Refer to The National Asphalt Pavement Associations' *Porous Asphalt Pavements for Stormwater Management: Design, Construction and Maintenance Guide* for detailed design guidance.
- Porous asphalt systems can be used where the underlying in-situ subsoils have an infiltration rate of at least 0.5 inches per hour. During construction and preparation of the subgrade, special care must be taken to avoid compaction of the soils.
- Porous asphalt should typically be used in applications where the pavement receives tributary runoff only from impervious areas. It is recommended that the ratio of the contributing impervious area to the porous asphalt surface area be no greater than 3:1. Porous asphalt systems should be sized for a minimum drawdown time of 24 hours and a maximum drawdown time of 3 days.
- If runoff is coming from adjacent pervious areas, it is important that those areas be fully stabilized to reduce sediment loads and prevent clogging of the porous asphalt surface.
- It is recommended that the subsoil of the porous asphalt have a slope of 0% and the surface have a slope 0.5% or less, if possible.
- A minimum of 2 feet of clearance is required between the bottom of the gravel base course and underlying bedrock.

- To protect groundwater from potential contamination, runoff form designated hotspot land uses or activities must not be infiltrated. Porous asphalt should not be used for manufacturing or industrial sites, where there is potential for high concentrations of soluble pollutants and heavy metals. In addition, porous asphalt is not suitable in areas with karst geology without adequate geotechnical testing and approval by qualified individuals and in accordance with local requirements.

- Porous asphalt can be used independently or in conjunction with other stormwater management system components to effectively infiltrate, bypass, or detain and time-release all required storm events.

- The surface of the subgrade should be lined with or an 8-inch layer of sand (ASTM C-33 concrete sand or GADOT Fine Aggregate Size No. 10) and completely flat to promote infiltration across the entire surface.

- Porous asphalt must use some method to convey larger storm event flows to the conveyance system. One option is to use storm drain inlets set slightly above the elevation of the pavement. This would allow for some ponding above the surface, but would accept bypass flows that are too large to be infiltrated by the porous asphalt. This would also address situations in which the surface clogs.

- In addition, credit can be taken for the runoff volume infiltrated from other impervious areas using the methodology in Section 3.1.

- For the purpose of sizing downstream conveyance and structural control system, porous asphalt surface areas can be estimated as 35% impervious.

- The cross-section typically consists of two layers. The aggregate reservoir can sometimes be avoided or minimized if the sub-grade is sandy and there is adequate time to infiltrate the necessary runoff volume into the sandy soil without by-passing the water quality volume. A description for each of the layers is presented below:
  - » **Porous Asphalt Layer** – The porous asphalt layer consists of a porous mixture of asphalt. This layer is usually 4-8 inches deep depending on the required bearing strength, pavement design requirements, and manufacturer's specifications.
  - » **Reservoir Layer** –The reservoir gravel base course consists of washed, bank-run gravel, 1.5-2.5 inches in diameter with a void space of about 40% (GADOT No.3 Stone). The depth of this layer depends on the desired storage volume, which is typically the water quality volume ($WQ_v$) at a minimum. Typical depths for the reservoir layer range from 2-4 feet. Aggregate contaminated with soil should not be used. A porosity value (void space/total volume) of 0.40 should be used in calculations unless aggregate-specific data exist.
  - » **Filter Fabric** – Filter fabric can be used in certain applications, as site conditions warrant. General guidance for the use of filter fabrics is below. Actual use should be under the guidance of a Georgia licensed engineer.Geotextiles consisting of permeable materials should line the sides of the aggregate base to prevent migration of adjacent soils into it, which may lead to permeability and storage capacity reduction.
  - » Geotextiles consisting of permeable materials should line the sides of the aggregate base to prevent migration of adjacent soils into it, which may lead to permeability and storage capacity reduction.

    Geotextiles are not recommended under the aggregate base in an infiltration design because they can accumulate fine particulates that inhibit infiltration.



BACK TO TOC

» Geogrids may be used on top of subgrade soils for additional structural support, especially in very weak, saturated soils. All manufacturer requirements must be followed in design and installation.

» Underlying Soil – Porous asphalt should not generally be used in fill soils. The underlying soil should have an infiltration capacity of at least 0.50 in/hr, as initially determined from NRCS soil textural classification, and subsequently confirmed by field geotechnical tests. The minimum geotechnical testing is one test hole per 5000 square feet, with a minimum of two borings per facility (taken within the proposed limits of the facility). Test borings are recommended to determine the soil classification, seasonal high groundwater table elevation, and impervious substrata, to produce an initial estimate of permeability. Often a double-ring infiltrometer test is done at subgrade elevation to determine the impermeable layer.

» Pit excavation should be limited to the width and depth specified in the design. Excavated material should be placed away from the open trench as to not jeopardize the stability of the trench sidewalls. The bottom of the excavated trench should not be loaded in a way that causes compaction, but should be scarified prior to placement of reservoir base material. The sides of the trench should be trimmed of all large roots. The sidewalls should be uniform with no voids and scarified prior to backfilling. All porous asphalt systems should be protected during site construction and constructed after upstream areas have been stabilized.

» An observation well consisting of perforated PVC pipe 4-6 inches in diameter should be placed at the downstream end of the practice and protected during construction. The well should be used to determine actual infiltration rates for use in final design of the porous asphalt system.

» A warning sign should be placed at the facility that states, "Porous pavement used on this site to reduce pollution. Do not resurface with non-porous material. Call (XXX) XXX-XXXX for more information."

» If OGFC is used, consult the *GDOT Manual on Drainage Design for Highways* for design specifications.

### 4.17.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of the porous asphalt.**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for porous asphalt. Create a rough layout of the porous asphalt dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the porous asphalt.**

Consider whether the porous asphalt is intended to:

» Meet a runoff reduction* target or water quality (treatment) target. For information on the sizing of a BMP utilizing the runoff reduction approach, see Step 3A. For information on the sizing of the BMP utilizing the water quality treatment approach, see Step 4.
**Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*

» Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)

» Provide a possible solution to a drainage problem

» Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3A, 3B, and 3C for a runoff reduction approach, or skip Step 3 and complete Steps 4A and 4B for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) Calculate the Stormwater Runoff Reduction Target Volume

Calculate the Runoff Reduction Volume using the following formula:

$$RR_V = (P)\,(R_V)\,(A) / 12$$

*Where*:

**$RR_V$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the practice:

Using the $RR_V$ calculated, determine the minimum Volume of the Practice (VP)

$$(VP_{MIN}) \geq RR_v\,(\text{target}) / (RR\%)$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice
**$VP_{MIN}$** = Minimum storage volume required to provide Runoff Reduction Target Volume ($ft^3$)
**$RR_v$ (target)** = Runoff Reduction Target Volume ($ft^3$)

(Step 3B) Determine the storage volume of the practice and the Pretreatment Volume

To determine the actual volume provided in the porous asphalt, use the following equation:

$$VP = (VBL) * (N)$$

*Where*:

**VP** = Volume provided (temporary storage)
**VBL** = Volume of Base Layer
**N** = Porosity

To determine the porosity, a qualified licensed professional should be consulted to determine the proper porosity based on the engineered soils used. Most gravel has a value of 0.40.

Provide pretreatment by using a grass filter strip or pea gravel diaphragm, 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 may also be desired to reduce flow velocities or 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. Refer to Section 4.9 for design criteria for a grass channel and Section 4.29 for vegetated filter strips.

(Step 3C) Determine whether the minimum storage volume was met.

When the $VP \geq VP_{MIN}$, then the Runoff Reduction requirements are met for this practice. Proceed to Step 5.

When the $VP < VP_{MIN}$, then the BMP must be sized according to the $WQ_V$ treatment method (See Step 4).

(Step 4A) Calculate the Target Water Quality Volume

Calculate the Water Quality Volume using the following formula:

$$WQ_V = (1.2)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$WQ_V$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

Where:

**I** = new impervious area of the contributing drainage area (%)

**A** = Site area (total drainage area) ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4B) **If using the practice for Water Quality treatment, determine the footprint of the infiltration practice and the Pretreatment Volume required.**

The peak rate of discharge for the water quality design storm is needed for sizing of off-line diversion structures (see Subsection 3.1.7). If designing off-line, follow steps (a) through (d below):

(a) Using $WQ_v$, compute CN
(b) Compute time of concentration using TR-55 method
(c) Determine appropriate unit peak discharge from time of concentration
(d) Compute $Q_{wq}$ from unit peak discharge, drainage area, and $WQ_v$.

To determine the minimum surface area of the pervious concrete, use the following formula:

$$A_f = (WQ_v)(d_{pc} + d_{rl})/[(k_{pc} * d_{pc} * t_{pc})+(k_{rl} * d_{rl} * t_{rl})]$$

*Where*:

**$A_f$** = surface area of pervious ($ft^2$)
**$WQ_v$** = Water Quality Volume ($ft^3$)
**$d_{pc}$** = porous asphalt depth (ft)
**$d_{rl}$** = reservoir layer depth (ft)
**$k_{pc}$** = coefficient of permeability for porous asphalt (ft/day)
**$k_{rl}$** = coefficient of permeability for reservoir layer (ft/day)
**$t_{pc}$** = drain time of porous asphalt (days)
**$t_{rl}$** = drain time of reservior layer (days)

(Step 5) **Size underdrain system (if applicable)**
See Subsection 4.2.5.3 (*Physical Specifications/Geometry*)

### 4.17.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for inspection and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

## 4.18 Proprietary Systems

**Description**: Proprietary systems are manufactured stormwater BMPs and treatment systems available from commercial vendors; these systems are designed to treat stormwater runoff and/or provide water quantity control.

**LID / GI Consideration**: Proprietary systems may be considered to be low impact development or green infrastructure practices.

*Note: It is the policy of this Manual not to recommend any specific commercial vendors for proprietary systems. However, this subsection is being included in order to provide communities with a rationale for approving the use of a proprietary system or practice in their jurisdictions.*

### KEY CONSIDERATIONS

**DESIGN CRITERIA**

- Design criteria, such as drainage area, slope, soils, flow velocity, storage area, permanent pool depth, and inlet/outlet considerations are based on manufacturer recommendations.

**ADVANTAGES / BENEFITS**

- Proprietary systems can be chosen or designed for a site's specific stormwater runoff characteristics and/or design constraints.
- Often, proprietary systems are well-suited for use on urban development sites where larger or above-ground BMPs are not an option, or for stormwater retrofit projects.
- Can be used as pretreatment for other BMPs
- Can replace a conventional junction or inlet structure
- Some designs require minimal drop between inlet and outlet.

**DISADVANTAGES / LIMITATIONS**

- Dissolved pollutants may not be effectively removed by proprietary systems.
- Proprietary systems may not achieve the 80% TSS removal target alone.
- Performance is dependent on design and maintenance of individual units.

**ROUTINE MAINTENANCE REQUIREMENTS**

- Maintenance requirements for a proprietary system should be obtained from the manufacturer.
- Frequency of inspection and maintenance is dependent on land use, climatological conditions, and the system's design.
- Failure to provide adequate inspection and maintenance can result in the re-suspension of accumulated solids.
- Ensure maintenance access to proprietary systems when designing each site.

**STORMWATER MANAGEMENT SUITABILITY**

★ Runoff Reduction
★ Water Quality
★ Channel Protection
★ Overbank Flood Protection
★ Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

**IMPLEMENTATION CONSIDERATIONS**

varies Land Requirement
varies Capital Cost
varies Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *Yes*

**Soils:** *Proprietary systems can be installed in almost any structure, soil, or terrain.*

**Other Considerations:** *Install as an off-line device unless the system can be sized to handle a small drainage area.*

**RUNOFF REDUCTION CREDIT**

- Runoff reduction is not typically provided by proprietary systems.

BACK TO TOC

### 4.18.1 General Description

There are many types of commercially-available proprietary stormwater BMPs for both water quality treatment and quantity control. These systems include:

- Hydrodynamic systems such as gravity and vortex separators
- Filtration systems
- Catch basin media inserts
- Chemical treatment systems
- Package treatment plants
- Prefabricated detention structures

Many proprietary systems are useful on small sites and space-limited areas where there is not enough land or room for other stormwater treatment alternatives. Proprietary systems can be used as pretreatment in a treatment train. However, proprietary systems are often more costly than other alternatives and may have high maintenance requirements. Perhaps the largest difficulty in using a proprietary system is the lack of adequate independent performance data, particularly for use in Georgia conditions. Below are general guidelines that should be followed before considering the use of a proprietary commercial system.

### 4.18.2 Guidelines for Using Proprietary Systems

A proprietary system should have a demonstrated ability to meet the stormwater management goals and uses for which it is intended. This means that the system should provide:

1. Independent third-party scientific verification of the ability of the proprietary system to meet water quality treatment objectives and/or to provide water quantity control (channel or flood protection)
2. A proven record of longevity in the field
3. Proven ability to function in Georgia conditions (e.g., climate, rainfall patterns, soil types)

For a propriety system to meet (1) above for water quality goals, the following monitoring criteria should be met for supporting studies:

- Samples should be taken for at least 15 storm events.
- The study should be independent or independently verified (i.e., may not be conducted by the vendor or designer without third-party verification).
- The study should be conducted in the field, as opposed to laboratory testing.
- Field monitoring should be conducted using standard protocols that require proportional sampling both upstream and downstream of the device.
- Concentrations reported in the study should be flow-weighted.
- The propriety system or device should have been in place for at least one year at the time of monitoring.

Although local data is preferred, data from other regions can be accepted as long as the design accounts for local conditions.

Local governments may submit a proprietary system to further scrutiny based on the performance of similar practices. A poor performance record or high failure rate is valid justification for not allowing the use of a proprietary system or device. Consult your local review authority for more information regarding the use of proprietary systems.

As an example for a proprietary system evaluation guideline, the Metropolitan North Georgia Water Planning District has developed the *Post-Construction Stormwater Technology Assessment Protocol (PCSTAP).*

#### Introduction

These guidelines, however, were not intended to be testing protocols or procedures for evaluating the performance of a proprietary technology or product.  Furthermore, the lack of consistent review and evaluation of monitoring and performance data has been a source of frustration for local governments, vendors and the development community.

As local governments are being asked to review and approve emerging stormwater treatment technologies, a consistent testing protocol and a process for evaluating and accepting proprietary stormwater treatment systems is necessary.

The objective of this protocol is to provide local governments and other entities with an assessment tool to use, if they so choose, as a starting point for evaluating a particular technology's effectiveness in removing pollutants from stormwater runoff for an intended application and to compare test results with vendor performance claims.

**Purpose**

This guidance document's primary purpose is to establish a testing protocol and process for evaluating and reporting on the performance and appropriate uses of proprietary stormwater treatment technologies and systems for addressing post-construction stormwater runoff. It is not intended for use in evaluation of erosion and sedimentation control technologies or products for use during construction or land-disturbing activities. See the Georgia Soil & Water Conservation Commission's *Manual for Sedimentation and Erosion Control in Georgia* for information on assessing E&S technologies.

Stormwater treatment technologies and products that have been tested according to this protocol can receive consideration to have their results evaluated and made available publicly on the Metro Water District website (www.northgeorgiawater.org). The review of vendor data and subsequent determinations and public dissemination is not intended to be an approval process or an endorsement of any product by the Metropolitan North Georgia Water Planning District. Purchasers and users of any technologies or products presented by a manufacturer or other entity using this protocol should make their own independent analyses and evaluations concerning the usefulness or value of any stormwater technologies or combinations of technologies in considering whether to use any particular technology or product for post-construction stormwater treatment. Local governments and other entities are free to use this information as part of their process to evaluate the suitability of these technologies or products.

Note: The current protocol is only set-up for collecting information on TSS removal. The protocol may be updated in the future to also provide information and evaluation on run-off reduction volumes.

For the latest protocol and other information, see: www.northgeorgiawater.org.

### 4.18.3 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for inspection and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.19 Rainwater Harvesting

(Source: Jones and Hunt, 2008)

**Description**: Rainwater harvesting is the ancient stormwater management practice of intercepting, diverting and storing stormwater runoff for later use. In a typical rainwater harvesting system, rainfall is collected from a gutter and downspout system, screened and "washed" and conveyed into an above- or below-ground storage tank or cistern. Once captured, stored water may be used for non-potable indoor or outdoor uses. In some cases, treatment may be required for indoor uses. If properly designed, rainwater harvesting systems can significantly reduce post-construction stormwater runoff rates, volumes and pollutant loads on development sites. Rainwater harvesting also helps reduce the demand on public water supplies, which, in turn, helps protect aquatic resources, such as groundwater aquifers, from drawdown and seawater intrusion.

## KEY CONSIDERATIONS

### DESIGN CRITERIA
- Rainwater harvesting systems should be sized based on the size of the contributing drainage area, local rainfall patterns and projected demand for harvested rainwater
- Pretreatment should be provided upstream of all rainwater harvesting systems to prevent leaves and other debris from clogging the system
- *Georgia Rainwater Harvesting Guidelines (2009)*

### ADVANTAGES / BENEFITS
- May reduce water bill
- Helps restore pre-development hydrology on development sites and reduces post-construction stormwater runoff rates, volumes and pollutant loads
- Can be used on almost any development site
- Reduces demand on public water supplies, which helps protect groundwater aquifers from drawdown and seawater intrusion
- Allows beneficial reuse of stormwater

### DISADVANTAGES / LIMITATIONS
- Stored rainwater should be used on a regular basis to maintain system storage capacity
- A pump may be required if rainwater harvesting system is below ground
- Not aesthetically pleasing in some cases
- If the system is inside, plumbing codes may be required. See Georgia Amendments to the International Plumbing Code (latest version)
- Should not be used with tar, gravel, and/or asbestos shingled roofs

### ROUTINE MAINTENANCE REQUIREMENTS
- Drain and clean out aboveground cistern
- Inspect health of vegetation receiving harvested rainwater to determine watering needs
- Remove leaves and debris from grated and screened inlet
- Inspect for erosion around the overflow discharge and repair as necessary
- Check for algae growth inside the cistern; if found, treat water to remove the algae
- Check pumping system to ensure it is working properly

### POLLUTANT REMOVAL
- varies **Total Suspended Solids**
- varies **Metals** - Cadmium, Copper, Lead, and Zinc removal
- varies **Nutrients** - Total Phosphorus / Total Nitrogen removal
- varies **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY
- ★ Runoff Reduction
- ★ Water Quality
- ★ Channel Protection
- Overbank Flood Protection
- Extreme Flood Protection

✓ suitable for this practice
★ based on demand

### IMPLEMENTATION CONSIDERATIONS
- L Land Requirement
- M Capital Cost
- H Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*

**Other Considerations:** *Trees near catchment area may cause clogging in the system.*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT
- Varies based on demand



BACK TO TOC

## 4.19.1 General Description

Rainwater harvesting is the ancient stormwater management practice of intercepting, diverting and storing rainfall for later use. In a typical rainwater harvesting system (**Figure 4.19-1**), rainfall is collected from a gutter and downspout system, screened, "washed," and conveyed into an above- or below-ground storage tank or cistern. Once captured stored water may be used for non-potable indoor or outdoor uses. If properly designed, rainwater harvesting systems can significantly reduce post-construction stormwater runoff rates, volumes and pollutant loads on development sites.

There are two basic types of rainwater harvesting systems:

1. Systems that are used to supply water for non-potable outdoor uses, such as landscape irrigation, car and building washing and firefighting; and
2. Systems that are used to supply water for non-potable indoor uses, such as laundry and toilet flushing.

Rainwater harvesting systems used to supply water for non-potable indoor uses are more complex and require separate plumbing, pressure tanks, pumps and backflow preventers. Additionally, the use of harvested rainwater for non-potable indoor uses may be restricted in some areas of Georgia, due to existing "development rules." Developers and their site planning and design teams are encouraged to consult with the local development review authority if they are interested in using harvested rainwater for non-potable indoor uses. If indoor non-potable use is planned, filtration followed by disinfection is required, and there are additional system components not pictured in **Figure 4.19-1**.

Whether it is used to supply water for non-potable indoor or outdoor uses, a well-designed rainwater harvesting system typically consists of five major components (**Figure 4.19-2**), including the collection and conveyance system (e.g., gutter and downspout system), pretreatment devices (e.g., leaf screens, first flush diverters, roof washers), the storage tank or cistern, the overflow pipe (which allows excess stormwater runoff to bypass the storage tank or cistern) and the distribution system (which may or may not require a pump, depending on site characteristics). When designing a rainwater harvesting system, site planning and design teams should consider each of these components, as well as the size of the contributing drainage area, local rainfall patterns and projected water demand, to determine how large the cistern or storage tank must be to provide enough water for the desired non-potable indoor or outdoor use.

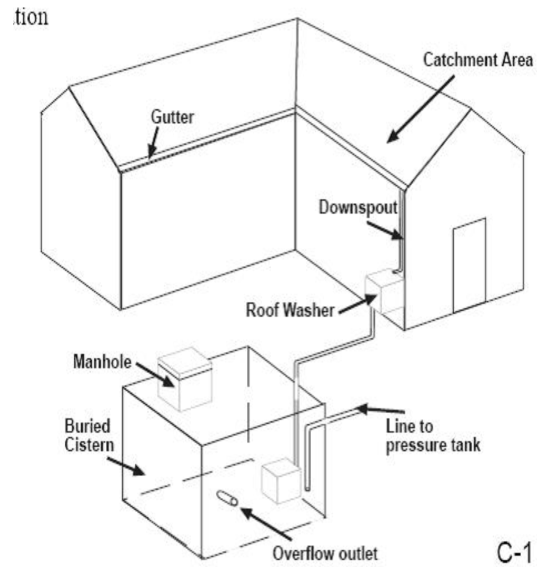



**Figure 4.19-1:** Rainwater Harvesting System
(Source: Rupp, 1998)


**Figure 4.19-2:** Major Components of a Rainwater Harvesting System
(Source: Jones and Hunt, 2008)

BACK TO TOC

### 4.19.2 Stormwater Management Suitability

The Center for Watershed Protection (Hirschman et al., 2008) recently documented the ability of rainwater harvesting systems to reduce annual stormwater runoff volumes and pollutant loads on development sites. Consequently, this low impact development practice has been assigned quantifiable stormwater management "credits" that can be used to help satisfy the stormwater management criteria presented in this manual:

- Runoff Reduction
  Subtract 75% of the storage volume provided by a rainwater harvesting system from the runoff reduction volume ($RR_v$) captured by the system.

- Water Quality
  Through the achieved runoff reduction volume provided by this practice, 100% of the pollutants associated with the achieved $RR_V$ are removed. Pollutant removal rates are otherwise not provided for this practice.

- Channel Protection
  Proportionally adjust the post-development runoff curve number (CN) to account for the runoff reduction provided by a rainwater harvesting system when calculating the channel protection volume ($CP_v$).

- Overbank Flood Protection
  Proportionally adjust the post-development runoff CN to account for the runoff reduction provided by a rainwater harvesting system when calculating the overbank peak discharge ($Q_{p25}$) on a development site.

- Extreme Flood Protection
  Proportionally adjust the post-development runoff CN to account for the runoff reduction provided by a rainwater harvesting system when calculating the extreme peak discharge ($Q_f$) on a development site.

Only 75% of the storage volume provided by a rainwater harvesting system can be subtracted from the runoff reduction volume (RRv) that is captured by the system due to the fact that some of the harvested rainwater may not be used between consecutive storm events.

In order to "receive" stormwater runoff and be eligible for these "credits," it is recommended that rainwater harvesting systems satisfy the planning and design criteria outlined below.

*Credit for the volume of runoff reduced in the rainwater harvesting system may be taken in the overbank flood protection and extreme flood protection calculations. If the practice is designed to provide Runoff Reduction for Water Quality compliance, then the practice is given credit for Channel Protection and Flood Control requirements by allowing the designer to compute an Adjusted CN (see Subsection 3.1.7.5 for more information).*

### 4.19.3 Pollutant Removal Capabilities

The amount of pollutant removal varies and is based on the type of rainwater harvesting system used.

For additional information on data and pollutant removal capabilities for rainwater harvesting systems, see the National Pollutant Removal Performance Database (Version 3) available at www.cwp.org, the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org, and the Georgia Rainwater Harvesting Guidelines (2009).

### 4.19.4 Application and Site Feasibility Criteria

The criteria listed in **Table 4.19-1** should be evaluated to determine whether or not a rainwater harvesting system is appropriate for use on a development site. It is important to note that rainwater harvesting systems have few constraints that impede their use on development sites.

Table 4.19-1: Factors to Consider When Evaluating the Overall Feasibility of Using a Rainwater Harvesting System on a Development Site

| Site Characteristic | Criteria |
|---|---|
| Drainage Area | No restrictions |
| Area Required | Varies according to the size of the contributing drainage area and the dimensions of the rain tank or cistern used to store the harvested rainwater. |
| Slope | No restrictions, although placing rainwater harvesting systems at higher elevations may reduce or eliminate pumping requirements. |
| Minimum Head | N/A |
| Minimum Depth to Water Table | N/A |
| Water Table | N/A |
| Soils | N/A |

**Site Applicability**

Rainwater harvesting systems can be used on a wide variety of development sites in rural, suburban and urban areas. They are especially well suited for use on commercial, institutional, municipal and multi-family residential buildings on urban and suburban development and redevelopment sites. When compared with other low impact development practices, rainwater harvesting systems have a moderate construction cost, a relatively high maintenance burden and require a relatively small amount of surface area. Although they can be expensive to install, rainwater harvesting systems are often a component of “green buildings,” such as those that achieve certification in the Leadership in Energy and Environmental Design (LEED) Green Building Rating System. Refer to the *Georgia Rainwater Harvesting Guidelines (2009)* document for a detailed discussion of beneficial reuse and associated rule and requirements of harvested rainwater.

## 4.19.5 Planning and Design Criteria

*The following criteria should be considered minimum standards for the design of a rainwater harvesting system. Consult with the local authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.19.5.1 LOCATION AND LAYOUT

Rainwater harvesting systems may be installed on nearly any development site. However, placing storage tanks or cisterns at higher elevations may reduce or eliminate pumping requirements

### 4.19.5.2 GENERAL DESIGN

- Distribution systems may be gravity fed or may include a pump to provide the energy necessary to convey harvested rainwater from the storage tank to its final destination. Rainwater harvesting systems used to provide water for non-potable outdoor uses typically use gravity to feed watering hoses through a tap and spigot arrangement.
- Staff working on larger sites may use vehicles to transport stormwater storage containers to specific sites for watering outside of the immediate vicinity of the collection area.
- Rain barrels (i.e., small storage tanks capable of storing less than 100 gallons of stormwater runoff) rarely provide enough storage capacity to accommodate the stormwater runoff volume generated by the target runoff reduction rainfall event. Consequently, they should not be used as part of a rainwater harvesting system, except on small drainage areas of 2,500 square feet or less in size.

### 4.19.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

Rainwater harvesting systems should be designed to provide at least enough storage for the stormwater runoff volume generated by the target runoff reduction rainfall event (e.g., 85th percentile rainfall event). The required size of a rainwater harvesting system is governed by several factors, including the size of the contributing drainage area, local rainfall patterns and projected demand for harvested rainwater. Site planning and design teams should calculate the projected water demand and then conduct water balance calculations, based on the size of the contributing drainage area and local precipitation data, to size a rainwater harvesting system.

BACK TO TOC

A rainwater harvesting model, such as the one provided by North Carolina State University (NCSU, 2008) at: http://www.bae.ncsu.edu/topic/waterharvesting, can be used to design a rainwater harvesting system, provided that the precipitation data being used in the model reflects local rainfall patterns and distributions and has been approved by the local development review authority prior to use.

### 4.19.5.4 PRETREATMENT/INLETS

Pretreatment is needed to remove debris, dust, leaves and other materials that accumulate on rooftops, and may cause clogging within a rainwater harvesting system. Pretreatment devices that may be used include leaf screens, roof washers and first-flush diverters, each of which are described briefly below:

- Leaf Screens: Leaf screens are mesh screens installed either in the gutter or downspout that are used to remove leaves and other large debris from rooftop runoff. Leaf screens must be regularly cleaned to work effectively. If not regularly maintained, they can become clogged and prevent rainwater from flowing into the storage tank.

- First Flush Diverters: First flush diverters direct the initial pulse of stormwater runoff away from the storage tank and into an adjacent pervious area. While leaf screens effectively remove larger debris such as leaves and twigs from harvested rainwater, first flush diverters can be used to remove smaller contaminants such as dust, pollen and animal feces.

- Roof Washers: Roof washers are placed just ahead of storage tanks and are used to filter small debris from the harvested rainwater. Roof washers consist of a small tank, usually between 25-50 gallons in size, with leaf strainers and filters with openings as small as 30 microns (TWDB, 2005). The filter functions to remove very small particulate matter from harvested rainwater. All roof washers must be cleaned on a regular basis. Without regular maintenance, they may not only become clogged and prevent rainwater from entering the storage tank, but may become breeding grounds for bacteria and other pathogens.

Rooftop drainage systems (e.g., gutter and downspout systems) should be designed as they would be for a building designed without a rainwater harvesting system. Drainage system components leading to the cistern should have a minimum slope of 2% to ensure that harvested rainwater is actually conveyed into the storage tank.

### 4.19.5.5 OUTLET STRUCTURES

- An overflow pipe should be provided to allow stormwater runoff to bypass the storage tank or cistern when it reaches its storage capacity. The overflow pipe should have a conveyance capacity that is equal to or greater than that of the inflow pipe and should direct excess stormwater runoff to another low impact development practice, such as a vegetated filter strip (Section 4.29), grass channel (Section 4.9) or bioretention areas (Section 4.2).

- All overflow pipes should be directed away from buildings to prevent damage to building foundations

### 4.19.5.6 SAFETY FEATURES

- Storage tanks (also known as cisterns) are the most important and often the most expensive component of a rainwater harvesting system. Storage tanks can be constructed from a variety of materials, including wood, plastic, fiberglass, concrete, and galvanized metal. Site planning and design teams should choose an appropriate cistern for the intended application and should ensure that it has been sealed with a water safe, non-toxic substance.

- All storage tanks should be opaque or otherwise protected from direct sunlight to inhibit algae growth. They should also be screened to discourage mosquito breeding and reproduction, but should be accessible for use, cleaning, inspection and maintenance.

- The quality of harvested rainwater will vary according to the rooftop material. For example, water harvested from certain types of rooftops, such as those constructed of asphalt, tar, gravel and treated wood shingles, should be avoided or only be used for non-potable outdoor uses, as these materials may leach toxic compounds into stormwater runoff.

BACK TO TOC

#### 4.19.5.7 LANDSCAPING

Landscaping requirements for the rainwater harvesting system is generally minimal. Rainwater harvesting systems typically drain to planted vegetation as a method of watering. Vegetation should be non-invasive and native plants. During dry periods, additional watering may be necessary to keep plants healthy. Check with local jurisdictions for any restrictions on using harvested rainwater.

#### 4.19.5.8 CONSTRUCTION CONSIDERATIONS

To help ensure that rainwater harvesting systems are successfully installed on a development site, site planning and design teams should consider the following recommendations:

- Rainwater harvesting systems may be installed on development and redevelopment sites after building rooftops and their drainage systems (e.g., gutter and downspout systems) have been constructed.

### 4.19.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of rainwater harvesting**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for rainwater harvesting. Create a rough layout of the rainwater harvesting dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of rainwater harvesting.**

Consider whether the rainwater harvesting is intended to:

- Meet a runoff reduction* target. For information on the sizing of a BMP utilizing the runoff reduction approach, see Step 3A.
  **Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*
- Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)
- Provide a possible solution to a drainage problem
- Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

If the rainwater harvesting system will be used to supply water for toilet and/or urinal flushing, provide filtering and treatment in accordance with the Georgia Rainwater Harvesting Guidelines (2009) and the plumbing features in accordance with the Georgia Amendments to the International Plumbing Code (latest version).

*Complete Step 3A, 3B, and 3C for a runoff reduction approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) **Calculate the Stormwater Runoff Reduction Target Volume**

Calculate the Runoff Reduction Volume using the following formula:

$$RR_v = (P)\,(R_v)\,(A)\,/\,12$$

*Where*:

**$RR_v$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_v$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

BACK TO TOC

The runoff reduction percentage (or credit) provided by the practice should be determined by the local jurisdiction. As guidance, if the demand for the harvested rainwater volume ($VP_{Demand}$) is ≥ VP and VP ≥ $VP_{MIN}$, then the RR% should 100%.

Using the $RR_V$ calculated above, determine the minimum Volume of the Practice (VP)

$$(VP_{MIN}) \geq RR_v \text{ (target)} / (RR\%)$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice
**$VP_{MIN}$** = Minimum storage volume required to provide Runoff Reduction Target Volume ($ft^3$)
**$RR_v$ (target)** = Runoff Reduction Target Volume ($ft^3$)

(Step 3B) Determine the storage volume of the practice and the Pretreatment Volume
The actual volume provided (VP) for rainwater harvesting is equal to the size of the rainwater harvesting tank.

(Step 3C) Determine whether the minimum storage volume was met
When $VP_{Demand} \geq VP_{MIN}$ and and VP ≥ $VP_{MIN}$, then the Runoff Reduction requirements are met.

(Step 4) Determine the drainage area from the roof and required $RR_V$ to each downspout
Assess roof to determine the percentage of the entire roof and the specific area of the roof that drains to each downspout. Use this area along with the $RR_V$ calculation in Step 3 to estimate the volume for each downspout.

(Step 5) Determine the number and size of the systems
Based on the volume calculated in Step 4, determine site constrains, cistern costs, and appropriate size of the rainwater harvesting system.

(Step 6) Size the overflow structure
Calculate the peak roof runoff rate to each downspout for the 2-, 10-, and 100-year storm events. Size the overflow for each cistern so that it can pass the larger storm events. The overflow should be designed in such a way that the discharge does not cause any erosion or scour.

(Step 7) Select and size treatment components, as needed, when the harvested rainwater will be used to supply water for toilet and/or urinal flushing.

### 4.19.7 Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.



BACK TO TOC

# 4.20 Regenerative Stormwater Conveyance

**Description**: Regenerative stormwater conveyance (RSC) provides treatment, infiltration 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.

**LID/GI Considerations**: By repairing and restoring eroded and damaged drainage channels by mimicking natural drainage patterns, regenerative stormwater conveyance provide water quality volume treatment for total suspended solids dissolved pollutants, while also providing mitigation of thermal impacts of stormwater runoff.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Ideally, channel slopes should be less than 10%; however, RSC designs can be adapted for slopes exceeding 10%.
- Natural drainage patterns should be matched as closely as possible.
- Stabilize the outfall from the RSC to receiving waters.
- RSCs can be designed to receive runoff from up to 50 acres, however typical drainage areas range from 10-30 acres.
- The storage volume of the pools should be large enough and include storage volume above the seasonally high groundwater table so that the practice is not inundated by groundwater.
- Pools should drain to their design (ponding) levels within 72 hours from the end of a storm event.
- The outlet of the storage facility should be sloped to prevent conditions that promote standing water.
- RSC should not be used to treat runoff from hot spots.

### ADVANTAGES / BENEFITS

- When designed correctly, RSCs are safe and aesthetically pleasing and may potentially increase the natural value of the site.
- RSC systems provide high total suspended solids and soluble pollutant removal rates.
- Effective at restoring highly eroded areas such as outfalls, ditches, and channels.

### DISADVANTAGES / LIMITATIONS

- RSC is a new type of BMP and significant data is not available making performance of the BMP uncertain.
- RSCs are unfamiliar to most designers due to the recent use of this practice.
- Design of RSC is an iterative practice.
- Construction of an RSC system can be time and labor intensive.

### ROUTINE MAINTENANCE REQUIREMENTS

- There are intensive maintenance requirements during establishment of the practice
- Check for erosion or "end-cutting" of weirs and riffle structures and stable water levels in pools.
- Remove invasive vegetation.
- Bare or eroding areas in the contributing drainage area or around the RSC channel should be immediately stabilized.
- Remove and replace dead plants.
- Once every 2 to 3 years, remove accumulated sediment in pools.
- Prune and weed the practice four times per year.

### STORMWATER MANAGEMENT SUITABILITY

- Runoff Reduction
- ✓ Water Quality
- Channel Protection
- Overbank Flood Protection
- Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- M Land Requirement
- H Capital Cost
- M Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Not recommended*
**Roadway Projects:** *Yes*

**Soils:** *RSC systems can be installed in hydrologic soil group A, B, C, or D soils.*

**Other Considerations:** *Vegetation is essential to the function of RSC systems. The use of native plants is recommended.*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 0% Runoff Reduction Credit is provided by this practice

### POLLUTANT REMOVAL

- 80% **Total Suspended Solids**
- 70/70% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- N/A **Metals** - Cadmium, Copper, Lead, and Zinc removal
- N/A **Pathogens** – Fecal Coliform

## 4.20.1 General 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. Both woody and herbaceous plants in the ponding area provide vegetative uptake of runoff and pollutants, and also serve to stabilize the surrounding soils. See **Figure 4.20-1** for a design schematic of a regenerative stormwater conveyance system.

For additional information about the design, construction and pollutant removal provided by RSC systems, visit Anne Arundel County: http://www.aacounty.org/DPW/Watershed/StepPoolStorm-Conveyance.cfm.

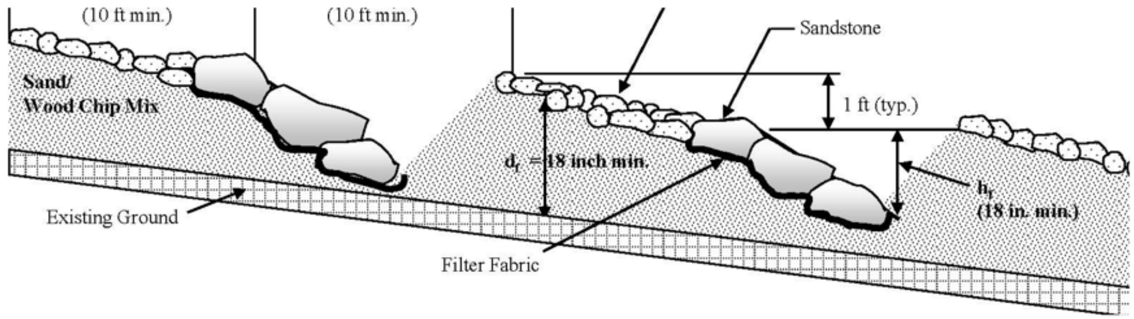



**Figure 4.20-1** Typical RSC Profile
(Source: Anne Arundel County, Maryland, June 2009)

## 4.20.2 Stormwater Management Suitability

- Runoff Reduction
  RRSC systems are not designed to provide runoff reduction. The substrate provides storage and gradual release, especially of 'first flush' events. RSC systems must be installed as part of a treatment train with other BMPs that provide runoff reduction, if it is desired.

- 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 ($WQ_v$) flowing to the RSC system. In addition, RSC is effective at removing 70% of the total phosphorus and nitrogen found in the water. RSC is not effective at removing metal or pathogens from the water.

- 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. (See Subsection 4.1.6.)

- 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.

### 4.20.3 Pollutant Removal Capabilities

Regenerative stormwater conveyance systems are presumed to be able to remove 80% of the total suspended solids (TSS) load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Other pollutants that RSC systems can remove include Phosphorus and Nitrogen. Metals, including Cadmium, Copper, Lead, and Zinc, and Pathogens, such as Fecal Coliform, are not presumed to be removed by a RSC system. See **Table 4.1.2-3** for pollutant removal rates.

### 4.20.4 Application and Site Feasibility Criteria

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 $CP_v$, $Q_{p25}$, and $Q_f$. Designers should consider the importance of native herbaceous and woody vegetation in the function of an RSC system, and ensure that the development area can support the necessary landscaping.

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **NO**
- Regional Stormwater Control – **NO**

**Physical Feasibility –** Physical Constraints at Project Site

- **Drainage Area** – 50 acres or less, but typically 10-30 acres
- **Space Required** – A rough rule of thumb is that RSC systems comprise approximately 0.5–3% of the contributing drainage area.
- **Site Slope** – Drainage channel slopes should be 10% or less.
- **Minimum 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** – RSC systems can be installed in hydrologic soil group A, B, C or D soils.

**Other Constraints/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
- **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.

**Coastal Areas**

- **Poorly Drained Soils and Shallow Water Table** — RSC systems may be installed in any soil type and where there is a shallow water table as long as the shallow pools of an RSC system drain to the designed ponding levels within 72 hours of a rain event.
- **Flat Terrain** – Adequate slope from inlet to outfall of the RSC system must be provided to generate flow.



BACK TO TOC

## 4.20.5 Planning and Design Criteria

Before designing the regenerative stormwater conveyance system, assess site conditions to determine its applicability. Check with the local stormwater authority to ensure that RSC is applicable to the particular site. The following information is needed prior to initiating design:

- Existing and proposed site and location maps and field reviews
- Topographic map with 1-foot minimum contour intervals
- Existing and proposed impervious and pervious areas
- Dimensions and profiles of existing drainage channels for retrofit projects
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site
- Design data from nearby storm sewer structures
- Average and 10 and 100 year water surface elevation of nearby water systems and depth to the seasonally high groundwater
- Infiltration testing of native soils at the proposed bottom elevation of the RSC system

*The following criteria are to be considered **minimum** standards for the design of a regenerative stormwater conveyance system. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.20.5.1 LOCATION AND LAYOUT

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.

### 4.20.5.2 GENERAL DESIGN

- A RSC consists of the following:
  1. A sequence of pools, riffles, and cascades to assist in treating, detaining, and conveying storm flow.
  2. Organic/mulch layer to protect planting media.
  3. Native soils to infiltrate the treated runoff, (see description of infiltration trenches, Section 4.12, for infiltration criteria).
  4. A grade control structure and settling pool should be used if the slope of the channel is greater than 5%.
- A RSC design may include some of the following:
  - » Pretreatment maybe required to keep sediment and large debris out of the practice.
  - » A series of riffle cobbles and boulders, shallow pools, and sand and wood chips.

### 4.20.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- Recommended total length of grade control structures and pools is less than 10 feet.
- 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 8-20 times the depth of the grade control structure. 10 feet preferred.
- The West Virginia design manual recommends using the following equation to determine the length of the grade control structure for RSC:

$$L_{GCS} = L_{pool} = \frac{L_{RSC\ Path}}{\frac{\Delta E}{2}}$$

*Where*:

$L_{GCS}$ = Length of grade control structure
$L_{pool}$=Length of pool
$L_{RSC\ Path}$=Length of the RSC flow path
$\Delta E$=Change in elevation of the RSC practice.

- 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. To stabilize the sand layer, 1 foot of bank-run gravel should be placed below the sand layer. A 1 foot layer of gravel should be placed on top of the sand layer to stabilize the grade control structures. 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.
- The mulch layer should consist of 3 to 4 inches of triple-shredded hardwood mulch. This provides additional benefits such as removing sediment and metals and retaining soil moisture.
- 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 4.20-1** to size the cobble stones based on the velocity of flow.

Table 4.20-1: Cobble Diameter Based on Flow Velocity

| Cobble Diameter, inches | Allowable velocity (ft/s) |
|---|---|
| 4 | 5.8 |
| 5 | 6.4 |
| 6 | 6.9 |
| 7 | 7.4 |
| 8 | 7.9 |
| 9 | 8.4 |
| 10 | 8.8 |
| 11 | 9.2 |
| 12 | 9.6 |
| 15 | 10.4 |

- If the native soils are not suitable for planting, then an engineered soil mix should be provided that meets the following specifications:
  - » **Texture**: Sandy loam or loamy sand
  - » **Sand Content**: Soils should contain 35%-60% clean, washed sand
  - » **Topsoil Content**: Soils should contain 20%-30% topsoil
  - » **Organic Matter Content**: Soils should contain 10%-25% organic matter
  - » **Clay**: Soils should contain less than 15%
  - » **Infiltration Rate**: Soils should have an infiltration rate of at least 0.50 inches per hour (in/hr), although an infiltration rate of between 2 and 4 in/hr is preferred
  - » **Phosphorus Index (P-Index)**: Soils should have a P-Index of less than 30
  - » **Exchange Capacity (CEC)**: Soils should have a CEC that exceeds 10 milliequivalents (meq) per 100 grams of dry weight
  - » **pH**: Soils should have a pH of 6-8
  - » For additional information on the soils for a Regenerative Stormwater Conveyance, refer to Appendix D.

#### 4.20.5.4 PRETREATMENT/INLETS

A grass filter strip or channel can be used for pretreatment. The length of the grass channel or width of the grass filter strip depends on the drainage area, land use, and channel slope. Design guidance on grass channels for pretreatment can be found in Section 4.9 (*Grass Channel*) and filter strips can be found in Section 4.29 (*Vegetated Filter Strip*).

#### 4.20.5.5 OUTLET STRUCTURES

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.

#### 4.20.5.6 SAFETY FEATURES

RSC generally does not require any special safety features, and fencing the RSC facility is not generally desired.

#### 4.20.5.7 LANDSCAPING

- Landscaping is critical to the performance and function of the RSC; the vegetation filters and transpires runoff and the root systems encourage infiltration.
- Vegetation should be selected to match the look and maintenance effort desired by the local community and those responsible for maintaining the facility.
- The RSC should be vegetated to resemble a terrestrial forest ecosystem, with a mature tree canopy, subcanopy of understory trees, shrub layer, and herbaceous ground cover. Three species each of trees, shrubs, and grass/herbaceous species should be planted to avoid creating a monoculture.
- Woody vegetation should not be specified at inflow locations.
- Plants should be installed prior to mulch.
- Choose plants based on factors such as whether they are native or not, resistance to drought and inundation, cost, aesthetics, maintenance, etc. Planting recommendations for RSCs are as follows:
  - » Native plant species should be specified over non-native species.
  - » Vegetation should be selected based on a specified zone of hydric tolerance.
  - » A selection of trees with an understory of shrubs and herbaceous materials should be provided.

Additional information and guidance on the appropriate woody and herbaceous species appropriate for RSC in Georgia, and their planting and establishment, can be found in

.

#### 4.20.5.8 CONSTRUCTION CONSIDERATIONS

- Construction equipment should be restricted from the regenerative stormwater conveyance system to prevent compaction of the native soils.
- A dense and vigorous vegetative cover should be established over the contributing pervious drainage areas before runoff can be accepted into the facility. Otherwise sediment from the stormwater runoff will clog the pores in the planting media and native soils.

## 4.20.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of a RSC.**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a RSC. Create a rough layout of the RSC dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the RSC.**

Consider whether the RSC is intended to:

- Meet the water quality (treatment) target. For information on the sizing of the BMP utilizing the water quality treatment approach, see Step 3.
- Provide a possible solution to a drainage problem
- Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3 for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of this method.*

(Step 3) **Calculate the Target Water Quality Volume**

Calculate the Water Quality Volume using the following formula:

$$WQ_V = (1.2)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$WQ_V$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

Using the $WQ_V$ calculated above, determine the actual size and Volume of the Practice (VP) as shown in Step 4. Note that $VP_{TOTAL}$, calculated in Step 4, must be greater than or equal to $WQ_V$.

(Step 4) **Determine the storage volume of the practice and the Pre-treatment Volume.**

The total volume provided (VP) is calculated by the following equation:

$$VP_{TOTAL} = VP_{SAND} + \Sigma\, VP_{POOLS}$$

*Where*:

**$VP_{TOTAL}$** = Total volume provided by the RSC system

**$VP_{SAND}$** = 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

To determine the porosity, a qualified licensed professional should be consulted to determine the proper porosity based on the sand used. Most sand has a porosity value of 0.40.

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

(Step 5) **Determine whether the $WQ_v$ was met.**

When the $VP_{Total} \geq WQ_v$, then proceed to Step 6.

When the $VP_{Total} < WQ_v$, then proceed to Step 7.

(Step 6) **Design grade control structure, pools, and cascades as described in Subsection 4.20.5. Design and Planning Criteria.**

(Step 7) **Prepare Vegetation and Landscaping Plan.**

A landscaping plan for a dry or wet swale should be prepared to indicate how the RSC system will be stabilized and established with vegetation.

See Subsection 4.20.5.7 (*Landscaping*) and Appendix D for more details.

## 4.20.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.21 Sand Filters

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

**LID/GI Consideration:** Sand filters have low land requirement and may be incorporated to complement the natural landscape.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Typically requires 2 to 6 feet of head, depending on the type of sand filter
- Maximum contributing drainage area of 10 acres for surface sand filter; 2 acres for perimeter sand filter
- Sand filter media with underdrain system

### ADVANTAGES / BENEFITS

- Applicable to small drainage areas
- Good for highly impervious areas
- Good retrofit capability

### DISADVANTAGES / LIMITATIONS

- High maintenance burden
- Not recommended for areas with high sediment content in stormwater or clay/silt runoff areas
- Relatively costly
- Possible odor problems

### MAINTENANCE REQUIREMENTS

- Inspect for clogging – rake first inch of sand
- Remove sediment from forebay and chamber
- Replace sand filter media as needed

### POLLUTANT REMOVAL

80% **Total Suspended Solids**

50% **Metals** - Cadmium, Copper, Lead, and Zinc removal

50/25% **Nutrients** - Total Phosphorus / Total Nitrogen removal

40% **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- Runoff Reduction
- ✓ Water Quality
- ★ Channel Protection
- Overbank Flood Protection
- Extreme Flood Protection

✓ suitable for this practice

★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

L **Land Requirement**

H **Capital Cost**

H **Maintenance Burden**

**Residential Subdivision Use:** *No*
**High Density/Ultra-Urban:** *Yes*
**Drainage Area:** *2-10 acres max.*

**Soils:** *No restrictions*

**Other Considerations:** Typically needs to be combined with other controls to provide water quantity control

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 0% of the runoff reduction volume provided
- Accepts Hotspot Runoff: *Yes (requires impermeable liner)*



BACK TO TOC

## 4.21.1 General Description

Sand filters (also referred to as filtration basins) are BMPs that capture and temporarily store stormwater runoff and pass it through a filter bed of sand. Most sand filter systems consist of two-chamber structures. The first chamber is a sediment forebay or sedimentation chamber, which removes floatables and heavy sediments. The second is the filtration chamber, which removes additional pollutants by filtering the runoff through a sand bed. The filtered runoff is typically collected and returned to the conveyance system, though it can also be partially or fully exfiltrated into the surrounding soil in areas with porous soils.

Because they have few site constraints beside head requirements, sand filters can be used on development sites where the use of other structural controls may be precluded. However, sand filter systems can be relatively expensive to construct and maintain.

There are two primary sand filter system designs, the surface sand filter and the perimeter sand filter. Below are descriptions of these filter systems:

- Surface Sand Filter – The surface sand filter is a ground-level open air structure that consists of a pretreatment sediment forebay and a filter bed chamber. This system can treat drainage areas up to 10 acres in size and is most commonly located off-line. Surface sand filters can be designed as an excavation with earthen embankments or as a concrete or block structure.
- Perimeter Sand Filter – The perimeter sand filter is an enclosed filter system typically constructed just below grade in a vault along the edge of an impervious area such as a parking lot. The system consists of a sedimentation chamber and a sand bed filter. Runoff flows into the structure through a series of inlet grates located along the top of the control.

A third design variant, the underground sand filter, is intended primarily for extremely space limited and highly dense areas and is thus only considered when local communities allow. Underground sand filters require additional planning for access, maintenance, and incorporation with the stormwater management plan.

**Surface Sand Filter**

**Perimeter Sand Filter**

**Figure 4.21-1** Sand Filter Examples

BACK TO TOC

## 4.21.2 Stormwater Management Suitability

Sand filter systems are designed primarily as off-line systems for stormwater quality (i.e., the removal of stormwater pollutants) and will typically need to be used in conjunction with another BMP to provide downstream channel protection, overbank flood protection, and extreme flood protection, if required. However, under certain circumstances, filters can provide limited runoff quantity control, particularly for smaller storm events. If used for smaller drainage areas, or used to provide limited quantity control, sand filters could be used as an on-line system provided proper design planning for erosion and scour is considered.

- Runoff Reduction Volume
  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. Subsection 4.21.3 provides median pollutant removal efficiencies that can be used for planning and design purposes.

- 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 ($Q_p$) 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.

## 4.21.3 Pollutant Removal Capabilities

Both the surface and perimeter sand filters are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed sand filters can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- *Total Suspended Solids* – 80%
- *Total Phosphorus* – 50%
- *Total Nitrogen* – 25%
- *Fecal Coliform* – 40%
- *Heavy Metals* – 50%

For additional information and data on pollutant removal capabilities for sand filters, see the National Pollutant Removal Performance Database (Version 3) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

BACK TO TOC

### 4.21.4 Application and Site Feasibility Criteria

Sand filter systems are well suited for highly impervious areas where land available for BMPs is limited. Sand filters should primarily be considered for new construction or retrofit opportunities for commercial, industrial, and institutional areas where the sediment load is relatively low, such as: parking lots, driveways, loading docks, gas stations, garages, airport runways/taxiways, and storage yards. Sand filters may also be feasible and appropriate in some multi-family or higher density residential developments.

To avoid rapid clogging and failure of the filter media, the use of sand filters should be avoided in areas with less than 50% impervious cover, or high sediment yield sites with clay/silt soils. Special attention should also be given to the topsoil and sod layers, if being used (See **Figure 4.21-2**).

The following basic criteria should be evaluated to ensure the suitability of a sand filter facility for meeting stormwater management objectives on a site or development.

**General Feasibility**

- Suitable for Residential Subdivision Usage – **NO**
- Suitable for High Density/Ultra Urban Areas – **YES**
- Regional Stormwater Control – **NO**

**Physical Feasibility -** Physical Constraints at Project Site

- **Drainage Area** – 10 acres maximum for surface sand filter; 2 acres maximum for perimeter sand filter
- **Space Required** – Function of available head at site
- **Site Slope** – No more than 6% slope across filter location
- **Minimum Head** – Elevation difference needed at a site from the inflow to the outflow: 5 feet for surface sand filters; 2 to 3 feet for perimeter sand filters
- **Minimum Depth to Water Table** – For a surface sand filter with exfiltration (earthen structure), 2 feet are required between the bottom of the sand filter and the elevation of the seasonally high water table
- **Soils** – No restrictions; Group "A" soils generally required to allow exfiltration (for surface sand filter earthen structure)

**Other Constraints/Considerations**

- **Aquifer Protection** – Do not allow exfiltration of filtered hotspot runoff into groundwater
- **Landscaping** -
  - ❏ Check with your local review authority to see if the planning of a grass cover or turf over a sand filter is allowed.
  - ❏ Do not plant trees or provide shade within 15 feet of filtering area or where leaf litter will collect and clog filtering area.
  - ❏ Do not locate plants to block maintenance access to the facility.
  - ❏ Sod areas with heavy flows that are not stabilized with erosion control mats.
  - ❏ Divert flows temporarily from seeded areas until stabilized.
  - ❏ Planting on any area requiring a filter fabric should include material selected with care to insure that no tap roots will penetrate the filter fabric.

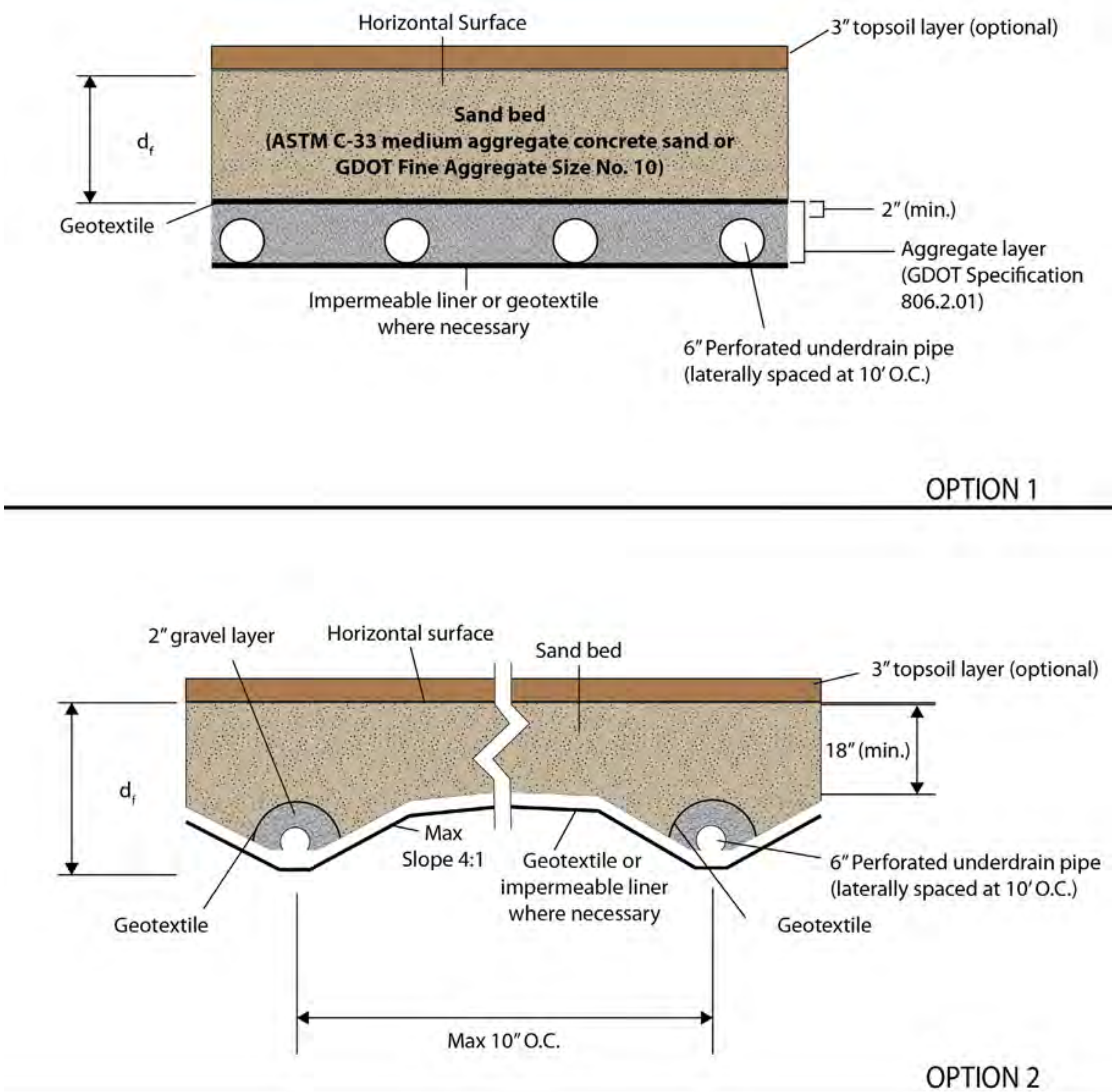



**Figure 4.21-2** Typical Sand Filter Media Cross Sections
(Source: GDOT Drainage Manual, 2014)

## 4.21.5 Planning and Design Criteria

*The following criteria are to be considered* ***minimum*** *standards for the design of a sand filter facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

### 4.21.5.1 LOCATION AND LAYOUT

- Surface sand filters should have a contributing drainage area of 10 acres or less. The maximum drainage area for a perimeter sand filter is 2 acres.
- Sand filter systems are generally applied to land uses with a high percentage of impervious surfaces. Sites with less than 50% imperviousness or high clay/silt sediment loads must not use a sand filter without adequate pretreatment due to potential clogging and failure of the filter bed. Any disturbed areas within the sand filter facility drainage area should be identified and stabilized. Filtration controls should only be constructed after the construction site is stabilized.
- Surface sand filters are generally used in an off-line configuration where the water quality volume ($WQ_v$) is diverted to the filter facility through the use of a flow diversion structure and flow splitter. Stormwater flows greater than the $WQ_v$ are diverted to other controls or downstream using a diversion structure or flow splitter.

- Perimeter sand filters are typically sited along the edge, or perimeter, of an impervious area such as a parking lot.
- Sand filter systems are designed for intermittent flow and must be allowed to drain and reaerate between rainfall events. They should not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

#### 4.21.5.2 GENERAL DESIGN

- **Surface Sand Filter**
  A surface sand filter facility consists of a two-chamber open-air structure, which is located at ground-level. The first chamber is the sediment forebay (a.k.a sedimentation chamber) while the second chamber houses the sand filter bed. Flow enters the sedimentation chamber where settling of larger sediment particles occurs. Runoff is then discharged from the sedimentation chamber through a perforated standpipe into the filtration chamber. After passing though the filter bed, stormwater runoff is collected by a perforated pipe and gravel underdrain system. **Figure 4.21-3** provides plan view and profile schematics of a surface sand filter.
- **Perimeter Sand Filter**
  A perimeter sand filter facility is a vault structure located just below grade level. Runoff enters the device through inlet grates along the top of the structure into the sedimentation chamber. Runoff is discharged from the sedimentation chamber through a weir into the filtration chamber. After passing though the filter bed, runoff is collected by a perforated pipe and gravel underdrain system. **Figure 4.21-4** provides plan view and profile schematics of a perimeter sand filter.
- **Underground Sand Filter**
  An underground sand filter is in an underground vault structure designed for high-density land use or ultra-urban applications where there is not enough space for a surface sand filter or other BMP. Due to its location below the surface, underground sand filters have a high maintenance burden and should only be used where adequate inspection and maintenance can be ensured. **Figure 4.21-5** provides plan view and profile schematics of an underground sand filter.

#### 4.21.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- **Surface Sand Filter**
  - » The entire treatment system (including the sedimentation chamber) must temporarily hold at least 75% of the $WQ_v$ prior to filtration. **Figure 4.21-6** illustrates the distribution of the treatment volume (0.75 WQv) among the various components of the surface sand filter, including
    - $V_s$ – volume within the sedimentation basin
    - $V_f$ – volume within the voids in the filter bed
    - $V_{f\text{-}temp}$ – temporary volume stored above the filter bed
    - $A_s$ – the surface area of the sedimentation basin
    - $A_f$ – surface area of the filter media
    - $h_s$ – height of water in the sedimentation basin
    - $h_f$ – average height of water above the filter media
    - $d_f$ – depth of filter media
  - » The sedimentation chamber must be sized to at least 25% of the computed $WQ_v$ and have a length-to-width ratio of at least 2:1. Inlet and outlet structures should be located at opposite ends of the chamber.
  - » The filter area is sized based on the principles of Darcy's Law. A coefficient of permeability (k) of 3.5 ft/day for sand should be used. The filter bed is typically designed to completely drain in 40 hours or less.
  - » The filter media consists of an 18-inch 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. Three inches of topsoil are placed over the sand bed. Permeable filter fabric is placed both above and below the sand bed to prevent clogging of the sand filter and the underdrain system. **Figure 4.21-2** illustrates a typical media cross section.

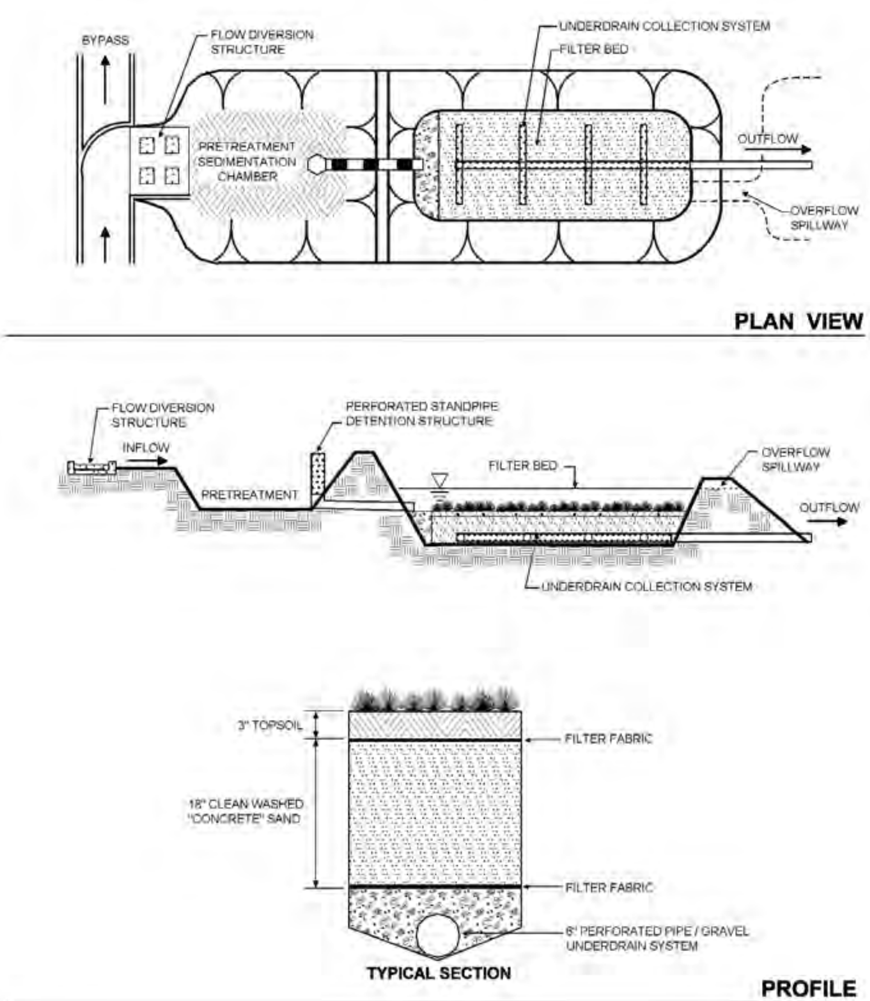



**Figure 4.21-3** Schematic of Surface Sand Filter
(Source: Center for Watershed Protection)

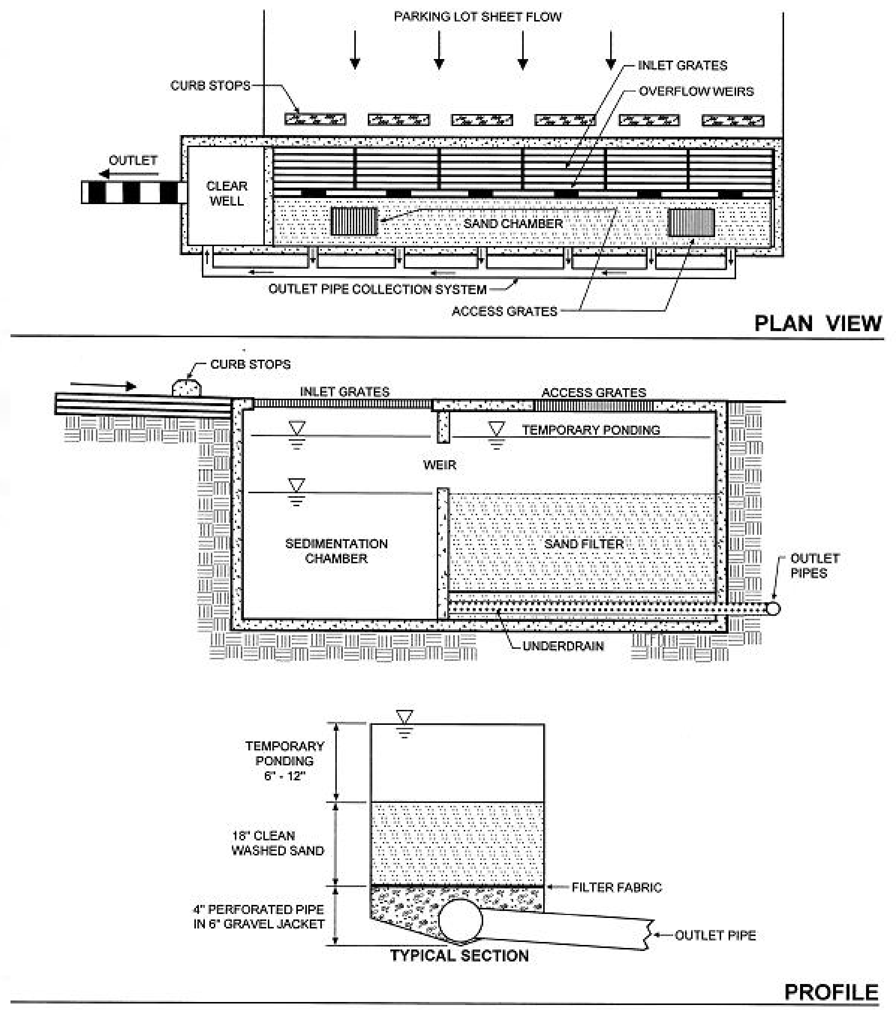



**Figure 4.21-4** Schematic of Perimeter Sand Filter
(Source: Center for Watershed Protection)

BACK TO TOC

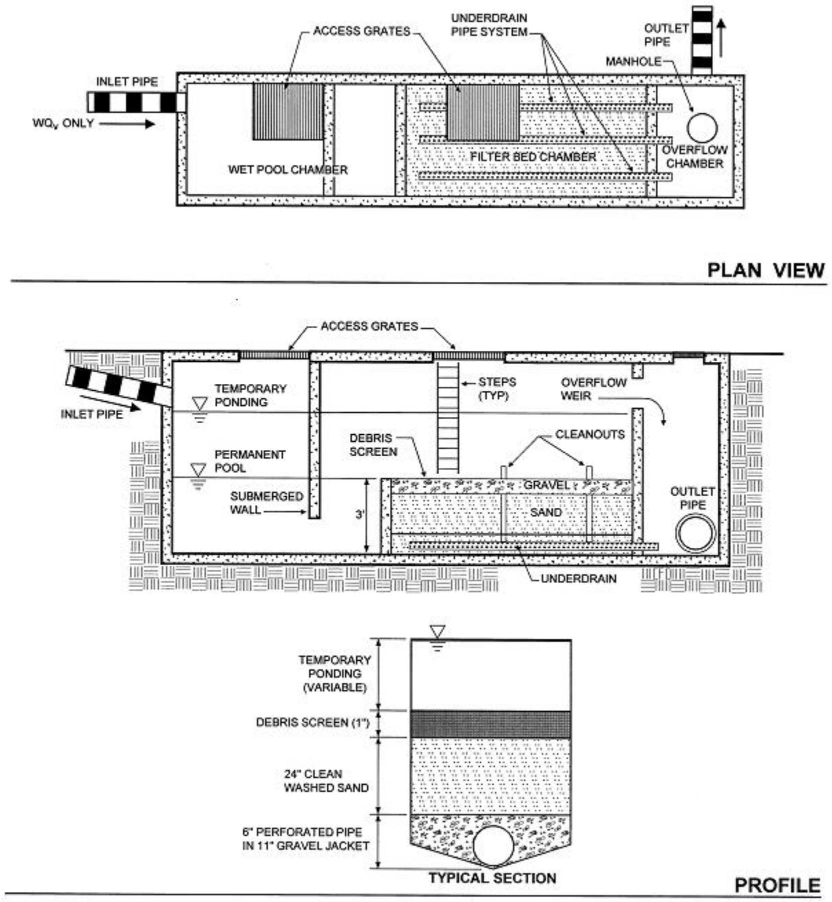



**Figure 4.21-5** Schematic of Underground Sand Filter
(Source: Center for Watershed Protection

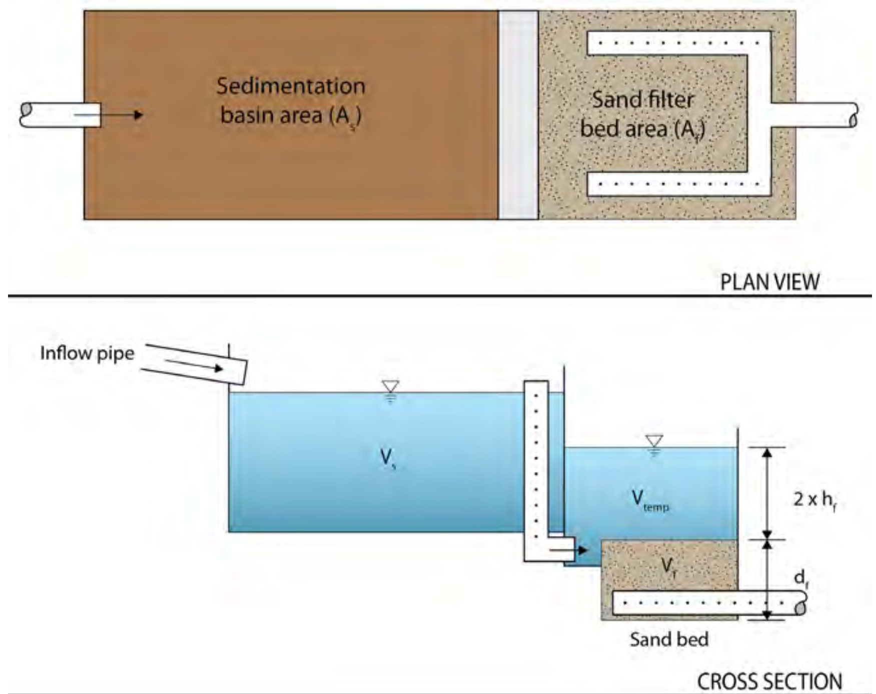



**Figure 4.21-6** Surface Sand Filter Volumes
Source: GDOT Drainage Manual, 2014

» The filter bed is typically equipped with a minimum 6-inch perforated PVC pipe (AASHTO M 252) underdrain in a gravel layer. The underdrain must have a minimum grade of 1/8-inch per foot (1% slope). Holes should be 3/8-inch diameter and spaced approximately 6 inches on center. Gravel should be clean washed aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches with a void space of about 40% (GDOT No.3 Stone). Aggregate contaminated with soil shall not be used.

» 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 structures before installation of the underdrain system and filter media.

- **Perimeter Sand Filter**

» The entire treatment system (including the sedimentation chamber) must temporarily hold at least 75% of the WQv prior to filtration. **Figure 4.21-7** illustrates the distribution of the treatment volume (0.75 $WQ_v$) among the various components of the perimeter sand filter, including:

- $V_w$ – wet pool volume within the sedimentation basin
- $V_f$ – volume within the voids in the filter bed
- $V_{temp}$ – temporary volume stored above the filter bed
- $A_s$ – the surface area of the sedimentation basin
- $A_f$ – surface area of the filter media
- $h_f$ – average height of water above the filter media (1/2 $h_{temp}$)
- $d_f$ – depth of filter media

» The sedimentation chamber must be sized to at least 50% of the computed $WQ_v$.

» The filter area is sized based on the principles of Darcy's Law. A coefficient of permeability (k) of 3.5 ft/day for sand should be used. The filter bed is typically designed to completely drain in 40 hours or less.

» The filter media should consist of a 12- to 18-inch 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. **Figure 4.21-2** illustrates a typical media cross section.

» The perimeter sand filter is typically equipped with a minimum 4 inch perforated PVC pipe (AASHTO M 252) underdrain in a gravel layer. The underdrain must have a minimum grade of 1/8 inch per foot (1% slope). Holes should be 3/8-inch diameter and spaced approximately 6 inches on center. A permeable filter fabric should be placed between the gravel layer and the filter media. Gravel should be clean washed aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches with a void space of about 40% (GDOT No.3 Stone). Aggregate contaminated with soil shall not be used.

### 4.21.5 PRETREATMENT/INLETS

- Pretreatment of runoff in a sand filter system is provided by the sedimentation chamber.
- Inlets to surface sand filters are to be provided with energy dissipators. Exit velocities from the sedimentation chamber must be nonerosive.
- **Figure 4.21-8** shows a typical inlet pipe from the sedimentation basin to the filter media basin for the surface sand filter.

### 4.21.5.5 OUTLET STRUCTURES

Outlet pipe is to be provided from the underdrain system to the facility discharge. Due to the slow rate of filtration, outlet protection is generally unnecessary (except for emergency overflows and spillways).

### 4.21.5.6 EMERGENCY SPILLWAY

An emergency or bypass spillway must be included in the surface sand filter to safely pass flows that exceed the design storm flows. The spillway prevents filter water levels from overtopping the embankment and causing structural damage. The emergency spillway should be located so that downstream buildings and structures will not be impacted by spillway discharges.

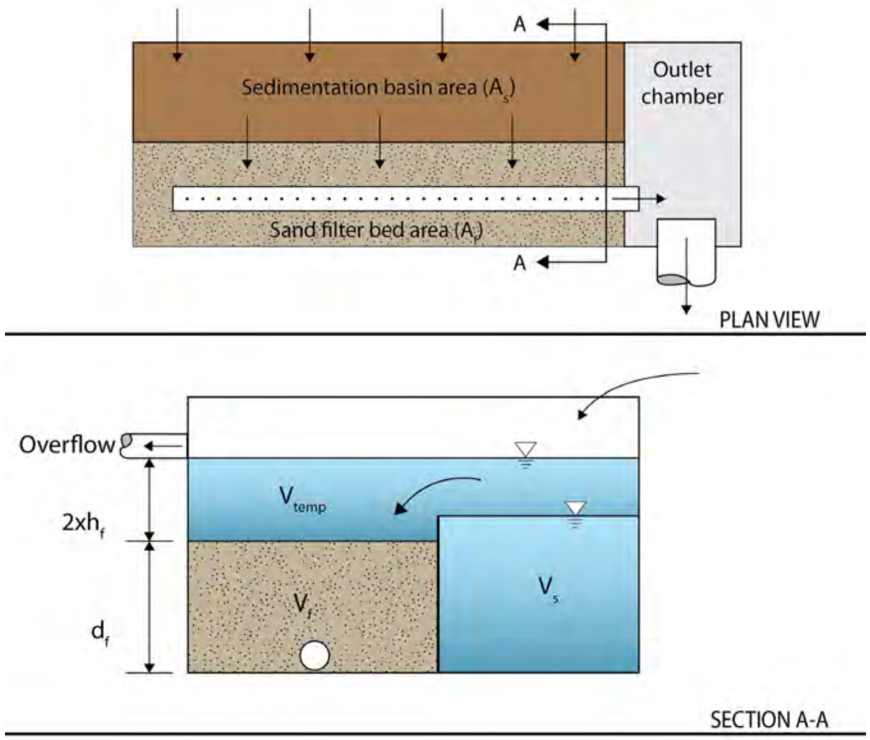



**Figure 4.21-7** Perimeter Sand Filter Volumes
(Source: GDOT Drainage Manual, 2014)

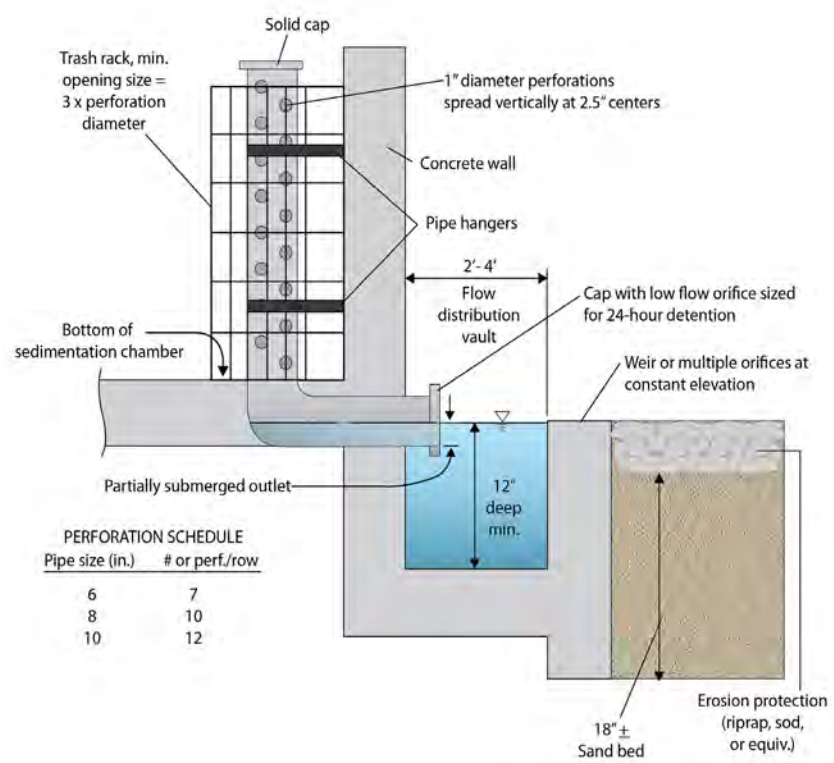



**Figure 4.21-8** Surface Sand Filter Perforated Stand-Pipe
(Source: GDOT Drainage Manual, 2014)

#### 4.21.5.7 MAINTENANCE ACCESS

Adequate access must be provided for all sand filter systems for inspection and maintenance, including the appropriate equipment and vehicles. Access grates to the filter bed need to be included in a perimeter sand filter design. Facility designs must enable maintenance personnel to easily replace upper layers of the filter media.

#### 4.21.5.8 SAFETY FEATURES

Surface sand filter facilities can be fenced to prevent access. Inlet and access grates to perimeter sand filters may be locked.

#### 4.21.5.9 LANDSCAPING

Surface sand filters can be designed with a grass cover to aid in pollutant removal and prevent clogging. The grass should be capable of withstanding frequent periods of inundation and drought. The sand filter is covered with permeable topsoil and planted with grass in a landscaped area. Properly planted, these facilities can be designed to blend into natural surroundings. Vegetated filter strips and buffers should fit into and blend with surrounding area. Native grasses are preferable, if compatible.

#### 4.21.5.10 ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

**Physiographic Factors -** Local terrain design constraints

- **Low Relief** – Use of surface sand filter may be limited by low head
- **High Relief** – Filter bed surface must be level
- **Karst** – Use polyliner or impermeable membrane to seal bottom of earthen surface sand filter or use watertight structure

**Soils**

No restrictions

**Special Downstream Watershed Considerations**

- **Trout Stream** – Evaluate for stream warming; use shorter drain time (24 hours) or consider increasing media filter depth
- **Aquifer Protection** – Use polyliner or impermeable membrane to seal bottom of earthen surface sand filter or use watertight structure; no exfiltration of filter runoff into groundwater

## 4.21.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of a sand filter.**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a sand filter. Create a rough layout of the sand filter dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the sand filter.**

Consider whether the sand filter is intended to:

- » Meet a water quality (treatment) target. See Step 3 to size the BMP utilizing the water quality treatment approach.
- » Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)
- » Provide a possible solution to a drainage problem

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design. The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3 for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3) **Calculate the Target Water Quality Volume**

Calculate the Runoff Reduction Volume using the following formula:

$$WQ_v = (1.2)\,(R_v)\,(A) / 12$$

*Where*:

**$WQ_v$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches))
**$R_v$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to the practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4) **Compute $WQ_v$ peak discharge ($Q_{wq}$)**

The peak rate of discharge for the water quality design storm is needed for sizing of off-line diversion structures (see Subsection 3.1.7). If designing off-line, follow steps (a) through (d) below:

(a) Using $WQ_v$, compute CN
(b) Compute time of concentration using TR-55 method
(c) Determine appropriate unit peak discharge from time of concentration
(d) Compute $Q_{wq}$ from unit peak discharge, drainage area, and $WQ_v$

(Step 5) **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. Size low flow orifice, weir, or other device to pass $Q_{wq}$.

(Step 6) **Size filtration basin chamber.**

The filter area is sized using the following equation (based on Darcy's Law):

$$A_f = (WQ_v)(d_f) / [(k)(h_f + d_f)(t_f)]$$

*Where*:

$\mathbf{A_f}$ = surface area of filter bed ($ft^2$)
$\mathbf{d_f}$ = filter bed depth (typically 18 in, no more than 24 in)
**k** = coefficient of permeability of filter media (ft/day) use 3.5 ft/day for sand)
$\mathbf{h_f}$ = average height of water above filter bed (ft) (1/2 $h_{max}$, which varies based on site but $h_{max}$ is typically $\leq$ 6 feet)
$\mathbf{t_f}$ = design filter bed drain time (days), (1.67 days or 40 hours is recommended maximum)

Set preliminary dimensions of filtration basin chamber. See Subsection 4.21.5.3 (*Physical Specifications/Geometry*) for filter media specifications.

(Step 7) **Size sedimentation chamber.**

*Surface sand filter*: 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:

$$A_s = -(Q_o/w) * Ln(1-E)$$

*Where*:

$\mathbf{A_s}$ = sedimentation basin surface area ($ft^2$)
$\mathbf{Q_o}$ = rate of outflow = the $WQ_v$ over a 24-hour period ($ft^3/s$)
**w** = particle settling velocity (ft/sec)
**E** = trap efficiency

*Assuming*:

- » 90% sediment trap efficiency (0.9)
- » Particle settling velocity (ft/sec) = 0.0033 ft/sec for imperviousness $\geq$ 75%
- » Particle settling velocity (ft/sec) = 0.0004 ft/sec for imperviousness < 75%
- » Average of 24 hour holding period

*Then*:

$A_s = (0.066)(WQ_v)\ ft^2$ for I < 75%
$A_s = (0.0081)(WQ_v)\ ft^2$ for I > 75%

Set preliminary dimensions of sedimentation chamber.

*Perimeter sand filter*: The sedimentation chamber should be sized to at least 50% of the computed $WQ_v$. Use same approach as for surface sand filter.

(Step 8) Compute $V_{min}$

$$V_{min} = 0.75 * WQ_v$$

(Step 9) **Compute storage volumes within entire facility and sedimentation chamber orifice size.**

*Surface sand filter:*

$$V_{min} = 0.75\ WQ_v = V_s + V_f + V_{f\text{-}temp}$$

1. Compute $V_f$ = water volume within filter bed/gravel/pipe = $A_f * d_f * n$, where: n = porosity = 0.4 for most applications
2. Compute $V_{f\text{-}temp}$ = temporary storage volume above the filter bed = $2 * h_f * A_f$
3. Compute $V_s$ = volume within sediment chamber = $V_{min} - V_f - V_{f\text{-}temp}$
4. Compute $h_s$ = height in sedimentation chamber = $V_s/A_s$
5. 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.
6. Size orifice from sediment chamber to filter chamber to release $V_s$ within 24-hours at average release rate with 0.5 $h_s$ as average head.
7. Design outlet structure with perforations allowing for a safety factor of 10.
8. Size distribution chamber to spread flow over filtration media – level spreader weir or orifices.

*Perimeter sand filter:*

1. Compute $V_f$ = water volume within filter bed/gravel/pipe = $A_f * d_f * n$
2. Where: n = porosity = 0.4 for most applications
3. Compute $V_w$ = wet pool storage volume $A_s * 2$ feet minimum
4. Compute $V_{temp}$ = temporary storage volume = $V_{min} – (V_f + V_w)$
5. Compute $h_{temp}$ = temporary storage height = $V_{temp} / (A_f + A_s)$
6. Ensure $h_{temp} \geq 2 * h_f$, otherwise decrease $h_f$ and re-compute. Ensure dimensions fit available head and area – change as necessary in design iterations until all site dimensions fit.
7. Size distribution slots from sediment chamber to filter chamber.

(Step 10) **Design inlets, pretreatment facilities, underdrain system, and outlet structures**

See Subsection 4.21.5.4 through 4.21.5.8 for more details.

(Step 11) **Compute overflow weir sizes.**

*Surface sand filter:*

1. Size overflow weir at elevation $h_s$ in sedimentation chamber (above perforated stand pipe) to handle surcharge of flow through filter system from 25-year storm (see example).
2. Plan inlet protection for overflow from sedimentation chamber and size overflow weir at elevation hf in filtration chamber (above perforated stand pipe) to handle surcharge of flow through filter system from 25-year storm (see example).

*Perimeter sand filter*: Size overflow weir at end of sedimentation chamber to handle excess inflow, set at $WQ_v$ elevation.

See Appendix B-3 for a Sand Filter Design Example

### 4.21.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

BACK TO TOC

# 4.22 Site Reforestation/Revegetation

**Description**: Site reforestation/revegetation refers to the process of planting trees, shrubs and other native vegetation in disturbed pervious areas to restore them to their pre-development conditions. The process can be used to help establish mature native plant communities (e.g., forests) in pervious areas that have been disturbed by clearing, grading and other land disturbing activities.

**LID/GI Consideration**: Restoring sites back to their pre-developed conditions improves their ability to reduce post-construction stormwater runoff rates, volumes and pollutant loads. The process can also be used to provide restored habitat for high priority plant and animal species.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Ideal for use in pervious areas that have been disturbed by clearing, grading and other land disturbing activities
- Methods used for site reforestation/revegetation should achieve at least 75% vegetative cover one year after installation
- Reforested/revegetated areas should be protected in perpetuity as secondary conservation areas

### ADVANTAGES / BENEFITS

- Applicable to small drainage areas
- Good for highly impervious areas
- Good retrofit capability

### DISADVANTAGES / LIMITATIONS

- High maintenance burden
- Not recommended for areas with high sediment content in stormwater or clay/silt runoff areas
- Relatively costly
- Possible odor problems

### MAINTENANCE REQUIREMENTS

- Inspect for clogging – rake first inch of sand
- Remove sediment from forebay and chamber
- Replace sand filter media as needed

### POLLUTANT REMOVAL

- 80% **Total Suspended Solids**
- 50% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 50/25% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- 40% **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- [1]Runoff Reduction
- [1]Water Quality
- [1]Channel Protection
- [1]Overbank Flood Protection
- [1]Extreme Flood Protection

✓ suitable for this practice

★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- M Land Requirement
- L Capital Cost
- L Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *Yes.*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 0% of the runoff reduction volume provided (see note 1)

1 = helps restore pre-development hydrology, which implicitly reduces post-construction stormwater runoff rates, volumes and pollutant loads. Runoff reduction credit is given to other BMPs that use soil restoration to improve hydrologic soil groups. See guidance below for calculating $RR_v$ and other reductions.

BACK TO TOC

### 4.22.1 General Description

Site reforestation/revegetation refers to the process of planting trees, shrubs and other native vegetation in disturbed pervious areas to restore them to their pre-development conditions (**Figure 4.22-1**). The process can be used to help establish mature native plant communities (e.g., forests) in pervious areas that have been disturbed by clearing, grading and other land disturbing activities. Mature plant communities intercept rainfall, increase evaporation and transpiration rates, slow and filter stormwater runoff and help improve soil porosity and infiltration rates (Cappiella et al., 2006a), which leads to reduced post-construction stormwater runoff rates, volumes and pollutant loads. The site reforestation/revegetation process can also be used to provide restored habitat for high priority plant and animal species.

Areas that have been reforested or revegetated should be maintained in an undisturbed, natural state over time. These areas should be designated as conservation areas and protected in perpetuity through a legally enforceable conservation instrument (e.g., conservation easement, deed restriction). If properly maintained over time, these areas can help improve aesthetics on development sites, provide passive recreational opportunities and create valuable habitat for high priority plant and animal species.

To help create contiguous, interconnected green infrastructure corridors on development sites, site planning and design teams should strive to connect reforested or revegetated areas with one another and with other primary and secondary conservation areas through the use of nature trails, bike trails and other "greenway" areas.

### 4.22.2 Stormwater Management Suitability

The Center for Watershed Protection (Hirschman et al., 2008) documented the ability of the site reforestation/revegetation process to reduce annual stormwater runoff volumes and pollutant loads on development sites. Consequently, this low impact development practice can be used to help satisfy the reduce runoff volume and provide water quality improvements:

- Runoff Reduction
  Site reforestation/revegetation is an effective low impact development (LID) practice that can reduce post-construction stormwater runoff and improve water quality. When used to improve site areas and create conservation amenities; runoff reduction, lower post-developed flow rates, and lower discharge velocities are all benefits of reforestation or revegetation. Subtract 50% of any reforested/revegetated areas from the total site area and re-calculate the runoff reduction volume ($RR_v$) that applies to the development site.

- Water Quality Protection
  Site reforestation and/or revegetation helps restore pre-development hydrology, which implicitly reduces post-construction stormwater runoff rates in addition to runoff volumes and pollutant loads. Subtract 50% of any reforested/revegetated areas from the total site area and re-calculate the water quality volume ($WQ_v$) that applies to the development site.

**Figure 4.22-1:** Active Replanting of Native Trees in a Disturbed Pervious Area
(Source: Center for Watershed Protection)

- Channel Protection
  Assume that the post-development hydrologic conditions of any reforested/revegetated areas are equivalent to those of a similar cover type (e.g., meadow, brush, woods) in fair condition.

BACK TO TOC

- Overbank Flood Protection
  Assume that the post-development hydrologic conditions of any reforested/revegetated areas are equivalent to those of a similar cover type (e.g., meadow, brush, woods) in fair condition.

- Extreme Flood Protection
  Assume that the post-development hydrologic conditions of any reforested/revegetated areas are equivalent to those of a similar cover type (e.g., meadow, brush, woods) in fair condition. Reforested/revegetated areas can only be assumed to be in "fair" hydrologic condition due to the fact that it will take many years for them to mature and provide full stormwater management benefits.

**Stormwater Management Suitability when combined with Soil Restoration**

If site reforestation/revegetation can be combined with soil restoration (Section 4.23) on a development site, the following stormwater management benefits and incentives are available to help satisfy the requirements presented in this manual:

- Stormwater Runoff Reduction
  Subtract 100% of any restored and reforested/revegetated areas from the total site area *and* re-calculate the runoff reduction volume ($RR_v$) that applies to the development site.

- Water Quality Protection
  Subtract 100% of any restored and reforested/revegetated areas from the total site area *and* re-calculate the water quality volume ($WQ_v$) that applies to the development site.

- Channel Protection
  Assume that the post-development hydrologic conditions of any restored and reforested/revegetated areas are equivalent to those of a similar cover type (e.g., meadow, brush, woods) in good condition.

- Overbank Flood Protection
  Assume that the post-development hydrologic conditions of any restored and reforested/revegetated areas are equivalent to those of a similar cover type (e.g., meadow, brush, woods) in good condition.

- Extreme Flood Protection
  Assume that the post-development hydrologic conditions of any restored and reforested/revegetated areas are equivalent to those of a similar cover type (e.g., meadow, brush, woods) in good condition.

## 4.22.3 Applications and Site Feasibility Criteria

The criteria listed in **Table 4.22-1** should be evaluated to determine whether or not site reforestation/revegetation is appropriate for use on a development site.

**General Feasibility**

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Stormwater Control – NO

**Site Applicability**

Although it may be difficult to apply in urban areas, due to space constraints, site reforestation/revegetation can be used on a wide variety of development sites, including residential, commercial, industrial and institutional development sites in rural and suburban areas. When compared with other low impact development practices, it has a moderate construction cost, a relatively low maintenance burden and requires no additional surface area beyond that which will undergo the reforestation/revegetation process. It is ideal for use in pervious areas that have been disturbed by clearing, grading and other land disturbing activities.

Table 4.22-1: Factors to Consider When Evaluating the Overall Feasibility of Using Site Reforestation/ Revegetation on a Development Site

| Site Characteristic | Criteria |
|---|---|
| Drainage Area | N/A |
| Area Required | Reforested/revegetated areas should at least be 10,000 square feet in size in order to be eligible for the stormwater management "credits" assigned to this low impact development practice (2008, Center for Watershed Protection – Runoff Reduction Technical Memo). |
| Slope | Maximum 25% in the disturbed pervious area to be reforested/ revegetated. |
| Minimum Head | N/A |
| Minimum Depth to Water Table | No restrictions |
| Soils | Soils need to be capable of sustaining the vegetation proposed which will require significant amendments in most cases. |

### 4.22.4 Planning and Design Criteria

It is *recommended* that the reforestation/re-vegetation process used on a development site meet all of the following criteria to be eligible for the stormwater management "credits" described above:

#### General Planning and Design

- Reforested/revegetated areas should have a contiguous area of 10,000 square feet or more.
- Reforested/revegetated areas should not be disturbed after construction (except for disturbances associated with landscaping or removal of invasive vegetation).
- Reforested/revegetated areas should be protected, in perpetuity, from the direct impacts of the land development process by a legally enforceable conservation instrument (e.g., conservation easement, deed restriction).

#### Landscaping

- A soil test should be performed to determine what type of vegetation can be supported by the soils in the area to be reforested/ revegetated and/or what soil amendments will be required.
- A landscaping plan should be prepared by a qualified licensed professional for all reforested/revegetated areas. The landscaping plan should be reviewed and approved by the local development review authority prior to construction.
- Landscaping commonly used in site reforestation/revegetation efforts includes native trees, shrubs and other herbaceous vegetation. Because the goal of the site reforestation/revegetation process is to establish a mature native plant community (e.g., forest), managed turf cannot be used to landscape reforested/revegetated areas.
- Methods used for site reforestation/ revegetation should achieve at least 75 percent vegetative cover one year after installation.
- A long-term vegetation management plan should be developed for all reforested/ revegetated areas. The plan should clearly specify how the area will be maintained in an undisturbed, natural state over time. Plan should include method for watering during plant establishment period of one to two years. Turf management is not considered to be an acceptable form of vegetation management. Consequently, only reforested/revegetated areas that remain in an undisturbed, natural state are eligible for this "credit" (i.e., pervious areas consisting of managed turf are not eligible for this "credit").

BACK TO TOC

### 4.22.5 Construction Considerations

To help ensure that the site reforestation/revegetation process is successfully completed on a development site, site planning and design teams should consider the following recommendations: Document the condition of the reforested/revegetated area before, during and after the completion of the site reforestation/revegetation process.

- Document the condition of the reforested/ revegetated area before, during and after the completion of the site reforestation/ revegetation process.

- To help prevent soil compaction, heavy vehicular and foot traffic should be kept out of all reforested/revegetated areas before, during and after construction. This can typically be accomplished by clearly delineating reforested/ revegetated areas on all development plans and, if necessary, protecting them with temporary construction fencing.

- Simple erosion and sediment control measures, such as temporary seeding and erosion control mats, should be used on reforested/ revegetated areas. If the reforested/revegetated areas will "receive" any stormwater runoff from other portions of the development site, measures should be taken (e.g., silt fence, temporary diversion berm) to prevent it from compromising the reforestation/ revegetation effort.

- Construction contracts should contain a replacement warranty that covers at least three growing seasons to help ensure adequate growth and survival of the vegetation planted on the reforested/revegetated area.

### 4.22.6 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.23 Soil Restoration

(Source: http://www.towncountryltd.com)

**Description:** Soil restoration refers to the process of tilling and adding compost and other amendments to soils to restore them to their pre-development conditions, which improves their ability to reduce post-construction stormwater runoff rates, volumes and pollutant loads. The soil restoration process can be used to improve the hydrologic conditions of pervious areas that have been disturbed by clearing, grading and other land disturbing activities.

**LID/GI Consideration**: Increases the reduction in stormwater runoff rates, volumes and pollutant loads provided by other low impact development practices.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Ideal for use in pervious areas that have been disturbed by clearing, grading and other land disturbing activities
- To properly restore disturbed pervious areas, soil amendments should be added to existing soils to a minimum depth of 18 inches until an organic matter content of 8% to 12% is obtained. Depths greater than 18" should be amended when shrubs or trees are being installed.
- Restored pervious areas should be protected from future land disturbing activities

### ADVANTAGES / BENEFITS

- Helps restore pre-development hydrology on development sites and reduces post-construction stormwater runoff rates, volumes and pollutant loads
- Promotes plant growth and improves plant health, which helps reduce stormwater runoff rates, volumes and pollutant loads

### DISADVANTAGES / LIMITATIONS

- Should not be used on areas that have slopes of greater than 10%
- To help prevent soil erosion, landscaping should be installed immediately after the soil restoration process is complete

### STORMWATER MANAGEMENT SUITABILITY

- ● [1]Runoff Reduction
- ● [1]Water Quality
- ● [1]Channel Protection
- ● [1]Overbank Flood Protection
- ● [1]Extreme Flood Protection

✓ suitable for this practice

★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- **M** Land Requirement
- **L** Capital Cost
- **L** Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *Yes*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 0% of the runoff reduction volume provided (see note 1)

1 = helps restore pre-development hydrology, which implicitly reduces post-construction stormwater runoff rates, volumes and pollutant loads. See guidance below for calculating $RR_V$ and other reductions.

### 4.23.1 General Description

Soil restoration refers to the process of tilling and adding compost and other amendments to soils to restore them to their pre-development conditions. It is ideal for use on lawns and other pervious areas that have been disturbed by clearing, grading and other land disturbing activities. Organic compost (**Figure 4.23-1**) and other amendments can be tilled into soils in these areas to help create healthier, uncompacted soil matrices that have enough organic matter to support a diverse community of native trees, shrubs and other herbaceous plants.

Soil restoration can also be used to increase the stormwater management benefits provided by other low impact development practices, such as site reforestation/revegetation (Section 4.22), vegetated filter strips (Section 4.29), grass channels (Section 4.9) and simple downspout disconnection (Section 4.4), on sites that have soils with low permeabilities (i.e., hydrologic soil group C or D soils). The soil restoration process can be used to help increase soil porosity and improve soil infiltration rates on these sites, which improves the ability of these and other low impact development practices to reduce post-construction stormwater runoff rates, volumes and pollutant loads.

### 4.23.2 Stormwater Management Suitability

The Center for Watershed Protection (Hirschman et al., 2008) documented the ability of the soil restoration process to reduce annual stormwater runoff volumes and pollutant loads on development sites. Consequently, this low impact development practice can be used to help satisfy the stormwater requirements presented in this manual. Consider including soil restoration in all other stormwater management BMPs where applicable.

**Figure 4.23-1**: Organic Compost
(Source: http://www.organicgardeninfo.com)

- Runoff Reduction
  Soil restoration is one of the most effective low impact development (LID) practices that can be combined with other BMPs to reduce post-construction stormwater runoff and improve runoff quality. Like other LID practices, soil restoration becomes more effective the higher the infiltration rate increases. When used to improve native soils when paired with another BMP, runoff reduction percentages can increase from 15 to 25 percent. Subtract 50% of any restored pervious areas from the total site area and re-calculate the runoff reduction volume ($RR_v$) that applies to the development site.

BACK TO TOC

- Water Quality Protection
  Due to the soil amendments themselves, in addition to the runoff reduction benefits, soil restoration inherently improves water quality. Depending on the organic compounds and other amendments added, nutrient uptake and other pollutant removal processes can be achieved. Subtract 50% of any restored pervious areas from the total site area and re-calculate the water quality volume ($WQ_v$) that applies to the development site.

- Channel Protection
  Soil restoration helps restore pre-development hydrology, which implicitly reduces post-construction stormwater runoff rates in addition to runoff volumes and pollutant loads. Assume that the post-development hydrologic conditions of any restored pervious areas are equivalent to those of open space (e.g., lawns, parks, golf courses) in good condition

- Overbank Flood Protection
  Soil restoration helps restore pre-development hydrology, which implicitly reduces post-construction stormwater runoff rates in addition to runoff volumes and pollutant loads. Assume that the post-development hydrologic conditions of any restored pervious areas are equivalent to those of open space (e.g., lawns, parks, golf courses) in good condition.

- Extreme Flood Protection
  Soil restoration helps restore pre-development hydrology, which implicitly reduces post-construction stormwater runoff rates in addition to runoff volumes and pollutant loads. Assume that the post-development hydrologic conditions of any restored pervious areas are equivalent to those of open space (e.g., lawns, parks, golf courses) in good condition.

In order to be eligible for these "credits," it is recommended that restored pervious areas satisfy the planning and design criteria outlined below.

If any type of vegetation other than managed turf can be planted on a restored pervious area, site planning and design teams are encouraged to combine soil restoration with site reforestation/revegetation (Section 4.22) to further reduce post-construction stormwater runoff rates, volumes and pollutant loads.

When soil restoration is used to enhance the performance of other low impact development practices (e.g., site reforestation/revegetation, vegetated filter strips, grass channels), it may be "credited" by improving the soils characterization for purposes of determining the runoff reduction percentage.

## 4.23.3 Applications and Site Feasibility Criteria

The criteria listed in **Table 4.23-1** should be evaluated to determine whether or not soil restoration is appropriate for use on a development site.

### General Feasibility

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **YES**
- Regional Stormwater Control – **NO**

### Site Applicability

Soil restoration can be used on a wide variety of development sites, including residential, commercial, industrial and institutional development sites in rural, suburban and urban areas. When compared with other low impact development practices, it has a moderate construction cost, a relatively low maintenance burden and requires no additional surface area beyond that which will undergo the soil restoration process. It is ideal for use in pervious areas that have been disturbed by clearing, grading and other land disturbing activities.

Table 4.23-1: Factors to Consider When Evaluating the Overall Feasibility of Using Soil Restoration on a Development Site

| Site Characteristic | Criteria |
|---|---|
| Drainage Area | N/A |
| Area Required | No restrictions |
| Slope | Maximum 10% in the disturbed pervious area to be restored. |
| Minimum Head | N/A |
| Minimum Depth to Water Table | A minimum separation distance of 18 inches is recommended between the surface of a restored pervious area and the top of the water table. Depths greater than 18" should be amended when shrubs or trees are installed. |
| Soils | Pervious areas that have soils with low permeabilities (i.e. hydrologic soil group C or D soils) or that have been disturbed by land disturbing activities are good candidates for soil restoration. Areas that have permeable soils (i.e. hydrologic soil group A or B soils) and that have not been disturbed by land disturbing activities do not need to be restored. |

BACK TO TOC

### 4.23.4 Planning and Design Criteria

It is recommended that the soil restoration process used on a development site meet all of the following criteria to be eligible for the stormwater management "credits" described above:

#### General Planning and Design

- To avoid damaging existing root systems, soil restoration should not be performed in areas that fall within the drip line of existing trees.
- Compost should be incorporated into existing soils, using a rototiller or similar equipment, to a depth of 18 inches and at an application rate necessary to obtain a final average organic matter content of 8%-12%. Required application rates can be determined using a compost calculator, such as the one provided on the following website: http://www.soilsforsalmon.org/resources.htm. Other calculations are available online.
- Only well-aged composts that have been composted for a period of at least one year should be used to amend existing soils. Composts should be stable and show no signs of further decomposition.
- Composts used to amend existing soils should meet the following specifications (most compost suppliers will be able to provide this information):
  - **Organic Content Matter**: Composts should contain 35%-65% organic matter.
  - **Moisture Content**: Composts should have a moisture content of 40%-60%.
  - **Bulk Density**: Composts should have an "as-is" bulk density of 40-50 pounds per cubic foot (lb/cf). In composts that have a moisture content of 40%-60%, this equates to a bulk density range of 450-800 pounds per cubic yard (lb/cy), by dry weight.
  - **Carbon to Nitrogen (C:N) Ratio**: Composts should have a C:N Ratio of less than 25:1.
  - **pH**: Composts should have a pH of 6-8.
  - **Cation Exchange Capacity (CEC)**: Composts should have a CEC that exceeds 50 milliequivalents (meq) per 100 grams of dry weight.
  - **Foreign Material Content:** Composts should contain less than 0.5% foreign materials (e.g., glass, plastic), by weight.
  - **Pesticide Content:** Composts should be pesticide free.
- The use of biosolids (except Class A biosolids) and composted animal manure to amend existing soils is not recommended.
- It is recommended that composts used to amend existing soils be provided by a member of the U.S. Composting Seal of Testing Assurance program. Additional information on the Seal of Testing Assurance program is available on the following website: http://www.compostingcouncil.org.

#### Landscaping

- Vegetation commonly planted on restored pervious areas includes turf, shrubs, trees and other herbaceous vegetation. Although managed turf is most commonly used, site planning and design teams are encouraged to use trees, shrubs and/or other native vegetation to help establish mature native plant communities (e.g., forests) in restored pervious areas.
- Methods used to establish vegetative cover within a restored pervious area should achieve at least 75 percent vegetative cover one year after installation.
- To help prevent soil erosion and sediment loss, landscaping should be installed immediately after the soil restoration process is complete. Temporary irrigation may be needed to quickly establish vegetative cover on a restored pervious area.

### 4.23.5 Construction Considerations

To help ensure that the soil restoration process is successfully completed on a development site, site planning and design teams should consider the following recommendations:

- To help minimize compaction, heavy vehicular and foot traffic should be kept out of all restored pervious areas during and after construction. This can typically be accomplished by clearly delineating soil restoration areas on all development plans and, if necessary, protecting them with temporary construction fencing.
- Simple erosion and sediment control measures, such as temporary seeding and erosion control mats, should be used on restored pervious areas that exceed 2,500 square feet in size. If the restored pervious areas will "receive" any stormwater runoff from other portions of the development site, measures should be taken (e.g., silt fence, temporary diversion berm) to prevent it from compromising the soil restoration effort.
- Test pits or a rod penetrometer can be used to verify that soil amendments have reached a depth of 18 inches.
- Construction contracts should contain a replacement warranty that covers at least three growing seasons to help ensure adequate growth and survival of the vegetation planted on a restored pervious area.

### 4.23.6 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

**Additional Resources**

Stenn, H. 2007. *Building Soil: Guidelines and Resources for Implementing Soil Quality and Depth BMP T5.13 in Washington Department of Ecology (WDOE) Stormwater Management Manual for Western Washington*. Public Works Department. Snohomish County, WA. Available Online: http://www.soilsforsalmon.org/resources.htm.

Washington Department of Ecology (WDOE). 2005. "BMP T5.13: Post-Construction Soil Quality and Depth." *Stormwater Management Manual for Western Washington*. Volume 5: Runoff Treatment BMPs. Washington Department of Ecology. Water Quality Program. Available Online: http://www.ecy.wa.gov/programs/wq/stormwater/manual.html.

Pennsylvania Department of Environmental Protection (PA DEP). 2006. "BMP 6.7.3: Soil Amendment and Restoration." *Pennsylvania Stormwater Best Management Practices Manual*. Pennsylvania Department of Environmental Protection. Bureau of Watershed Management. Available Online: http://www.depweb.state.pa.us/watershedmgmt/site/default.asp.

# 4.24 Stormwater Planters/Tree Boxes

(Source: Center for Watershed Protection)

**Description:** Stormwater planters are landscape planter boxes that are specially designed to receive post-construction stormwater runoff. They consist of planter boxes that are equipped with waterproof liners, filled with an engineered soil mix, and planted with trees, shrubs, and other herbaceous vegetation. Stormwater planters are designed to capture and temporarily store stormwater runoff in the engineered soil mix, where runoff is subject to the hydrologic processes of evaporation and transpiration before being conveyed back into the storm drain system through an underdrain.

**LID/GI Considerations:** Use of a stormwater planter provides measurable reductions in post-construction stormwater runoff rates, volumes, and pollutant loads on development sites. Stormwater planters and/or tree boxes can take the place of traditional landscaped areas, thus minimizing development space required to treat stormwater runoff.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Stormwater planters should be designed to completely drain within 24 hours of the end of a rainfall event.
- A maximum ponding depth of 6 inches is recommended within stormwater planters to help prevent nuisance ponding conditions.
- Unless a shallow water table is found on the development site, stormwater planter planting beds should be at least 2 feet deep.

### ADVANTAGES / BENEFITS

- Helps restore pre-development hydrology on development sites and reduces post-construction stormwater runoff rates, volumes, and pollutant loads
- Can be integrated into development plans as attractive landscaping features
- Particularly well-suited for urban development sites

### DISADVANTAGES / LIMITATIONS

- Can only be used to "receive" runoff from small drainage areas of 2,500 square feet or less

### ROUTINE MAINTENANCE REQUIREMENTS

- Water to promote plant growth and survival.
- Inspect stormwater planters following rainfall events. Plant replacement vegetation in any eroded areas.
- Prune and weed stormwater planter.
- Remove accumulated trash and debris.

### POLLUTANT REMOVAL

80% **Total Suspended Solids**

60/60% **Nutrients** - Total Phosphorus / Total Nitrogen removal

N/A **Metals** - Cadmium, Copper, Lead, and Zinc removal

80% **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

✓ Runoff Reduction

✓ Water Quality

Channel Protection

Overbank Flood Protection

Extreme Flood Protection

✓ suitable for this practice

★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

L Land Requirement

H Capital Cost

M Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *In urban areas, check clear and safety zones for guidance*

Soils: *The soils used within stormwater planter planting beds should be an engineered soil mix.*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 50% of the runoff reduction volume provided

BACK TO TOC

## 4.24.1 General Description

Stormwater planters are small, underdrained bio-retention areas (Section 4.2) that are designed to fit within landscape planter boxes (**Figure 4.24-1**); they consist of landscape planter boxes equipped with waterproof liners, filled with an engineered soil mix and planted with trees, shrubs, and other herbaceous vegetation. Stormwater planters are designed to capture and temporarily store stormwater runoff in the engineered soil mix, where runoff is subject to the hydrologic processes of evaporation and transpiration before being conveyed back into the storm drain system through an underdrain. This allows stormwater planters to provide measurable reductions in post-construction stormwater runoff rates, volumes, and pollutant loads.

A primary concern associated with the design of a stormwater planter (**Figure 4.24-2**) is its storage capacity, which directly influences its ability to reduce stormwater runoff rates, volumes, and pollutant loads. Site planning and design teams should strive to design stormwater planters that can accommodate the stormwater runoff volume generated by the target runoff reduction rainfall event (e.g., 85th percentile rainfall event). If this cannot be accomplished, due to site characteristics or constraints, site planning and design teams should consider using stormwater planters in combination with other runoff reducing low impact development practices, such as dry wells (Section 4.7) and rainwater harvesting (Section 4.19), to supplement the stormwater management benefits provided by the planters.

**Figure 4.24-1:** Various Stormwater Planters

(Sources: Center for Watershed Protection, City of Portland, OR, 2008)

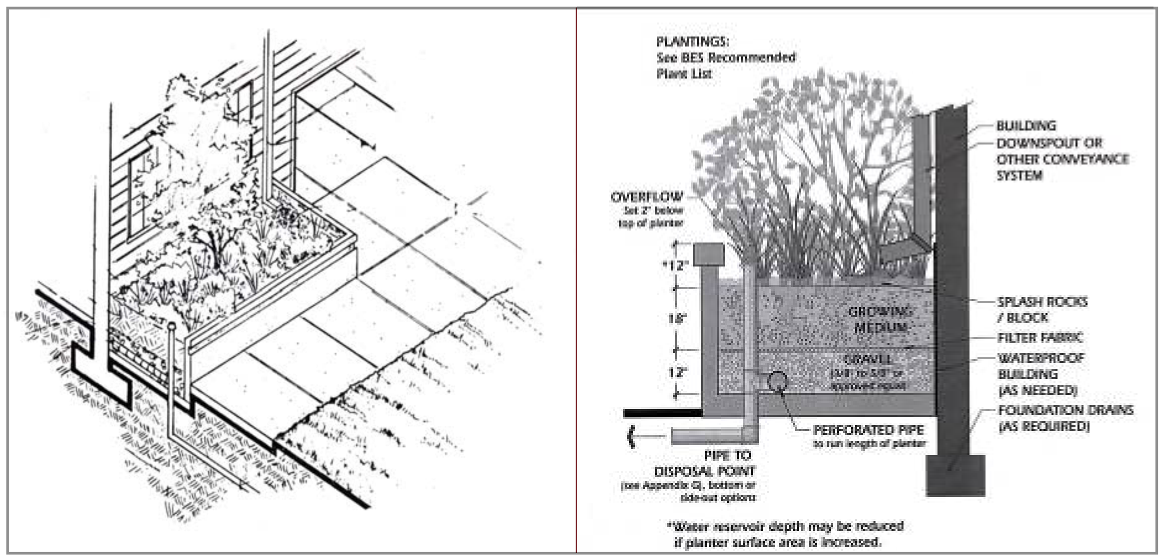



**Figure 4.24-2:** Stormwater Planters

(Source: City of Portland, OR, 2004)

### 4.24.2 Stormwater Management Suitability

The Center for Watershed Protection (Hirschman et al., 2008) recently documented the ability of stormwater planters to reduce annual stormwater runoff volumes and pollutant loads on development sites.

- Stormwater Runoff Reduction
  Subtract 50% of the storage volume provided by a stormwater planter from the runoff reduction volume ($RR_V$) conveyed through the stormwater planter.

- Water Quality Protection
  If installed as per the recommended design criteria and properly maintained, 80% total suspended solids removal will be applied to the water quality volume ($WQ_V$) flowing to the stormwater planter.

- Channel Protection
  Proportionally adjust the post-development runoff curve number (CN) to account for the runoff reduction provided by a stormwater planter when calculating the channel protection volume ($CP_V$) on a development site (see Subsection 3.1.7.5).

- Overbank Flood Protection
  Proportionally adjust the post-development runoff CN to account for the runoff reduction provided by a stormwater planter when calculating the overbank peak discharge ($Q_{p25}$) on a development site (see Subsection 3.1.7.5).

- Extreme Flood Protection
  Proportionally adjust the post-development runoff CN to account for the runoff reduction provided by a stormwater planter when calculating the extreme peak discharge ($Q_f$) on a development site (see Subsection 3.1.7.5).

### 4.24.3 Pollutant Removal Capabilities

Stormwater planters are presumed to remove 80% of the total suspended solids (TSS) load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Stormwater planters also remove 60% of the Phosphorus and Nitrogen, and 80% of fecal coliform. Stormwater planters are not presumed to remove metals such as Cadmium, Copper, Lead, and Zinc in contributing runoff.

### 4.24.4 Application and Site Feasibility Criteria

Stormwater planters are typically used on commercial, institutional, and industrial development sites. Because they can be constructed immediately adjacent to buildings and other structures, they are ideal for use in urban areas. Although they are well-suited to receive rooftop runoff, they can also be used to treat stormwater runoff from other small impervious and pervious drainage areas, such as sidewalks, plazas, and small parking lots (**Figure 4.24-1**). When compared with other low impact development practices, stormwater planters have a moderate construction cost and maintenance burden, while requiring a relatively small amount of surface area.

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **YES**
- Regional Stormwater Control – **NO**

**Physical Feasibility –** Physical Constraints at Project Site

- Drainage Area – The size of the contributing drainage area should be 2,500 square feet or less.

- Flow path – The length of flow path in contributing drainage areas should be 150 feet or less in pervious drainage areas and 75 feet or less in impervious drainage areas.

- Space Required – Stormwater planter surface area requirements vary according to the size of the contributing drainage area. In general, stormwater planters require about 5% of the size of their contributing drainage areas.

- Site Slope – Although stormwater planters may be used on development sites with slopes of up to 6%, they should be designed with slopes that are as close to flat as possible to help ensure that stormwater runoff is evenly distributed over the planting bed.

- Minimum Depth to Water Table – 2 feet

- Minimum Head – 2 feet

- **Soils** – Stormwater planters should be designed to completely drain within 24 hours of the end of a rainfall event.

#### Other Constraints/Considerations

- **Hot spots** – Because they are constructed with waterproof liners and under/overdrains, stormwater planters may be used to treat hot spot runoff.

- **Damage to existing structures and facilities** – Stormwater planters should not be used in areas where their operation may create a risk for basement flooding, interfere with subsurface sewage disposal systems, or negatively affect other underground structures. Stormwater planters should be designed so that overflow drains away from buildings to prevent damage to building foundations.

- **Proximity** – Stormwater planters may be used without restriction in areas that are:
  - » 10 feet from property lines
  - » 100 feet from private water supply wells
  - » 1,200 feet from public water supply wells
  - » 100 feet from septic systems
  - » 100 feet from surface waters
  - » 400 feet from public water supply surface waters

- **Trout Stream** – Use of a stormwater planter reduces a site's runoff pollutant load, as well as the volume and velocity of stormwater runoff, without significantly warming the water. Therefore, stormwater planters are an effective BMP for use where trout streams or other protected waters may receive stormwater runoff.

#### Coastal Areas

- **Poorly Draining Soils** – Since stormwater planters are equipped with waterproof liners and underdrains, the presence of poorly draining soils does not influence their use.

- **Well-Draining Soils** – Since stormwater planters are equipped with waterproof liners and underdrains, the presence of well-draining soils does not influence their use.

- **Flat Terrain** – It may be difficult to provide adequate drainage on flat terrain, which may cause stormwater runoff to pond in the stormwater planter for extended periods of time. Ensure that the underdrain will allow the stormwater planter to drain completely within 24 hours of the end of a rainfall event to prevent nuisance ponding conditions.

- **Shallow Water Table** – It may be difficult to provide 2 feet of clearance between the bottom of the stormwater planter and the top of the water table, which may cause stormwater runoff to pond in the stormwater planter. Designers can address this issue by reducing the depth of the planting bed to 18 inches or reducing the distance between the bottom of the stormwater planter and top of the water table to 12 inches and providing an adequately sized underdrain.

- **Tidally-influenced drainage system** – Tides may occasionally prevent stormwater runoff from being conveyed through a stormwater planter, particularly during high tide.

### 4.24.5 Planning and Design Criteria

Before designing a stormwater planter, the following data is necessary:

- Existing and proposed site, topographic, and location maps, as well as field reviews
- Proposed site design, including buildings, parking lots, sidewalks, stairs, handicapped ramps, and landscaped areas
- Architectural roof plan for rooftop pitches and downspout locations
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site
- Information about downstream BMPs and receiving waters
- Design data from nearby storm sewer structures
- Water surface elevation of nearby water systems and depth to seasonally high groundwater table

*The following criteria are to be considered **minimum** standards for the design of a stormwater planter. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*



BACK TO TOC

#### 4.24.5.1 LOCATION AND LAYOUT

- Stormwater planters should be used to receive stormwater runoff from small drainage areas of 2,500 square feet or less. The stormwater runoff rates and volumes from larger contributing drainage areas typically become too large to be properly treated by stormwater planters.
- The length of the flow path within the contributing drainage area should be 150 feet or less for pervious drainage areas and 75 feet or less for impervious drainage areas. In contributing drainage areas with longer flow paths bioretention areas (Section 4.2) should be used to receive post-construction stormwater runoff.
- Stormwater planters should be designed to provide enough storage for the stormwater runoff volume generated by the target runoff reduction rainfall event (e.g., 85th percentile rainfall event).
- A minimum width, measured from inside wall to inside wall, of 18 inches is recommended for all stormwater planters.
- Although stormwater planters may be used on development sites with slopes up to 6%, they should be designed with slopes that are as close to flat as possible to help ensure that stormwater runoff is evenly distributed over the planting bed. Multiple planters can be constructed at varying elevations.
- Unless a shallow water table is found on the development site, the distance from the bottom of a stormwater planter to the top of the water table should be at least 2 feet. If a shallow water table is found on the development site, the distance from the bottom of a stormwater planter to the top of the water table may be reduced to 12 inches.

#### 4.24.5.2 GENERAL DESIGN

- Stormwater planters should be designed to completely drain within 24 hours of the end of a rainfall event. Where site characteristics allow, it is preferable to design stormwater planters to drain within 12 hours of the end of a rainfall event to help prevent nuisance ponding conditions.
- Stormwater planters may be designed with a maximum ponding depth of 12 inches, although a ponding depth of 6 inches is recommended to help prevent nuisance ponding conditions.
- A minimum of 2 inches of freeboard should be provided between the elevation of the maximum ponding depth and the top of the planter box.
- Unless a shallow water table is found on the development site, all stormwater planter planting beds should be at least 24 inches deep. If a shallow water table is found on the development site, the depth of the planting bed may be reduced to 18 inches.
- The soils used within stormwater planter beds should be an engineered soil mix that meets the following specifications:
  - » *Texture*: Sandy loam or loamy sand should be used.
  - » Sand Content: Soils should contain 85-88% clean, washed sand.
  - » Topsoil Content: Soils should contain 8-12% topsoil.
  - » Organic Matter Content: Soils should contain 3-5% organic matter.
  - » Infiltration Rate: Soils should have an infiltration rate of at least 0.25 inches per hour (in/hr), although an infiltration rate of between 1-2 in/hr is preferred.
  - » Phosphorus Index (P-Index): Soils should have a P-Index of less than 30.
  - » Exchange Capacity (CEC): Soils should have a CEC that exceeds 10 milliequivalents (meq) per 100 grams of dry weight.
  - » pH: Soils should have a pH of 6-8.
- The organic matter used within a stormwater planter bed should be a well-aged compost that meets the specifications outlined in Appendix D.
- All stormwater planters should be equipped with a waterproof liner to prevent damage to building foundations and other adjacent impervious surfaces. Waterproof liners should be 30 mil (0.030 inch) polyvinylchloride (PVC) or equivalent.
- Stormwater planters should be constructed of stone, concrete, brick, or another durable material. Chemically treated wood that can leach toxic chemicals and contaminate stormwater runoff should not be used to construct a stormwater planter.



BACK TO TOC

- Stormwater planters should be equipped with an underdrain consisting of a 4-inch perforated PVC (AASHTO M 252) pipe bedded in a 6-inch layer of clean, washed stone. The pipe should have 3/8-inch perforations, spaced 6 inches on center, and a minimum slope of 0.5%. The clean, washed stone should be ASTM D448 size No. 57 stone (i.e., 1-1/2 to 1/2 inches in diameter) and should be separated from the planting bed by a layer of permeable filter fabric or a thin, 2-4-inch layer of choker stone (i.e., ASTM D 448 size No. 8, 3/8" to 1/8" or ASTM D 448 size No. 89, 3/8" to 1/16").
- If permeable filter fabric is used, the filter fabric should be a non-woven geotextile with a permeability that is greater than or equal to the hydraulic conductivity of the overlying planting bed.
- Consideration should be given to the stormwater runoff rates and volumes generated by larger storm events (e.g., 25-year, 24-hour storm event) to help ensure that these larger storm events are able to safely bypass stormwater planters. An overflow system, such as an overdrain with an invert set slightly above the elevation of maximum ponding depth, should be designed to convey the stormwater runoff generated by these larger storm events safely out of the stormwater planter.

#### 4.24.5.3 PRETREATMENT/INLETS

- If used to receive non-rooftop runoff, consider preceding the stormwater planters with a pea gravel diaphragm or equivalent level spreader device (e.g., concrete sills, curb stops, curbs with "sawteeth" cut into them) to intercept stormwater runoff and distribute it evenly, as overland sheet flow, across the stormwater planter.

#### 4.24.5.4 OUTLET STRUCTURES

- An overflow system, such as an overdrain with an invert set slightly above the elevation of maximum ponding depth, should be designed to convey the stormwater runoff generated by larger storm events safely out of the stormwater planter.
- If designed to receive rooftop runoff, stormwater planters should be constructed with bypass structures and/or piping to ensure that stormwater runoff from larger storm events can be safely passed without damaging the practice or nearby property.

#### 4.24.7.5 SAFETY FEATURES

Except for stable outlet and bypass structures, stormwater planters generally do not require any special safety features. Fencing of stormwater planters is not generally desirable.

#### 4.24.5.6 LANDSCAPING

- A landscaping plan should be prepared for all stormwater planters. The landscaping plan should be reviewed and approved by the local development review authority prior to construction.
- Vegetation commonly planted in stormwater planters includes native trees, shrubs, and other herbaceous vegetation. When developing a landscaping plan, site planning and design teams should choose vegetation that will be able to stabilize soils and tolerate the stormwater runoff rates and volumes that will pass through the stormwater planter. Vegetation used in stormwater planters should also be able to tolerate both wet and dry conditions. See Appendix D for a list of grasses and other plants that are appropriate for use in stormwater planters in the State of Georgia.
- A mulch layer, consisting of 2-4 inches of fine shredded hardwood mulch or hardwood chips, should be included on the surface of the stormwater planter.
- Methods used to establish vegetative cover within a stormwater planter should achieve at least 75% vegetative cover one year after installation.
- To help prevent soil erosion and sediment loss, landscaping should be provided immediately after the stormwater planter has been installed. Temporary irrigation may be needed to quickly establish vegetative cover within a stormwater planter.

#### 4.24.5.7 CONSTRUCTION CONSIDERATIONS

To help ensure that stormwater planters are successfully installed, site planning and design teams should consider the following recommendations:

- If stormwater planters will be used to receive non-rooftop runoff, they should only be installed after their contributing drainage areas have been completely stabilized. To help prevent stormwater planter failure, stormwater runoff may be diverted around the stormwater planter until the contributing drainage area has been stabilized.
- To help prevent soil compaction, heavy vehicular and foot traffic should be kept out of stormwater planters before, during, and immediately after construction. This can typically be accomplished by clearly delineating stormwater planters on all development plans and, if necessary, protecting them with temporary construction fencing.
- Excavation for stormwater planters should be limited to the width and depth specified in the development plans. Excavated material should be placed away from the excavation so as not to jeopardize the stability of the side walls.
- Native soils along the bottom of the stormwater planter should be scarified or tilled to a depth of 3-4 inches prior to the placement of the choker stone and stormwater planter stone.
- The sides of all excavations should be trimmed of large roots that will hamper the installation of the permeable filter fabric used to line the sides and top of the stormwater planter.

#### 4.24.5.8 CONSTRUCTION AND MAINTENANCE COSTS

- The initial cost of a stormwater planter averages around $8 per square foot; however, the overall cost will vary depending on the type and size of vegetation and planters used.
- Maintenance costs average around $400-$500 per year for a 500-square-foot planter. These also vary depending on size and plant choice.

### 4.24.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of a stormwater planter or tree box.**
Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a stormwater planter or tree box. Create a rough layout of the stormwater planters or tree box dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the stormwater planters or tree box.**
Consider whether the stormwater planters or tree box is intended to:

- Meet a runoff reduction* target or water quality (treatment) target. For information on the sizing of a BMP utilizing the runoff reduction approach, see Step 3A. For information on the sizing of the BMP utilizing the water quality treatment approach, see Step 4A. **Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*
- Provide a possible solution to a drainage problem
- Enhance landscape and provide aesthetic qualities

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3A, 3B, and 3C for a runoff reduction approach, or skip Step 3 and complete Steps 4A and 4B for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3A) **Calculate the Stormwater Runoff Reduction Target Volume**
Calculate the Runoff Reduction Volume using the following formula:

$$RR_V = (P)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$RR_V$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, look up the appropriate runoff reduction percentage (or credit) provided by the practice:

Using the $RR_v$ calculated above, determine the minimum Volume of the Practice (VP)

$$(VP_{MIN}) \geq RR_v \text{ (target) / (RR\%)}$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice
**$VP_{MIN}$** = Minimum storage volume required to provide Runoff Reduction Target Volume ($ft^3$)
**$RR_v$ (target)** = Runoff Reduction Target Volume ($ft^3$)

(Step 3B) Determine the storage volume of the practice and the Pretreatment Volume

To determine the actual volume provided in the stormwater planter or tree box, use the following equation:

$$VP = (PV + VES\ (N))$$

*Where*:

**VP** = Volume provided (temporary storage)
**PV** = Ponding Volume
**VES** = Volume of Engineered Soils
**N** = Porosity

To determine the porosity, a qualified licensed professional should be consulted to determine the proper porosity based on the engineered soils used. Most soil media has a porosity of 0.25 and gravel a value of 0.40.

Provide pretreatment by using a grass filter strip or pea gravel diaphragm, 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 may also be desired to reduce flow velocities or 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. Refer to Section 4.9 for design criteria for a grass channel and Section 4.29 for vegetated filter strips.

(Step 3C) Determine whether the minimum storage volume was met

When the VP ≥ $VP_{MIN}$, then the Runoff Reduction requirements are met for this practice. Proceed to Step 5.

When the VP < $VP_{MIN}$, then the BMP must be sized according to the $WQ_v$ treatment method (See Step 4).

(Step 4A) Calculate the Target Water Quality Volume

Calculate the Water Quality Volume using the following formula:

$$WQ_v = (1.2)\ (R_v)\ (A) / 12$$

*Where*:

**$WQ_v$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_v$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)
**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4B) If using the practice for Water Quality treatment, determine the footprint of the stormwater planter practice

The peak rate of discharge for the water quality design storm is needed for sizing of off-line diversion structures (see Subsection 3.1.7). If designing off-line, follow steps (a) through (d) below:

(a) Using $WQ_v$, compute CN
(b) Compute time of concentration using TR-55 method
(c) Determine appropriate unit peak discharge from time of concentration
(d) Compute $Q_{wq}$ from unit peak discharge, drainage area, and $WQ_v$.

To determine the minimum surface area of the stormwater planter, use the following formula:

$$A_f = (WQ_v)\ (d_f)\ /\ [\ (k)\ (h_f + d_f)(t_f)]$$

*Where*:

**$A_f$** = surface area of ponding area ($ft^2$)
**$WQ_v$** = water quality volume ($ft^3$)
**$d_f$** = media depth (ft)
**k** = coefficient of permeability of planting media (ft/day) (use 1 ft/day for silt-loam if engineered soils is being used)
**$h_f$** = average height of water above stormwater planter bed (ft)
**$t_f$** = design planting media drain time (days) (1 day is recommended maximum)

(Step 5) Calculate the adjusted curve numbers for $CP_V$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm). See Subsection 3.1.7.5 for more information

(Step 6) Size flow diversion structure, if needed

If the contributing drainage area to stormwater planter exceeds the water quality treatment and/or storage capacity, a flow regulator (or flow splitter diversion structure) should be supplied to divert the $WQ_v$ (or $RR_v$) to the stormwater planter.

(Step 7) Design emergency overflow system

An overflow system, such as an overdrain with an invert set slightly above the elevation of maximum ponding depth, 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, 24-hour storm event.

(Step 8) Prepare site Vegetation and Landscaping Plan

A landscaping plan for the stormwater planter should be prepared to indicate how it will be established with vegetation.

See Subsection 4.24.5.6 (*Landscaping*) and Appendix D for more details.

### 4.24.7 Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.25 Stormwater Ponds

**Description:** Constructed stormwater retention basins that have a permanent pool (or micropool). Runoff from each rain event is detained and treated in the pool primarily through settling and biological uptake mechanisms.

**LID/GI Considerations:** Stormwater ponds are not generally considered low impact development or green infrastructure best management practices (BMPs).

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Minimum contributing drainage area of 25 acres; 10 acres for micropool extended detention pond
- A sediment forebay or equivalent upstream pretreatment facility should be provided.
- Minimum length-to-width ratio for the pond is 1.5:1
- Depth of the permanent pool should not exceed 8 feet.
- Side slopes to the pond should not exceed 3:1 (h:v) without safety precautions or if mowing is anticipated and should terminate on a safety bench

### ADVANTAGES / BENEFITS

- Moderate to high removal rate of urban pollutants
- High community acceptance
- Opportunities for wildlife habitat

### DISADVANTAGES / LIMITATIONS

- Potential for thermal impacts/downstream warming
- Dam height restrictions for high-relief areas
- Pond drainage can be problematic for low-relief terrain
- Cost of dredging and disposal
- Dredging operations may require a stream buffer variance

### MAINTENANCE REQUIREMENTS

- Remove debris from inlet and outlet structures.
- Maintain side slopes and remove invasive vegetation.
- Monitor sediment accumulation and remove it periodically.

### POLLUTANT REMOVAL

80% **Total Suspended Solids**

50% **Metals** - Cadmium, Copper, Lead, and Zinc removal

50/30% **Nutrients** - Total Phosphorus / Total Nitrogen removal

70% **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- Runoff Reduction
- ✓ Water Quality
- ✓ Channel Protection
- ✓ Overbank Flood Protection
- ✓ Extreme Flood Protection

✓ suitable for this practice

★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- H Land Requirement
- L Capital Cost
- L Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Not recommended*
**Roadway Projects:** *Yes*

Soils: *Underlying soils of hydrologic group "C" or "D" should be adequate to maintain a permanent pool. Hydrologic group "A" soils generally require a pond liner; group "B" soils may require infiltration testing.*

L=Low **M**=Moderate **H**=High

### RUNOFF REDUCTION CREDIT

- 0% of the runoff reduction volume provided by this practice.



BACK TO TOC

## 4.25.1 General Description

Stormwater ponds (also referred to as retention ponds, wet ponds, or wet extended detention ponds) are constructed stormwater retention basins that have a permanent (dead storage) pool of water throughout the year. They can be created by excavating an existing natural depression or through the construction of embankments.

In a stormwater pond, runoff from each rain event is detained and treated in the pool through gravitational settling and biological uptake until it is displaced by runoff from the next storm. The permanent pool also serves to protect deposited sediments from resuspension. Above the permanent pool level, additional temporary storage (live storage) is provided for runoff quantity control. The upper stages of a stormwater pond are designed to provide extended detention of the 1-year, 24-hour rainfall event for downstream channel protection, as well as normal detention of larger storm events (25-year, 24-hour and, optionally, the 100-year, 24-hour storm event).

Stormwater ponds are among the most cost-effective and widely used stormwater practices. A well-designed and landscaped pond can be an aesthetic feature on a development site when planned, located, implemented, and maintained properly. However, ponds only work as designed until the deign sediment volume is reached. At that point, the sediment must be removed to maintain treatment performance and aquatic resource, channel and extreme flood protection.

There are several variants of stormwater pond design, the most common of which include the wet pond, the wet extended detention pond, and the micropool extended detention pond. In addition, multiple stormwater ponds can be placed in series or parallel to increase performance or meet site design constraints. Below are descriptions of each design variant:

- **Wet Pond** – Wet ponds are stormwater basins constructed with a permanent (dead storage) pool of water equal to the water quality volume. Stormwater runoff displaces the water already present in the pool. Temporary storage (live storage) can be provided above the permanent pool elevation for larger flows.

- **Wet Extended Detention (ED) Pond** – A wet extended detention pond is a wet pond where the water quality volume is split evenly between the permanent pool and extended detention (ED) storage provided above the permanent pool. During storm events, water is detained above the permanent pool and released over 24 hours. This design has similar pollutant removal to a traditional wet pond, but consumes less space.

- **Micropool Extended Detention (ED) Pond** – The micropool extended detention pond is a variation of the wet ED pond where only a small “micropool” is maintained at the outlet to the pond. The outlet structure is sized to detain the water quality volume for 24 hours. The micropool prevents resuspension of previously settled sediments and also prevents clogging of the low flow orifice.

- **Multiple Pond Systems** – Multiple pond systems consist of constructed facilities that provide water quality and quantity volume storage in two or more cells. The additional cells can create longer pollutant removal pathways and improved downstream protection.

**Figure 4.25-1** on the next page shows several examples of stormwater pond variants.

**Wet Pool**

**Wet ED Pond**

**Micropool ED Pond**

**Wet ED Pond**

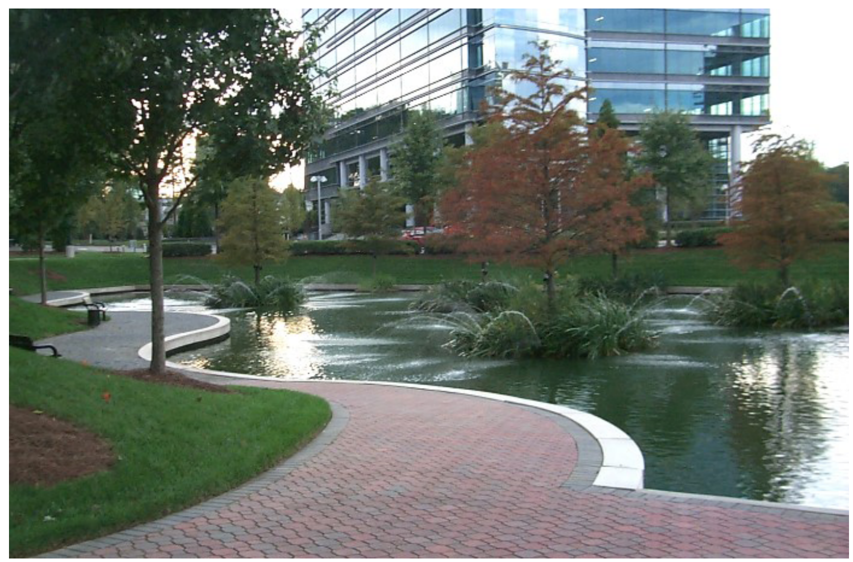


**Figure 4.25-1** Stormwater Pond Examples

**Figures 4.25-2** through **4.25-5** provide plan view and profile view schematics for the design of a wet pond, wet extended detention pond, micropool extended detention pond, and multiple pond system.

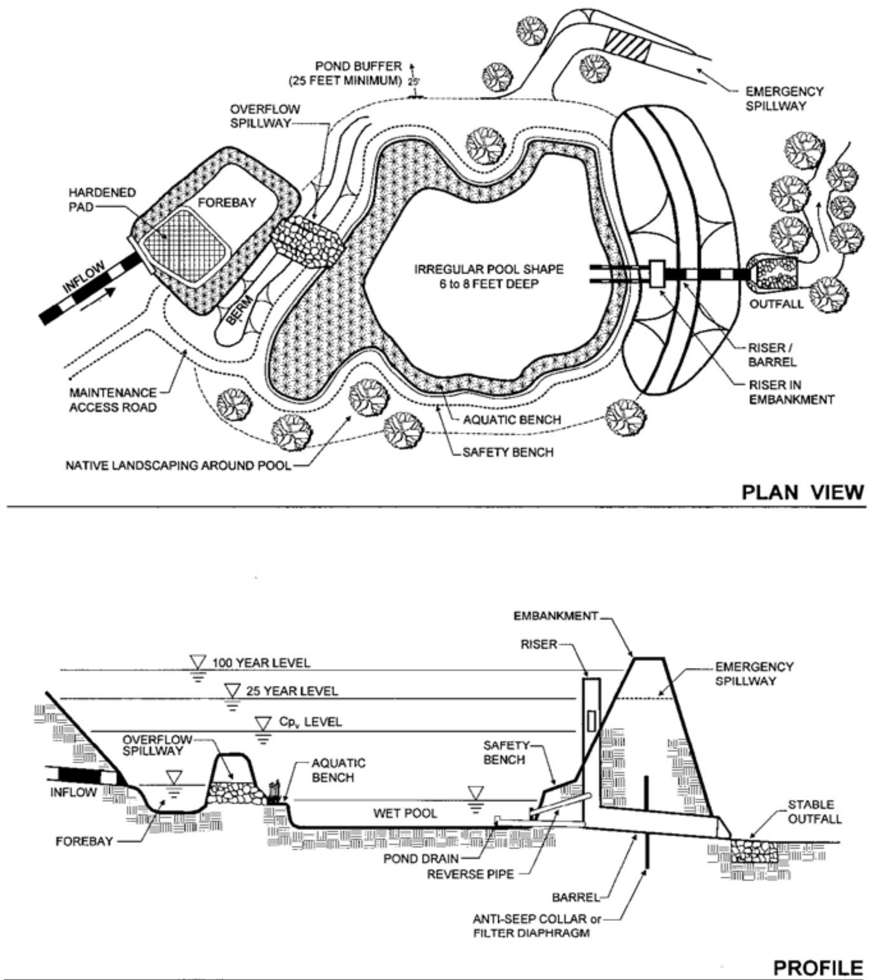



**Figure 4.25-2** Schematic of Wet Pond

(Source: Center for Watershed Protection)

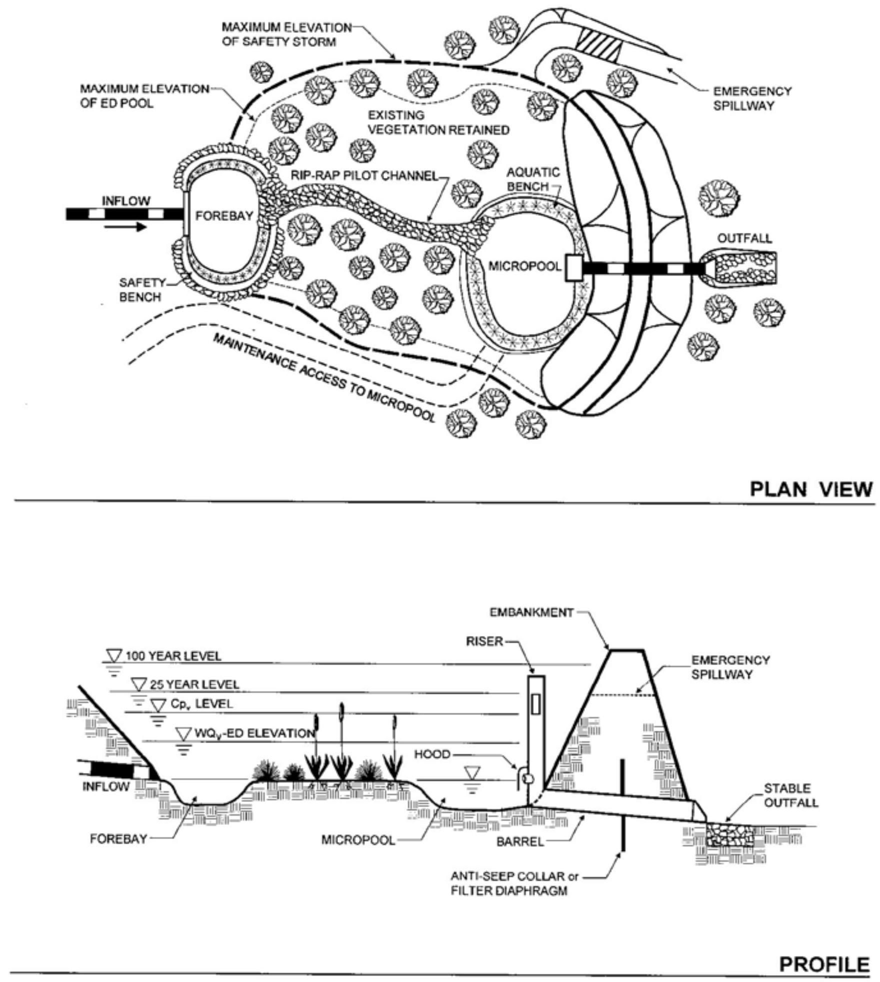



**Figure 4.25-3** Schematic of Wet Extended Detention Pond

(Source: Center for Watershed Protection)

BACK TO TOC

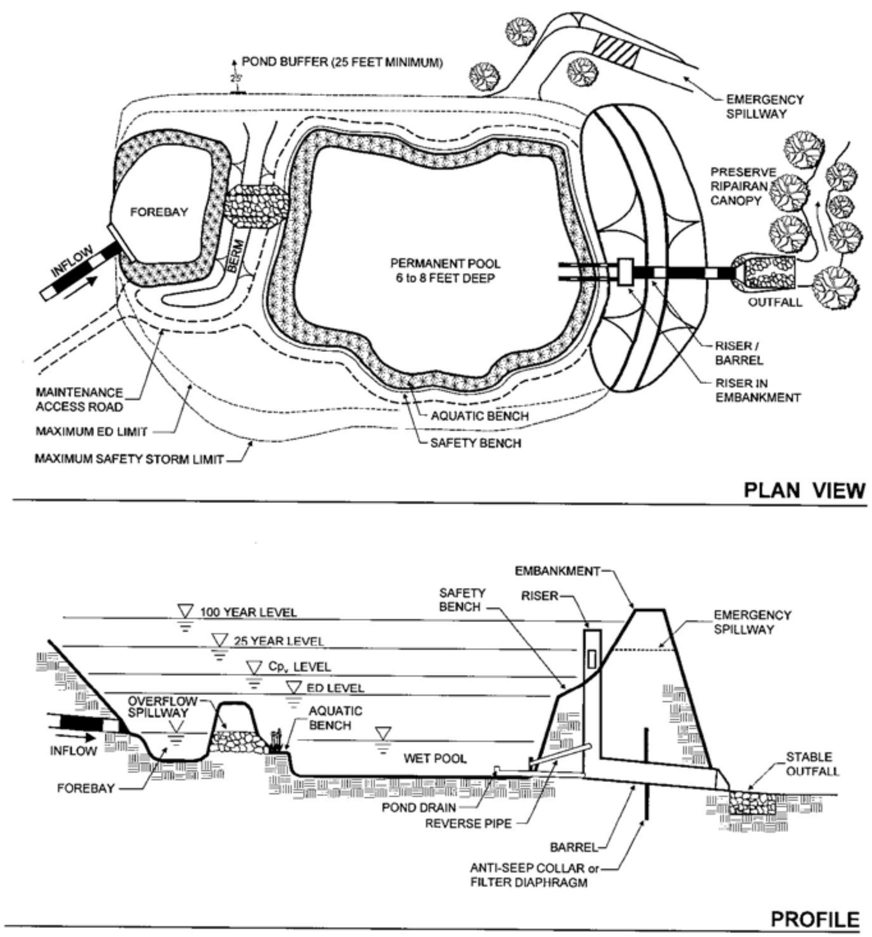



**Figure 4.25-4** Schematic of Micropool Extended Detention Pond

(Source: Center for Watershed Protection)

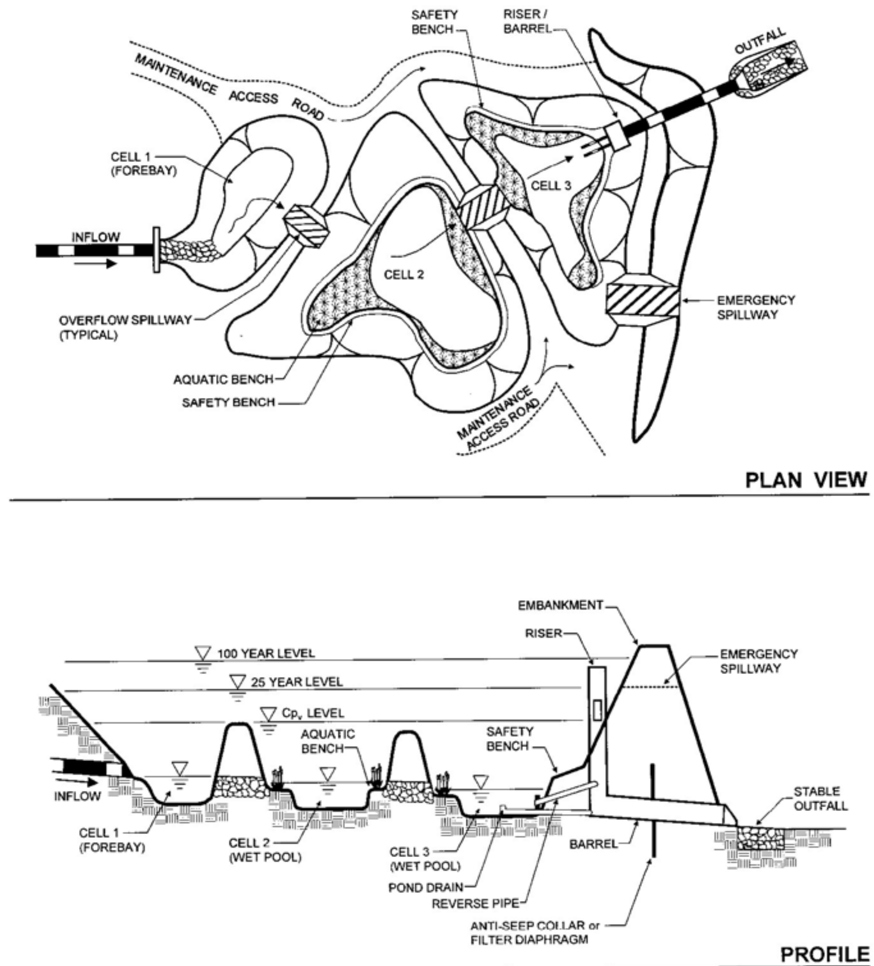



**Figure 4.25-5** Schematic of Multiple Pond System

(Source: Center for Watershed Protection)

BACK TO TOC

### 4.25.2 Stormwater Management Suitability

Stormwater ponds are designed to control both stormwater quantity and quality. Thus, a stormwater pond can be used to address most of the *unified stormwater sizing criteria* for a given drainage area.

- Runoff Reduction
  Stormwater 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
  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. Another mechanism for pollutant removal is 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.

- Channel Protection
  A portion of the storage volume above the permanent pool in a stormwater 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.

### 4.25.3 Pollutant Removal Capabilities

All of the stormwater pond design variants are presumed to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Undersized or poorly designed ponds can reduce TSS removal performance, as can excess sediment.

The following pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- *Total Suspended Solids – 80%*
- *Total Phosphorus – 50%*
- *Total Nitrogen – 30%*
- *Fecal Coliform – 70% (if no resident waterfowl population present)*
- *Heavy Metals – 50%*

For additional information and data on pollutant removal capabilities for stormwater ponds, see the National Pollutant Removal Performance Database (Version 3) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.25.4 Application and Site Feasibility Criteria

Stormwater ponds are generally applicable to most types of new development and redevelopment, and can be used in both residential and nonresidential areas. Ponds can also be used in retrofit situations.

*The following criteria should be evaluated to ensure the suitability of a stormwater pond for meeting stormwater management objectives on a site or development.*

**General Feasibility**

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – NO
- Regional Stormwater Control – YES

BACK TO TOC

**Physical Feasibility –** Physical Constraints at Project Site

- **Drainage Area** – A minimum of 25 acres is needed for a wet pond or wet ED pond to maintain a permanent pool, or 10 acres minimum for a micropool ED pond. A smaller drainage area may be acceptable with an adequate water balance and anti-clogging device.
- **Space Required** – Approximately 2-3% of the tributary drainage area
- **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 on-site from the inflow to the outflow
- **Minimum Depth to Water Table** – If used on a site with an underlying water supply aquifer, or when treating a hot spot, a separation distance of 2 feet is required between the bottom of the pond and the elevation of the seasonally high water table.
- **Soils** – Underlying soils of hydrologic group C or D should be adequate to maintain a permanent pool. Most group A soils and some group B soils will require a pond liner. *Evaluation of soils should be based upon an actual subsurface analysis and permeability tests.*

**Other Constraints/Considerations**

- **Hot spots** – Stormwater ponds can accept runoff from stormwater hot spots. Reduce potential groundwater contamination by preventing infiltration of hot spot runoff. Pretreatment should be provided for hot spot runoff, as well as a 2-4-foot separation distance from the water table.
- **Damage to existing structures and facilities** – SStormwater ponds should be designed to safely store and/or bypass the overbank flood ($Q_{p25}$) and extreme flood ($Q_f$) storms to prevent overflow or failure, which may cause damage to site structures and facilities.
- Minimum setback requirements for stormwater pond facilities (when not specified by local ordinance or criteria):
  - *Property line* – 10 feet
  - *Private well* – 100 feet; if a well is down-gradient from a hot spot land use then the minimum setback is 250 feet
  - *Septic system tank/leach field* – 50 feet
- **Trout Streams** – Consideration should be given to the thermal influence of stormwater pond outflows on downstream trout waters. A micropool ED pond is the best pond alternative, but wet ponds and wet ED ponds can be designed off-line and under shade to minimize their thermal impact. Limit $WQ_{v-ED}$ to 12 hours.
- **Low-relief** – Maximum normal pool depth is limited, so providing a pond drain can be problematic.
- **High-relief** – Embankment heights are restricted.
- **Karst** – Requires clay or polyliner to sustain a permanent pool of water and protect aquifers. Ponding depth is limited. Geotechnical tests may be required.
- **Swimming Area / Shellfish** – Design for geese prevention. Provide 48-hour ED for maximum coliform die-off.
- **Aquifer Protection** – Reduce potential groundwater contamination by preventing infiltration of hot spot runoff. A liner may be required for type A and B soils. Pretreat hot spots and provide 2-4-foot separation distance from the groundwater table.

**Coastal Areas**

- **Poorly Draining Soils** – Poorly draining soils do not inhibit a stormwater pond's ability to temporarily store and treat stormwater runoff.
- **Flat Terrain** – Consider stormwater wetlands as an alternative stormwater management (Section 4.26) practice in areas with flat terrains and a shallow water table.

- **Shallow Water Table** – If used on a site with an underlying water supply aquifer or when treating a hot spot, a separation distance of 2 feet is required between the bottom of the pond and the elevation of the seasonally high water table. Otherwise, the elevation of the stormwater pond bottom can be below the seasonally high water table, as this will help to maintain a permanent pool within the BMP.

## 4.25.5 Planning and Design Criteria

Before designing the stormwater pond, the following data is necessary:

- Existing and proposed site, topographic, and location maps, as well as field reviews
- Impervious and pervious areas and other means to determine the land use data as needed
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site
- Design data for nearby storm sewer structures
- Water surface elevation of nearby water systems and the depth to the seasonally high groundwater table
- Desired sediment storage volume and cleanout frequency

*The following criteria are to be considered minimum standards for the design of a stormwater pond facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.25.5.1 LOCATION AND LAYOUT

- Stormwater ponds should have a minimum contributing drainage area of 25 acres for a wet pond or wet ED pond to maintain a permanent pool. For a micropool ED pond, the minimum drainage area is 10 acres. A smaller drainage area can be considered when water availability can be confirmed (such as from a groundwater source or in areas with a high water table). In these cases a water balance may be performed. Ensure that an appropriate anti-clogging device is provided for the pond outlet.
- A stormwater pond should be sited such that the site topography allows for maximum runoff storage at minimum excavation or construction costs. Pond siting should also take into account the location and use of other site features, such as buffers and undisturbed natural areas, and should attempt to aesthetically "fit" the facility into the landscape. Bedrock close to the surface may prevent excavation.
- Stormwater ponds should not be located on steep (>15%) or unstable slopes.
- Stormwater ponds cannot be located within a stream or any other navigable waters of the U.S., including wetlands, without obtaining a Section 404 permit under the Clean Water Act, and any other applicable state permit.
- All utilities should be located outside of the pond/basin site.

### 4.25.5.2 GENERAL DESIGN

- A well-designed stormwater pond consists of the following components.
  1. One or more permanent pools of water
  2. An overlying zone in which runoff control volumes are stored
  3. Shallow littoral zone (aquatic bench) along the edge of the permanent pool that acts as a biological filter
- In addition, all stormwater pond designs should include a sediment forebay at their inflow to allow heavier sediments to drop out of suspension before runoff enters the permanent pool.
- Additional pond design features include an emergency spillway, maintenance access, safety bench, pond buffer, and appropriate native landscaping.

#### 4.25.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

In general, pond designs are unique for each site and application. However, there are a number of geometric ratios and limiting depths for ponds that must be observed for adequate pollutant removal, ease of maintenance, and improved safety.

- Permanent pool volume is typically sized as follows:
  - » **Standard wet ponds:** 100% of the water quality treatment volume (1.0 $WQ_v$)
  - » **Wet ED ponds:** 50% of the water quality treatment volume (0.5 $WQ_v$)
  - » **Micropool ED ponds:** Volume should be approximately 0.1 inch per impervious acre draining to the pond.
- Proper geometric design is essential to prevent hydraulic short-circuiting (unequal distribution of inflow), which results in failure of the pond to achieve adequate levels of pollutant removal. The minimum length-to-width ratio for the permanent pool shape is 1.5:1, and should ideally be greater than 3:1 to avoid short-circuiting. In addition, ponds should be wedge-shaped when possible so that flow enters the pond and gradually spreads out, improving the sedimentation process. Baffles, pond shaping, and/or islands can be added within the permanent pool to increase the flow path.
- Maximum depth of the permanent pool generally should not exceed 8 feet to avoid stratification and anoxic conditions. Minimum depth for the pond bottom should be 3-4 feet. Deeper depths near the outlet will yield cooler bottom water discharges that may help to mitigate downstream thermal effects.
- Side slopes to the pond usually should not exceed 3:1 (h:v) without safety precautions or if mowing is anticipated and should terminate on a safety bench (see **Figure 4.25-6**). The safety bench requirement may be waived if slopes are 4:1 or flatter.
- The perimeter of all deep pool areas (4 feet or deeper) should be surrounded by two benches: safety and aquatic. For larger ponds, a safety bench extends approximately 15 feet outward from the normal water edge to the toe of the pond side slope. The maximum slope of the safety bench should be 6%. An aquatic bench extends inward from the normal pool edge (15 feet on average) and has a maximum depth of 18 inches below the normal pool water surface elevation (see **Figure 4.25-6**).
- The contours and shape of the permanent pool should be irregular to provide a more natural landscaping effect.

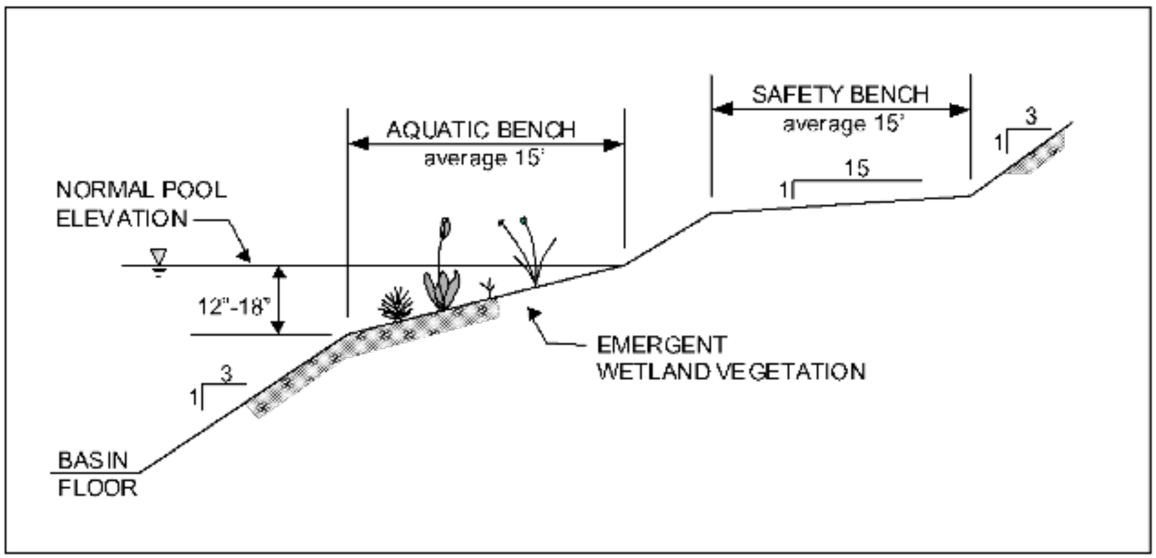



**Figure 4.25-6** Typical Stormwater Pond Geometry Criteria

#### 4.25.5.4 PRETREATMENT/INLETS

- Each pond should have a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the stormwater flow prior to dispersal in a larger permanent pool. The forebay should consist of a separate cell, formed by an acceptable barrier. A forebay is to be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. In some design configurations, the pretreatment volume may be located within the permanent pool.
- The forebay is sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4-6 feet deep. The pretreatment storage volume is part of the total $WQ_v$ requirement and may be subtracted from $WQ_v$ for permanent pool sizing.
- Forebays should contain a fixed vertical sediment depth marker to measure sediment deposition over time. The bottom of the forebay may be hardened (using concrete, paver blocks, etc.) to make sediment removal easier.
- Inflow channels are to be stabilized with flared aprons, or the equivalent. Inlet pipes to the pond can be partially submerged. Exit velocities from the forebay must be non-erosive.

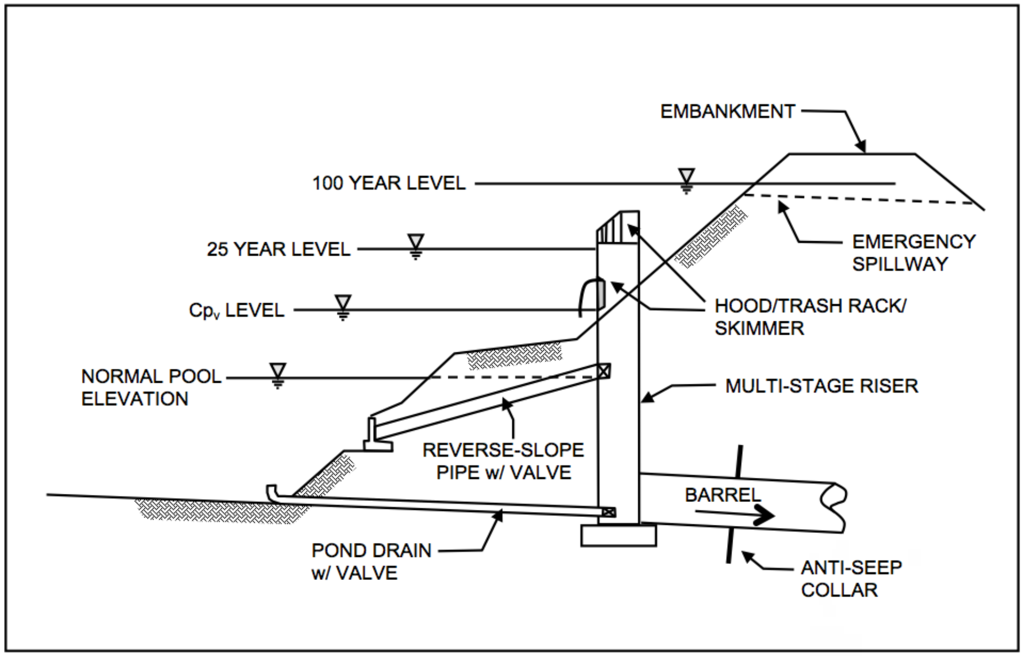



**Figure 4.25-7** Typical Pond Outlet Structure

#### 4.25.5.5 OUTLET STRUCTURES

- Flow control from a stormwater pond is typically accomplished with the use of a concrete or corrugated metal riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the base of the pond with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (see **Figure 4.25-7**). The riser should be located within the embankment for maintenance access, safety, and aesthetics.
- A number of outlets at varying depths in the riser provide internal flow control for routing of the water quality, channel protection, and overbank flood protection runoff volumes. The number of orifices can vary and is usually a function of pond design.
  - » For example, a wet pond riser configuration is typically comprised of a channel protection outlet (usually an orifice) and overbank flood protection outlet (often a slot or weir). The channel protection orifice is sized to release the channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water stream basins). Since the water quality volume is fully contained in the permanent pool, no orifice sizing is necessary for this volume. As runoff from a water quality event enters the wet pond, it simply displaces that same volume through the channel protection orifice. Thus an off-line wet pond providing only water quality treatment can use a simple overflow weir as the outlet structure.



BACK TO TOC

- » Flow control from a stormwater pond is typically accomplished with the use of a concrete or corrugated metal riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the base of the pond with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (see **Figure 4.25-7**). The riser should be located within the embankment for maintenance access, safety, and aesthetics.
- » A number of outlets at varying depths in the riser provide internal flow control for routing of the water quality, channel protection, and overbank flood protection runoff volumes. The number of orifices can vary and is usually a function of pond design.

- The water quality outlet (for a wet ED or micropool ED pond) and channel protection outlet should be fitted with adjustable gate valves or other mechanism(s) that can be used to adjust detention time.
- Higher flows (overbank and extreme flood protection) pass through openings or slots protected by trash racks further up on the riser.
- After entering the riser, flow is conveyed through the barrel and discharged downstream. Anti-seep collars should be installed on the outlet barrel to reduce the potential for pipe failure.
- Riprap, plunge pools or pads, or other energy dissipators should be placed at the outlet of the barrel to prevent scouring and erosion. If a pond daylights to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance.
- Each pond must have a bottom drain pipe with an adjustable valve that can completely or partially drain the pond within 24 hours. This requirement may be waived for coastal areas, where positive drainage is difficult to achieve due to very low relief.
- The pond drain should be sized one pipe size larger than the calculated design diameter. The drain valve is typically a hand-wheel activated knife or gate valve. Valve controls should be located inside of the riser at a point where they will not normally be inundated and can be operated in a safe manner.

See the design procedures in Subsection 4.25.6 as well as Section 3.3 (*Storage Design*) and Section 3.4 (*Outlet Structures*) for additional information and specifications on pond routing and outlet works.

#### 4.25.5.6 EMERGENCY SPILLWAYS

- An emergency spillway should be included in the stormwater pond design to safely pass the extreme flood flow. The spillway prevents pond water levels from overtopping the embankment and causing structural damage. The emergency spillway must be located so that downstream structures will not be impacted by spillway discharges.
- A minimum of 1 foot of freeboard must be provided, measured from the top of the water surface elevation for the extreme flood to the lowest point of the dam embankment, not counting the emergency spillway.

#### 4.25.5.7 MAINTENANCE ACCESS

- A maintenance right-of-way or easement must be provided to a pond from a public or private road. Maintenance access should be at least 12 feet wide, have a maximum slope of no more than 15%, and be appropriately stabilized to support maintenance equipment and vehicles.
- The maintenance access must extend to the forebay, safety bench, riser, and outlet and, to the extent feasible, be designed to allow vehicles to turn around.
- Access to the riser should be provided by lockable manhole covers, and manhole steps within easy reach of valves and other controls.

BACK TO TOC

#### 4.25.5.8 SAFETY FEATURES

- Based on pond embankment height and pond storage volume, it may have to be designed to State of Georgia rules for dam safety. Check the latest rules and corresponding laws.
- Fencing of ponds is not generally desirable, but may be required by the local review authority. A preferred method is to manage the contours of the pond through the inclusion of a safety bench (see above) to eliminate drop-offs and reduce the potential for accidental drowning. In addition, the safety bench may be landscaped to deter access to the pool.
- The principal spillway opening should not permit access by small children. Ponds with endwalls above pipe outfalls greater than 48 inches in diameter should be fenced to prevent access. Warning signs should be posted near the pond to prohibit swimming and fishing in the facility.

#### 4.25.5.9 LANDSCAPING

- Aquatic vegetation can play an important role in pollutant removal in a stormwater pond, including enhancing the appearance of the pond, stabilizing side slopes, serving as wildlife habitat, and temporarily concealing unsightly trash and debris. Therefore, wetland plants should be encouraged in a pond design, along the aquatic bench (fringe wetlands), safety bench, and side slopes (ED ponds), and within shallow areas of the pool itself. The best elevations for establishing wetland plants, either through transplantation or volunteer colonization, are within 6 inches (plus or minus) of the normal pool elevation. Additional information on establishing wetland vegetation and appropriate wetland species for Georgia can be found in Appendix D (*Planting and Soil Guidance*).
- Woody vegetation may not be planted on the embankment or allowed to grow within 15 feet of the toe of the embankment or 25 feet from the principal spillway structure.
- A pond buffer should be provided that extends 25 feet outward from the maximum water surface elevation of the pond. The pond buffer should be contiguous with other buffer areas that are required by existing regulations (e.g., stream buffers) or that are part of the overall stormwater management concept plan. No structures should be located within the buffer, and an additional setback to permanent structures may be provided.
- Existing trees should be preserved in the buffer area during construction. It is desirable to locate forest conservation areas adjacent to ponds. To discourage resident geese populations, the buffer can be planted with trees, shrubs and native ground covers.
- The soils of a pond buffer are often severely compacted during construction to ensure stability. The density of these compacted soils is so great that it effectively prevents root penetration and, therefore, may lead to premature mortality or loss of vigor. Consequently, it is advisable to excavate large, deep holes around the proposed planting sites and backfill these with uncompacted topsoil.
- Fish such as Gambusia can be stocked in a pond to aid in mosquito prevention.
- A fountain or solar-powered aerator may be used for oxygenation of water in the permanent pool.
- Compatible multi-objective use of stormwater ponds is strongly encouraged to leverage additional benefits.

#### 4.25.5.10 CONSTRUCTION CONSIDERATIONS

- Construction equipment should be restricted from the stormwater pond area to prevent compaction of the native soils.
- Contributing drainage areas should be properly stabilized with the appropriate erosion and sediment controls or permanent seeding before allowing stormwater runoff to drain to the stormwater pond.

### 4.25.5.11 CONSTRUCTION AND MAINTENANCE COSTS

A recent study (Brown and Schueler, 1997) estimated that construction costs for stormwater ponds could be calculated with the following equation:

$$C = 24.5V^{0.705}$$

*Where*:

**C** = Construction cost and

**V** = Volume in the pond to include the 10-year storm ($ft^3$).

Using this equation, typical construction costs are:

- \$45,700 for a 1 acre-foot facility
- \$232,000 for a 10 acre-foot facility
- \$1,170,000 for a 100 acre-foot facility

BACK TO TOC

## 4.25.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of a stormwater pond.**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a stormwater pond. Create a rough layout of the stormwater pond dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the stormwater pond.**

Consider whether the stormwater pond is intended to:

- Meet a water quality (treatment) target. See Step 3 to size the BMP utilizing the water quality treatment approach.
- Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)
- Provide a possible solution to a drainage problem

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3 for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3) **Calculate the Target Water Quality Volume**

Calculate the Water Quality Volume using the following formula:

$$WQ_v = (1.2)\ (R_v)\ (A) / 12$$

*Where*:

**$WQ_v$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_v$** = Volumetric runoff coefficient which can be found by:

$$\mathbf{R_v = 0.05+0.009(I)}$$

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4) **Determine the pretreatment volume.**

A sediment forebay should be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4-6 feet deep. The forebay storage volume counts toward the total $WQ_v$ requirement and may be subtracted from the $WQ_v$ for subsequent calculations.

(Step 5) **Determine permanent pool volume (and water quality ED 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 e extended detention volume to remainder of $WQ_v$

(Step 6) **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 $WQ_{v\text{-}ED}$ elevation for wet ED and micropool ED ponds)

(Step 7) **Compute extended detention orifice release rate(s) and size(s), and establish the $CP_v$ elevation.**

*Wet Pond*: The $CP_v$ elevation is determined from the stage-storage relationship and the orifice is then sized to release the channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water stream basins). The channel protection orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool, is a recommended design. The orifice diameter may be reduced to 1 inch if internal orifice protection is used (i.e., an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.

*Wet ED Pond and Micropool ED Pond*: Based on the elevations established in Step 6 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 water quality orifice should have a minimum diameter of 3 inches and be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool, is a recommended design. Adjustable gate valves can also be used to achieve this equivalent diameter. 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 channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams).

(Step 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.

(Step 9) **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 ($Q_f$).

At final design, provide safe passage for the 100-year, 24-hour rainfall event.

(Step 10) **Investigate potential pond 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.

(Step 11) **Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.**

See Subsection 4.25.5.4 to 4.25.5.8 for more details.

(Step 12) **Prepare 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 Subsection 4.25.5.9 (*Landscaping*) and Appendix D for more details.

See Appendix B-1 for a Stormwater Pond Design Example

### 4.25.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

**Additional Resources**

Brown, W., and T. Schueler. 1997. *The Economics of Stormwater BMPs in the Mid-Atlantic Region*. Prepared for the Chesapeake Research Consortium, Edgewater, MD, by the Center for Watershed Protection, Ellicott City, MD.

# 4.26 Stormwater Wetlands

**Description:** Constructed wetland systems used for stormwater management. Runoff volume is both stored and treated in the wetland facility which consists of a shallow impoundment with a permanent pool designed to mimic natural wetlands.

**LID/GI Considerations:** Wetlands should not be designed close to the source of runoff as LID dictates because it is not practical or cost effective. However wetlands employ several LID/GI characteristics, for example mimicking natural systems and providing infiltration and evapotranspiration.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Minimum contributing drainage area of 25 acres; 5 acres for pocket wetland
- Two design variations (level 1 and level 2) to achieve different pollutant removal rates
- Outflow hydrograph should mimic the existing conditions hydrograph, where applicable
- Design should include water balance analysis and landscaping

### ADVANTAGES / BENEFITS

- Good nutrient removal for level 2
- Provides natural wildlife habitat
- Relatively low maintenance costs
- Provides moderate to high removal of many of the pollutants of concern typically contained in post-construction stormwater runoff
- Ideal for use in flat terrain and in areas with high groundwater

### DISADVANTAGES / LIMITATIONS

- Requires large land area
- Needs continuous baseflow for viable wetland
- Sediment regulation is critical to sustain wetlands
- Provides minimal reduction of post-construction stormwater runoff volumes
- More costly than some BMPs
- Difficulties in maintaining the permanent pool may arise

### MAINTENANCE REQUIREMENTS

- Plant replacement vegetation in any eroded areas.
- Remove invasive vegetation
- Monitor sediment accumulation and remove periodically
- A valve will be required to dewater the wetland

### POLLUTANT REMOVAL

Level 1

- 80% **Total Suspended Solids**
- 50% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 40/30% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- 70% **Pathogens** – Fecal Coliform

Level 2

- 85% **Total Suspended Solids**
- 60% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 75/55% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- 85% **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- Runoff Reduction
- ✓ Water Quality
- ✓ Channel Protection
- ✓ Overbank Flood Protection
- ✓ Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- M/H Land Requirement
- M Capital Cost

Maintenance Burden:

- M Shallow Wetland
- M ED Shallow Wetland
- M/H Pocket Wetland
- M Pond/Wetland

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *No*
**Drainage Area:** *Minimum contributing drainage area of 25 acres; 5 acres for pocket wetland*
**Roadway Projects:** *Not applicable*

**Soils:** *Hydrologic group 'A' and 'B' soils may require a liner (not relevant for pocket wetland)*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- Level 1: 0% - Runoff Reduction Credit
- Level 2: 0% - Runoff Reduction Credit
- Accepts Hotspot Runoff: *Yes, 2 feet of separation distance required to water table*



BACK TO TOC

## 4.26.1 General Description

Stormwater wetlands (also referred to as constructed wetlands) are constructed shallow marsh systems that are designed to both treat urban stormwater and control runoff volumes (see Figure 4.26-1 for examples). As stormwater runoff flows through the wetland facility, pollutant removal is achieved through settling and uptake by marsh vegetation.

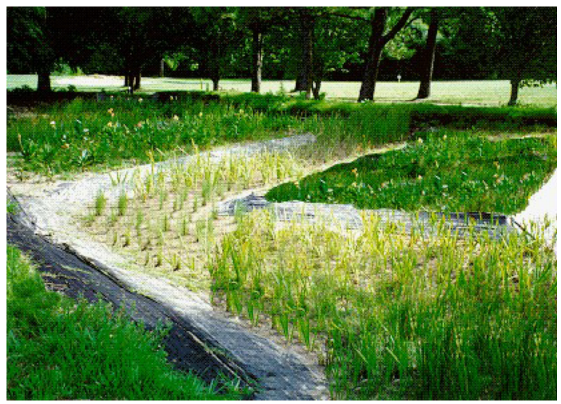
Wetlands are among the most effective stormwater practices in terms of pollutant removal and also offer aesthetic value and wildlife habitat. Constructed stormwater wetlands differ from natural wetland systems in that they are engineered facilities designed specifically for the purpose of treating stormwater runoff and typically have less biodiversity than natural wetlands both in terms of plant and animal life. However, as with natural wetlands, stormwater wetlands require a continuous base flow or a high water table to support aquatic vegetation.

Wetlands are divided into two levels. Level 1 stormwater wetlands can be used to meet water quality volume, channel protection volume, and the 25-year, 24-hour storm event. A riser is used to create the pool and a small orifice is placed in the riser above the bottom of the wetland to create a shallow permanent pool. This allows the wetland to store additional runoff for a short period of time. Storm events that are greater than the design volume can be released through the top of the riser and /or through an emergency spill way channel.

**Shallow Wetland**

**Shallow ED Wetland**

**Newly Constructed Shallow Wetland**



**Pocket Wetland**

**Figure 4.26-1** Stormwater Wetland Examples

BACK TO TOC

Level 2 stormwater wetlands are mainly used to meet water quality requirements. They cannot be used for extended detention, so the outlet structure should be simplified. Level 2 wetlands can be installed parallel to wet detention ponds to meet detention requirements and help maintain the wetland permanent pool level.

There are several design variations of the stormwater wetland, each differing in the relative amounts of shallow and deep water, and dry storage above the wetland. These include shallow wetland, extended detention shallow wetlands, pond/wetland systems, and pocket wetlands. Below are descriptions of each design variant:

- Shallow Wetland (Level 2) – In the shallow wetland design, most of the water quality treatment volume is in the relatively shallow high marsh or low marsh depths. The only deep portions of the shallow wetland design are the forebay at the inlet to the wetland, and the micropool at the outlet. One disadvantage of this design is that, since the pool is very shallow, a relatively large amount of land is typically needed to store the water quality volume.

- Extended Detention (ED) Shallow Wetland (Level 1)– The extended detention (ED) shallow wetland design is the same as the shallow wetland; however, part of the water quality treatment volume is provided as extended detention above the surface of the marsh and released over a period of 24 hours. This design can treat a greater volume of stormwater in a smaller space than the shallow wetland design. In the extended detention wetland option, plants that can tolerate both wet and dry periods need to be used in the ED zone.

- Pond/Wetland Systems (Level 1) – The pond/wetland system has two separate cells: a wet pond and a shallow marsh. The wet pond traps sediment and reduces runoff velocities prior to entry into the wetland, where stormwater flows receive additional treatment. Less land is required for a pond/wetland system than for the shallow wetland or the ED shallow wetland systems.

- Pocket Wetland (Level 2) – A pocket wetland is intended for drainage areas of 5-10 acres and typically requires excavation down to the water table for a reliable water source to support the wetland system.

*Certain types of wetlands, such as submerged gravel wetland systems are not recommended for general use to meet stormwater management goals due to limited performance data. They may be applicable in special or retrofit situations where there are severe limitations on what can be implemented. Please see a further discussion in Section 4..27.*

### 4.26.2 Stormwater Management Suitability

Similar to stormwater ponds, stormwater wetlands are designed to control both stormwater quantity and quality. Thus, a stormwater wetland can be used to address all of the unified stormwater sizing criteria for a given drainage area.

- Runoff Reduction
  None. 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 (Hirschman et al., 2008, Strecker et al., 2004).

- 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. Subsection 4.26.3 provides median pollutant removal efficiencies that can be used for planning and design purposes.

- 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.

## 4.26.3 Pollutant Removal Capabilities

The pollution removal rates below may be used for design purposes. It should be noted that for this BMP there are two different types of Stormwater Wetlands that can be constructed. Level 1 wetlands are a more traditional design and can be used to meet $WQ_V$, $CP_v$, $Q_{P25}$, and $Q_f$ requirements. Level 2, on the other hand, has a different design configuration that includes modifications after nearly 20 years of research. Both of these levels will be explained in greater detail in this section.

For additional information and data on pollutant removal capabilities for stormwater wetlands, see the National Pollutant Removal Performance Database (3rd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

## 4.26.4 Application and Site Feasibility Criteria

Stormwater wetlands are generally applicable to most types of new development and redevelopment, and can be utilized in both residential and nonresidential areas. However, due to the large land requirements, wetlands may not be practical in higher density areas.

*The following criteria should be evaluated to ensure the suitability of a stormwater wetland for meeting stormwater management objectives on a site or development.*

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **Land requirements may preclude use**
- Regional Stormwater Control – **YES**

**Physical Feasibility** - Physical Constraints at Project Site

- **Drainage Area** – A minimum of 25 acres and a positive water balance are needed to maintain wetland conditions; 5 acres for pocket wetlands
- **Space Required** – Approximately 3 to 5% of the tributary drainage area
- **Site Slope** – There should be no more than 8% slope across the wetland site for level 1 wetlands. Level 2 wetlands should be generally flat.
- **Minimum Head** – Elevation difference needed at a site from the inflow to the outflow: 3 to 5 feet; 2 to 3 feet for pocket wetland
- **Minimum Depth to Water Table** – If used on a site with an underlying water supply aquifer or when treating a hotspot, a separation distance of 2 feet is recommended between the bottom of the wetland and the elevation of the seasonally high water table; pocket wetlands are typically below the water table.
- **Soils** – Permeable soils are not well-suited for a constructed stormwater wetland without a high water table. Underlying soils of hydrologic group "C" or "D" should be adequate to maintain wetland conditions. Most group "A" soils and some group "B" soils will require a liner. *Evaluation of soils should be based upon a subsurface analysis and permeability tests.*
- **Setback requirements:**
  - » *Property lines* - 10 feet (this is for site development projects only)
  - » *Private wells* - 100 feet; if well is downgradient from a hotspot land use then the minimum setback is 250 feet
  - » *Septic system tank/leach field* - 50 feet
  - » *Airports* - 5 miles

BACK TO TOC

### Other Constraints/Considerations

- **Trout Streams** – Consideration should be given to the thermal influence of stormwater wetland outflows on downstream trout waters.

- **Coastal Areas** - Several site characteristics commonly encountered in coastal Georgia may present challenges to site planning and design teams interested in using stormwater wetlands to manage post-construction stormwater runoff on a development site. **Table 4.26-1** identifies these common site characteristics and describes how they influence the use of stormwater wetlands on development sites. The table also provides site planning and design teams with some ideas about how they can work around these potential constraints.

Table 4.26-1: Challenges Associated with Using Stormwater Wetlands in Coastal Georgia

| Site Characteristic | How it Influences the Use of Stormwater Wetlands | Potential Solutions |
|---|---|---|
| Poorly draining soils, such as hydrologic soil group C and D soils | • Since stormwater wetlands are designed to have a permanent water surface, the presence of poorly drained soils does not influence their use on development sites. In fact, the presence of poorly draining soils may help maintain a permanent water surface within a stormwater wetland. | |
| Well draining soils, such as hydrologic soil group A and B soils | • May be difficult to maintain a permanent water surface within a stormwater wetland.<br>• May allow stormwater pollutants to reach groundwater aquifers with greater ease. | • Install a liner to maintain a permanent water surface.<br>• At stormwater hotspots and in areas known to provide groundwater recharge to water supply aquifers, install a liner to prevent pollutants from reaching underlying groundwater aquifers.<br>• In areas that are not considered to be stormwater hotspots and areas that do not provide groundwater recharge to water supply aquifers, use non-underdrained bioretention areas (Section 4.2) and infiltration practices (Section 4.12) to significantly reduce stormwater runoff volumes. |
| Flat terrain | • Makes it difficult, if not impossible, to provide a drain at the bottom of a stormwater wetland. | • Eliminate the use of drains, if necessary. |
| Shallow water table | • Makes it easier to maintain a permanent water surface within a stormwater wetland<br>• May allow stormwater pollutants to reach groundwater aquifers with greater ease. | • Excavation below the water table to create a stormwater wetland is acceptable, but any storage volume found below the water table should not be counted when determining the total storage volume provided by the stormwater wetland.<br>• At stormwater hotspots and in areas known to provide groundwater recharge to water supply aquifers, install a liner to prevent pollutants from reaching underlying groundwater aquifers.<br>• Use bioretention areas (Section 4.2) and infiltration practices (Section 4.12) with liners and underdrains to intercept and treat stormwater runoff at stormwater hotspots and in areas known to provide groundwater recharge to water supply aquifers. |
| Tidally-influenced drainage system | • May occasionally prevent stormwater runoff from being conveyed through a stormwater wetland, particularly during high tide. | • Maximize the use of low impact development practices in these areas to reduce stormwater runoff rates, volumes and pollutant loads.<br>• Consider the use of bubbler aeration and proper fish stocking to maintain nutrient cycling and healthy oxygen levels in stormwater wetlands located in these areas. |

BACK TO TOC

## 4.26.5 Planning and Design Criteria

*The following criteria are to be considered minimum standards for the design of a stormwater wetland facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.26.5.1 LOCATION AND LAYOUT

- A continuous base flow or high water table is required to support wetland vegetation. A water balance must be performed to demonstrate that a stormwater wetland can withstand a 30-day drought at summer evaporation rates without completely drawing down.
- Wetland siting should also take into account the location and use of other site features such as natural depressions, buffers, and undisturbed natural areas, and should attempt to aesthetically "fit" the facility into the landscape. Bedrock close to the surface may prevent excavation.
- Stormwater wetlands cannot be located within navigable waters of the U.S., including wetlands, without obtaining a Section 404 permit under the Clean Water Act, and any other applicable State permit. In some isolated cases, a wetlands permit may be granted to convert an existing degraded wetland in the context of local watershed restoration efforts.
- If a wetland facility is not used for overbank flood protection, it should be designed as an off-line system to bypass higher flows rather than passing them through the wetland system.
- All utilities should be located outside of the wetland site.

### 4.26.5.2 GENERAL DESIGN

- A well-designed stormwater wetland consists of:
  1. Shallow marsh areas of varying depths with wetland vegetation,
  2. Permanent micropool(s)
  3. An overlying zone where runoff control volumes are stored.
- Pond/wetland systems also include a stormwater pond facility (see Section 4.25, *Stormwater Ponds*, for pond design information).
- In addition, *all wetland designs must include a sediment forebay at the inflow* to the facility to allow heavier sediments to drop out of suspension before the runoff enters the wetland marsh.
- Additional pond design features include an *emergency spillway, maintenance access, safety bench, wetland buffer, and appropriate wetland vegetation and native landscaping.*

**Figures 4.26-2** through **4.26-5** in Subsection 4.26.8 provide plan view and profile schematics for the design of a shallow wetland, ED shallow wetland, pond/wetland system, and pocket wetland.

Table 4.26-2 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% | 75% |
| Low Land | Depth | 0"+ | N/A |
| | % of Total Volume | 50% | N/A |

BACK TO TOC

#### 4.26.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

In general, wetland designs are unique for each site and application. However, there are a number of geometric ratios and limiting depths for the design of a stormwater wetland that must be observed for adequate pollutant removal, ease of maintenance, and improved safety.

The stormwater wetland should be designed with the recommended proportion of "depth zones." Each of the four wetland design variants has depth zone allocations that are given as a percentage of the stormwater wetland surface area. Target allocations are found in **Table 4.26-2**. The four basic depth zones are:

- Deepwater zone
  Includes the outlet micropool and deepwater channels through the wetland facility. This zone supports little emergent wetland vegetation, but may support submerged or floating vegetation.

- Low marsh zone
  This zone is suitable for the growth of several emergent wetland plant species.

- High marsh zone
  This zone will support a greater density and diversity of wetland species than the low marsh zone. The high marsh zone should have a higher surface area to volume ratio than the low marsh zone.

Table 4.26-3 Design Criteria for Level 1 and 2 Stormwater Wetlands

| Criteria | Level 1 | Level 2 |
|---|---|---|
| $WQ_v$ | Use rainfall depth of 1.2 | Use rainfall depth of 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 be generally flat, use several cells if needed; use a maximum drop of 1' between cells |
| Normal flow path (distance from inlet to oulet) | 1:1 | 1.5:1 |
| Normal flow path (distance from inlet to outlet) | 1:1 | 1.5:1 |
| Dry weather flow path | Not required | 5:1 |
| Vegetation | Can use herbaceous only | Include woody vegetation (trees and shrubs) |
| Average wetland depth | Can be >1 | Should be < 1 |
| Extended detention | Limit to 1' vertically | Not allowed |

- Semi-wet zone
  This zone includes areas above the permanent pool that are inundated during larger storm events; it supports a number of species that can survive flooding.

- Micro-topology is important to Level 2 wetland designs. Planting peninsulas are the preferred method, but the following list can be used to enhance micro-topology:
  - Snags
  - Inverted root wads
  - Tree peninsulas
  - Coir fiber islands
  - Internal pools
  - Cobble sand weirs

It is recommended that a stream restoration specialist be consulted for additional guidance on these items.

BACK TO TOC

#### 4.26.5.4 PRETREATMENT/INLETS

- Sediment regulation is critical to sustain stormwater wetlands. A wetland facility should have a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the stormwater flow prior to dispersal into the wetland. The forebay should consist of a separate cell, formed by an acceptable barrier. A forebay is to be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the wetland facility.
- The forebay is sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 feet deep. The pretreatment storage volume is part of the total WQv requirement and may be subtracted from WQv for wetland storage sizing.
- A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.
- Inflow channels are to be stabilized with flared riprap aprons, or the equivalent. Inlet pipes to the pond can be partially submerged. Exit velocities from the forebay must be non-erosive.

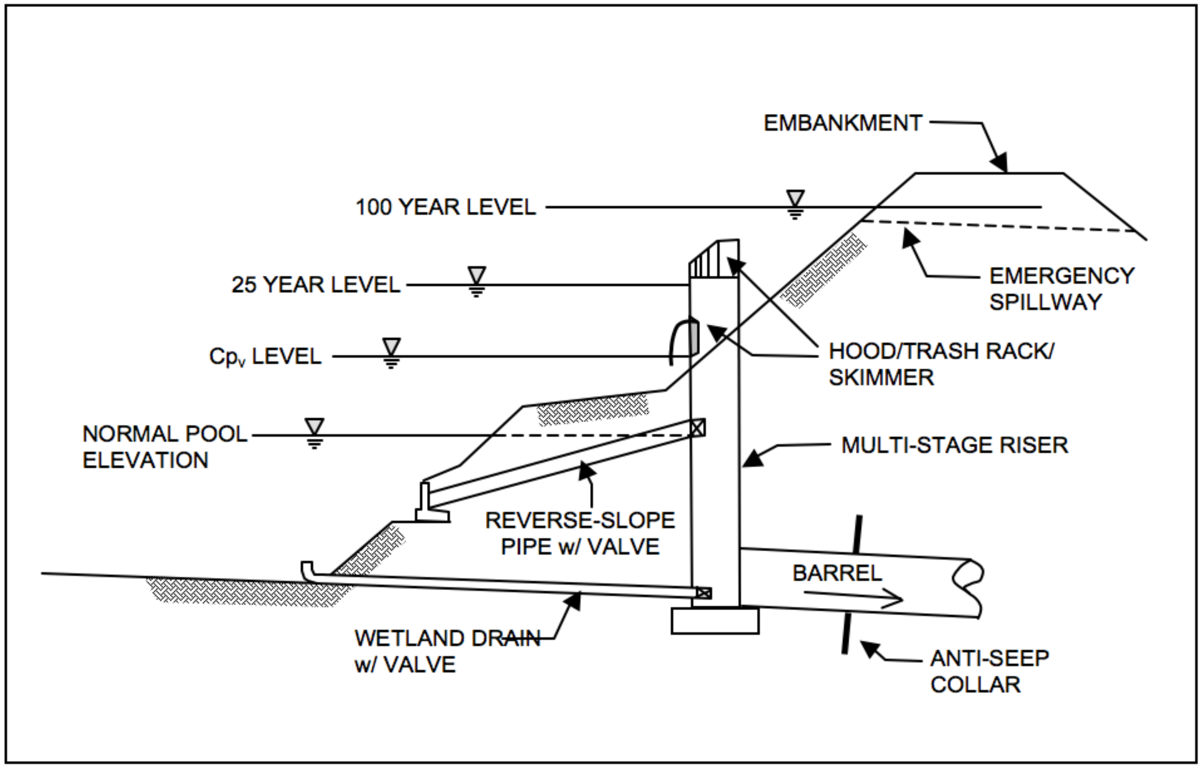



**Figure 4.26-6** Typical Wetland Facility Outlet Structure

#### 4.26.5.5 OUTLET STRUCTURES

- Flow control from a stormwater wetland is typically accomplished with the use of a concrete or corrugated metal riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the base of the micropool with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (see Figure 4.26-6) The riser should be located within the embankment for maintenance access, safety and aesthetics.
- A number of outlets at varying depths in the riser provide internal flow control for routing of the water quality, channel protection, and overbank flood protection runoff volumes. The number of orifices can vary and is usually a function of the pond design.

For shallow and pocket wetlands, the riser configuration is typically comprised of a channel protection outlet (usually an orifice) and overbank flood protection outlet (often a slot or weir). The channel protection orifice is sized to release the channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams). Since the water quality volume is fully contained in the permanent pool, no orifice sizing is necessary for this volume. As runoff from a water quality event enters the wet pond, it simply displaces that same volume through the channel protection orifice. Thus an off-line shallow or pocket wetland providing only water quality treatment can use a simple overflow weir as the outlet structure.

In the case of an extended detention (ED) shallow wetland, there is generally a need for an additional outlet (usually an orifice) that is sized to pass the extended detention water quality volume that is surcharged on top of the permanent pool. Flow will first pass through this orifice, which is sized to release the water quality ED volume in 24 hours. The preferred design is a reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond. The next outlet is sized for the release of the channel protection storage volume. The outlet (often an orifice) invert is located at the maximum elevation associated with the extended detention water quality volume and is sized to release the channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams).

Alternative hydraulic control methods to an orifice can be used and include the use of a broad-crested rectangular, V-notch, proportional weir, or an outlet pipe protected by a hood that extends at least 12 inches below the normal pool.

- The water quality outlet and channel protection outlet should be fitted with adjustable gate valves or other mechanism that can be used to adjust detention time.
- Higher flows (overbank and extreme flood protection) flows pass through openings or slots protected by trash racks further up on the riser.
- After entering the riser, flow is conveyed through the barrel and is discharged downstream. Anti-seep collars should be installed on the outlet barrel to reduce the potential for pipe failure.
- Riprap, plunge pools or pads, or other energy dissipators are to be placed at the outlet of the barrel to prevent scouring and erosion. If a wetland facility daylights to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. See Section 5.5 (Energy Dissipation Design) for more guidance.
- The wetland facility must have a bottom drain pipe located in the micropool with an adjustable valve that can completely or partially dewater the wetland within 24 hours. (This requirement may be waived for coastal areas, where positive drainage is difficult to achieve due to very low relief)
- The wetland drain should be sized one pipe size greater than the calculated design diameter. The drain valve is typically a handwheel activated knife or gate valve. Valve controls shall be located inside of the riser at a point where they (a) will not normally be inundated and (b) can be operated in a safe manner.

See the design procedures in Subsection 4.26.6 as well as Section 3.3 (*Storage Design*) and Section 3.4 (*Outlet Structures*) for additional information and specifications on pond routing and outlet works.

#### 4.26.5.6 EMERGENCY SPILLWAY

- An emergency spillway shall be included in the stormwater wetland design to safely pass flows that exceed the design storm flows. The spillway prevents the wetland's water levels from overtopping the embankment and causing structural damage. The emergency spillway must be located so that downstream structures will not be impacted by spillway discharges.
- A minimum of 1 foot of freeboard must be provided, measured from the top of the water surface elevation for the extreme flood to the lowest point of the dam embankment, not counting the emergency spillway.

BACK TO TOC

#### 4.26.5.7 MAINTENANCE ACCESS

- A maintenance right of way or easement must be provided to the wetland facility from a public or private road. Maintenance access should be at least 12 feet wide, have a maximum slope of no more than 15%, and be appropriately stabilized to withstand maintenance equipment and vehicles.
- The maintenance access must extend to the forebay, safety bench, riser, and outlet and, to the extent feasible, be designed to allow vehicles to turn around.
- Access to the riser is to be provided by lockable manhole covers, and manhole steps within easy reach of valves and other controls.

#### 4.26.5.8 SAFETY FEATURES

- All embankments and spillways must be designed to State of Georgia guidelines for dam safety.
- Fencing of wetlands is not generally desirable, but may be required by the local review authority. A preferred method is to manage the contours of deep pool areas through the inclusion of a safety bench (see above) to eliminate dropoffs and reduce the potential for accidental drowning.
- The principal spillway opening should not permit access by small children, and endwalls above pipe outfalls greater than 48 inches in diameter should be fenced to prevent a hazard.

#### 4.26.5.9 LANDSCAPING

- A landscaping plan should be provided that indicates the methods used to establish and maintain wetland coverage. Minimum elements of a plan include: delineation of landscaping zones, selection of corresponding plant species, planting plan, sequence for preparing wetland bed (including soil amendments, if needed) and sources of plant material.
- Landscaping zones include low marsh, high marsh, and semi-wet zones. The low marsh zone ranges from 6 to 18 inches below the normal pool. This zone is suitable for the growth of several emergent plant species. The high marsh zone ranges from 6 inches below the pool up to the normal pool. This zone will support greater density and diversity of emergent wetland plant species. The high marsh zone should have a higher surface area to volume ratio than the low marsh zone. The semi-wet zone refers to those areas above the permanent pool that are inundated on an irregular basis and can be expected to support wetland plants.
- The landscaping plan should provide elements that promote greater wildlife use within the wetland and buffers.
- Woody vegetation may not be planted on the embankment or allowed to grow within 15 feet of the toe of the embankment and 25 feet from the principal spillway structure.
- A wetland buffer shall extend 25 feet outward from the maximum water surface elevation, with an additional 15-foot setback to structures. The wetland buffer should be contiguous with other buffer areas that are required by existing regulations (e.g., stream buffers) or that are part of the overall stormwater management concept plan. No structures should be located within the buffer, and an additional setback to permanent structures may be provided.
- Existing trees should be preserved in the buffer area during construction. It is desirable to locate forest conservation areas adjacent to ponds. To discourage resident geese populations, the buffer can be planted with trees, shrubs and native ground covers.
- The soils of a wetland buffer are often severely compacted during the construction process to ensure stability. The density of these compacted soils is so great that it effectively prevents root penetration and therefore may lead to premature mortality or loss of vigor. Consequently, large and deep holes should be excavated around the proposed planting sites and backfill these with uncompacted topsoil.

Guidance on establishing wetland vegetation can be found in Appendix D (*Planting and Soil Guidance*).

#### 4.26.5.10 ADDITIONAL SITE SPECIFIC DESIGN CRITERIA AND ISSUES

- **Physiographic Factors -** Local terrain design constraints
  - » **Low Relief** – Providing wetland drain can be problematic
  - » **High Relief** – Embankment heights restricted
  - » **Karst** – Requires poly or clay liner to sustain a permanent pool of water and protect aquifers; limits on ponding depth; geotechnical tests may be required
- **Soils**
  - » Hydrologic group "A" soils and some group "B" soils may require liner (not relevant for pocket wetland)
- **Special Downstream Watershed Considerations**
  - » **Trout Stream** – Design wetland offline and provide shading to reduce thermal impact; limit $WQ_{v\text{-}ED}$ to 12 hours
  - » **Aquifer Protection** – Prevent possible groundwater contamination by preventing infiltration of hotspot runoff. May require liner for type "A" soils; pretreat hotspots; 2 to 4 foot separation distance from water table.
  - » **Swimming Area/Shellfish** – Design for geese prevention (see Appendix D); provide 48-hour ED for maximum coliform die-off.

BACK TO TOC

## 4.26.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of a stormwater wetland.**

Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a stormwater wetland. Create a rough layout of the stormwater wetland dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the stormwater wetland.**

Consider whether the stormwater wetland is intended to:

» Meet a water quality (treatment) target. See Step 3 to size the BMP utilizing the water quality treatment approach.

» Be "oversized" to include partial credit for storage capacity for other stormwater requirements (Channel Protection Volume ($CP_v$)

» Provide a possible solution to a drainage problem

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3 for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3) **Calculate the Target Water Quality Volume**

Calculate the Water Quality Volume using the following formula:

$$WQ_v = (1.2)\,(R_v)\,(A) / 12$$

*Where*:

**$WQ_v$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_v$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4) **Determine the pretreatment (forebay volume)**

Size the forebay. The forebay storage volume counts toward the total runoff reduction volume and may be subtracted for the following calculation. Note that the forebay volume should be at least 10% of the total water quality volume.

(Step 5) **Determine the inlet/outlet design and emergency overflow**

Check water surface elevations for all design storm events to ensure the stormwater wetland can pass these flows safely. 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.

(Step 6) Prepare Site Vegetation and Landscaping Plan

A landscaping plan for the submerged gravel wetland should be prepared to indicate how it will be established with vegetation. See Subsection 4.26.5 (*Landscaping*) and Appendix D for more details.

### 4.26.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

## 4.26.8 Example Schematics

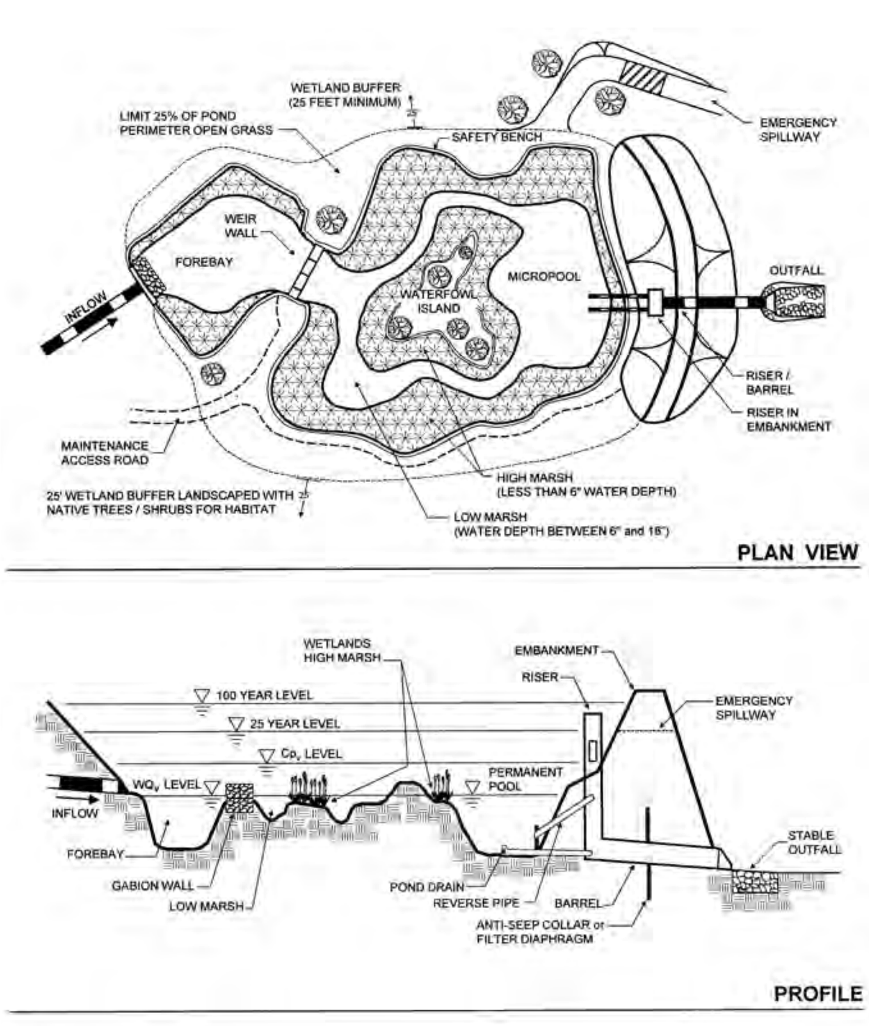



**Figure 4.26-2** Schematic of Shallow Wetland

(Source: Center for Watershed Protection)

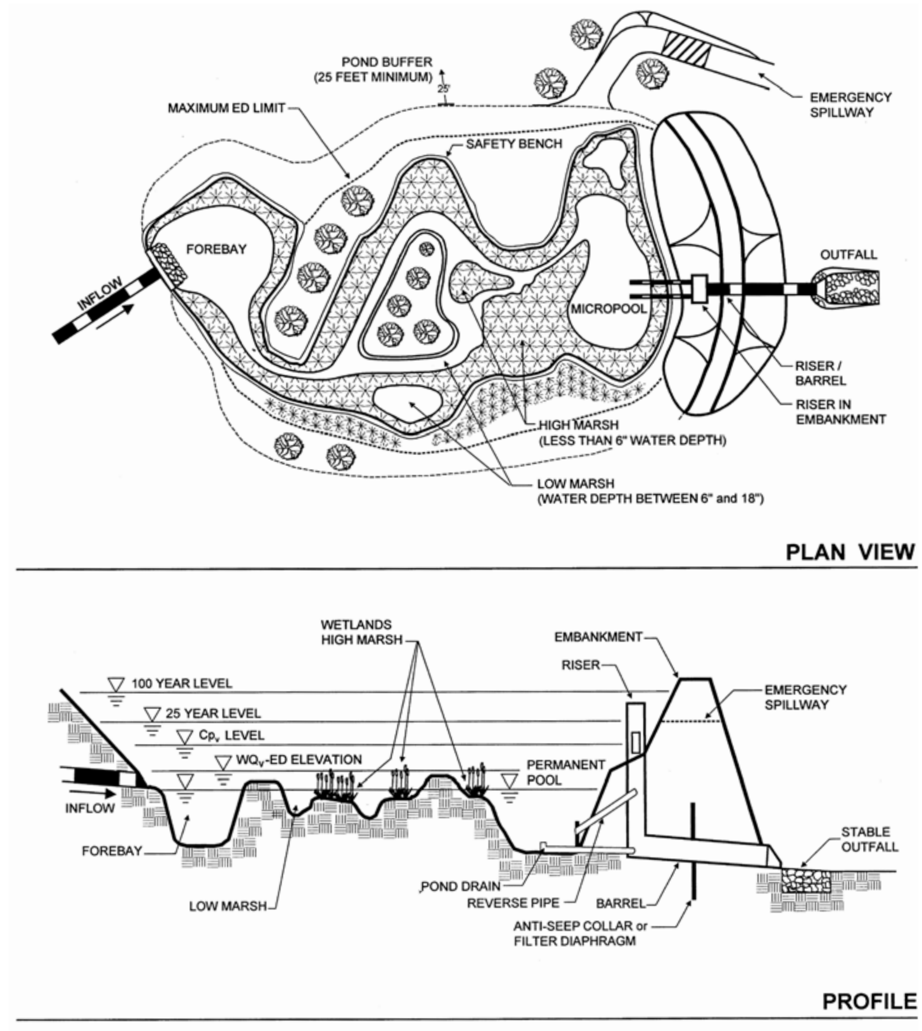



**Figure 4.26-3** Schematic of Extended Detention Shallow Wetland

(Source: Center for Watershed Protection)

BACK TO TOC

### 4.26.8-2 Example Schematics

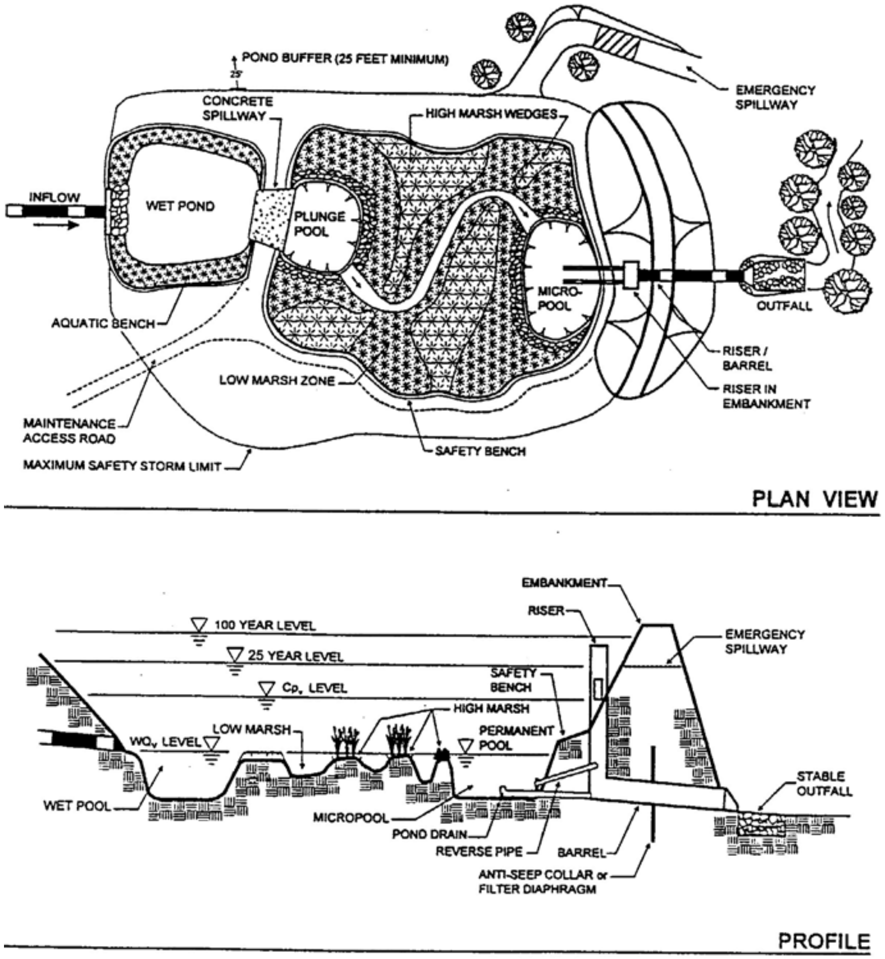



Figure 4.26-4 Schematic of Pond/Wetland System

(Source: Center for Watershed Protection)

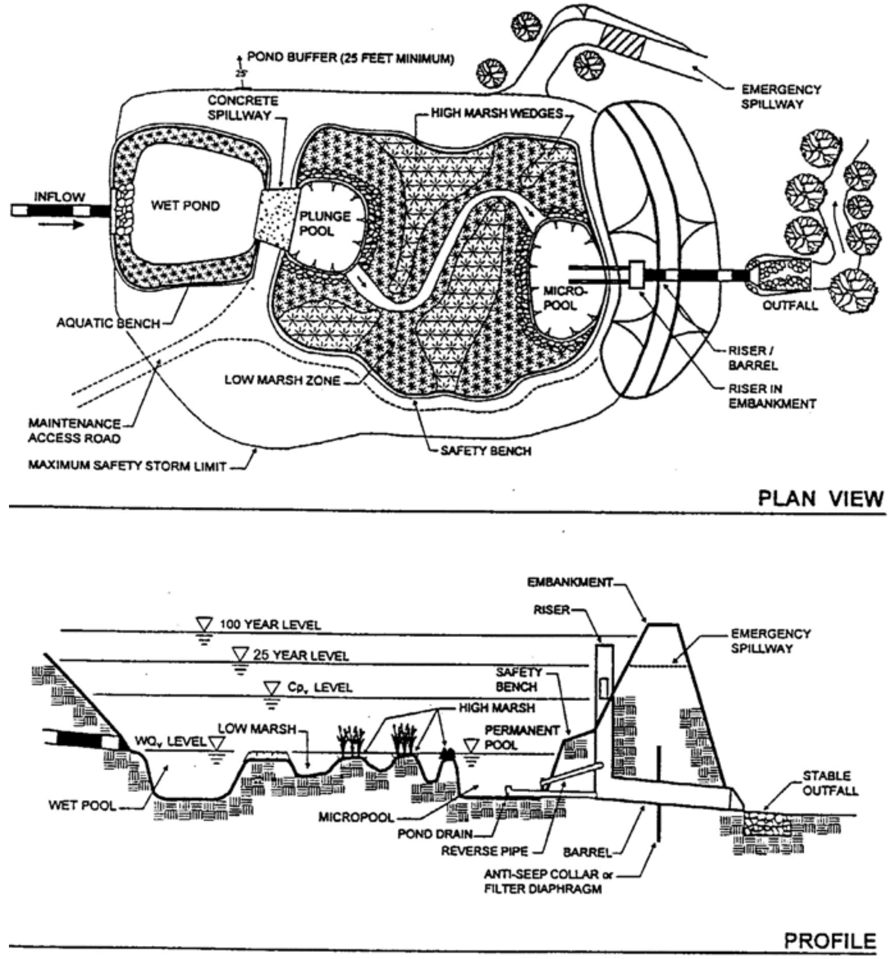



Figure 4.26-5 Schematic of Pocket Wetland

(Source: Center for Watershed Protection)

BACK TO TOC

# 4.27 Submerged Gravel Wetlands

**Description:** One or more cells filled with crushed rock designed to support wetland plants. Stormwater flows subsurface through the root zone of the constructed wetland, where pollutant removal takes place.

**LID/GI Considerations:** Submerged gravel wetlands are designed with a small footprint area and provide high total suspended solids and pollutant removal rates for highly impervious areas.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Submerged gravel wetlands should be designed as off-line systems sized to handle only water quality volume.
- Submerged gravel wetland systems need sufficient drainage area to maintain vegetation.
- The local slope should be relatively flat (<4%).
- Elevation drop from the inlet to the outlet is required to ensure that hydraulic conveyance by gravity is feasible (generally about 2-5 feet).
- All submerged gravel wetland designs should include a sediment forebay or equivalent pretreatment measures to prevent sediment and debris from entering and clogging the gravel bed.
- Unless they receive hot spot runoff, submerged gravel wetland systems can be allowed to intersect the groundwater table.

### ADVANTAGES / BENEFITS

- High total suspended solids removal
- High removal rate of urban pollutants
- Useful in space-limited applications
- Can be located in low-permeability soils with a high groundwater table

### DISADVANTAGES / LIMITATIONS

- High maintenance requirements
- Drainage through the wetland can be problematic for low-relief terrain.

### ROUTINE MAINTENANCE REQUIREMENTS

- Ensure that inlets and outlets for each submerged gravel wetland cell are free from debris and not clogged.
- Check for sediment buildup in the gravel bed.
- If sediment buildup is preventing flow through the wetland, remove gravel and sediment from the cell, replace it with clean gravel, and replant vegetation.

### POLLUTANT REMOVAL

- 80% **Total Suspended Solids**
- 50% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 50/20% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- 70% **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- Runoff Reduction
- ✓ Water Quality
- Channel Protection
- Overbank Flood Protection
- Extreme Flood Protection

✓ suitable or this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- L Land Requirement
- H Capital Cost
- H Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *No*

**Soils:** *Submerged gravel wetlands can be used in almost all soils and geology, but HSG C or D soils are preferred to maintain submerged flow.*

**Other Considerations:** *Extra space is recommended for pretreatment and to keep the practice from clogging due to sediment and debris.*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 0% Runoff Reduction Credit is provided by this practice.



BACK TO TOC

## 4.27.1 General Description

The submerged gravel wetland system consists of one or more treatment cells that are filled with crushed rock or gravel and designed to allow stormwater to flow subsurface through the root zone of the constructed wetland. The outlet from each cell is set at an elevation to keep the rock or gravel submerged. Wetland plants are rooted in the media, where they can directly take up pollutants. In addition, algae and microbes thrive on the surface area of the rocks. In particular, the anaerobic conditions on the bottom of the filter can foster the denitrification process. Although widely used for wastewater treatment in recent years, only a handful of submerged gravel wetland systems have been designed to treat stormwater. Mimicking the pollutant removal ability of nature, this BMP relies on the pollutant-stripping ability of plants and soils to remove pollutants from runoff. Typical schematics for a submerged gravel wetland are shown in **Figure 4.27-1** and **Figure 4.27-2.**

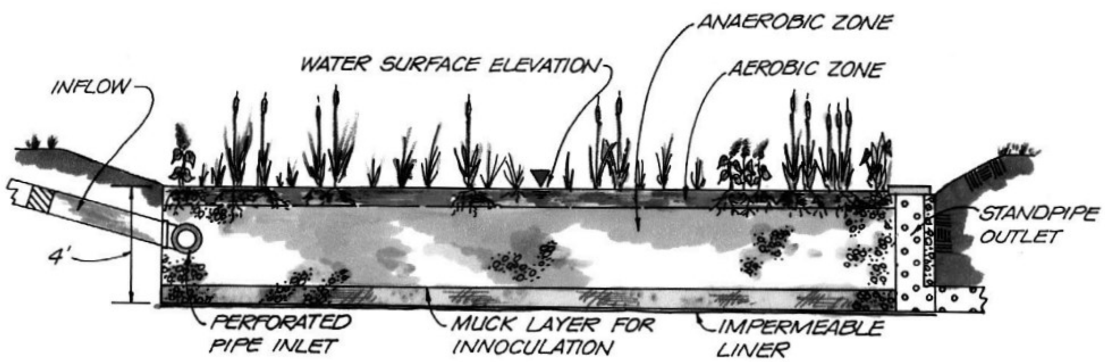



**Figure 4.27-1** Schematic of Submerged Gravel Wetland System

(Source: Center for Watershed Protection; Roux Associates Inc.)

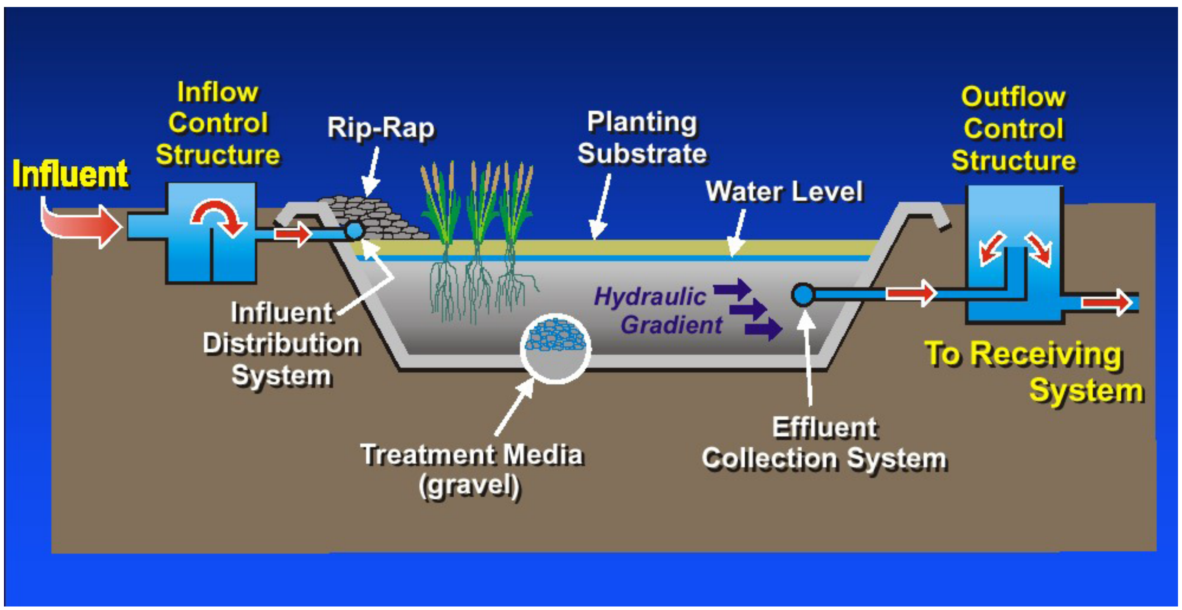



**Figure 4.27-2** Schematic of Submerged Gravel Wetland System

(Sources: Center for Watershed Protection; Roux Associates Inc.)

BACK TO TOC

## 4.27.2 Stormwater Management Suitability

- Runoff Reduction
  Submerged gravel wetlands provide minimal stormwater volume runoff reduction. Another BMP should be used in a treatment train with submerged gravel wetlands to provide runoff reduction.

- Water Quality
  If installed as per the recommended design criteria and properly maintained, 80% total suspended solids removal will be applied to the water quality volume ($WQ_v$) flowing to the submerged gravel wetland.

- Channel Protection
  Submerged gravel wetlands do not provide channel protection. Another BMP should be used in a treatment train with submerged gravel wetlands to provide channel protection or runoff reduction. (See Subsection 4.1.6.) Additionally, the submerged gravel wetlands should be designed off-line or with a bypass for higher flows.

- Overbank Flood Protection
  Submerged gravel wetlands do not provide overbank flood protection. Another BMP should be used in a treatment train with submerged gravel wetlands to provide overbank flood protection or runoff reduction. Additionally, the submerged gravel wetlands should be designed off-line or with a bypass for higher flows. (See Subsection 3.1.5.)

- Extreme Flood Protection
  Submerged gravel wetlands do not provide extreme flood protection. Another BMP should be used in a treatment train with submerged gravel wetlands to provide extreme flood protection or runoff reduction. Additionally, the submerged gravel wetlands should be designed off-line or with a bypass for higher flows. (See Subsection 3.1.5.)

## 4.27.3 Pollutant Removal Capabilities

The pollution removal efficiency of the submerged gravel wetland is similar to a typical wetland. Recent data show a TSS removal rate in excess of the 80% goal. This reflects the settling environment of the gravel media. These systems also exhibit removals of about 50% TP, 20% TN, and 50% Zn. The growth of algae and microbes among the gravel media has been determined to be the primary removal mechanism of the submerged gravel wetland.

The following pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- *Total Suspended Solids* – 80%
- *Total Phosphorus* – 50%
- *Total Nitrogen* – 20%
- *Fecal Coliform* – 70%
- *Heavy Metals* – 50%

## 4.27.4 Application and Site Feasibility Criteria

The submerged gravel wetland system is similar to a regular stormwater wetland; however, it is filled with crushed rock or gravel and designed to allow stormwater to flow through the root zone of the constructed wetland. The outlet from each cell is set at an elevation to keep the rock or gravel submerged. Wetland plants are rooted in the rock media, where they can directly take up pollutants. In addition, algae and bacteria grow in the rock media and provide an additional avenue for pollutant removal through biological uptake. Mimicking the pollutant removal ability of nature, this structural control relies on the pollutant-stripping ability of plants and bacteria to remove pollutants from runoff.

*The following criteria should be evaluated to ensure the suitability of submerged gravel wetlands for meeting stormwater management objectives on a site.*

**General Feasibility**

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Stormwater Control – NO

BACK TO TOC

**Physical Feasibility –** Physical Constraints at Project Site

- **Drainage Area** – In general, submerged gravel wetlands should be used on sites with a minimum drainage area of 1 acre to ensure submerged flow conditions. The maximum drainage area for a submerged gravel wetland is 5 acres.

- **Space Required** – Additional space is recommended for pretreatment measures to prevent sediment and debris from entering and clogging the gravel bed.

- **Site Slope** – The local slope should be relatively flat (<4%). While there is no minimum slope requirement, there does need to be enough elevation drop from the inlet to the outlet to ensure that hydraulic conveyance by gravity is feasible (generally about 2-5 feet).

- **Minimum Depth to Water Table** – Unless they receive hot spot runoff, submerged gravel wetland systems can be allowed to intersect the groundwater table. If a submerged gravel wetland receives hot spot runoff and has an underlying water supply aquifer, a liner and a separation distance of 2 feet is required between the bottom of the gravel and the elevation of the seasonally high water table to prevent groundwater contamination.

- **Soils** – Submerged gravel wetlands can be used in almost all soils and geology, with minor design adjustments for regions of karst (i.e., limestone) topography or in rapidly percolating soils such as sand. In these areas, submerged gravel wetlands should be designed with an impermeable liner to prevent groundwater contamination and sinkhole formation.

**Other Constraints/Considerations**

- **Hot spots** – Submerged gravel wetlands without a liner should not be used to treat hot spots that generate higher concentrations of hydrocarbons, trace metals, or toxicants than are found in typical stormwater runoff, which could contaminate groundwater.

- **Proximity** – The following is a list of specific setback requirements for the location of a submerged gravel wetland:
  - » 10 feet from building foundations
  - » 10 feet from property lines
  - » 100 feet from private water supply wells
  - » 100 feet from open water (measured from edge of water)
  - » 200 feet from public water supply reservoirs (measured from edge of water)
  - » 1,200 feet from public water supply wells

- **Trout Stream** – In cold water streams, submerged gravel wetlands should be designed to detain stormwater for a relatively short time (i.e., less than twelve hours) to minimize the potential amount of stream warming that occurs in the practice.

**Coastal Areas**

- **Poorly Draining Soils, such as hydrologic soil groups C and D** — Since they would normally be equipped with waterproof liners, the presence of poorly draining soils does not influence the use of submerged gravel wetlands on development sites.

- **Well-draining soils, such as hydrologic soil group A and B** – Since they are equipped with waterproof liners, the presence of well-draining soils does not influence the use of submerged gravel wetlands on development sites.

- **Flat Terrain** – The presence of flat terrain does not preclude the use of a submerged gravel wetland.

- **Shallow Water Table** – Except in the case of hot spot runoff, the base of the submerged gravel wetland can intersect the groundwater table.

- **Tidally-influenced drainage system** – Saltwater intrusion of the submerged gravel wetland should not be allowed.

## 4.27.5 Planning and Design Criteria

- Existing and proposed site, topographic, and location maps, as well as field reviews
- Impervious and pervious areas and other means used to determine the land use data
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site
- Design data from nearby storm sewer structures
- Water surface elevation of nearby water systems as well as the depth to seasonally high groundwater

*The following criteria are to be considered minimum standards for the design of a submerged gravel wetland. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.27.5.1 LOCATION AND LAYOUT

Submerged gravel wetlands are generally applied to land uses with a high percentage of impervious surfaces. Submerged gravel wetlands should be located upstream or downstream of other BMPs providing runoff reduction, channel protection volume ($CP_v$), overbank flood protection ($Q_{p25}$) and extreme flood protection ($Q_f$). See Subsection 4.1.6 for more information on the use of multiple BMPs in a treatment train.

### 4.27.5.2 GENERAL DESIGN

Pretreated stormwater enters via piped or overland flow and discharges into the gravel-filled chamber. A perforated pipe (4-6-inch diameter preferred) at the base of the gravel layer allows for flow-through conditions and maintains a constant water surface elevation. Discharges that exceed the $WQ_v$ exit to a stable outfall at non-erosive velocities.

### 4.27.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- Gravel layer should be 18-48 inches thick of clean washed uniformly graded material with a porosity of 40%.
- Provide storage for 75% of the submerged gravel wetland drainage area $WQ_v$. Maximum ponding depth should not be higher than the level of wetland vegetation.
- Use the porosity of the gravel media when calculating submerged gravel wetland storage volumes.
- The gravel layer should be 4 feet or less.
- A flow splitter may be required to redirect the $WQ_v$ to the submerged gravel wetland.
- Optional earth berms may separate multiple treatment cells.
- Observation wells should consist of 6-inch diameter perforated pipe and be at least 6 inches above grade.
- Maintenance right-of-way or drainage easement should be at least 20 feet wide.

Maximum slope of access easement should be 15%, and the driveway path should be at least 12 feet wide. This driveway path should be able to support maintenance vehicles and equipment.

### 4.27.5.4 PRETREATMENT/INLETS

- A sediment forebay or pretreatment is important for controlling and the amount of sediment entering a submerged gravel wetland.
- Forebays should be designed to remove sediment from stormwater runoff prior to entering the submerged gravel wetland.
- The forebay should be a separated from the wetland by a barrier and located at each inlet to the gravel wetland, except if the inlet supplies 10% or less of the inflow to the gravel wetland.
- The forebay should be sized to contain 0.1 inches of impervious area of the $WQ_v$. This volume may be subtracted from the $WQ_v$ for wetland storage sizing. A device to measure vertical sediment depth maybe may be installed to measure sediment accumulation over time.

### 4.27.5.5 OUTLET STRUCTURES

- For a submerged gravel wetland, the outlet structure can consist of a weir, orifice, outlet pipe, combination outlet, or other acceptable control structure.
- Small outlets that will be subject to clogging or are difficult to maintain are not acceptable.

BACK TO TOC

#### 4.27.5.6 SAFETY FEATURES

- A minimum of 6 inches of freeboard must be provided, measured from the top of the water surface elevation for the water quality volume, to the lowest point of the ground surface elevation, not counting the outlet weir.
- Stormwater should be conveyed to and from submerged gravel wetlands safely and to minimize erosion potential.

#### 4.27.5.7 LANDSCAPING

- It is recommended that native (local) wetland plant stock is used for establishing vegetation.
- A minimum of three types of wetland species should be provided.
- Mulch or topsoil may be placed on top of the rock media to establish vegetation. Note that using rock media to establish the wetland may require specific planting stock.
- Regular inspection and maintenance may be necessary until vegetation within the wetland is established. It may be necessary to replace some of the plants.

#### 4.27.5.8 CONSTRUCTION CONSIDERATIONS

- Construction equipment should be restricted from the submerged gravel wetland to prevent compaction of the native soils.
- Stabilize disturbed areas prior to runoff entering the constructed wetland.
- Protect the location of a submerged gravel wetland during construction. Divert surface runoff from the practice during grading. Any conveyance infrastructure should be blocked.
- Use wide-tracked equipment to minimize disturbance and compaction. If it is necessary to pump water during construction, discharge filtered water to a stable outlet.

#### 4.27.5.9 CONSTRUCTION AND MAINTENANCE COSTS

- An estimate for submerged gravel wetland construction costs is $5-6 per square foot. This includes sediment forebays.

## 4.27.6 Design Procedures

(Step 1) **Determine if the development site and conditions are appropriate for the use of a submerged gravel wetland.**
Consider the application and site feasibility criteria in this chapter. In addition, determine if site conditions are suitable for a submerged gravel wetland. Create a rough layout of the submerged gravel wetland dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 2) **Determine the goals and primary function of the submerged gravel wetland.**
Consider whether the submerged gravel wetland is intended to:

» Meet a water quality (treatment) target. See Step 3 to size the BMP utilizing the water quality treatment approach.

» Provide a possible solution to a drainage problem

Check with local officials and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply. In addition, consider if the best management practice has any special site-specific design conditions or criteria. List any restrictions or other requirements that may apply or affect the design.

The design of the BMP should be centered on the restrictions/requirements, goals, targets, and primary function(s) of the BMP, described in this section. By considering the primary function, as well as, topographic and soil conditions, the design elements of the practice can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.)

*Complete Step 3 for a water quality (treatment) approach. Refer to your local community's guidelines for any additional information or specific requirements regarding the use of either method.*

(Step 3) **Calculate the Target Water Quality Volume**
Calculate the Water Quality Volume using the following formula:

$$WQ_V = (1.2)\ (R_V)\ (A)\ /\ 12$$

*Where*:

**$WQ_V$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4) **Size flow diversion structure, if needed**
If the contributing drainage area to the submerged gravel wetland exceeds the water quality treatment and/or storage capacity, a flow regulator (or flow splitter diversion structure) should be supplied to divert the $WQ_v$ to the submerged gravel wetland.

(Step 5) Design stable outfall(s).

An overflow, such as an overdrain with an invert set slightly above the elevation of maximum ponding depth, 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.

(Step 6) Prepare Site Vegetation and Landscaping Plan

A landscaping plan for the submerged gravel wetland should be prepared to indicate how it will be established with vegetation. See Subsection 4.27.5.7 (*Landscaping*) and Appendix D for more details.

### 4.27.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.28 Underground Detention

**Description**: Detention storage located in underground tanks or vaults designed to provide water quantity control through detention and/or extended detention of stormwater runoff.

**LID/GI Considerations**: Underground detention facilities do not provide runoff reduction or water quality treatment and are not generally considered to be low impact development or green infrastructure

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- The maximum contributing drainage area to be served by a single underground detention vault or tank is 25 acres.
- Detention vaults should be constructed with a minimum 3,000 psi structural reinforced concrete.
- All construction joints must be provided with water stops.
- Cast-in-place wall sections must be designed as retaining walls.
- The maximum depth from finished grade to the vault invert should be 20 feet.
- The minimum pipe diameter for underground detention tanks is 36 inches.
- Underground detention vaults and tanks must meet structural requirements for overburden support and traffic loading, if appropriate.
- Riprap, plunge pools or pads, or other energy dissipators are to be placed at the end of the outlet to prevent scouring and erosion.
- A high flow bypass should be included in the underground detention system design to safely pass the extreme flood flow.

### ADVANTAGES / BENEFITS

- Ideal for highly urbanized areas where land is limited
- Can be used for stormwater quantity control downstream of other runoff reducing or water quality treating BMPs
- Some designs require minimal drop between inlet and outlet

### DISADVANTAGES / LIMITATIONS

- Not designed to provide storm water quality benefits
- Underground installation may make these systems difficult to maintain.
- Performance dependent on design and frequency of inspection and cleanout of unit
- Some designs may require a confined space entry for maintenance and repairs.

### ROUTINE MAINTENANCE REQUIREMENTS

- Adequate maintenance access must be provided for all underground detention systems.
- Remove any trash, debris, and sediment buildup in the underground vaults or tanks.
- Perform structural repairs to inlet and outlets, as needed.

### POLLUTANT REMOVAL

- 0% **Total Suspended Solids**
- 0% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 0/0% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- 0% **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- Runoff Reduction
- Water Quality
- ✓ Channel Protection
- ✓ Overbank Flood Protection
- ✓ Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- L Land Requirement
- H Capital Cost
- M Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *Yes*
**Roadway Projects:** *Not recommended*

**Soils:** *Geotechnical testing for the structural load bearing capacity of subsurface soils may be required prior to underground detention installation.*

**Other Considerations:** *Install a manhole on the downstream side to provide easy access for sampling of effluent.*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 0% Runoff Reduction Credit is provided by this practice.



BACK TO TOC

## 4.28.1 General Description

Detention vaults are box-shaped underground stormwater storage facilities typically constructed with reinforced concrete. Detention tanks are underground storage facilities typically constructed with large diameter metal or plastic pipe. Both serve as an alternative to surface detention for stormwater quantity control, particularly for space-limited areas where there is not adequate land for a dry detention basin or multi-purpose detention area.

Both underground vaults and tanks can provide channel protection through extended detention of the channel protection volume ($CP_v$) and overbank flood $Q_{p25}$ (and in some cases protection for the extreme flood $Q_f$). Basic storage design and routing methods are the same as for detention basins except that a bypass for high flows is typically included.

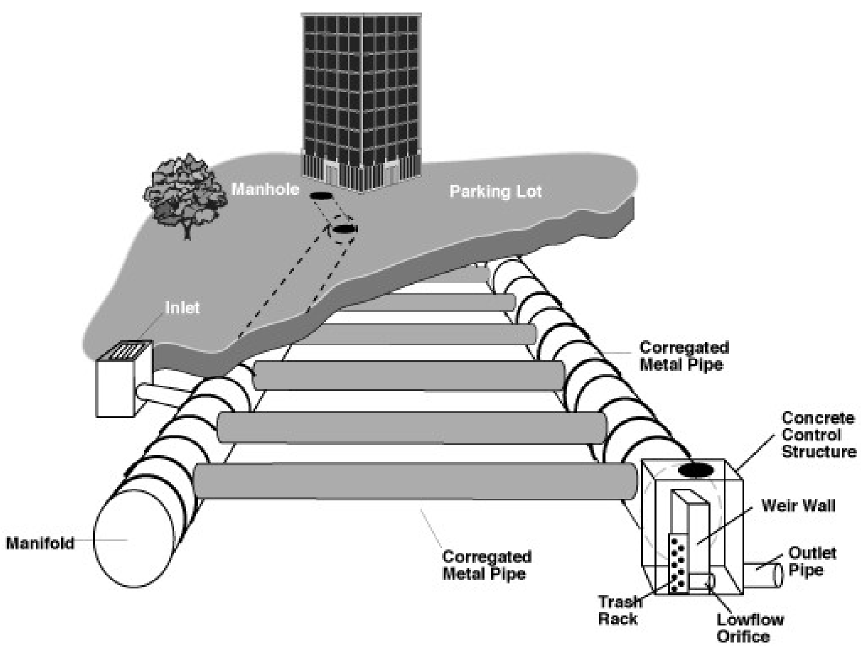



**Figure 4.28-1** Example Underground Detention Tank System

(Source: WDE, 2000)

Underground detention vaults and tanks are not intended for water quality treatment and must be used in a treatment train approach with other BMPs that provide treatment of the $WQ_v$ (see Subsection 4.1.6). This will prevent the underground vault or tank from becoming clogged with trash or sediment and significantly reduces the maintenance requirements for an underground detention system.

Prefabricated concrete vaults are available for commercial vendors. In addition, several pipe manufacturers have developed packaged detention systems. **Figures 4.28-1** and **Figure 4.28-2** show example design schematics for underground detention systems.

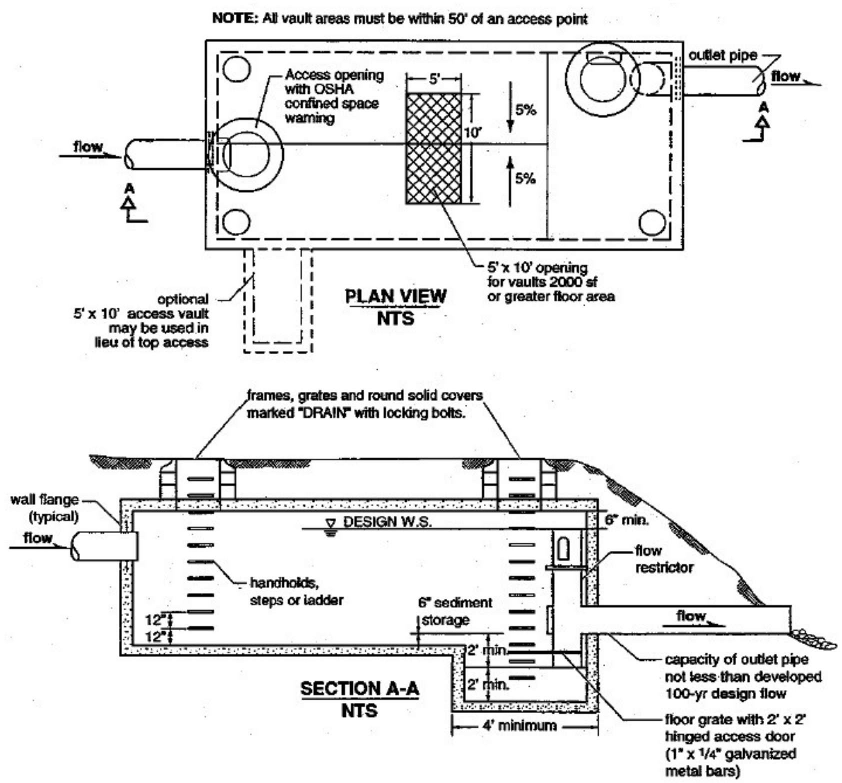



**Figure 4.28-2** Example Underground Detention Tank System

(Source: WDE, 2000)

BACK TO TOC

### 4.28.2 Stormwater Management Suitability

- Runoff Reduction
  Underground detention provides negligible stormwater volume runoff reduction unless modified to include an infiltration component. Another BMP should be used in a treatment train with underground detention to provide runoff reduction. See Subsection 4.1.6 for more information about using BMPs in series.

- Water Quality
  Underground detention provides minimal water quality volume ($WQ_v$) treatment. Another BMP should be used in a treatment train with underground detention to provide $WQ_v$ treatment. See Subsection 4.1.6 for more information about using BMPs in series.

- Channel Protection
  Underground detention 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 storm runoff volume.

- Overbank Flood Protection
  Underground detention is intended to provide overbank flood protection (peak flow reduction of the 25-year, 24-hour storm, $Q_{p25}$).

- Extreme Flood Protection
  Underground detention can be designed to control the extreme flood (100-year, 24-hour rainfall event peak flow, $Q_f$).

### 4.28.3 Pollutant Removal Capabilities

Underground detention does not provide measurable removal of total suspended solids, nutrient, metals, or organic matter. For additional information and data on pollutant removal capabilities for bioretention areas, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

### 4.28.4 Application and Site Feasibility Criteria

Underground detention systems are sized to provide extended detention of the channel protection volume over 24 hours and temporary storage of the runoff volume required to provide overbank flood ($Q_{p25}$) protection (i.e., reduce the post-development peak flow of the 25-year, 24-hour storm event to the pre-development rate). Due to the storage volume required, underground detention vaults and tanks are typically not used to control the 100-year storm ($Q_f$) except for very small drainage areas (<1 acre).

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Stormwater Control – YES
- Roadway Project – NO

Physical Feasibility – Physical Constraints at Project Site

- Drainage Area – The maximum contributing drainage area to be served by a single underground detention vault or tank is 25 acres.
- Space Required – Underground detention is installed underground; therefore, minimal surface area is required for the facility.
- Adequate maintenance access to each chamber must be provided for inspection and cleanout of underground detention units.
- Site Slope – Underground detention may be installed on sites with slopes up to 15%.
- Minimum Depth to Water Table – 2 feet
- Minimum Head – 4-8 feet
- Soils – Structural load bearing capacity of subsurface soils must be adequate to support the detention device and stormwater runoff.
- Check with manufacturer recommendations for additional site design constraints.

#### Other Constraints / Considerations

- Hot spots – Underground detention is well-suited for hot spot runoff.
- Damage to existing structures and facilities:
  - » Underground detention should not be used in areas where their operation may create a risk for basement flooding, interfere with subsurface sewage disposal systems, or affect other underground structures.
  - » Underground detention should be designed so that overflow drains away from buildings to prevent damage to building foundations.
- Trout Stream – Underground detention will not reduce thermal impacts of stormwater runoff, suspended solids, or soluble pollutants impacts. Therefore, they are not considered an effective means of protecting trout streams.

#### Coastal Areas

- Poorly Draining Soils — Poorly draining soils do not inhibit an underground detention facility's ability to temporarily store and treat stormwater runoff.
- Flat Terrain — Flat terrain and low site slopes do not interfere with the operation of underground detention.
- Shallow Water Table — Review manufacturer's instructions regarding groundwater elevation. Anti-flotation calculations may be required when large open chambers are installed at or below the water table.

## 4.28.5 Planning and Design Criteria

Before designing the underground detention system, the following data is necessary:

- Existing and proposed site, topographic, and location maps, as well as field reviews
- Impervious and pervious areas and other means used to determine the land use data
- Roadway and drainage profiles, cross sections, utility plans, and soil report for the site
- Design data from nearby storm sewer structures
- Water surface elevation of nearby water systems as well as the depth to seasonally high groundwater table

*The following criteria are to be considered minimum standards for the design of an underground detention system. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.28.5.1 LOCATION AND LAYOUT

Underground detention systems should be located downstream of other BMPs providing runoff reduction and/or treatment of the water quality volume ($WQ_v$). See Subsection 4.1.6 for more information on the use of multiple BMPs in a treatment train.

### 4.28.5.2 GENERAL DESIGN

- The maximum contributing drainage area to be served by a single underground detention vault or tank is 25 acres.
- Routing calculations must be used to demonstrate that the storage volume is adequate. See Section 3.3 (*Storage Design*) for procedures on the design of detention storage.
- Detention Vaults: Minimum 3,000 psi structural reinforced concrete may be used for underground detention vaults. All construction joints must be provided with water stops. Cast-in-place wall sections must be designed as retaining walls.
- Underground detention vaults and tanks must meet structural requirements for overburden support and traffic loading, if appropriate.
- Adequate maintenance access must be provided for all underground detention systems. Access must be provided over the inlet pipe and outflow structure. Access openings can consist of a standard frame, grate, and solid cover, or a removable panel.
- Vaults with widths of 10 feet or less should have removable lids.

BACK TO TOC

#### 4.28.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- Detention Tanks:  The minimum pipe diameter for underground detention tanks is 36 inches.
- The maximum depth from finished grade to the vault invert should be 20 feet.

#### 4.28.5.4 PRETREATMENT/INLETS

- A separate sediment sump or vault chamber sized to 0.1 inches times the impervious acres of contributing drainage should be provided at the inlet for underground detention systems that are in a treatment train with off-line water quality treatment BMPs.
- For $CP_v$ control, a low-flow orifice capable of releasing the channel protection volume over
- 24 hours must be provided. The channel protection orifice should have a minimum diameter of 3 inches and be adequately protected from clogging by an acceptable external trash rack. The orifice diameter may be reduced to 1 inch if internal orifice protection is used (i.e., an overperforated vertical stand pipe with 0.5-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve an equivalent diameter.

#### 4.28.5.5 OUTLET STRUCTURES

- For overbank flood protection, an additional outlet is sized for $Q_{p25}$ control (based upon hydrologic routing calculations) and can consist of a weir, orifice, outlet pipe, combination outlet, or other acceptable control structure. See Section 3.4 (*Outlet Structures*) for more information on the design of outlet works.
- Riprap, plunge pools or pads, or other energy dissipators should be placed at the end of the outlet to prevent scouring and erosion.
- A high-flow bypass should be included in the underground detention system design to safely pass the extreme flood flow ($Q_f$).

#### 4.28.5.6 SAFETY FEATURES

- Maintenance activities for an underground detention device may require a confined space entry.
- Vaults that are greater than 4 feet deep should be equipped with a safety ladder.

#### 4.28.5.7 CONSTRUCTION CONSIDERATIONS

- Newly installed underground detention should be inspected prior to being placed in service. Remove sediment and debris that may have collected in the system during delivery and installation.
- A minimum 20-foot wide maintenance right-of-way or drainage easement should be provided for the underground detention.

#### 4.28.5.8 CONSTRUCTION AND MAINTENANCE COSTS

- Material and installation costs for underground detention systems and vaults can vary based on the size, location, treatment requirements, and manufacturer.
- Typically, underground detention systems can range from approximately $12,000 for a small pipe and manifold system to over $60,000 for a multiple-chamber, high-volume, high-flow device.

## 4.28.6 Design Procedures

In general, site designers should perform the following design procedures when designing underground detention.

(Step 1) **Determine the goals and primary functions of the underground detention.**

- Underground detention can be designed to provide 24-hour detention of the channel protection volume ($CP_v$), and provide Overbank Flood ($Q_{p25}$) and Extreme Flood ($Q_f$) protection.
- Check with local officials and other agencies to determine if there are any additional watershed restrictions that may apply. In addition, consider if the underground detention has any special site-specific design conditions or criteria. List any other requirements that may apply to or affect the design.

(Step 2) **Determine if the development site and conditions are appropriate for the use of underground detention.**
Consider the application and site feasibility criteria in this chapter to determine if site conditions are suitable for underground detention. Create a rough layout of the proposed underground detention facility taking into consideration existing trees, utility lines, and other obstructions.

(Step 3) **Determine underground detention location and preliminary geometry.**
Ensure that there is adequate site area for installation of the underground detention facility, including maintenance access to the vault.

(Step 4) **Compute runoff control volumes and rates.**
Calculate $CP_v$, $Q_{p25}$, and $Q_f$, in accordance with the guidance presented in Subsection 3.1.5.

(Step 5) **Determine pretreatment volume..**
A separate sediment sump or vault chamber sized to 0.1 inches times the impervious acres of contributing drainage area should be provided at the inlet for underground detention systems that are in a treatment train with off-line water quality treatment BMPs.

(Step 6) **Calculate $CP_v$ release rates and water surface elevations.**
Set up a stage-storage-discharge relationship for the control structure for the 1-year, 24-hour rainfall event orifice. Size and determine the invert elevation of the $CP_v$ orifice to ensure that the channel protection volume is stored for at least 24 hours within the underground detention facility.

(Step 7) **Calculate $Q_{p25}$ and $Q_f$ release rates and water surface elevations.**
Set up a stage-storage-discharge relationship for the control structure for the 25- and 100-year, 24-hour storm orifices.

## 4.28.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

# 4.29 Vegetated Filter Strip


**Description**: Vegetated filter strips are uniformly graded and densely vegetated sections of land, engineered and designed to treat runoff from and remove pollutants through vegetative filtering and infiltration.

**LID/GI Considerations**: The main means for filtration through vegetated filter strip is where there is permeable soil. These practices work well as a pretreatment and when used with other types of structural stormwater BMPs.

## KEY CONSIDERATIONS

### DESIGN CRITERIA

- Runoff from an adjacent impervious area must be evenly distributed across the filter strip as sheet flow
- Can be used as part of the runoff conveyance system to provide pretreatment
- Slopes should be between 2-6%
- Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion.
- Uniform grading across filter strip to encourage sheet flow and prevent concentrated flows

### ADVANTAGES / BENEFITS

- Can provide groundwater recharge
- Reasonably low construction cost, effort, and changes to existing landscaping
- Works well for mitigating highway runoff pollution
- Works well alone or in series
- Adaptable to a variety of site conditions
- Flexible in design and layout
- Lower cost alternative

### DISADVANTAGES / LIMITATIONS

- Cannot alone achieve the 80% TSS removal target
- Large land requirement
- Requires periodic repair, regrading, and sediment removal to prevent channelization
- Vulnerable to erosion and concentrated flow
- Provides less runoff reduction than most BMPs

### ROUTINE MAINTENANCE REQUIREMENTS

- Mow grass to a height to maintain a dense vegetative cover
- Inspect for invasive species and remove as needed
- Inspect pea gravel diaphragm for clogging and remove sediment buildup
- Inspect vegetation for rills and gullies. Seed or sod bare areas.
- Sheet flow onto filter strips can be difficult to maintain, resulting in reconcentration of flow.
- Remove trash, debris, sediment, and dead grass.
- Reseed or resod as needed.

### POLLUTANT REMOVAL

- 60% **Total Suspended Solids**
- 40% **Metals** - Cadmium, Copper, Lead, and Zinc removal
- 20/20% **Nutrients** - Total Phosphorus / Total Nitrogen removal
- N/A **Pathogens** – Fecal Coliform

### STORMWATER MANAGEMENT SUITABILITY

- ★ Runoff Reduction
- ★ Water Quality
- ★ Channel Protection
- Overbank Flood Protection
- Extreme Flood Protection

✓ suitable for this practice
★ may provide partial benefits

### IMPLEMENTATION CONSIDERATIONS

- L Land Requirement
- H Capital Cost
- L Maintenance Burden

**Residential Subdivision Use:** *Yes*
**High Density/Ultra-Urban:** *No*
**Roadway Projects:** *Yes*

**Other Considerations:** *Use in a buffer system and to treat runoff from pervious areas*

L=Low M=Moderate H=High

### RUNOFF REDUCTION CREDIT

- 50% of the RRv conveyed to the practice (A & B hydrologic soils)
- 25% of the RRv conveyed to the practice (C & D hydrologic soils)

BACK TO TOC

### 4.29.1 General Description

Filter strips are uniformly graded and densely vegetated sections of land, engineered and designed to treat runoff and remove pollutants through vegetative filtering and infiltration. Filter strips are best suited to treating runoff from roads and highways, roof downspouts, very small parking lots, and pervious surfaces. They are also ideal components of the "outer zone" of a stream buffer, or as pretreatment for another structural stormwater practice. Filter strips can serve as a buffer between incompatible land uses, be landscaped in an aesthetically pleasing way, and provide groundwater recharge in areas with pervious soils. Filter strips are often used as a stormwater site design credit (see Subsection 2.2.4 for more information).

Filter strips rely on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from urban stormwater. There can also be a significant reduction in runoff volume for smaller flows that infiltrate pervious soils beneath the filter strip. To be effective, sheet flow must be maintained across the entire filter strip. Once runoff flow concentrates, it effectively short-circuits the filter strip and reduces any water quality benefits. Therefore, a flow spreader must normally be included in the filter strip design.

There are two different filter strip designs: a simple filter strip and a design that includes a permeable berm at the bottom. The presence of the berm increases contact time with the runoff, thus reducing the overall width of filter strip required to treat stormwater runoff. Filter strips must be designed to withstand the full range of storm events without eroding.

### 4.29.2 Stormwater Management Suitability

Filter strips are typically built for flowing moving perpendicular to and away from a roadway or parking lot. They must be designed to withstand a full range of storm events without eroding.

- Runoff Reduction
  Vegetated filter strips are an effective low impact development (LID) practice 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 the higher the infiltration rate of the native soils. A vegetated filter strip can be designed to provide 50% of the runoff reduction volume for type A and B hydrologic soils or 25% 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
  Use of vegetated 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.

- Channel Protection
  For smaller sites, a vegetated filter strip may be designed to capture the entire channel protection volume ($CP_v$). Given that a vegetated filter strip is typically designed to completely drain over 48-72 hours, the requirement of extended detention of the 1-year, 24-hour rainfall event runoff volume will be met. For larger sites, or where only the $WQ_v$ is diverted to the vegetated filter strip, another practice must be used to provide $CP_v$ extended detention.

- Overbank Flood Protection
  Another practice in conjunction with a vegetated filter strip will likely be required to reduce the post-development peak flow of the 25-year storm ($Q_p$) to pre-development levels (detention).

- Extreme Flood Protection
  Vegetated filter strips must provide flow diversion and/or be designed to safely pass extreme storm flows ($Q_f$) and protect the vegetation.

*Credit for the volume of runoff reduced in the vegetated filter strip may be taken in the overbank flood protection and extreme flood protection calculations. If the practice is designed to provide runoff reduction for water quality compliance, then the practice is given credit for channel protection and flood control requirements by allowing the designer to compute an Adjusted CN (see Subsection 3.1.7.5 for more information).*

BACK TO TOC

## 4.29.3 Pollutant Removal Capabilities

Pollutant removal from filter strips is highly variable and depends primarily on density of vegetation and contact time for filtration and infiltration. These, in turn, depend on soil and vegetation type, slope, and presence of sheet flow. Research on fecal coliform removal has been inconclusive, but suggests that filter strips are generally not effective BMPs for treating bacterial loads.

For additional information and data on pollutant removal capabilities for vegetated filter strips, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

## 4.29.4 Application and Site Feasibility Criteria

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. As a rule, flow concentrates within a maximum of 75 feet for impervious surfaces, and 150 feet for pervious surfaces (CWP, 1996). For longer flow paths, special provision must be made to ensure design flows spread evenly across the filter strip.

Because of design constraints, vegetated filter strips generally have a maximum drainage area of 5 acres, but 2 areas are preferred. Filter strips should be designed for slopes between 2-6%. Greater slopes than this 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.

**General Feasibility**

- Suitable for Residential Subdivision Usage – **YES**
- Suitable for High Density/Ultra Urban Areas – **NO**
- Regional Stormwater Control – **NO**
- Roadway Projects – **YES**

**Physical Feasibility -** Physical Constraints at Project Site

- **Drainage Area** – 5 acres or less, 2 acres preferred.
- **Space Required** – Rough rule of thumb ratio of drainage area to filter strip surface area required is 10:1.
- **Site Slope** – Slopes should be between 2-6% (perpendicular to the roadway).
- **Minimum Depth to Water Table** – A separation distance of 1-2 feet is recommended between the bottom of the vegetated filter strip and the elevation of the seasonally high water table.

**Other Constraints/Considerations**

- **Location requirements** – The following is a list of specific setback requirements for the location of a vegetated filter strip:
  - » Filter strips should be constructed outside the natural stream buffer area whenever possible to maintain a more natural buffer along the streambank.
  - » Filter strips should not be used on soils that cannot sustain a dense grass cover with high retardance. Designers should choose a grass that can withstand relatively high velocity flows at the entrances, and both wet and dry periods. See Appendix D for a list of appropriate grasses for use in Georgia.
  - » 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. 100 feet the recommended maximum strip length.



BACK TO TOC

## 4.29.5 Planning and Design Criteria

Before designing the vegetated filter strip, the following data is necessary:

- 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

*The following criteria are considered to be minimum standards for the design of a vegetated filter strip. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be met.*

### 4.29.5.1 LOCATION AND LAYOUT

Vegetated filter strips location and layout areas based on site constraints such as proposed and existing infrastructure, soils, existing vegetation, contributing drainage area, and utilities. Vegetated filter strips systems are designed for intermittent flow and must be allowed to drain and reaerate between rainfall events. They should not be used on sites with a continuous flow from groundwater, sump pumps, or other sources. Vegetated filter strip locations should be integrated into the site planning process and aesthetic and maintenance considerations should be taken into account in their siting and design. Elevations must be carefully worked out to ensure that the desired runoff flow enters the facility as sheet flow with no more than the maximum design depth and velocity.

### 4.29.5.2 GENERAL DESIGN

- Filter strips should be integrated within site designs.
- An effective flow spreader may use a pea gravel diaphragm at the top of the slope (ASTM D 448 size no. 6, 1/8" to 3/8"). The pea gravel diaphragm (a small trench running along the top of the filter strip) serves two purposes. First, it acts as a pretreatment device, settling out sediment particles before they reach the practice. Second it acts as a level spreader, maintaining sheet flow as runoff flows over the filter strip. Other types of flow spreaders include a concrete sill, curb stops, or curb and gutter with "sawteeth" cut into it.
- Ensure that flows in excess of the design flow move across or around the strip without damaging it. Often a bypass channel or overflow spillway with protected channel section is designed to handle higher flows.
- Maximum discharge loading per foot of filter strip width (perpendicular to flow path) is found using the Manning's equation:

$$q = \frac{0.00236}{n} Y^{\frac{5}{3}} S^{\frac{1}{2}}$$

**(Equation 4.29.1)**

*Where*:

**q** = discharge per foot of width of filter strip (cfs/ft)

**Y** = allowable depth of flow (inches; maximum flow depth of 2 inches)

**S** = slope of filter strip (percent)

**n** = Manning's "n" roughness coefficient *(n=0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)*

- The minimum length of a filter strip is:

$$W_{fMIN} = \frac{Q_{WQ}}{q}$$

**(Equation 4.29.2)**

*Where*:

**$W_{fMIN}$** = minimum filter strip width perpendicular to flow (feet)

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

q = discharge per foot of width of filter strip (cfs/ft)

BACK TO TOC

### Filter without Berm

- Size filter strip (parallel to flow path) for a contact time of 5 minutes minimum
- The equation for filter length is based on the SCS TR55 travel time equation (SCS, 1986):

$$L_f = \frac{(T_t)^{1.25}(P_{2-24})^{0.625}(S)^{0.5}}{0.338n}$$

**(Equation 4.29.3)**

*Where*:

**$L_f$** = length of filter strip parallel to flow path (ft)

**$T_t$** = travel time through filter strip (minutes)

**$P_{2-24}$** = 2-year, 24-hour rainfall depth (inches)

**S** = slope of filter strip (percent)

**n** = Manning's "n" roughness coefficient *(n=0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)*

Table 4.29-1 Vegetated Filter Strip Sizing Guidance

(Source: Claytor and Schueler, 1996)

| Parameter | Impervious Areas | | Pervious Areas (lawns, etc) | |
|---|---|---|---|---|
| Maximum inflow approach length (feet) | 35 | 75 | 75 | 100 |
| Filter strip minimum length (feet) | 15 | 25 | 12 | 18 |

### Filter Strips with Berm

- Size outlet pipes to ensure that the bermed area drains within 24 hours.
- Specify grasses resistant to frequent inundation within the shallow ponding limits.
- Berm material should be composed of sand, gravel, and sandy loam to encourage grass cover (Sand: ASTM C-33 fine aggregate concrete sand 0.02"-0.04", Gravel: AASHTO M-43 ½" to 1").
- Size filter strip to contain the WQv within the wedge of water backed up behind the berm.
- Maximum berm height is 12 inches.

### Filter Strips for Pretreatment

- A number of other structural controls, including vegetated filter strips and infiltration trenches, may utilize a filter strip as a pretreatment measure. The required length of the filter strip depends on the drainage area, imperviousness, and the filter strip slope. **Table 4.29-1** provides sizing guidance for bioretention filter strips for pretreatment.

### 4.29.5.3 PHYSICAL SPECIFICATIONS/GEOMETRY

- Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion.
- A minimum strip length of 15 feet should be used (25 feet is preferred).
- A maximum strip length of 100 feet is recommended.
- Slopes should be between 2-6% (perpendicular to the roadway).
- Acceptable velocities for filter strips should be less than 4 feet per second for grass and less than 1 foot per second for native herbaceous vegetation.

BACK TO TOC

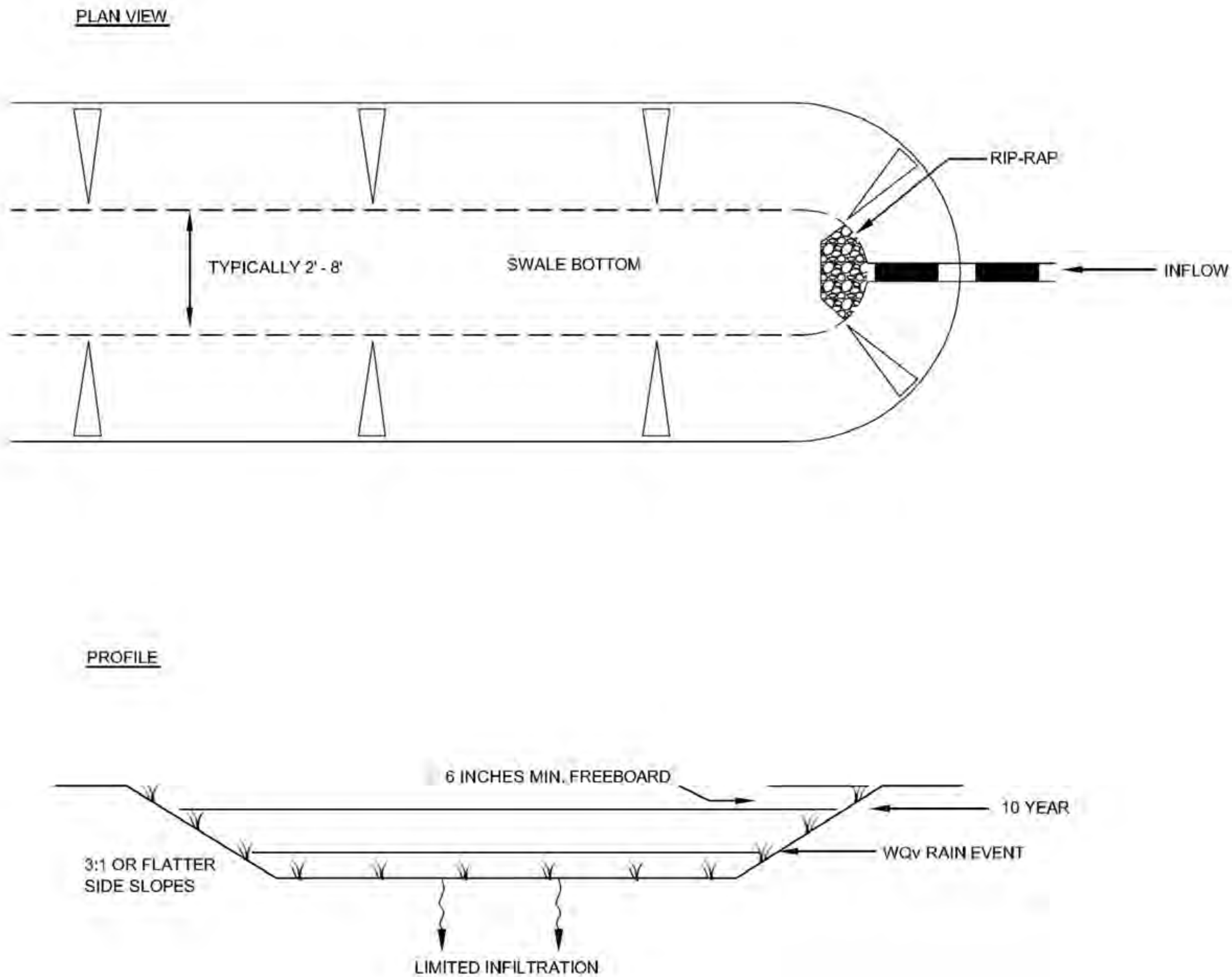



**Figure 4.29-1** Typical Vegetated Filter Strip

BACK TO TOC

## 4.29.6 Design Procedures

(Step 1) **Determine the goals and primary function of the vegetated filter strips.**

Consider whether vegetated filter strip is intended to:

- Meet a runoff reduction* target or water quality (treatment) target.
  **Note that minimum infiltration rates of the surrounding native soils must be acceptable and suitable when used in runoff reduction applications.*
- Be "oversized" to include partial credit for storage capacity for Channel Protection Volume ($CP_v$).
- Provide a possible solution to a drainage problem.
- Enhance landscape and provide aesthetic qualities.

Check with local officials and other agencies to determine if there are any additional watershed restrictions that may apply. In addition, consider if the vegetated filter strip has any special site-specific design conditions or criteria. List any other requirements that may apply to or affect the design.

The design of the vegetated filter strip should be centered on the restrictions/requirements, goals, targets, and primary function(s) of a vegetated filter strip. By considering the primary function, as well as, topographic and soil conditions, the design elements of the vegetated filter strip can be determined (i.e. planting media, underdrain, inlet/outlet, overflow, etc.).

(Step 2) **Determine if the development site and conditions are appropriate for the use of a vegetated filter strip**

Consider the application and site feasibility criteria in this chapter to determine if site conditions are suitable for a vegetated filter strip. Create a rough layout of the vegetated filter strip dimensions taking into consideration existing trees, utility lines, and other obstructions.

(Step 3) **Calculate the Target Water Quality Volume**

Calculate the Water Quality Volume using the following formula:

$$WQ_v = (1.2)\ (R_v)\ (A)\ /\ 12$$

*Where*:

**$WQ_v$** = Water Quality Volume ($ft^3$)
**1.2** = Target rainfall amount to be treated (inches)
**$R_v$** = Volumetric runoff coefficient which can be found by:

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

*Where*:

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 4) **Determine the maximum discharge loading per foot of filter strip.**

$$q = \frac{0.0237}{n} S^{\frac{1}{2}}$$

*Where*:

**q** = discharge per foot of width of filter strip (cfs/ft) based on $WQ_v$ storm event
**S =** slope of filter strip (percent)
**n =** Manning's "n" roughness coefficient
*(n=0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)*

(Step 5) Determine the minimum width (perpendicular to flow) of a filter strip.

$$W_{fMIN} = \frac{Q_{WQ}}{q}$$

**(Equation 4.29.5)**

*Where*:

**$W_{fMIN}$ =** min. filter strip width perpendicular to flow (ft)
**$Q_{WQ}$=**water quality volume peak flow ($ft^3/s$)
**q=**discharge per foot of width of filter strip (cfs/ft)

(Step 6) **Calculate the depth of flow of the stormwater runoff across the buffer to be sure it is <1 inch.**

$$D = (1.04*q^{0.6}*n^{0.6})/S^{0.3}$$

*Where*:

**D =** depth of flow (ft)

(Step 7) **Calculate the velocity of the stormwater runoff across the buffer to be sure it is < 2.0 fps.**

$$V = Q_{WQ}/(D* W_{fMIN})$$

(Step 8) **Size filter strip (parallel to flow path) for a contact time of at least 5 minutes.**

$$L_f = \frac{(T_t)^{1.25}(P_{2-24})^{0.625}(S)^{0.5}}{0.338*n}$$

**(Equation 4.29.4)**

*Where*:

**$L_f$** = length of filter strip parallel to flow path (ft)
**$T_t$** = travel time through filter strip (minutes)
**$P_{2-24}$** = 2-year, 24-hour rainfall depth (feet)
**S** = slope of filter strip (percent)
**n** = Manning's "n" roughness coefficient
*(n=0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)*

(Step 9) **To calculate the $RR_V$ credited for the practice (sized for $WQ_V$), Steps 5, 6, 7 and 8 have to be met, then proceed to Step 10. Otherwise proceed to Step 13.**

(Step 10) **Calculate the Stormwater Runoff Reduction Volume conveyed to the practice.**
Calculate the Runoff Reduction Volume using the following formula:

$$RR_V = (P)(RV)(A) / 12$$

*Where:*

**$RR_V$** = Runoff Reduction Target Volume ($ft^3$)
**P** = Target runoff reduction rainfall (inches)
**$R_V$** = Volumetric runoff coefficient which can be found by:

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

*Where:*

**I** = new impervious area of the contributing drainage area (%)

**A** = Area draining to this practice ($ft^2$)
**12** = Unit conversion factor (in/ft)

(Step 11) Calculate $RR_v$ credited.

Using **Table 4.1.3-2** - *BMP Runoff Reduction Credits*, lookup the appropriate runoff reduction percentage (or credit) provided by the practice:

$$RR_v \text{ (credited)} = RR_v(RR\%)$$

*Where*:

**RR%** = Runoff Reduction percentage, or credit, assigned to the specific practice

**$RR_v$ (credited)** = Runoff Reduction Volume provided by this practice ($ft^3$)

**$RR_v$** = $RR_v$ conveyed to the practice

(Step 12) Calculate the adjusted curve numbers for $CP_v$ (1-yr, 24-hour storm), $Q_{P25}$ (25-yr, 24-hour storm), and $Q_f$ (100-yr, 24-hour storm). See Subection 3.1.7.5 for more information

(Step 13) Prepare site Vegetation and Landscaping Plan

A landscaping plan for the vegetated filter strip should be prepared to indicate how it will be established with vegetation. See section Appendix D for more details.

### 4.29.7 Inspection and Maintenance Requirements

All best management practices require proper maintenance. Without proper maintenance, BMPs will not function as originally designed and may cease to function altogether. The design of all BMPs includes considerations for maintenance and maintenance access. For additional information on inspection and maintenance requirements, see Appendix E.

## References


Anne Arundel County. 2012. *Step Pool Storm Conveyance (SPSC) Systems.* Anne Arundel County, Maryland.

ASTM International. 2005. *Standard Practice for Determination of Deadloads and Live Loads Associated with Green Roof Systems.* Standard E2397.05. ASTM International. West Conshohocken, PA.

ASTM International. 2006. *Standard Guide for Selection, Installation and Maintenance of Plants for Green Roof Systems.* Standard E2400-06. ASTM International. West Conshohocken, PA.

Atlanta Regional Commission (ARC). 2001. *Georgia Stormwater Management Manual, Volume 2.* Atlanta Regional Commission, Atlanta, GA.

Bureau of Environmental Services. 2008. *Portland Stormwater Management Manual.* City of Portland, OR.

Cappiella, K., T. Schueler and T. Wright. 2006. *Urban Watershed Forestry Manual. Part 2: Conserving and Planting Trees at Development Sites.* NA-TP-01-06. US Department of Agriculture. Forest Service. Northeastern Area. State and Private Forestry. Newtown Square, PA.

Cappiella, K., T. Schueler, J. Tomlinson and T. Wright. 2006. *Urban Watershed Forestry Manual. Part 3: Urban Tree Planting Guide.* NA-TP-01-06. US Department of Agriculture. Forest Service. Northeastern Area. State and Private Forestry. Newtown Square, PA.

Chesapeake Stormwater Network. 2015. About. Chesapeake Stormwater Network, Ellicott City, MD.

Green Building Alliance. 2013. *Stormwater Planters.* Green Building Alliance. Pittsburgh, PA.

Georgia Department of Transportation. 2014. *Manual on Drainage Design for Highways.* Atlanta, GA.

Knox County, Tennessee. 2008. Knox County *Tennessee Stormwater Management Manual.* Knox County, Tennessee.

Maryland Department of the Environment (MDE). 2009. *Environmental Site Design, Chapter 5.* Baltimore, Maryland.

Minnesota Pollution Control Agency (MPCA). 2006. *Minnesota Stormwater Manual.* Minnesota Pollution Control Agency.

NCHRP. 2013. *Synthesis 444: Pollutant Load Reductions for Total Maximum Daily Loads for Highways.* National Cooperative Highway Research Program, Transportation Research Board, National Research Council.

Novotney, M., P. Sturm, C. Swann, and J. Tasillo. 2008. *Downspout Disconnection in the City of Baltimore, Maryland.* City of Baltimore, Maryland. Center for Watershed Protection, Ellicott City, MD.

Oregon Department of Transportation. *Appendix E – Bioslopes.* GDOT, Salem, Oregon.

Pennsylvania Department of Environmental Protection. 2006. *Pennsylvania Stormwater Best Management Practices Manual.* Pennsylvania DEP.

Schueler, T., D. Hirschman, M. Novotney, and J. Zielinski. 2007. *Urban Stormwater Retrofit Practices. Manual 3: Urban Subwatershed Restoration Manual Series.* Center for Watershed Protection (CWP). Ellicott City, MD.

United States Environmental Protection Agency. 2014. *National Menu of Stormwater Best Management Practices.* United States Environmental Protection Agency. Washington, D.C.

Washington State Department of Transportation. 2006. *Technology Evaluation and Engineering Report.* Washington State Department of Transportation (WSDOT).

West Virginia Department of Environmental Protection. 2015. *West Virginia Stormwater Management and Design Guidance Manual.* West Virginia Department of Environmental Protection.

# 5. Stormwater Drainage System Design

## 5.1 Stormwater Drainage Design Overview

### 5.1.1 Stormwater Drainage System Design

#### 5.1.1.1 INTRODUCTION

Stormwater drainage design is an integral component of both site and overall stormwater management design. Good drainage design must strive to maintain compatibility and minimize interference with existing drainage patterns; control flooding of property, structures and roadways for design flood events; and minimize potential environmental impacts on stormwater runoff.

Stormwater collection systems must be designed to provide adequate surface drainage while at the same time meeting other stormwater management goals such as runoff reduction, water quality, streambank channel protection, habitat protection and groundwater recharge.

#### 5.1.1.2 DRAINAGE SYSTEM COMPONENTS

In every location there are two stormwater drainage systems, the minor system and the major system. Three considerations largely shape the design of these systems: flooding, public safety and water quality.

The minor drainage system is designed to remove stormwater from areas such as streets and sidewalks for public safety reasons. The minor drainage system consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, and small underground pipe systems which collect stormwater runoff and transport it to structural control facilities, pervious areas and/or the major drainage system (i.e., natural waterways, large man-made conduits, and large water impoundments).

Paths taken by runoff from very large storms are called major systems. The major system (designed for the less frequent storm up to the 100-yr level) consists of natural waterways, large man-made conduits, and large water impoundments. In addition, the major system includes some less obvious drainageways such as overload relief swales and infrequent temporary ponding areas. The major system includes not only the trunk line system that receives the water from the minor system, but also the natural backup system which functions in case of overflow from or failure of the minor system. Overland relief must not flood or damage houses, buildings or other property.

The major/minor concept may be described as a 'system within a system' for it comprises two distinct but conjunctive drainage networks. The major and minor systems are closely interrelated, and their design needs to be done in tandem and in conjunction with the design of structural stormwater controls and the overall stormwater management concept and plan.

This chapter is intended to provide design criteria and guidance on several drainage system components, including street and roadway gutters, inlets and storm drain pipe systems (Section 5.2); culverts (Section 5.3); vegetated and lined open channels (Section 5.4); and energy dissipation devices for outlet protection (Section 5.5). The rest of this section covers important considerations to keep in mind in the planning and design of stormwater drainage facilities.

#### 5.1.1.3 CHECKLIST FOR DRAINAGE PLANNING AND DESIGN

The following is a general procedure for drainage system design on a development site.

1. Analyze topography
   a. Check off-site drainage pattern. Where is water coming onto the site? Where is water leaving the site?
   b. Check on-site topography for surface runoff and storage, and infiltration
      1. Determine runoff pattern; high points, ridges, valleys, streams, and swales. Where is the water going?
      2. Overlay the grading plan and indicate watershed areas; calculate square footage (acreage), points of concentration, low points, etc.
   c. Check potential drainage outlets and methods
      1. On-site (structural control, receiving water)
      2. Off-site (highway, storm drain, receiving water, regional control)
      3. Natural drainage system (swales)
      4. Existing drainage system (drain pipe)

BACK TO TOC

2. Analyze other site conditions.
   a. Land use and physical obstructions such as walks, drives, parking, patios, landscape edging, fencing, grassed area, landscaped area, tree roots, etc.
   b. Soil type determines the amount of water that can be absorbed by the soil.
   c. Vegetative cover will determine the amount of slope possible without erosion.
3. Analyze areas for probable location of drainage structures and facilities.
4. Identify the type and size of drainage system components that are required. Design the drainage system and integrate with the overall stormwater management system and plan.

## 5.1.2 Key Issues in Stormwater Drainage Design

### 5.1.2.1 INTRODUCTION

The traditional design of stormwater drainage systems has been to collect and convey stormwater runoff as rapidly as possible to a suitable location where it can be discharged. This Manual takes a different approach wherein the design methodologies and concepts of drainage design are to be integrated with the objectives for water quantity and quality control in the stormwater management minimum standards. This means that:

- Stormwater conveyance systems are to remove water efficiently enough to meet flood protection criteria and level of service requirements, and
- These systems are to complement the ability of the site design and structural stormwater controls to mitigate the major impacts of urban development.

*The following are some of the key issues in integrating water quantity and quality control consideration in stormwater drainage design.*

### 5.1.2.2 GENERAL DRAINAGE DESIGN CONSIDERATIONS

- Stormwater systems should be planned and designed so as to generally conform to natural drainage patterns and discharge to natural drainage paths within a drainage basin. These natural drainage paths should be modified as necessary to contain and safely convey the peak flows generated by the development.
- Runoff must be discharged in a manner that will not cause adverse impacts on downstream properties or stormwater systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. If the developer wishes to change discharge points he or she must demonstrate that the change will not have any adverse impacts on downstream properties or stormwater systems.
- It is important to ensure that the combined minor and major system can handle blockages and flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor systems and/or major structures occurs during these periods, the risk to life and property could be significantly increased.
- In establishing the layout of stormwater networks, it is essential to ensure that flows will not discharge onto private property during flows up to the major system design capacity.

### 5.1.2.3 STREET AND ROADWAY GUTTERS

- Gutters are efficient flow conveyance structures. This is not always an advantage if removal of pollutants and reduction of runoff is an objective. Therefore, impervious surfaces should be disconnected hydrologically where possible and runoff should be allowed to flow across pervious surfaces or through grass channels. Gutters should be used only after other options have been investigated and only after runoff has had as much chance as possible to infiltrate and filter through vegetated areas.
- It may be possible not to use gutters at all, or to modify them to channel runoff to off-road pervious areas or open channels. For example, curb opening type designs take roadway runoff to smaller feeder grass channels. Care should be taken not to create erosion problems in off-road areas. Protection during construction, establishment of strong stands of grass, and active maintenance may be necessary in some areas.



BACK TO TOC

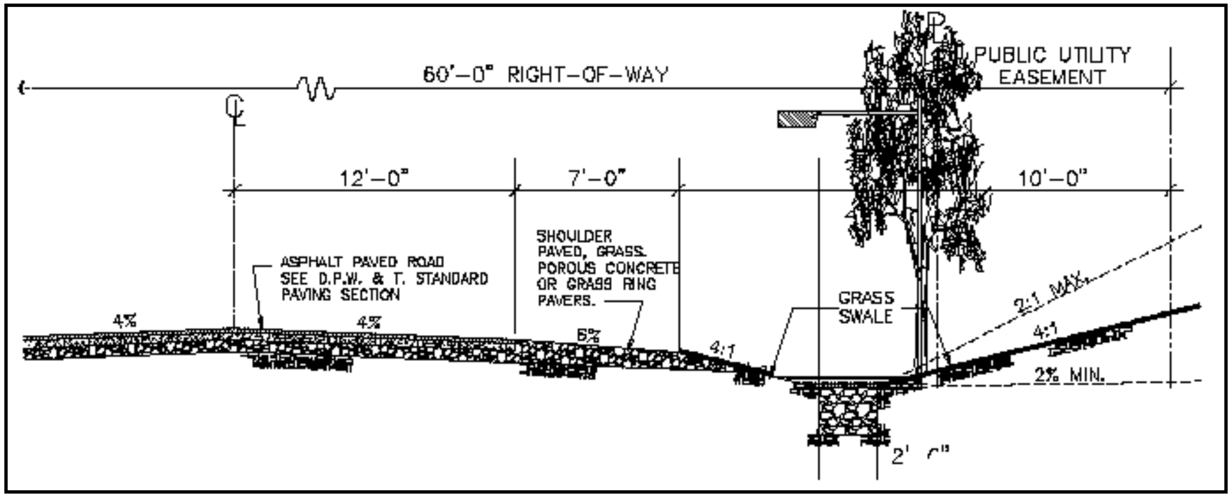



**Figure 5.1-1** Alternate Roadway Section without Gutters
(Source: Prince George's County, MD, 1999)

- Use road cross sections that include grass channels or swales instead of gutters to provide for pollution reduction and reduce the impervious area required. **Figure 5.1-1** illustrates a roadway cross section that eliminates gutters for residential neighborhoods. Flow can also be directed to center median strips in divided roadway designs. To protect the edge of pavement, ribbons of concrete can be used along the outer edges of asphalt roads.

#### 5.1.2.4 INLETS AND DRAINS

- Inlets should be located to maximize overland flow path, take advantage of pervious areas, and seek to maximize vegetative filtering and infiltration. For example, it might be possible to design a parking lot so that water flows into vegetated areas prior to entering the nearest inlet.

- Inlet location should not compromise safety or aesthetics. It should not allow for standing water in areas of vehicular or pedestrian traffic, but should take advantage of natural depression storage where possible.

- Inlets should be located to serve as overflows for structural stormwater controls. For example, a bioretention device in a commercial area could be designed to overflow to a catch basin for larger storm events.

- The choice of inlet type should match its intended use. A sumped inlet may be more effective supporting water quality objectives.

- Use several smaller inlets instead of one large inlet in order to:

  1. Prevent erosion on steep landscapes by intercepting water before it accumulates too much volume and velocity.
  2. Provide a safety factor. If a drain inlet clogs, the other surface drains may pick up the water.
  3. Improve aesthetics. Several smaller drains will be less obvious than one large drain.
  4. Spacing smaller drain inlets will give surface runoff a better chance of reaching the drain. Water will have farther to travel to reach one large drain inlet.

#### 5.1.2.5 STORM DRAIN PIPE SYSTEMS (STORM SEWERS)

- The use of better site design practices (and corresponding site design credits) should be considered to reduce the overall length of a piped stormwater conveyance system.

- Shorter and smaller conveyances can be designed to carry runoff to nearby holding areas, natural conservation areas, or filter strips (with spreaders at the end of the pipe).

- Ensure that storms in excess of pipe design flows can be safely conveyed through a development without damaging structures or flooding major roadways. This is often done through design of both a major and minor drainage system. The minor (piped) system carries the mid-frequency design flows while larger runoff events may flow across lots and along streets as long as it will not cause property damage or impact public safety.

### 5.1.2.6 CULVERTS

- Culverts can serve double duty as flow retarding structures in grass channel design. Care should be taken to design them as storage control structures if depths exceed several feet, and to ensure safety during flows.
- Improved inlet designs can absorb considerable slope and energy for steeper sloped designs, thus helping to protect channels.

### 5.1.2.7 OPEN CHANNELS

- Open channels provide opportunities for reduction of flow peaks and pollution loads. They may be designed as wet or dry enhanced swales or grass channels.
- Channels can be designed with natural meanders improving both aesthetics and pollution removal through increase of contact time.
- Grass channels generally provide better habitat than hardened channel sections, though studies have shown that riprap interstices provide significant habitat as well. Velocities should be carefully checked at design flows and the outer banks at bends should be specifically designed for increased shear stress.
- Compound sections can be developed that carry the annual flow in the lower section and higher flows above them. **Figure 5.1-2** illustrates a compound section that carries the 2-year and 10-year flows within banks. This reduces channel erosion at lower flows, and meandering, self-forming low flow channels that attack banks. The shelf in the compound section should have a minimum 1:12 slope to ensure drainage.

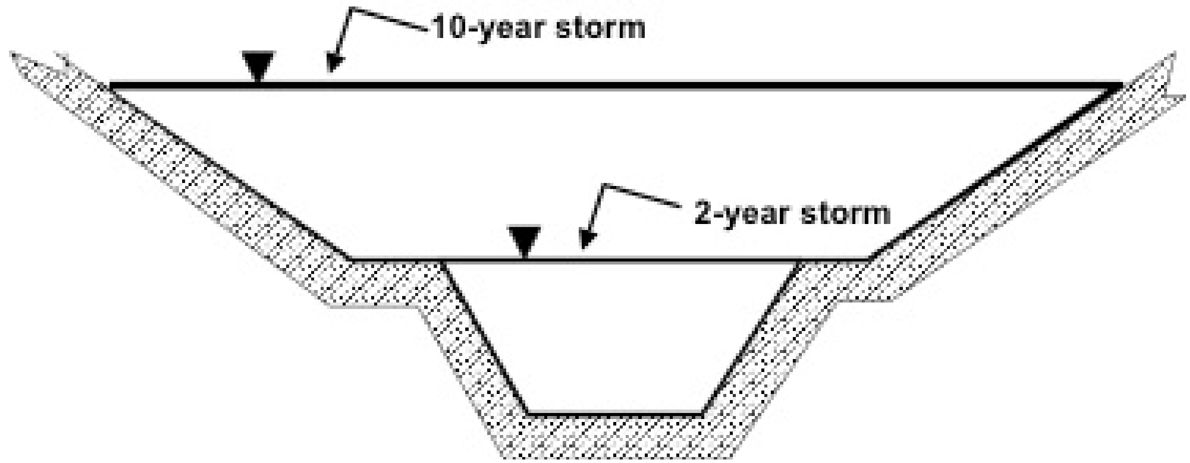



**Figure 5.1-2** Compound Channel

- Flow control structures can be placed in the channels to increase residence time. Higher flows should be calculated using a channel slope that goes from the top of the cross piece to the next one if it is significantly different from the channel bottom for normal depth calculations. Channel slope stability can also be ensured through the use of grade control structures that can serve as pollution reduction enhancements if they are set above the channel bottom. Regular maintenance is necessary to remove sediment and keep the channels from aggrading and losing capacity for larger flows.

### 5.1.2.8 ENERGY DISSIPATORS

- Energy dissipaters should be designed to return flows to non-eroding velocities to protect downstream channels.
- Care must be taken during construction that design criteria are followed exactly. The designs presented in this Manual have been carefully developed through model and full-scale tests. Each part of the criteria is important to the proper function.

### 5.1.3 Design Storm Recommendations

Listed below are the design storm recommendations for various stormwater drainage system components to be designed and constructed in accordance with the minimum stormwater management standards. Some jurisdictions may require the design of both a minor and major stormwater conveyance system, sized for two different storm frequencies. Please consult your local review authority to determine the local requirements. *It is recommended that the full build-out conditions be used to calculate flows for the design storm frequencies below.*

**Storm Drainage Systems**

Includes storm drainage systems and pipes that do not convey runoff under public roadways, sometimes called lateral closed systems.

- 10- to 25-year design storm (for pipe and culvert design)
- 10- to 25-year design storm (for inlet design)
- 50-year design storm (for sumped inlets, unless overflow facilities are provided)

**Roadway Culvert Design**

Cross drainage facilities that transport storm runoff under roadways.

- 25- to 100-year design storm, or in accordance with GDOT requirements, whichever is more stringent. (Criteria to be taken into consideration when selecting design flow include roadway type, depth of flow over road, structures and property subject to flooding, emergency access, and road replacement costs)

**Open Channel Design**

Open channels include all channels, swales, etc.

- 25-year design storm

Channels may be designed with multiple stages (e.g., a low flow channel section containing the 2-year to 5-year flows, and a high flow section that contains the design discharge) to improve stability and better mimic natural channel dimensions. Where flow easements can be obtained and structures kept clear, overbank areas may also be designed as part of a conveyance system wherein floodplain areas are designed for storage and/or conveyance of larger storms.

**Energy Dissipation Design**

Includes all outlet protection facilities.

- 25-year design storm

**Check Storm**

Used to estimate the runoff that is routed through the drainage system and stormwater management facilities to determine the effects on the facilities, adjacent property, floodplain encroachment and downstream areas.

- 100-year design storm, or as required by the Georgia Safe Dams Act.

## 5.2 Minor Drainage System Design

### 5.2.1 Overview

#### 5.2.1.1 INTRODUCTION

Minor stormwater drainage systems, also known as convenience systems, quickly remove runoff from areas such as streets and sidewalks for public safety purposes. The minor drainage system consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, and small underground pipe systems which collect stormwater runoff and transport it to structural control facilities, pervious areas and/or the major drainage system (i.e., natural waterways, large man-made conduits, and large water impoundments).

This section is intended to provide criteria and guidance for the design of minor drainage system components including:

- Street and roadway gutters
- Stormwater inlets
- Storm drain pipe systems

Ditch, channel and swale design criteria and guidance are covered in Section 5.4, *Open Channel Design*.

Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for grate, curb and combination inlets are based on information contained in HEC-12 (USDOT, FHWA, 1984). Storm drain system design is based on the use of the Rational Formula.

#### 5.2.1.2 GENERAL CRITERIA

**Design Frequency**

See Section 5.1 or the local review authority for design storm requirements for the sizing of minor storm drainage system components.

**Flow Spread Limits**

Catch basins shall be spaced so that the spread in the street for the 25-year design flow shall not exceed the following, as measured from the face of the curb:

- 8 feet if the street is classified as a Collector or Arterial street (for 2-lane streets spread may extend to one-half of the travel lane; for 4-lane streets spread may extend across one travel lane)
- 16 feet at any given section, but in no case greater than 10 feet on one side of the street, if the street is classified as a Local or Sub-Collector street

### 5.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in **Table 5.2-1** will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 5.2-1 Symbols and Definitions

| Symbol | Definition | Units |
|---|---|---|
| a | Gutter depression | in |
| A | Area of cross section | $ft^2$ |
| d or D | Depth of gutter flow at the curb line | ft |
| D | Diameter of pipe | ft |
| $E_O$ | Ratio of frontal flow to total gutter flow $Q_w/Q$ | - |
| g | Acceleration due to gravity (32.2 $ft/s^2$) | $ft/s^2$ |
| h | Height of curb opening inlet | ft |
| H | Head Loss | ft |
| K | Loss coefficient | - |
| L or $L_T$ | Length of curb opening inlet | ft |
| L | Pipe length | ft |
| n | Roughness coefficient in the modified Manning's formula for triangular gutter flow | - |
| P | Perimeter of grate opening, neglecting bars and side against curb | ft |
| Q | Rate of discharge in guttter | cfs |
| $Q_i$ | Intercepted flow | cfs |
| $Q_s$ | Gutter capacity above the depressed section | cfs |
| S or $S_X$ | Cross Slope - Traverse Slope | ft/ft |
| S or $S_L$ | Longitudinal slope of pavement | ft/ft |
| $S_f$ | Friction slope | ft/ft |
| $S_W$ | Depression section slope | ft/ft |
| T | Top width of water surface (spread on pavement) | ft |
| $T_S$ | Spread above depressed section | ft |
| V | Velocity of flow | ft/s |
| W | Width of depression for curb opening inlets | ft |
| Z | T/d, reciprocal of the the cross sope | - |

BACK TO TOC

## 5.2.3 Street and Roadway Gutters

Effective drainage of street and roadway pavements is essential to the maintenance of the roadway service level and to traffic safety. Water on the pavement can interrupt traffic flow, reduce skid resistance, increase potential for hydroplaning, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity. The design of these elements is dependent on storm frequency and the allowable spread of stormwater on the pavement surface.

This section presents design guidance for gutter flow hydraulics originally published in HEC-12, Drainage of Highway Pavements and AASHTO's Model Drainage Manual.

### 5.2.3.1 FORMULA

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = [0.56 / n] \; S_x^{5/3} \; S^{1/2} \; T^{8/3} \qquad (5.2.1)$$

*Where*:

**Q**= gutter flow rate, cfs
**$S_x$** =pavement cross slope, ft/ft
**n** = Manning's roughness coefficient
**S** = longitudinal slope, ft/ft
**T** = width of flow or spread, ft

### 5.2.3.2 NOMOGRAPH

**Figure 5.2-1** is a nomograph for solving **Equation 5.2.1**. Manning's n values for various pavement surfaces are presented in **Table 5.2-2** below.

### 5.2.3.3 MANNING'S N TABLE

Table 5.2-2 Manning's n Values for Street and Pavement Gutters

| Type of Gutter or Pavement | Range of Manning's n |
|---|---|
| Concrete gutter, trowled finish | 0.012 |
| Ashphalt pavement: | |
| Smooth Texture | 0.013 |
| Rough Texture | 0.016 |
| Concrete gutter with asphalt pavement: | |
| Smooth | 0.013 |
| Rough | 0.015 |
| Concrete pavement: | |
| Float Finish | 0.014 |
| Broom Finish | 0.016 |
| For gutters with small slopes, where sediment may accumulate, increase above values of n by | 0.002 |

Note: Estimates are by the Federal Highway Administration
Source: USDOT, FHWA, HDS-3 (1961)

### 5.2.3.4 UNIFORM CROSS SLOPE

The nomograph in **Figure 5.2-1** is used with the following procedures to find gutter capacity for uniform cross slopes:

Condition 1: Find spread, given gutter flow.

(Step 1) Determine input parameters, including longitudinal slope (S), cross slope ($S_x$), gutter flow (Q), and Manning's n.

(Step 2) Draw a line between the S and $S_x$ scales and note where it intersects the turning line.

(Step 3) Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the product of Q and n.

(Step 4) Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

BACK TO TOC

**Condition 2**: Find gutter flow, given spread.

(Step 1) Determine input parameters, including longitudinal slope (S), cross slope ($S_x$), spread (T), and Manning's n.

(Step 2) Draw a line between the S and $S_x$ scales and note where it intersects the turning line.

(Step 3) Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or $Q_n$ from the intersection of that line on the capacity scale.

(Step 4) For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times n ($Q_n$) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

### 5.2.3.5 COMPOSITE GUTTER SECTIONS

**Figure 5.2-2** in combination with **Figure 5.2-1** can be used to find the flow in a gutter with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

**Figure 5.2-3** provides a direct solution of gutter flow in a composite gutter section. The flow rate at a given spread or the spread at a known flow rate can be found from this figure. **Figure 5.2-3** involves a complex graphical solution of the equation for flow in a composite gutter section. Typical of graphical solutions, extreme care in using the figure is necessary to obtain accurate results.

**Condition 1**: Find spread, given gutter flow.

(Step 1) Determine input parameters, including longitudinal slope (S), cross slope ($S_x$), depressed section slope ($S_w$), depressed section width (W), Manning's n, gutter flow (Q), and a trial value of gutter capacity above the depressed section ($Q_s$).

(Step 2) Calculate the gutter flow in W ($Q_w$), using the equation:

$$Q_w = Q - Q_s \qquad (5.2.2)$$

(Step 3) Calculate the ratios $Q_w/Q$ or $E_o$ and $S_w/S_x$ and use **Figure 5.2-2** to find an appropriate value of W/T.

(Step 4) Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3.

(Step 5) Find the spread above the depressed section ($T_s$) by subtracting W from the value of T obtained in Step 4.

(Step 6) Use the value of Ts from Step 5 along with Manning's n, S, and $S_x$ to find the actual value of $Q_s$ from **Figure 5.2-1**.

(Step 7) Compare the value of $Q_s$ from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Qs and return to Step 1.

**Condition 2**: Find gutter flow, given spread.

(Step 1) Determine input parameters, including spread (T), spread above the depressed section ($T_s$), cross slope ($S_x$), longitudinal slope (S), depressed section slope ($S_w$), depressed section width (W), Manning's n, and depth of gutter flow (d).

(Step 2) Use **Figure 5.2-1** to determine the capacity of the gutter section above the depressed section ($Q_s$). Use the procedure for uniform cross slopes, substituting $T_s$ for T.

(Step 3) Calculate the ratios W/T and $S_w/S_x$, and, from **Figure 5.2-2**, find the appropriate value of Eo (the ratio of $Q_w/Q$).

(Step 4) Calculate the total gutter flow using the equation:

$$Q = Q_s / (1 - E_o) \qquad (5.2.3)$$

*Where*:

**Q** = gutter flow rate, cfs

$\mathbf{Q_s}$ = flow capacity of the gutter section above the depressed section, cfs

$\mathbf{E_o}$ = ratio of frontal flow to total gutter flow ($Q_w/Q$)

(Step 5) Calculate the gutter flow in width (W), using **Equation 5.2.2**.



BACK TO TOC

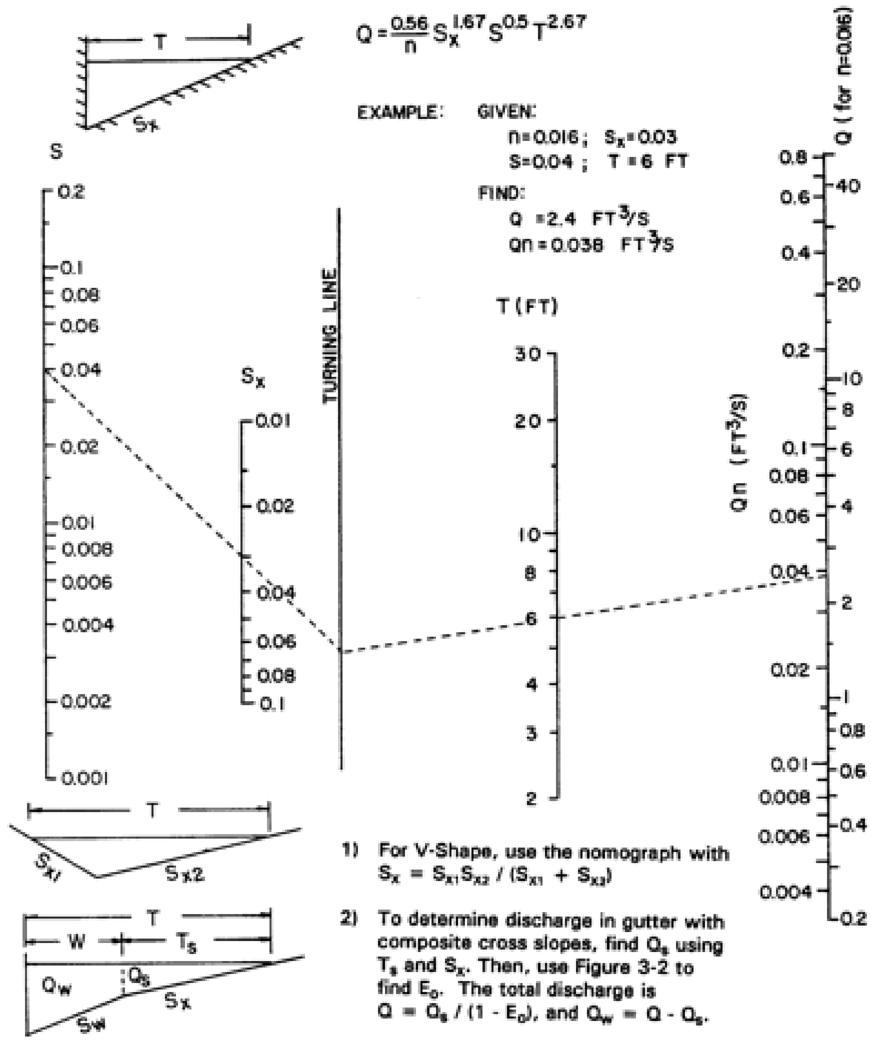



**Figure 5.2-1** Flow in Triangular Gutter Sections
(Source: AASHTO Model Drainage Manual, 2005)

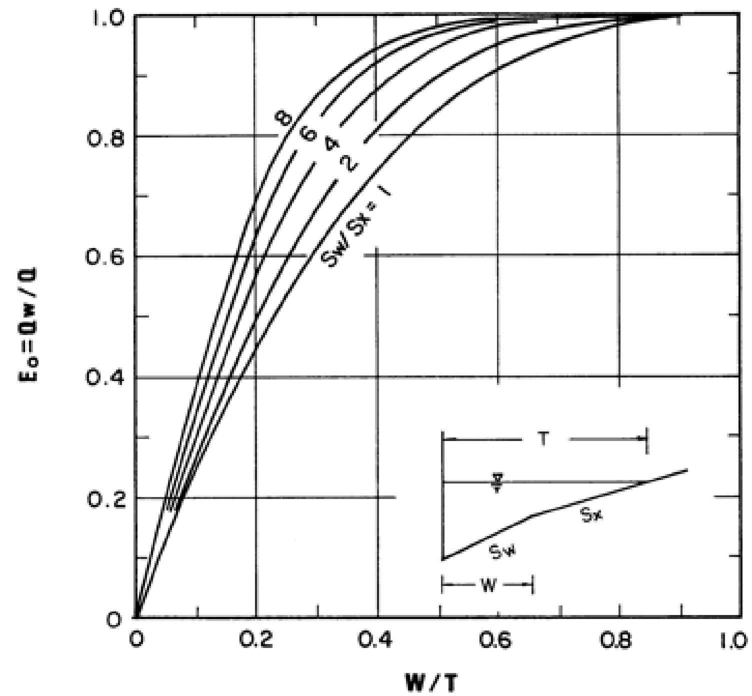



**Figure 5.2-2** Ratio of Frontal Flow to Total Gutter Flow
(Source: AASHTO Model Drainage Manual, 2005)

### 5.2.3.6 EXAMPLES

Example 1

*Given*: **T** = 8 ft   $\mathbf{S_x}$ = 0.025 ft/ft

**n** = 0.015   **S** = 0.01 ft/ft

*Find*: (a) Flow in gutter at design spread

(b) Flow in width (W = 2 ft) adjacent to the curb

*Solution*:

(a) From **Figure 5.2-1**, Qn = 0.03

Q = Qn/n = 0.03/0.015 = 2.0 cfs

(b) T = 8 - 2 = 6 ft

$(Qn)_2$ = 0.014 (**Figure 5.2-1**)

(flow in 6-foot width outside of width (W))

Q = 0.014/0.015 = 0.9 cfs

$Q_w$ = 2.0 - 0.9 = 1.1 cfs

Flow in the first 2 ft adjacent to the curb is 1.1 cfs and 0.9 cfs in the remainder of the gutter.

Example 2

*Given*:

$\mathbf{T_s}$ = 6 - 1.5 = 4.5 ft   **T** = 6 ft

$\mathbf{S_w}$ = 0.0833 ft/ft   **W** = 1.5 ft

$\mathbf{S_x}$ = 0.03 ft/ft   **n** = 0.014

**S** = 0.04 ft/ft

*Find*: Flow in the composite gutter

*Solution*:

(1) Use **Figure 5.2-1** to find the gutter section capacity above the depressed section.

$Q_s n$ = 0.038

$Q_s$ = 0.038/0.014 = 2.7 cfs

(2) Calculate W/T = 1.5/6 = 0.25 and $S_w/S_x$ = 0.0833/0.03 = 2.78

Use **Figure 5.2-2** to find

$E_o$ = 0.64

(3) Calculate the gutter flow using **Equation 5.4.2.3:**

**Q = 2.7/(1 - 0.64) = 7.5 cfs**

(4) Calculate the gutter flow in width, W, using **Equation 5.2.2:**

$\mathbf{Q_w = 7.5 - 2.7 = 4.8}$ **cfs**

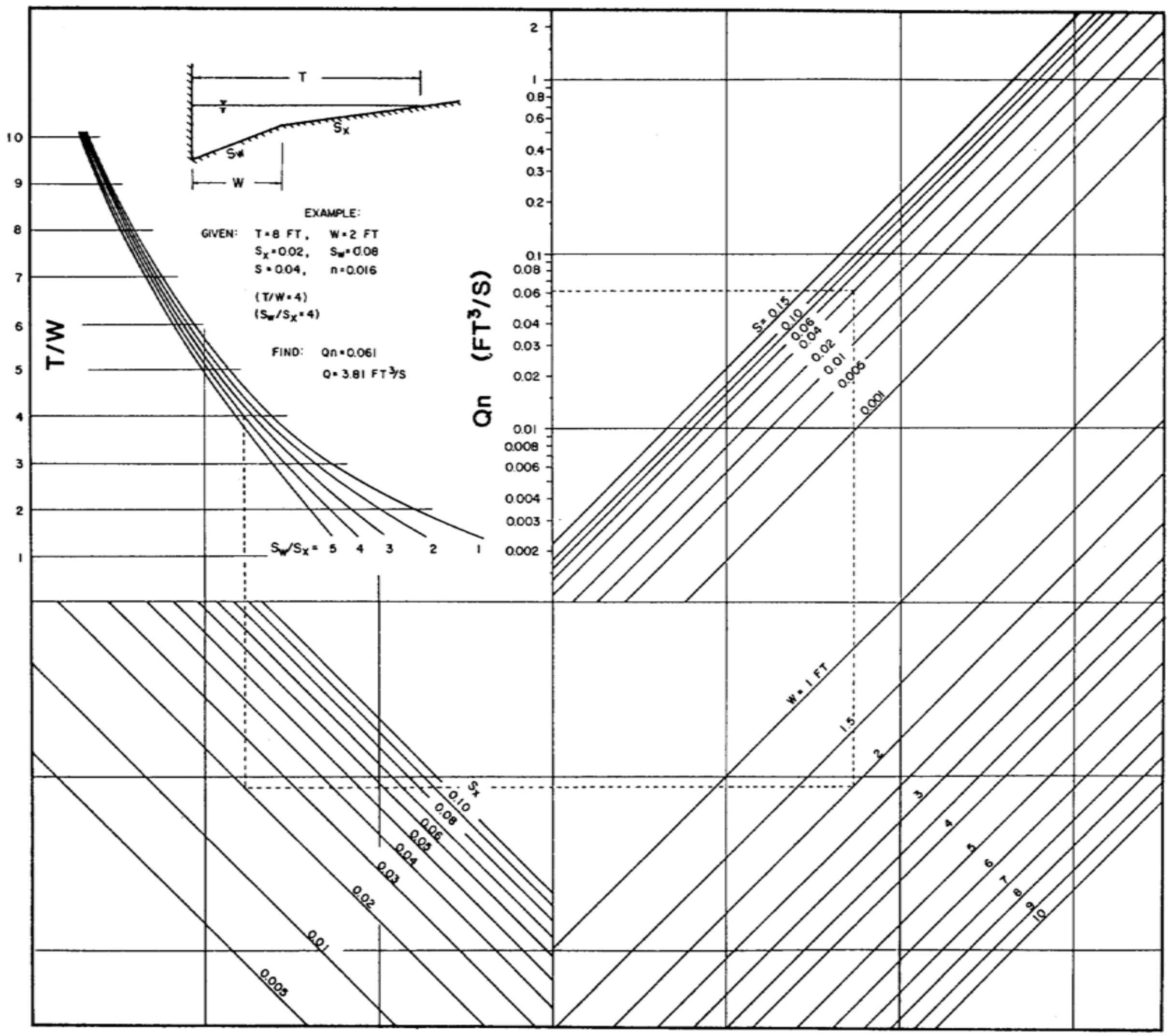



Figure 5.2-3 Flow in Composite Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

## 5.2.4 Stormwater Inlets

Inlets are drainage structures used to collect surface water through grate or curb open-ings and convey it to storm drains or direct outlet to culverts. Grate inlets subject to traffic should be bicycle safe and be load-bearing adequate. Appropriate frames should be provided.

Inlets used for the drainage of highway surfaces can be divided into three major classes:

1. Grate Inlets – These inlets include grate inlets consisting of an opening in the gutter covered by one or more grates, and slotted inlets consisting of a pipe cut along the longitudinal axis with a grate or spacer bars to form slot openings.
2. Curb-Opening Inlets – These inlets are vertical openings in the curb covered by a top slab.
3. Combination Inlets – These inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.

Inlets may be classified as being on a continuous grade or in a sump. The term "continuous grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sump" condition exists when the inlet is located at a low point and water enters from both directions.

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

The design of grate inlets will be discussed in Section 5.2.5, curb inlet design in Section 5.2.6, and combination inlets in Section 5.2.7.

## 5.2.5 Grate Inlet Design

### 5.2.5.1 GRATE INLETS ON GRADE

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is small.

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. They also handle debris better than other grate inlets but the vanes of the grate must be turned in the proper direction.

Table 5.2-3 Grate Debris Handling Efficiencies

| Rank | Grate | Longitudinal Slope | |
|---|---|---|---|
| | | (0.005) | (0.04) |
| 1 | CV - 3-1/4 - 4-1/4 | 46 | 61 |
| 2 | 30 - 3-1/4 - 4 | 44 | 55 |
| 3 | 45 - 3-1/4 - 4 | 43 | 48 |
| 4 | P - 1-7/8 | 32 | 32 |
| 5 | P - 1-7/8 - 4 | 18 | 28 |
| 6 | 45 - 2-1/4 - 4 | 16 | 23 |
| 7 | Recticuline | 12 | 16 |
| 8 | P - 1-1/8 | 9 | 20 |

Source: "Drainage of Highway Pavements" (HEC-12), Federal Highway Administration, 1984.

Where debris is a problem, consideration should be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was made to qualita-tively simulate field conditions. **Table 5.2-3** presents the results of debris handling efficiencies of several grates.

The ratio of frontal flow to total gutter flow, $E_o$, for straight cross slope is ex-pressed by the following equation:

$$E_o = Q_w/Q = 1 - (1 - W/T)^{2.67} \qquad (5.2.4)$$

*Where*:

**Q** = total gutter flow, cfs
$Q_w$ = flow in width W, cfs
**W** = width of depressed gutter or grate, ft
**T** = total spread of water in the gutter, ft

**Figure 5.2-2** provides a graphical solution of $E_o$ for either depressed gutter sections or straight cross slopes. The ratio of side flow, $Q_s$, to total gutter flow is:

$$Q_s/Q = 1 - Q_w/Q = 1 - E_o \qquad (5.2.5)$$

The ratio of frontal flow intercepted to total frontal flow, $R_f$, is expressed by the following equation:

$$R_f = 1 - 0.09\ (V - V_0) \qquad (5.2.6)$$

*Where*:

**V** = velocity of flow in the gutter, ft/s
(using Q from **Figure 5.2-1**)
$V_o$ = gutter velocity where splash-over first occurs, ft/s
(from **Figure 5.2-4**)

This ratio is equivalent to frontal flow interception efficiency. **Figure 5.2-4** provides a solution of **equation 5.2.6**, which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use **Figure 5.2-4** is total gutter flow divided by the area of flow. The ratio of side flow intercepted to total side flow, Rs, or side flow interception efficiency, is expressed by:

$$Rs = 1 / [1 + (0.15V^{1.8}/S_xL^{2.3})] \qquad (5.2.7)$$

*Where*:

**L** = length of the grate, ft
$S_x$ = pavement cross slope, ft/ft

**Figure 5.2-5** provides a solution to equation **5.2.7**.

The efficiency, E, of a grate is expressed as:

$$E = R_fE_o + R_s(1 - E_o) \qquad (5.2.8)$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q[R_fE_o + R_s(1 - E_o)] \qquad (5.2.9)$$

The following example illustrates the use of this procedure.

*Given*:

**W** = 2 ft | **T** = 8 ft

**$S_x$** = 0.025 ft/ft | **S** = 0.01 ft/ft

**$E_o$** = 0.69 | **Q** = 3.0 cfs

**V** = 3.1 ft/s | **Gutter depression** = 2 in

*Find*:

Interception capacity of:

(1) a curved vane grate, and

(2) a reticuline grate 2-ft long and 2-ft wide

*Solution*:

From **Figure 5.2-4** for Curved Vane Grate, $R_f$ = 1.0

From **Figure 5.2-4** for Reticuline Grate, $R_f$ = 1.0

From **Figure 5.2-5** $R_s$ = 0.1 for both grates

From **Equation 5.2.9**:

$$Q_i = 3.0[1.0 \text{ X } 0.69 + 0.1(1 - 0.69)] = 2.2 \text{ cfs}$$

For this example, the interception capacity of a curved vane grate is the same as that for a reticuline grate for the sited conditions.

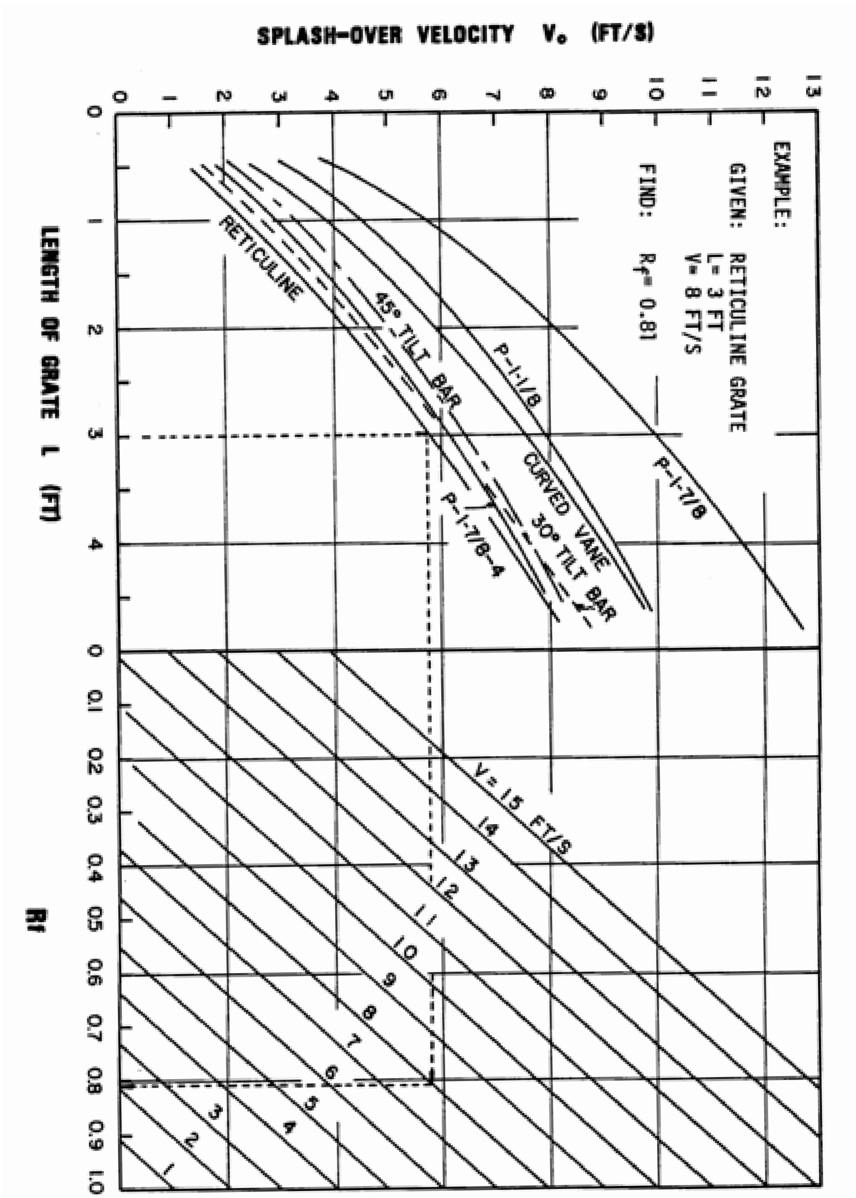



**Figure 5.2-4** Grate Inlet Frontal Flow Interception Efficiency
(Source: HEC-12, 1984)

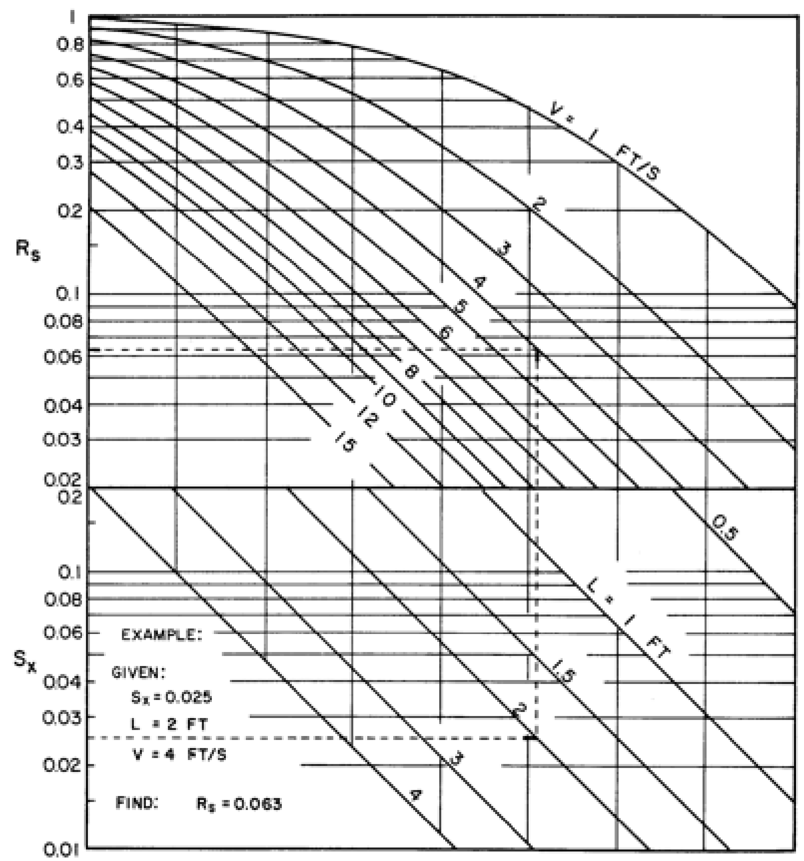



**Figure 5.2-5** Grate Inlet Side Flow Interception Efficiency
(Source: HEC-12, 1984)

BACK TO TOC

### 5.2.5.2 GRATE INLETS IN SAG

A grate inlet in a sag operates as a weir up to a certain depth, depending on the bar configuration and size of the grate, and as an orifice at greater depths. For a standard gutter inlet grate, weir operation continues to a depth of about 0.4 feet above the top of grate and when depth of water exceeds about 1.4 feet, the grate begins to operate as an orifice. Between depths of about 0.4 feet and about 1.4 feet, a transition from weir to orifice flow occurs.

The capacity of grate inlets operating as a weir is:

$$Q_i = CPd^{1.5} \qquad (5.2.10)$$

*Where*:

**P** = perimeter of grate excluding bar widths and the side against the curb, ft

**C** = 3.0

**d** = depth of water above grate, ft

and as an orifice is:

$$Q_i = CA(2gd)^{0.5} \qquad (5.2.11)$$

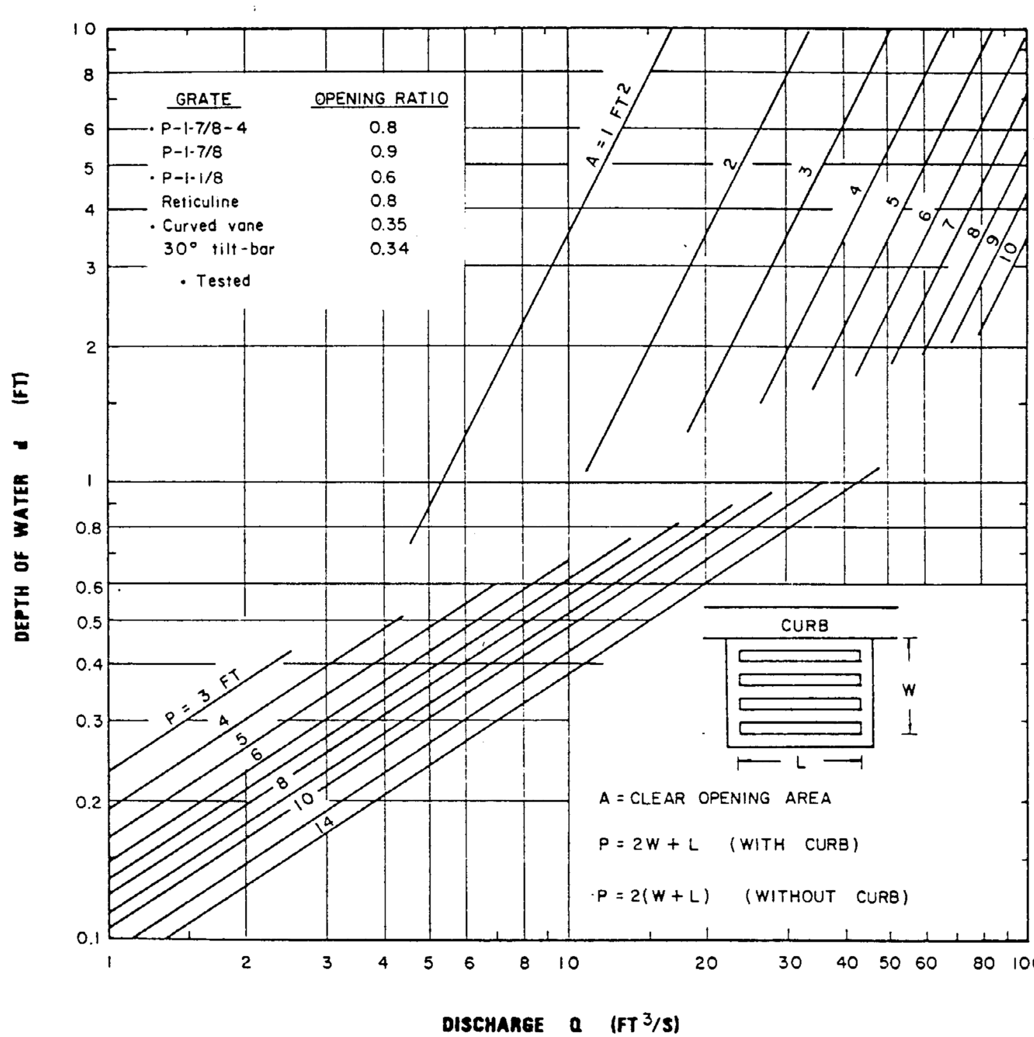
*Where*:

**C** = 0.67 orifice coefficient

**A** = clear opening area of the grate, ft$^2$

**g** = 32.2 ft/s$^2$

**Figure 5.2-6** is a plot of **equations 5.2.10** and **5.2.11** for various grate sizes. The effects of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used. The following example illustrates the use of this figure.




*Reference: USDOT, FHWA, HEC-12 (1984).*

**Figure 5.2-6** Grate Inlet Capacity in Sag Conditions
(Source: HEC-12, 1984)

BACK TO TOC

**Example**

*Given*: A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point; allow for 50% clogging of the grate.

**$Q_b$** = 3.6 cfs
**Q** = 8 cfs, 25-year storm
**T** = 10 ft, design
**$S_x$** = 0.05 ft/ft
**d** = $TS_x$ = 0.5 ft

*Find*: Grate size for design Q. Check spread at S = 0.003 on approaches to the low point.

*Solution*: From **Figure 5.2-6**, a grate must have a perimeter of 8 ft to intercept 8 cfs at a depth of 0.5 ft.

Some assumptions must be made regarding the nature of the clogging in order to com-pute the capacity of a partially clogged grate. If the area of a grate is 50% covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50%. For example if a 2-ft x 4-ft grate is clogged so that the effective width is 1 ft, then the perimeter, P = 1 + 4 + 1 = 6 ft, rather than 8 ft, the total perimeter, or 4 ft, half of the total perimeter. The area of the opening would be reduced by 50% and the perimeter by 25%.

Therefore, assuming 50% clogging along the length of the grate, a 4 x 4, a 2 x 6, or a 3 x 5 grate would meet requirements of an 8-ft perimeter 50% clogged.

Assuming that the installation chosen to meet design conditions is a double 2 x 3 ft grate, for 50% clogged conditions: P = 1 + 6 + 1 = 8 ft

For 25-year flow: d = 0.5 ft (from **Figure 5.2-6**)
The American Society of State Highway and Transportation Officials (AASHTO) geometric policy recommends a gradient of 0.3% within 50 ft of the level point in a sag vertical curve.

Check T at S = 0.003 for the design and check flow (from **Figure 5.2-1**):

Q = 3.6 cfs,
T = 8.2 ft (25-year storm)

Thus a double 2 x 3-ft grate 50% clogged is adequate to intercept the design flow at a spread that does not exceed design spread, and spread on the approaches to the low point will not exceed design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet, or curb-opening inlet in a sag where ponding can occur, and flanking inlets on the low gradient approaches.

## 5.2.6 Curb Inlet Design

### 5.2.6.1 CURB INLETS ON GRADE

Following is a discussion of the procedures for the design of curb inlets on grade. Curb-opening inlets are effective in the drainage of highway pave-ments where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of curb-opening inlet required for total interception of gutter flow on a pave-ment section with a straight cross slope is determined using **Figure 5.2-7**. The efficiency of curb-opening inlets shorter than the length required for total interception is determined using **Figure 5.2-8**.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, $S_e$, in the following equation:

$$S_e = S_x + S_w E_o \qquad (5.2.12)$$

*Where*:

$S_x$ = pavement cross slope, ft/ft

$S_w$ = gutter cross slope = (a/12W), ft/ft

a = gutter depression, in

W = gutter width, ft

$E_o$ = ratio of flow in the depressed section to total gutter flow

It is apparent from examination of **Figure 5.2-7** that the length of curb open-ing required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

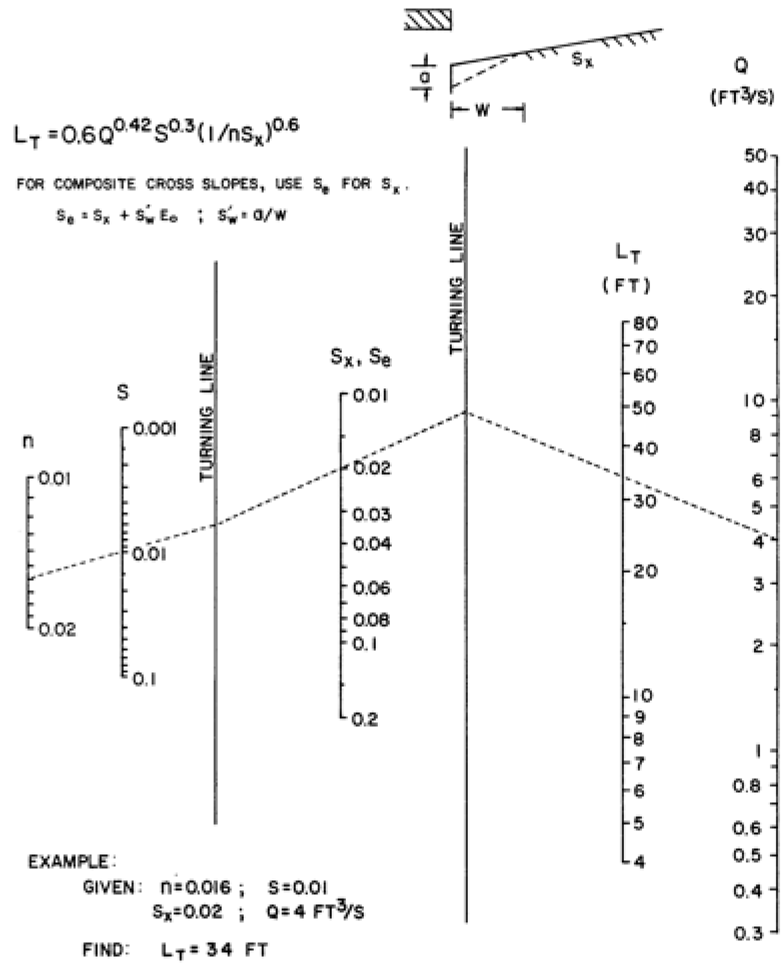



**Figure 5.2-7** Curb-Opening and Slotted Drain Inlet Length for Total Interception
(Source: HEC-12, 1984)

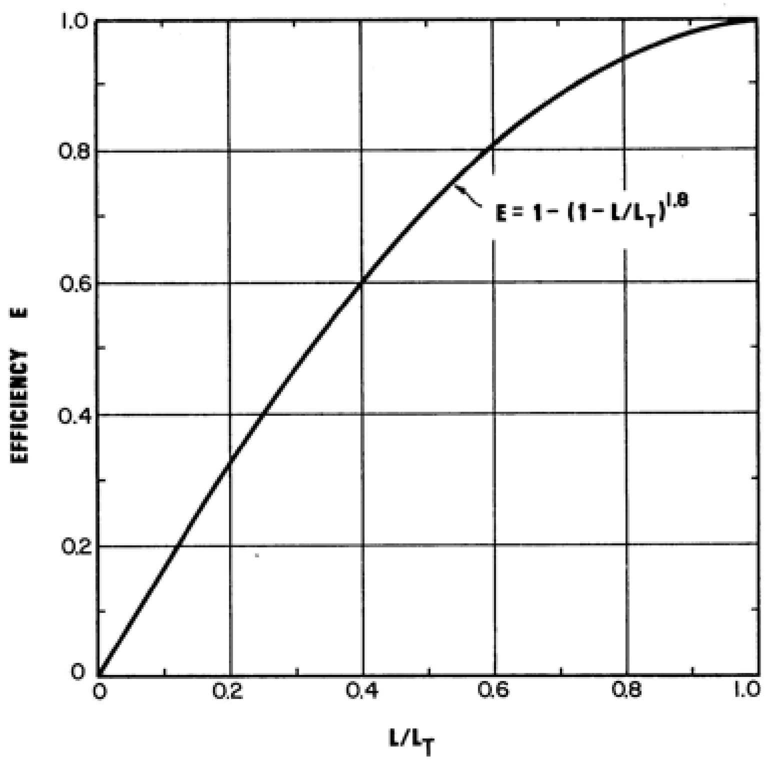



**Figure 5.2-8** Curb-Opening and Slotted Drain Inlet Interception Efficiency
(Source: HEC-12, 1984)

### Design Steps

Steps for using **Figures 5.2-7** and **5.2-8** in the design of curb inlets on grade are given below.

(Step 1) Determine the following input parameters:
Cross slope = Sx (ft/ft)
Longitudinal slope = S (ft/ft)
Gutter flow rate = Q (cfs)
Manning's n = n
Spread of water on pavement = T (ft) from **Figure 5.2-1**

(Step 2) Enter **Figure 5.2-7** using the two vertical lines on the left side labeled n and S. Locate the value for Manning's n and longitudinal slope and draw a line connecting these points and extend this line to the first turning line.

(Step 3) Locate the value for the cross slope (or equivalent cross slope) and draw a line from the point on the first turning line through the cross slope value and extend this line to the second turning line.

(Step 4) Using the far right vertical line labeled Q locate the gutter flow rate. Draw a line from this value to the point on the second turning line. Read the length re-quired from the vertical line labeled $L_T$.

(Step 5) If the curb-opening inlet is shorter than the value obtained in Step 4, **Figure 5.2-8** can be used to calculate the efficiency. Enter the x-axis with the $L/L_T$ ratio and draw a vertical line upward to the E curve. From the point of intersection, draw a line horizontally to the intersection with the y-axis and read the efficiency value.

### Example

*Given*:

$S_x$ = 0.03 ft/ft n = 0.016
S = 0.035 ft/ft Q = 5 cfs
$S_w$ = 0.083 (a = 2 in, W = 2 ft)

*Find*:

(1) $Q_i$ for a 10-ft curb-opening inlet
(2) $Q_i$ for a depressed 10-ft curb-opening inlet with a = 2 in, W = 2 ft, T = 8 ft (**Figure 5.2-1**)

*Solution*:

(1) From **Figure 5.2-7**, LT = 41 ft, L/LT = 10/41 = 0.24
From Figure 5.2-8, E = 0.39, Qi = EQ = 0.39 x 5 = 2 cfs
(2) Qn = 5.0 x 0.016 = 0.08 cfs

$S_w/S_x$ = (0.03 + 0.083)/0.03 = 3.77
T/W = 3.5 (from **Figure 5.2-3**)
T = 3.5 x 2 = 7 ft
W/T = 2/7 = 0.29 ft
$E_o$ = 0.72 (from **Figure 5.2-2**)

Therefore, $S_e = S_x + S_wE_o$ = 0.03 + 0.083(0.72) = 0.09

From **Figure 5.2-7**, LT = 23 ft, L/LT = 10/23 = 0.4
From **Figure 5.2-8**, E = 0.64, $Q_i$ = 0.64 x 5 = 3.2 cfs

The depressed curb-opening inlet will intercept 1.6 times the flow intercepted by the undepressed curb opening and over 60% of the total flow.

#### 5.2.6.2 CURB INLETS IN SUMP

For the design of a curb-opening inlet in a sump location, the inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The capacity of curb-opening inlets in a sump location can be determined from **Figure 5.2-9**, which accounts for the operation of the inlet as a weir and as an orifice at depths greater than 1.4h. This figure is applicable to depressed curb-opening inlets and the depth at the inlet includes any gutter depression. The height (h) in the figure assumes a vertical orifice opening (see sketch on **Figure 5.2-9**). The weir portion of **Figure 5.2-9** is valid for a depressed curb-opening inlet when d $\leq$ (h + a/12).

The capacity of curb-opening inlets in a sump location with a vertical orifice opening but without any depression can be determined from **Figure 5.2-10**. The capacity of curb-opening inlets in a sump location with other than vertical orifice openings can be determined by using **Figure 5.2-11**.

**Design Steps**

Steps for using **Figures 5.2-9, 5.2-10**, and **5.2-11** in the design of curb-opening inlets in sump locations are given below.

(Step 1) Determine the following input parameters:
**Cross slope** = $S_x$ (ft/ft)
**Spread of water on pavement** = T (ft) from **Figure 5.2-1**
**Gutter flow rate** = Q (cfs) or dimensions of curb-opening inlet [L (ft) and H (in)]
**Dimensions of depression** if any [a (in) and W (ft)]

(Step 2) To determine discharge given the other input parameters, select the appropriate figure (**5.2-9, 5.2-10**, or **5.2-11** de-pending on whether the inlet is in a de-pression and if the orifice opening is vertical).

(Step 3) To determine the discharge (Q), given the water depth (d), locate the water depth value on the y-axis and draw a hori-zontal line to the appropriate perimeter (p), height (h), length (L), or width x length (hL) line. At this intersection draw a vertical line down to the x-axis and read the discharge value.

(Step 4) To determine the water depth given the discharge, use the procedure described in Step 3 except enter the figure at the value for the discharge on the x-axis.

BACK TO TOC

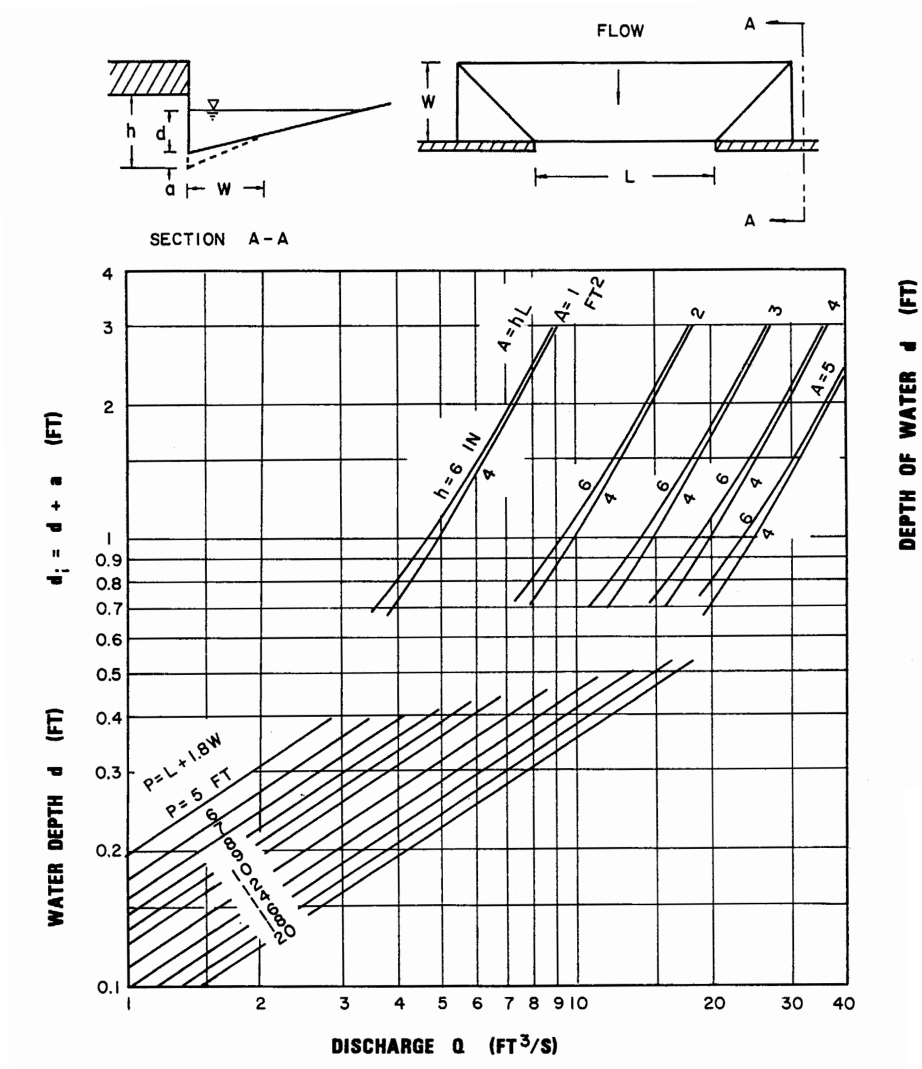



Figure 5.2-9 Depressed Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 2005)

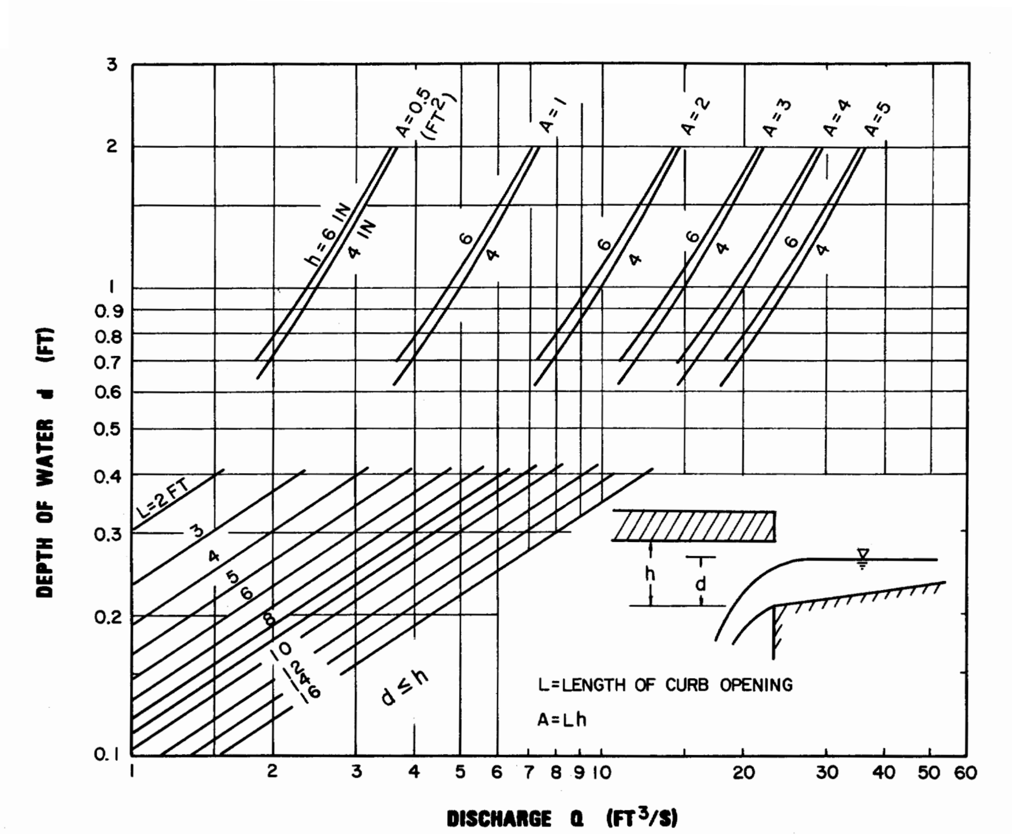



Figure 5.2-10 Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 2005)

BACK TO TOC

**Example**

*Given*: Curb-opening inlet in a sump location

$L = 5$ ft

$h = 5$ in

(1) Undepressed curb opening

$S_x = 0.05$ ft/ft

$T = 8$ ft

(2) Depressed curb opening

$S_x = 0.05$ ft/ft

$a = 2$ in

$W = 2$ ft

$T = 8$ ft

*Find*: Discharge $Q_i$

*Solution*:

(1) $d = TS_x = 8 \times 0.05 = 0.4$ ft

$d < h$

From **Figure 5.2-10**, $Q_i = 3.8$ cfs

(2) $d = 0.4$ ft

$h + a/12 = (5 + 2/12)/12 = 0.43$ ft

Since $d < 0.43$ the weir portion of **Figure 5.2-9** is applicable (lower portion of the figure).

$P = L + 1.8W = 5 + 3.6 = 8.6$ ft

From **Figure 5.2-9**, $Q_i = 5$ cfs

At $d = 0.4$ ft, the depressed curb-opening inlet has about 30% more capacity than an inlet without depression.

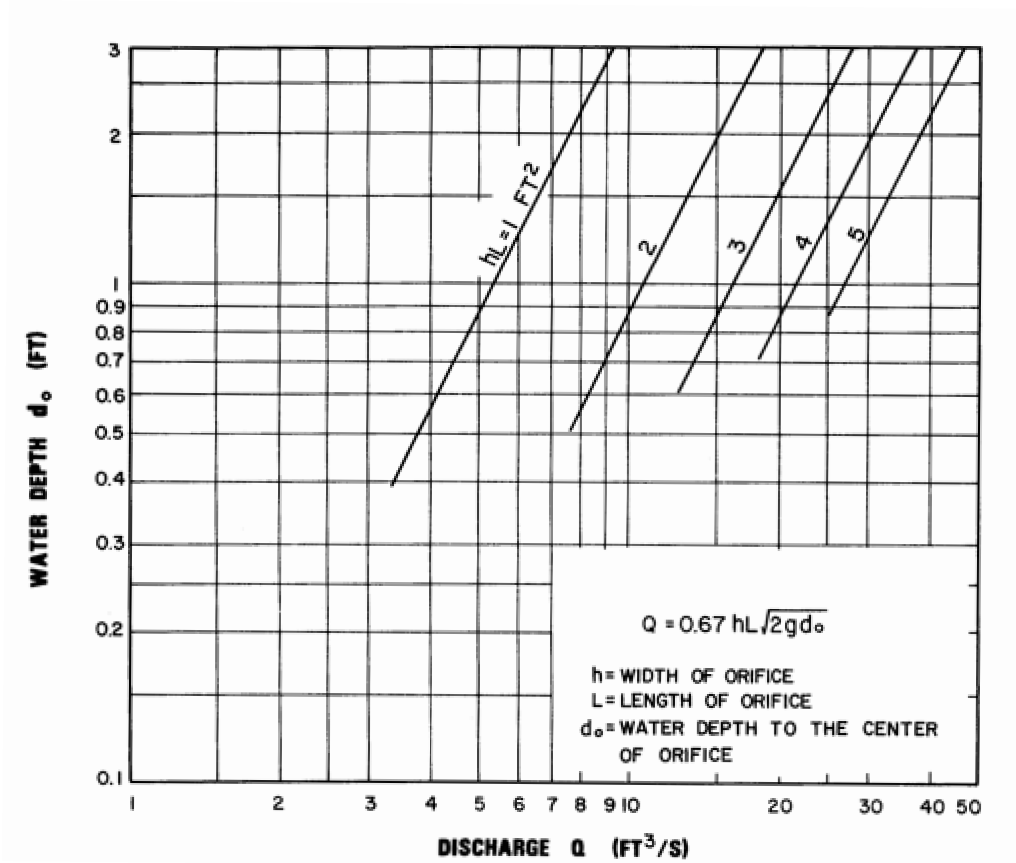



**Figure 5.2-11** Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats

(Source: AASHTO Model Drainage Manual, 2005)

## 5.2.7 Combination Inlets

### 5.2.7.1 COMBINATION INLETS ON GRADE

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus capacity is computed by neglecting the curb opening inlet and the design procedures should be followed based on the use of **Figures 5.2-4, 5.2-5** and **5.2-6**.

### 5.2.7.2 COMBINATION INLETS IN SUMP

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be con-centrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, it is generally appropriate to estimate the capacity of a combination inlet at a sump by neglecting the grate inlet capacity. Assuming complete clogging of the grate, Figures **5.2-9**, **5.2-10**, and **5.2-11** for curb-opening inlets should be used for design.

## 5.2.8 Storm Drain Pipe Systems

### 5.2.8.1 INTRODUCTION

Storm drain pipe systems, also known as storm sewers, are pipe conveyances used in the minor stormwater drainage system for transporting runoff from roadway and other inlets to outfalls at structural stormwater controls and receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainageways and/or vegetated open channels is not feasible.

### 5.2.8.2 GENERAL DESIGN PROCEDURE

The design of storm drain systems generally follows these steps:

(Step 1) Determine inlet location and spacing as outlined earlier in this section.

(Step 2) Prepare a tentative plan layout of the storm sewer drainage system including:

a. Location of storm drains
b. Direction of flow
c. Location of manholes
d. Location of existing facilities such as water, gas, or underground cables

(Step 3) Determine drainage areas and compute runoff using the Rational Method

(Step 4) After the tentative locations of inlets, drain pipes, and outfalls (including tailwaters) have been determined and the inlets sized, compute of the rate of discharge to be carried by each storm drain pipe and determine the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from upstream of a line to downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the pipe serving that discharge is sized, and the process is repeated for the next run downstream. The storm drain system design computation form (**Figure 5.2-12**) can be used to summarize hydrologic, hydraulic and design computations.

(Step 5) Examine assumptions to determine if any adjustments are needed to the final design.

It should be recognized that the rate of discharge to be carried by any particular section of storm drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. It is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

### 5.2.8.3 DESIGN CRITERIA

Storm drain pipe systems should conform to the following criteria:

- For ordinary conditions, storm drain pipes should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The Manning Formula is recommended for capacity calculations.
- The maximum hydraulic gradient should not produce a velocity that exceeds 15 ft/s.
- The minimum desirable physical slope should be 0.5% or the slope that will produce a velocity of 2.5 feet per second when the storm sewer is flowing full, whichever is greater.
- If the potential water surface elevation exceeds 1 foot below ground elevation for the design flow, the top of the pipe, or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the hydraulic grade line.

BACK TO TOC

# STORM DRAIN COMPUTATION SHEET

COMPUTED ______ DATE ______

CHECKED ______ DATE ______

ROUTE ______

SECTION ______

COUNTY ______

| STATION | | LENGTH FEET | DRAINAGE AREA "A" (ACRES) | | RUNOFF COEFFICIENT "C" | "A" • "C" | | FLOW TIME MIN. | | RAINFALL INTENSITY "I" in/hour | TOTAL RUNOFF C.I.A. = Q (c.f.s.) | DIAMETER PIPE inches | CAPACITY FULL c.f.s. | VELOCITY f.p.s. | | INVERT ELEV | | MANHOLE INVERT DROP | SLOPE OF DRAIN ft/ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| FROM | TO | | INCREMENT | TOTAL | | INCREMENT | TOTAL | TO UPPER END | IN SECTION | | | | | FLOWING FULL | DESIGN FLOW | UPPER END | LOWER END | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | |

BACK TO TOC

### 5.2.8.4 CAPACITY CALCULATIONS

**Formulas for Gravity and Pressure Flow**

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning's Formula, expressed by the following equation:

$$V = [1.486\ R^{2/3}S^{1/2}]/n \qquad (5.2.13)$$

*Where*:

**V** = mean velocity of flow, ft/s
**R** = the hydraulic radius, ft -defined as the area of flow divided by the wetted flow surface or wetted perimeter (A/WP)
**S** = the slope of hydraulic grade line, ft/ft
**n** = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$Q = [1.486\ AR^{2/3}S^{1/2}]/n \qquad (5.2.14)$$

*Where*:

**Q** = rate of flow, cfs
**A** = cross sectional area of flow, $ft^2$

For pipes flowing full, the above equations become:

$$V = [0.590\ D^{2/3}S^{1/2}]/n \qquad (5.2.15)$$

$$Q = [0.463\ D^{8/3}S^{1/2}]/n \qquad (5.2.16)$$

*Where*:

**D** = diameter of pipe, ft

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$$H_f = [2.87\ n^2V^2L]/[S^{4/3}] \qquad (5.2.17)$$

$$H_f = [29\ n^2V^2L]/[(R^{4/3})\ (2g)] \qquad (5.2.18)$$

*Where*:

$H_f$ = total head loss due to friction, ft
**n** = Manning's roughness coefficient
**D** = diameter of pipe, ft
**L** = length of pipe, ft
**V** = mean velocity, ft/s
**R** = hydraulic radius, ft
**g** = acceleration of gravity = 32.2 ft/sec$^2$

### 5.2.8.5 NOMOGRAPHS AND TABLE

The nomograph solution of Manning's formula for full flow in circular storm drain pipes is shown in **Figures 5.2-13, 5.2-14**, and **5.2-15**. **Figure 5.2-16** has been provided to solve the Manning's equation for partially full flow in storm drains.

### 5.2.8.6 HYDRAULIC GRADE LINES

All head losses in a storm sewer system are considered in computing the hydraulic grade line to determine the water surface elevations, under design conditions in the various inlets, catch basins, manholes, junction boxes, etc.

Hydraulic control is a set water surface elevation from which the hydraulic calculations are begun. All hydraulic controls along the alignment are established. If the control is at a main line upstream inlet (inlet control), the hydraulic grade line is the water surface elevation minus the entrance loss minus the difference in velocity head. If the control is at the outlet, the water surface is the outlet pipe hydraulic grade line.

### Design Procedure - Outlet Control

The head losses are calculated beginning from the control point upstream to the first junction and the procedure is repeated for the next junction. The computation for an outlet control may be tabulated on **Figure 5.2-17** using the following procedure:

(Step 1) Enter in Column 1 the station for the junction immediately upstream of the outflow pipe. Hydraulic grade line computations begin at the outfall and are worked upstream taking each junction into consideration.

(Step 2) Enter in Column 2 the outlet water surface elevation if the outlet will be sub-merged during the design storm or 0.8 diameter plus invert elevation of the outflow pipe, whichever is greater.

(Step 3) Enter in Column 3 the diameter ($D_o$) of the outflow pipe.

(Step 4) Enter in Column 4 the design discharge ($Q_o$) for the outflow pipe.

(Step 5) Enter in Column 5 the length ($L_o$) of the outflow pipe.

(Step 6) Enter in Column 6 the friction slope ($S_f$) in ft/ft of the outflow pipe. This can be determined by using the following formula:

$$\mathbf{S_f = (Q^2)/K} \qquad \mathbf{(5.2.19)}$$

*Where*:

$\mathbf{S_f}$ = friction slope
$\mathbf{K}$ = $[1.486\ AR^{2/3}]/n$

(Step 7) Multiply the friction slope ($S_f$) in Column 6 by the length ($L_o$) in Column 5 and enter the friction loss ($H_f$) in Column 7. On curved alignments, calculate curve losses by using the formula $H_c = 0.002\ (\Delta)(V_o^2/2g)$, where $\Delta$ = angle of curvature in degrees and add to the friction loss.

(Step 8) Enter in Column 8 the velocity of the flow ($V_o$) of the outflow pipe.

(Step 9) Enter in Column 9 the contraction loss ($H_o$) by using the formula:

$$\mathbf{H_o = [0.25\ V_o^2)]/2g}, \textit{ where } g = 32.2 \text{ ft/s}^2$$

**(Step 10)** Enter in Column 10 the design discharge (Qi) for each pipe flowing into the junction. Neglect lateral pipes with inflows of less than 10% of the mainline outflow. Inflow must be adjusted to the mainline outflow duration time before a comparison is made.

**(Step 11)** Enter in Column 11 the velocity of flow ($V_i$) for each pipe flowing into the junction (for exception see Step 10).

**(Step 12)** Enter in Column 12 the product of $Q_i$ x $V_i$ for each inflowing pipe. When several pipes inflow into a junction, the line producing the greatest $Q_i$ x $V_i$ product is the one that should be used for expansion loss calculations.

**(Step 13)** Enter in Column 13 the controlling expansion loss ($H_i$) using the formula:

$$\mathbf{H_i = [0.35\ (V_1^2)]/2g}$$

**(Step 14)** Enter in Column 14 the angle of skew of each inflowing pipe to the outflow pipe (for exception, see Step 10).

**(Step 15)** Enter in Column 15 the greatest bend loss ($H$ ) calculated by using the formula $H$ = $[KV_i^2)]/2g$ where K = the bend loss coefficient corresponding to the various angles of skew of the inflowing pipes.

**(Step 16)** Enter in Column 16 the total head loss ($H_t$) by summing the values in Column 9 ($H_o$), Column 13 ($H_i$), and Column 15 ($H\Delta$).

**(Step 17)** If the junction incorporates adjusted surface inflow of 10% or more of the mainline outflow, i.e., drop inlet, increase $H_t$ by 30% and enter the adjusted $H_t$ in Column 17.

**(Step 18)** If the junction incorporates full diameter inlet shaping, such as standard man-holes, reduce the value of $H_t$ by 50% and enter the adjusted value in Column 18.

**(Step 19)** Enter in Column 19 the FINAL H, the sum of $H_f$ and $H_t$, where $H_t$ is the final adjusted value of the $H^t$.

**(Step 20)** Enter in Column 20 the sum of the elevation in Column 2 and the Final H in Column 19. This elevation is the potential water surface elevation for the junction under design conditions.

**(Step 21)** Enter in Column 21 the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration in Column 20. If the potential water surface elevation exceeds 1 foot below ground elevation for the design flow, the top of the pipe or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the Hydraulic Grade Line (H.G.L.).

**(Step 22)** Repeat the procedure starting with Step 1 for the next junction upstream.

**(Step 23)** At last upstream entrance, add $V_1^2/2g$ to get upstream water surface elevation.

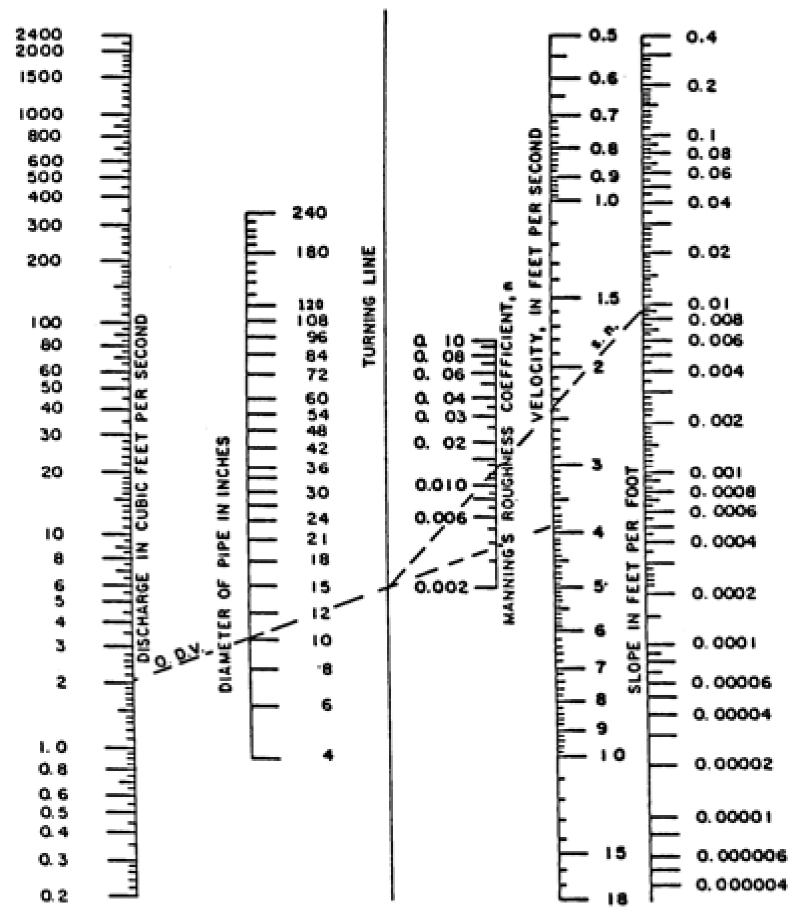



**Figure 5.2-13** Nomograph for Solution of Manning's Formula for Flow in Storm Sewers
(Source: AASHTO Model Drainage Manual, 2005)

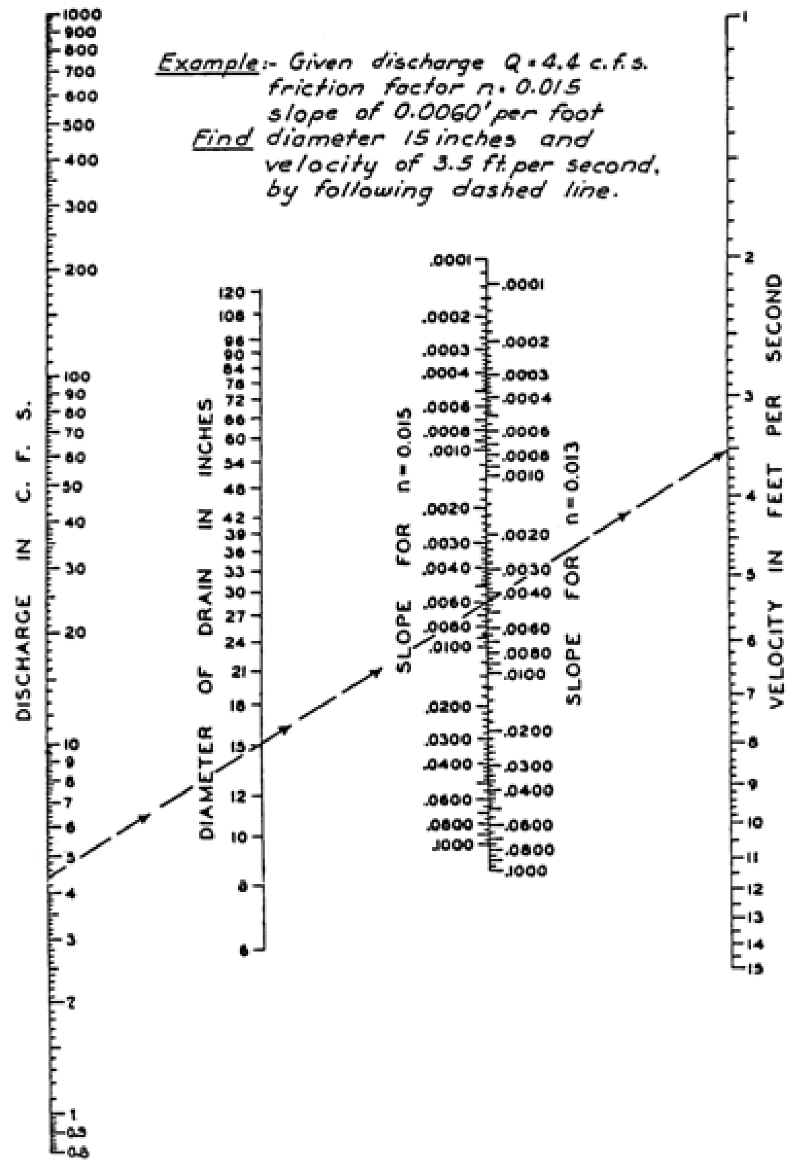



Figure 5.2-14 Nomograph for Computing Required Size of Circular Drain, Flowing Full n = 0.013 or 0.015

(Source: AASHTO Model Drainage Manual, 2005)

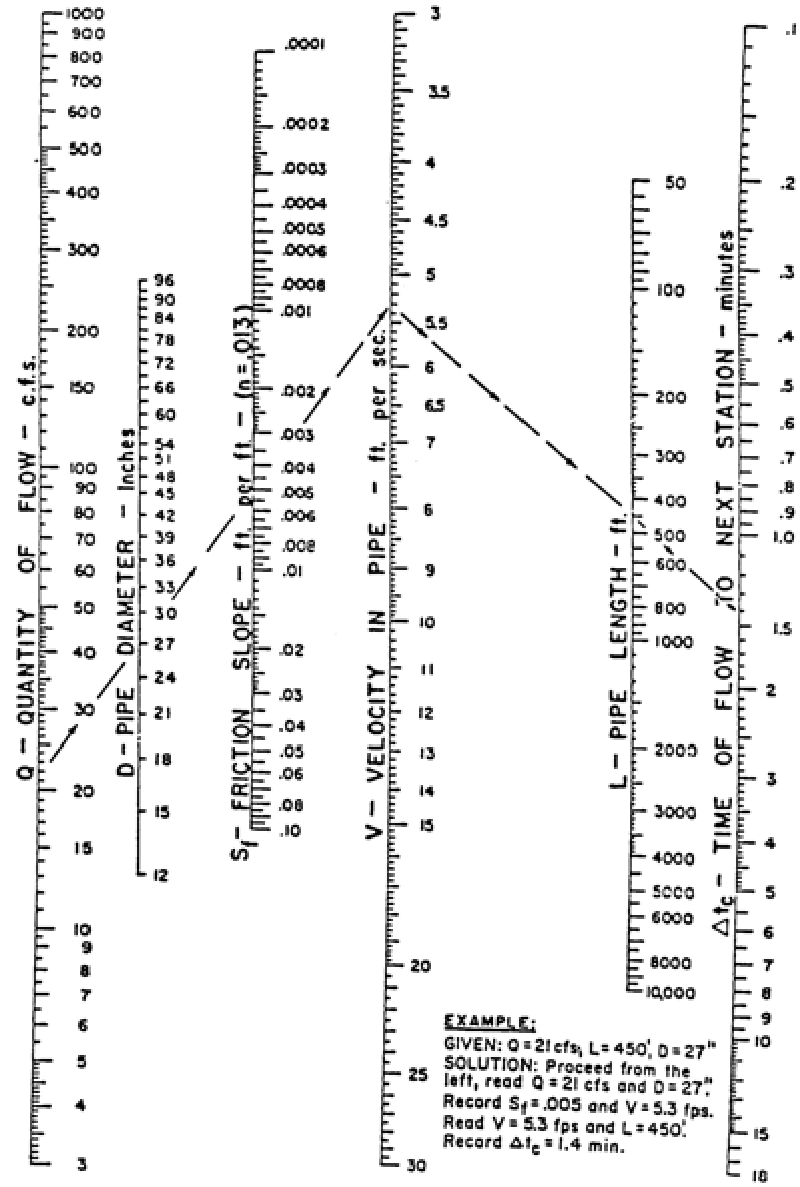
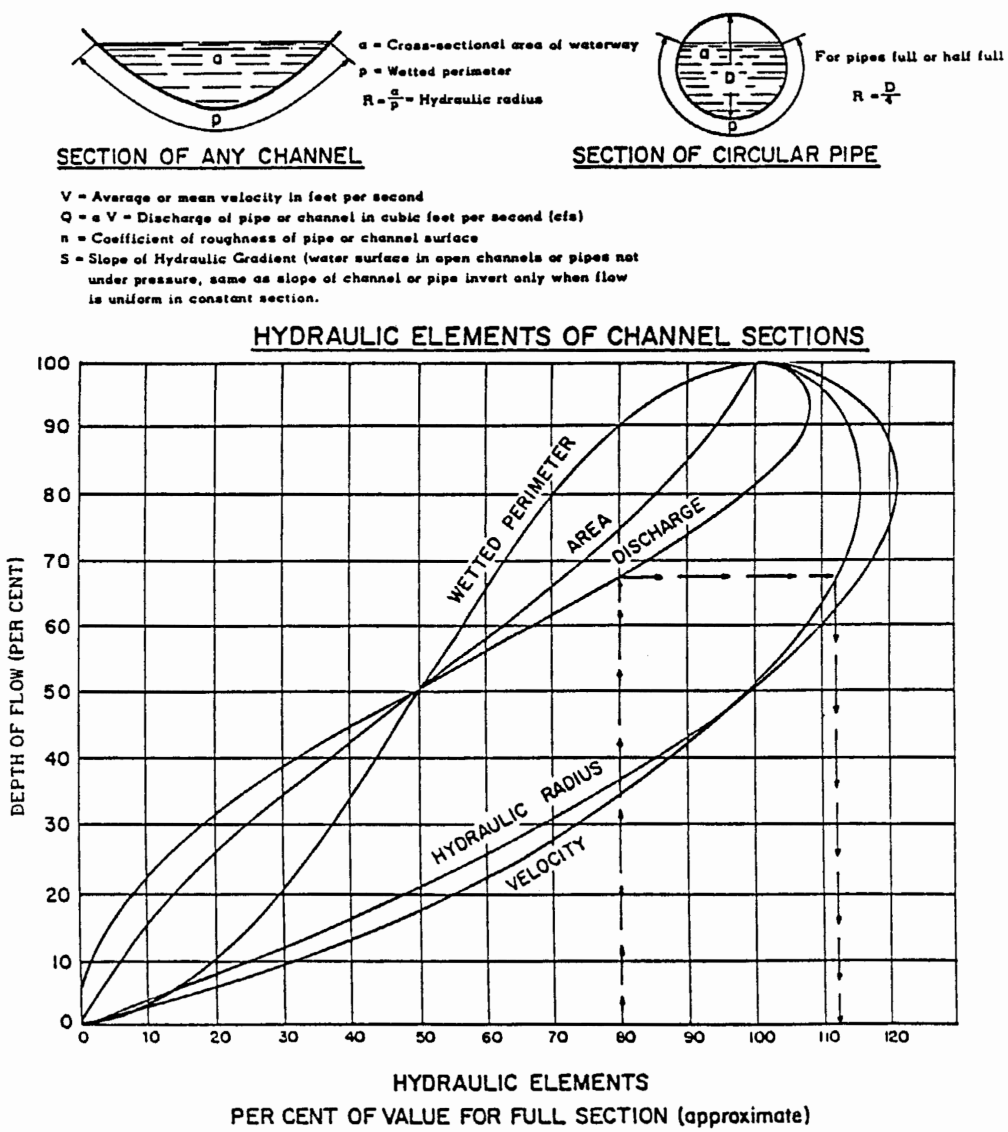



Figure 5.2-15 Concrete Pipe Flow Nomograph

(Source: AASHTO Model Drainage Manual, 2005)




**V** = Average of mean velocity in feet per second

**Q** = Discharge of pipe or channel in cubic feet per second

**S** = Slope of hydraulic grade line

**Figure 5.2-16** Values of Various Elements of Circular Section for Various Depths of Flow

(Source: AASHTO Model Drainage Manual, 2005)

HYDRAULIC GRADE LINE PROJECT

| STATION | Outlet Water Surface Elev. | $D_o$ | $Q_o$ | $L_o$ | $S_{f_o}$ % | $H_f$ | JUNCTION LOSS | | | | | | | | | | | | FINAL H | Inlet Water Surface Elev. | Rim Elev. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | $V_o$ | $H_o$ | $Q_l$ | $V_l$ | $Q_l V_l$ | $\frac{V_l^2}{2g}$ | $H_l$ | ANGLE | $H_\Delta$ | $H_t$ | 1.3 $H_t$ | 0.5 $H_t$ | | | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) | (21) |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | |

$H_l = 0.35 \frac{V_l^2}{2g}$ $H_o = 0.25 \frac{V_o^2}{2g}$ $H_\Delta = K \frac{V_l^2}{2g}$

FINAL $H = H_f + H_t$

$H_t = H_o + H_l + H_\Delta$

| | | |
|---|---|---|
| 90° K = 0.70 | 50° K = 0.47 | 20° K = 0.16 |
| 80° K = 0.66 | 40° K = 0.38 | 15° K = 0.10 |
| 70° K = 0.61 | 30° K = 0.28 | |
| 60° K = 0.55 | 25° K = 0.22 | |

**Figure 5.2-17** Hydraulic Grade Line Computation Form

(Source: AASHTO Model Drainage Manual, 2005)

BACK TO TOC

### 5.2.8.7 MINIMUM GRADE

All storm drains should be designed such that velocities of flow will not be less than 2.5 feet per second at design flow or lower, with a minimum slope of 0.5%. For very flat flow lines the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. Upper reaches of a storm drain system should have flatter slopes than slopes of lower reaches. Progressively increasing slopes keep solids moving toward the outlet and deter settling of particles due to steadily increasing flow streams.

The minimum slopes are calculated by the modified Manning's formula:

$$S = [(nV)^2]/[2.208R^{4/3}] \qquad (5.2.20)$$

*Where*:

**S** = the slope of the hydraulic grade line, ft/ft
**n** = Manning's roughness coefficient
**V** = mean velocity of flow, ft/s
**R** = hydraulic radius, ft (area dived by wetted perimeter)

### 5.2.8.8 STORM DRAIN STORAGE

If downstream drainage facilities are undersized for the design flow, a structural stormwater control may be needed to reduce the possibility of flooding. The required storage volume can also be provided by using larger than needed storm drain pipe sizes and restrictors to control the release rates at manholes and/or junction boxes in the storm drain system. The same design criteria for sizing structural control storage facilities are used to determine the storage volume required in the system (see Section 3.3 for more information).

# 5.3 Culvert Design

## 5.3.1 Overview

A *culvert* is a short, closed (covered) conduit that conveys stormwater runoff under an embankment, usually a roadway. The primary purpose of a culvert is to convey surface water, but properly designed it may also be used to restrict flow and reduce downstream peak flows. In addition to the hydraulic function, a culvert must also support the embankment and/or roadway, and protect traffic and adjacent property owners from flood hazards to the extent practicable.

Most culvert design is empirical and relies on nomographs and "cookbook procedures." The purpose of the section is to provide an overview of culvert design criteria and procedures.

## 5.3.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual the symbols listed in **Table 5.3-1** will be used. These symbols were selected because of their wide use.

## 5.3.3 Design Criteria

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following design criteria should be considered for all culvert designs as applicable.

### 5.3.3.1 FREQUENCY FLOOD

See Section 5.1 or the local review authority for design storm requirements for the sizing of culverts.

The 100-year frequency storm shall be routed through all culverts to be sure building structures (e.g., houses, commercial buildings) are not flooded or increased damage does not occur to the highway or adjacent property for this design event.

### 5.3.3.2 VELOCITY LIMITATIONS

Both minimum and maximum velocities should be considered when designing a culvert. The maximum velocity should be consistent with channel stability requirements at the culvert outlet. The maximum allowable velocity for corrugated metal pipe is 15 feet per second. There is no specified maximum allowable velocity for reinforced concrete pipe, but outlet protection shall be provided where discharge velocities will cause erosion problems. To ensure self-cleaning during partial depth flow, a minimum velocity of 2.5 feet per second, for the 2-year flow, when the culvert is flowing partially full is required.

Table 5.3-1 Symbols and Definitions

| Symbol | Definition | Units |
|---|---|---|
| A | Area of cross section of flow | $ft^2$ |
| B | Barrel width | ft |
| $C_d$ | Overtopping discharge coefficient | - |
| D | Culvert Diameter or barrel depth | in or ft |
| d | Depth of flow | ft |
| $d_c$ | Critical depth of flow | ft |
| $d_u$ | Uniform depth of flow | ft |
| g | Acceleration of gravity | $ft/s^2$ |
| $H_f$ | Depth of pool or head, above the faction section of invert | ft |
| $h_o$ | Height of hydraulic grade line above outlet invert | ft |
| HW | Headwater depth above invert of culvert (depth from inlet invert to upstream total energy grade line) | ft |
| $K_e$ | Inlet loss coefficient | - |
| L | Length of culvert | ft |
| N | Number of barrels | - |
| Q | Rate of discharge | cfs |
| S | Slope of culvert | ft/ft |
| TW | Tailwater depth above invert of culvert | ft |
| V | Mean velocity of flow | ft/s |
| $V_c$ | Critical velocity | ft/s |

BACK TO TOC

#### 5.3.3.3 BUOYANCY PROTECTION
Headwalls, endwalls, slope paving or other means of anchoring to provide buoyancy protection should be considered for all flexible culverts.

#### 5.3.3.4 LENGTH AND SLOPE
The culvert length and slope should be chosen to approximate existing topography and, to the degree practicable, the culvert invert should be aligned with the channel bottom and the skew angle of the stream, and the culvert entrance should match the geometry of the roadway embankment. The maximum slope using concrete pipe is 10% and for CMP is 14% before pipe-restraining methods must be taken. Maximum drop in a drainage structure is 10 feet.

#### 5.3.3.5 DEBRIS CONTROL
In designing debris control structures it is recommended that the Hydraulic Engineering Circular No. 9 entitled Debris Control Structures be consulted.

#### 5.3.3.6 HEADWATER LIMITATIONS
Headwater is water above the culvert invert at the entrance end of the culvert. The allowable headwater elevation is that elevation above which damage may be caused to adjacent property and/or the roadway and is determined from an evaluation of land use upstream of the culvert and the proposed or existing roadway elevation. It is this allowable headwater depth that is the primary basis for sizing a culvert.

The following criteria related to headwater should be considered:

- The allowable headwater is the depth of water that can be ponded at the upstream end of the culvert during the design flood, which will be limited by one or more of the following constraints or conditions:
    1. Headwater be nondamaging to upstream property
    2. Ponding depth be no greater than the low point in the road grade
    3. Ponding depth be no greater than the elevation where flow diverts around the culvert
    4. Elevations established to delineate floodplain zoning
    5. 18-inch (or applicable) freeboard requirements

- The following HW/D criteria:
    1. For drainage facilities with cross-sectional area equal to or less than 30 $ft^2$, HW/D should be equal to or less than 1.5
    2. For drainage facilities with cross-sectional area greater than 30 $ft^2$, HW/D should be equal to or less than 1.2

- The headwater should be checked for the 100-year flood to ensure compliance with flood plain management criteria and for most facilities the culvert should be sized to maintain flood-free conditions on major thoroughfares with 18-inch freeboard at the low-point of the road.

- The maximum acceptable outlet velocity should be identified (see Subsection 5.4.3).

- Either the headwater should be set to produce acceptable velocities, or stabiliza-tion or energy dissipation should be provided where these velocities are ex-ceeded.

- In general, the constraint that gives the lowest allowable headwater elevation establishes the criteria for the hydraulic calculations.

- Other site-specific design considerations should be addressed as required.

#### 5.3.3.7 TAILWATER CONSIDERATIONS
The hydraulic conditions downstream of the culvert site must be evaluated to determine a tailwater depth for a range of discharge. At times there may be a need for calculating backwater curves to establish the tailwater conditions. The following conditions must be considered:

- If the culvert outlet is operating with a free outfall, the critical depth and equival-ent hydraulic grade line should be determined.

- For culverts that discharge to an open channel, the stage-discharge curve for the channel must be determined. See Section 5.4, *Open Channel Design*.

- If an upstream culvert outlet is located near a downstream culvert inlet, the head-water elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.

BACK TO TOC

- If the culvert discharges to a lake, pond, or other major water body, the expected high water elevation of the particular water body may establish the culvert tailwater.

#### 5.3.3.8 STORAGE

If storage is being assumed or will occur upstream of the culvert, refer to Subsection 5.3.4.6 regarding storage routing as part of the culvert design.

#### 5.3.3.9 CULVERT INLETS

Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient Ke, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in **Table 5.3-2**.

#### 5.3.3.10 INLETS WITH HEADWALLS

Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, providing embankment protection against erosion, providing protection from buoyancy, and shortening the length of the required structure. Headwalls are required for all metal culverts and where buoyancy protection is necessary. If high headwater depths are to be encountered, or the approach velocity in the channel will cause scour, a short channel apron should be provided at the toe of the headwall. This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.

#### 5.3.3.11 WINGWALLS AND APRONS

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow.

#### 5.3.3.12 IMPROVED INLETS

Where inlet conditions control the amount of flow that can pass through the culvert, improved inlets can greatly increase the hydraulic performance of the culvert.

#### 5.3.3.13 MATERIAL SELECTION

Reinforced concrete pipe (RCP) is generally recommended for use (1) under a roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams. RCP and any other Georgia Department of Transportation approved pipe material may be used as allowed by local regulations. **Table 5.3-3** gives recommended Manning's n values for different materials.

#### 5.3.3.14 CULVERT SKEWS

Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval.

#### 5.3.3.15 CULVERT SIZES

The minimum allowable pipe diameter shall be 18".

#### 5.3.3.16 WEEP HOLES

Weep holes are sometimes used to relieve uplift pressure. Filter materials should be used in conjunction with the weep holes in order to intercept the flow and prevent the formation of piping channels. The filter materials should be designed as an underdrain filter so as not to become clogged and so that piping cannot occur through the pervious material and the weep hole.

#### 5.3.3.17 OUTLET PROTECTION

See Section 5.5 for information on the design of outlet protection. Outlet protection should be provided for the 25-year storm.

#### 5.3.3.18 EROSION AND SEDIMENT CONTROL

Erosion and sediment control shall be in accordance with the latest approved Soil Erosion and Sediment Control Ordinance for the municipality. See also the *Manual for Erosion and Sediment Control in Georgia* for design standards and details related to erosion and sediment control.

#### 5.3.3.19 ENVIRONMENTAL CONSIDERATIONS

Where compatible with good hydraulic engineering, a site should be selected that will permit the culvert to be constructed to cause the least impact on the stream or wetlands. This selection must consider the entire site, including any necessary lead channels.

Table 5.3-2 Inlet Coefficients

| Type of Structure and Design of Entrance | Coefficient $K_e$ |
|---|---|
| **Pipe, Concrete** | |
| Projecting from fill, socket end (grove-end) | 0.2 |
| Projecting from fill, square cut end | 0.5 |
| Headwall or headwall and wingwalls | |
| Socket end of pipe (grove-end) | 0.2 |
| Square-edge | 0.5 |
| Rounded [radius = 1/12(D)] | 0.2 |
| Mitered to conform to fill slope | 0.7 |
| *End-Section conforming to fill slope | 0.5 |
| Beveled edges, 33.7° or 45° bevels | 0.2 |
| Side- or slope-tapered inlet | 0.2 |
| **Pipe, or Pipe-Arch, Corrugated Metal**[1] | |
| Projecting from fill (no headwall) | 0.9 |
| Headwall or headwall and wingwalls square-edge | 0.5 |
| Mitered to fill slope, paved or unpaved slope | 0.7 |
| *End-section conforming to fill slope | 0.5 |
| Beveled edges, 33.7° or 45° bevels | 0.2 |
| Side- or slope-tapered inlet | 0.2 |
| **Box, Reinforced Concrete** | |
| Headwall parallel to embankment (no wingwalls) | |
| Square-edged on 3 edges | 0.5 |
| Rounded on 3 edges to radius of [1/12(D)] or beveled edges on 3 sides | 0.2 |
| Wingwalls at 30° to 75° to barrel | |
| Square-edged at crown | 0.4 |
| Crown edge rounded to radius of [1/12(D)] or beveled top edge | 0.2 |
| Wingwalls at 10° or 25° to barrel | |
| Square-edged at crown | 0.5 |
| Wingwalls parallel (extension of sides) | |
| Square-edged at crown | 0.7 |
| Side- or slope-tapered inlet | 0.2 |

[1] Although laboratory tests have not been completed on $K_e$ values for High-Density Polyethylene (HDPE) pipes, the $K_e$ values for corrugated metal pipes are recommended for HDPE pipes

* Note: End Section conforming to fill slope, made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both inlet and outlet control.

Source: HDS No. 5, 2012, C.6 (216)

Table 5.3-3 Manning's n Values for Culverts

| Type of Conduit | Wall & Joint Description | Manning's n |
|---|---|---|
| Concrete Pipe | Smooth | 0.010 - 0.011 |
| Concrete Box | Good joints, smooth finished walls<br>Poor joints, rough, unfinished walls | 0.012<br>0.015 |
| Corrugated Metal Pipes and Boxes Annular Corrugations | 2 2/3- by 1/2 inch corrugations<br>5- by 1-inch corrugations<br>3- by 1-inch corrugations<br>6- by 2-inch structural plate<br>9- by 2-1/2 inch structural plate | 0.022 - 0.027<br>0.025 - 0.026<br>0.027 - 0.028<br>0.033 - 0.035<br>0.033 - 0.037 |
| Corrugated Metal Pipes, Helical Corrugations | 2 2/3-by 1/2-inch corrugated 24-inch plate width<br>6-by 1-inch | 0.011 - 0.023<br>0.022 - 0.025 |
| Spiral Rib Metal Pipe | 3/4 by 3/4 in recesses at 12 inch spacing, good joints | 0.012 - 0.013 |
| High Density Polyethylene (HDPE) | Corrugated Smooth Liner<br>Corrugated | 0.009 - 0.015<br>0.018 - 0.025 |
| Polyvinyl Chloride (PVC) | | 0.009 - 0.011 |

Source: HDS No. 5, 2012

Note: For further information concerning Manning n values for selected culverts consult Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS No. 5, page B.6 (208).

## 5.3.4 Design Procedures

### 5.3.4.1 TYPES OF FLOW CONTROL

There are two types of flow conditions for culverts that are based upon the location of the control section and the critical flow depth:

» Inlet Control – Inlet control occurs when the culvert barrel is capable of conveying more flow that the inlet will accept. This typically happens when a culvert is operating on a steep slope. The control section of a culvert is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.

» Outlet Control – Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions.

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control, see the FHWA Hydraulic Design of Highway Culverts, HDS-5, 2012.

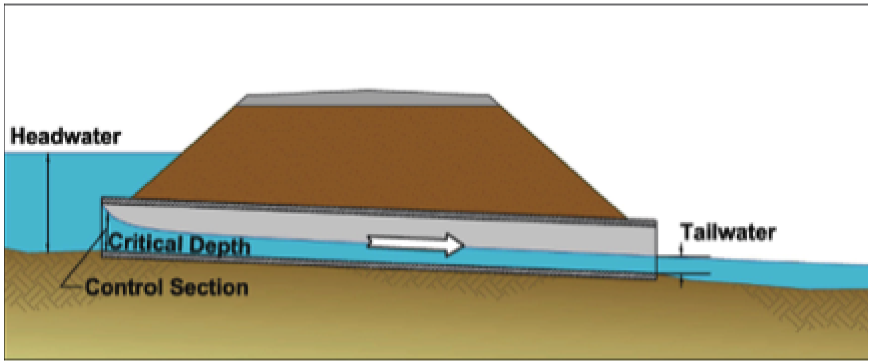



Figure 5.3-1(a) Typical Inlet Control Flow Section
(Adapted from: HDS-5, 2012)

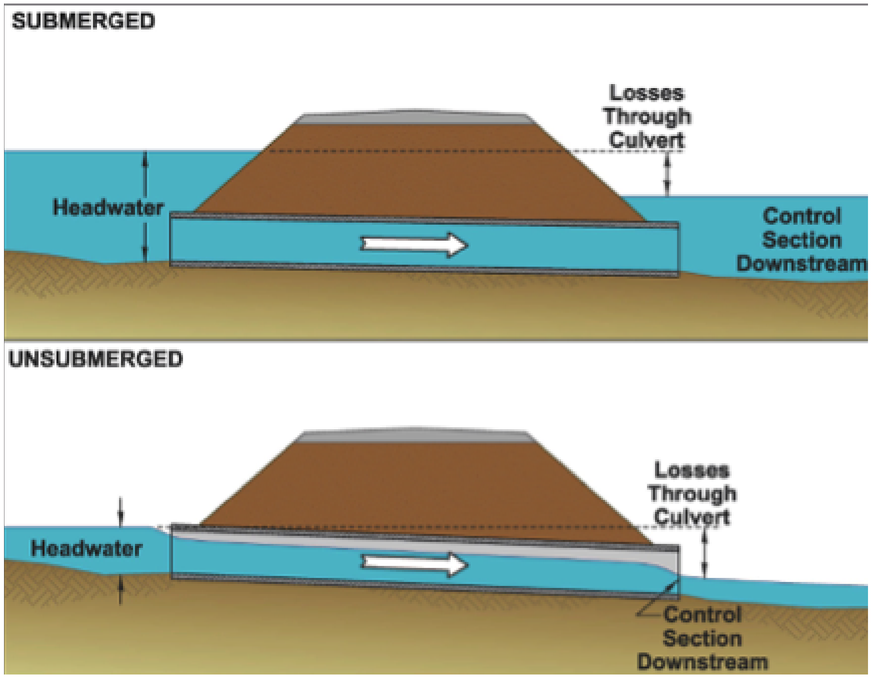



Figure 5.3-1(b) Typical Outlet Control Flow Conditions
(Adapted from: HDS-5, 2012)

### 5.3.4.2 PROCEDURES

There are two procedures for designing culverts: manual use of inlet and outlet control nomographs, and the use computer programs such as HY8. It is recommended that the HY8 computer model or equivalent be used for culvert design. The computer software package HYDRAIN, which includes HY8, uses the theoretical basis from the nomographs to size culverts. In addition, this software can evaluate improved inlets, route hydrographs, consider road overtopping, and evaluate outlet streambed scour. By using water surface profiles, this procedure is more accurate in predicting backwater effects and outlet scour.

### 5.3.4.3 NOMOGRAPHS

The use of culvert design nomographs requires a trial and error solution. Nomograph solutions provide reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and

BACK TO TOC

outlet scour require additional, separate computations beyond what can be obtained from the nomographs. **Figures 5.3-2(a)** and **(b)** show examples of an inlet control and outlet control nomograph for the design of concrete pipe culverts. For other culvert designs, refer to the complete set of nomographs in Appendix C.

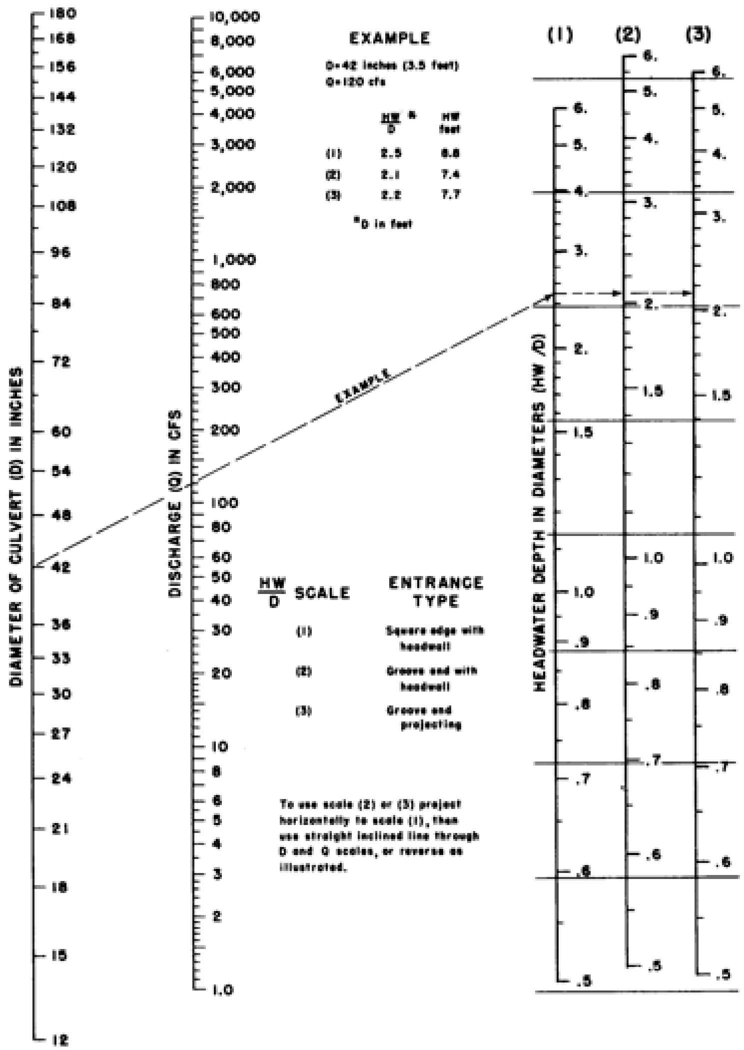



Figure 5.3-2(a) Headwater Depth for Concrete Pipe Culvert with Inlet Control

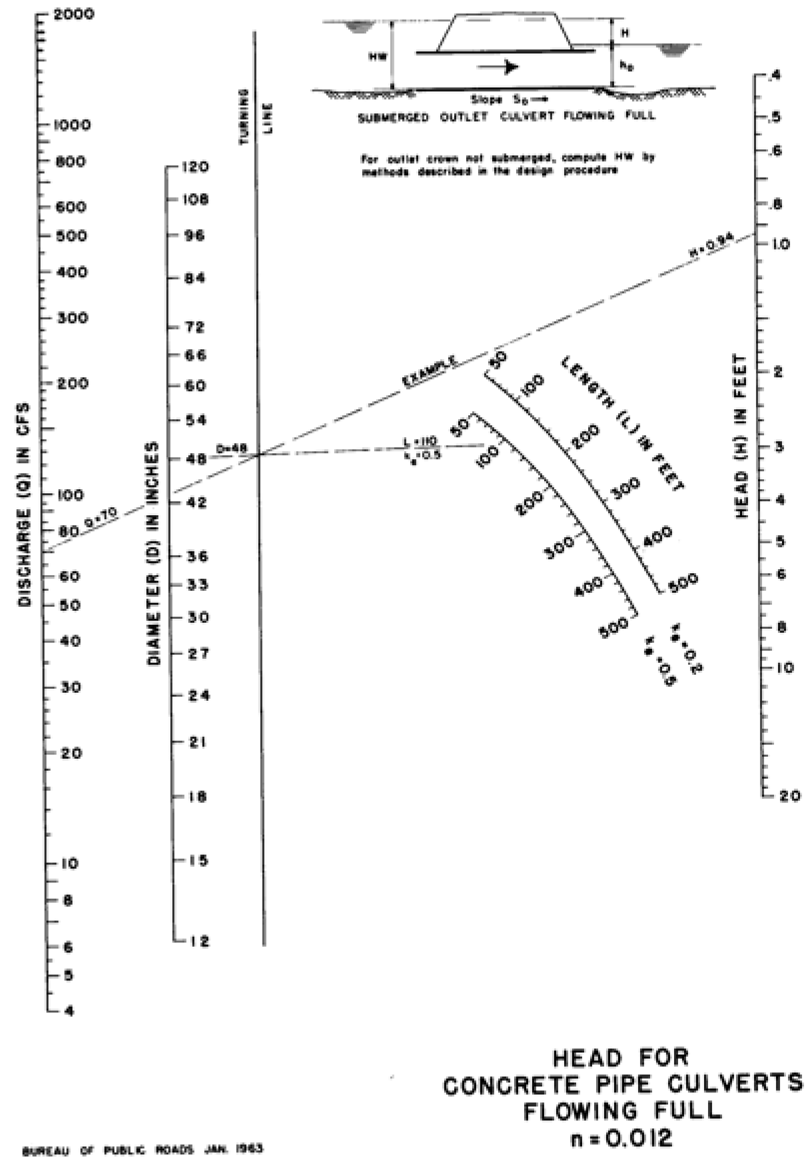



Figure 5.3-2(b) Head for Concrete Pipe Culverts Flowing Full

### 5.3.4.4 DESIGN PROCEDURE

The following design procedure requires the use of inlet and outlet nomographs.

(Step 1) List design data:

Q = discharge (cfs) L = culvert length (ft)

S = culvert slope (ft/ft) TW= tailwater depth (ft)

V = velocity for trial diameter (ft/s)

$K_e$= inlet loss coefficient

HW= allowable headwater depth for the design storm (ft)

(Step 2) Determine trail culvert size by assuming a trial velocity 3 to 5 ft/s and computing the culvert area, A = Q/V. Determine the culvert diameter (inches).

(Step 3) Find the actual HW for the trial size culvert for both inlet and outlet control.

- For **inlet control**, enter inlet control nomograph with D and Q and find HW/D for the proper entrance type.
- Compute HW and, if too large or too small, try another culvert size before computing HW for outlet control.
- For **outlet control** enter the outlet control nomograph with the culvert length, entrance loss coefficient, and trial culvert diameter.
- To compute HW, connect the length scale for the type of entrance condition and culvert diameter scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge through the turning point to the head loss scale H. Compute the headwater elevation HW from the equation:

$$HW = H + h_o \quad LS \qquad (5.3.1)$$

*Where*:

**$h_o$** = ½ (critical depth + D), or tailwater depth, whichever is greater

**L** = culvert length

**S** = culvert slope

(Step 4) Compare the computed headwaters and use the higher HW nomograph to determine if the culvert is under inlet or outlet control.

- If **inlet control** governs, then the design is complete and no further analysis is required.
- If **outlet control** governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet control nomographs. Since the smaller size of culvert had been selected for allowable HW by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

(Step 5) Calculate exit velocity and if erosion problems might be expected, refer to Section 5.5 for appropriate energy dissipation designs.

### 5.3.4.5 PERFORMANCE CURVES - ROADWAY OVERTOPPING

A performance curve for any culvert can be obtained from the nomographs by repeating the steps outlined above for a range of discharges that are of interest for that particular culvert design. A graph is then plotted of headwater versus discharge with sufficient points so that a curve can be drawn through the range of interest. These curves are applicable through a range of headwater, velocities, and scour depths versus discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes. Such computations are made much easier by the use of computer programs.

To complete the culvert design, roadway overtopping should be analyzed. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed.

The overall performance curve can be determined as follows:

(Step 1) Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. The flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated.

(Step 2) Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

(Step 3) When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and **equation 5.3.2** to calculate flow rates across the roadway.

$$Q = C_d L (HW)^{1.5} \qquad (5.3.2)$$

*Where*:

**Q** = overtopping flow rate ($ft^3/s$)
**$C_d$** = overtopping discharge coefficient
**L** = length of roadway (ft)
**HW** = upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown (ft)

Note: See **Figure 5.3-3** for guidance in determining a value for $C_d$. For more information on calculating overtopping flow rates see pages 39 - 42 in HDS No. 5.

(Step 4) Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

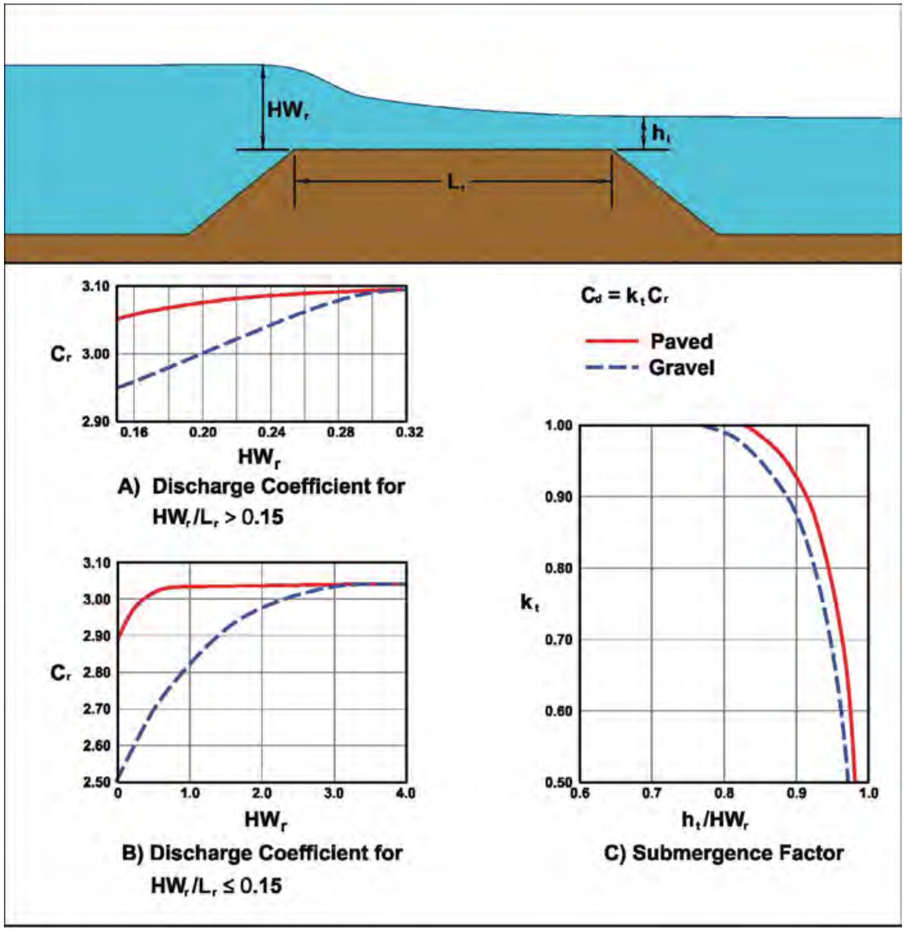



**Figure 5.3-3 Discharge Coefficients for Roadway Overtopping**
(Source: HDS No. 5, 2012)

#### 5.3.4.6 STORAGE ROUTING

A significant storage capacity behind a highway embankment attenuates a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert, and its size, may be reduced considerably. If significant storage is anticipated behind a culvert, the design should be checked by routing the design hydrographs through the culvert to determine the discharge and stage behind the culvert. See subsection 5.3.7 and Section 3.3 for more information on routing. Additional routing procedures are outlined in Hydraulic Design of Highway Culverts, Section V - Storage Routing, HDS No. 5, Federal Highway Administration.

*Note: Storage should be taken into consideration only if the storage area will remain available for the life of the culvert as a result of purchase of ownership or right-of-way or an easement has been acquired.*

## 5.3.5 Culvert Design Example

#### 5.3.5. INTRODUCTION

The following example problem illustrates the procedures to be used in designing culverts using the nomographs.

#### 5.3.5.2 EXAMPLE

Size a culvert given the following example data, which were determined by physical limitations at the culvert site and hydraulic procedures described elsewhere in this handbook.

#### 5.3.5.3 EXAMPLE DATA

**Input Data**

- **Discharge for 2 yr flood** = 35 cfs
- **Discharge for 25 yr flood** = 70 cfs
- **Allowable Hw for 25 yr discharge** = 5.25 ft
- **Length of culvert** = 100 ft
- **Natural channel invert elevations - inlet** = 15.50 ft, **outlet** = 14.30 ft
- **Culvert slope** = 0.012 ft/ft
- **Tailwater depth for 25 yr discharge** = 3.5 ft
- **Tailwater depth is the normal depth in downstream channel**
- **Entrance type** = Groove end with headwall

#### 5.3.5.4 COMPUTATIONS

1. Assume a culvert velocity of 5 ft/s. Required flow area = 70 cfs/5 ft/s = 14 ft$^2$ (for the 25 yr recurrence flood).
2. The corresponding culvert diameter is about 48 in. This can be calculated by using the formula for area of a circle: Area = (3.14D$^2$)/4 or D = (Area times 4/3.14)$^{0.5}$. Therefore: D = ((14 sq ft x 4)/3.14)$^{0.5}$ x 12 in/ft) = 50.7 in
3. A grooved end culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 5.3-1), with a pipe diameter of 48 inches and a discharge of 70 cfs; read a HW/D value of 0.93.
4. The depth of headwater (HW) is (0.93) x (4) = 3.72 ft, which is less than the allowable headwater of 5.25 ft. Since 3.72 ft is considerably less than 5.25 try a small culvert.
5. Using the same procedures outlined in steps 4 and 5 the following results were obtained.
   - 42-inch culvert – HW = 4.13 ft
   - 36-inch culvert – HW = 4.98 ft
   - Select a 36-inch culvert to check for outlet control.
6. The culvert is checked for outlet control by using **Figure 5.3-2**.
   - With an entrance loss coefficient Ke of 0.20, a culvert length of 100 ft, and a pipe diameter of 36 in., an H value of 2.8 ft is determined. The headwater for outlet control is computed by the equation: HW = H + $h_o$ LS
   - Compute $h_o$
     - $h_o$ = $T_w$ or ½ (critical depth in culvert + D), whichever is greater.
     - $h_o$ = 3.5 ft or $h_o$ = ½ (2.7 + 3.0) = 2.85 ft
   - Therefore: $h_o$ = 3.5 ft
   - The headwater depth for outlet control is:

     HW = H + $h_o$ LS = 2.8 + 3.5 (100) x (0.012) = 5.10 ft

7. Since HW for inlet outlet (5.10 ft) is greater than the HW for inlet control (4.98 ft), outlet control governs the culvert design. Thus, the maximum headwater expected for a 25 year recurrence flood is 5.10 ft, which is less than the allowable headwater of 5.25 ft.
8. Estimate outlet exit velocity. Since this culvert is on outlet control and discharges into an open channel downstream with tailwater above culvert, the culvert will be flowing full at the flow depth in the channel. Using the design peak discharge of 70 cfs and the area of a 36-inch or 3.0-foot diameter culvert the exit velocity will be:

» $Q = VA$

» Therefore: $V = 70 / (3.14(3.0)^2)/4 = 9.9$ ft/s

» With this high velocity, consideration should be given to provide an energy dissipator at the culvert outlet. See Section 5.5 (*Energy Dissipation Design*).

9. Check for minimum velocity using the 2-year flow of 35 cfs.

» Therefore: $V = 35 / (3.14(3.0)^2/4 = 5.0$ ft/s > minimum of 2.5 - OK

10. The 100-year flow should be routed through the culvert to determine if any flooding problems will be associated with this flood.

**Figure 5.3-4** provides a convenient form to organize culvert design calculations.

| PROJECT: | STATION: ______ | **CULVERT DESIGN FORM** |
|---|---|---|
| | SHEET ______ OF ______ | DESIGNER / DATE: ______ / ______ |
| | | REVIEWER / DATE: ______ / ______ |

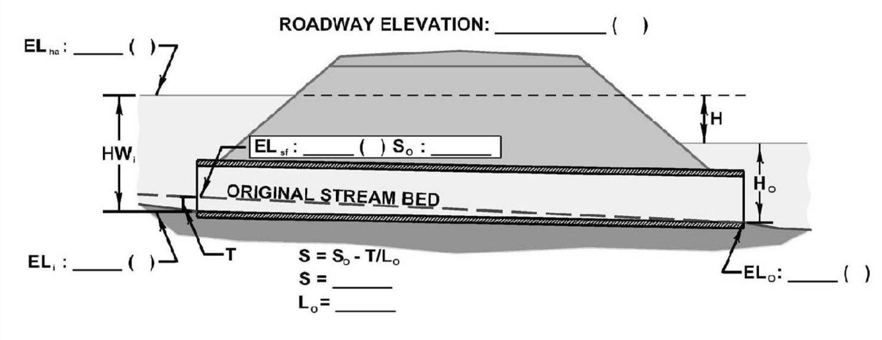
HYDROLOGICAL DATA

See Add'l Shts.

- ☐ METHOD: ______
- ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______
- ☐ CHANNEL SHAPE: ______
- ☐ ROUTING: ______ ☐ OTHER: ______

**DESIGN FLOWS/TAILWATER**

| R.I. (YEARS) | FLOW (cfs) | TW (ft) |
|---|---|---|
| | | |
| | | |




| **CULVERT DESCRIPTION:** MATERIAL – SHAPE – SIZE – ENTRANCE | Total Flow Q (cfs) | Flow Per Barrel Q / N (1) | **HEADWATER CALCULATIONS** INLET CONTROL $HW_i/D$ (2) | $HW_i$ | T (3) | $EL_{hi}$ (4) | OUTLET CONTROL TW (5) | $d_c$ | $\frac{d_c+D}{2}$ | $h_o$ (6) | $k_e$ | H (7) | $EL_{ho}$ (8) | Control Headwater Elevation | Outlet Velocity | Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | |

**TECHNICAL FOOTNOTES:**

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i$ / D = HW / D OR $HW_i$ / D FROM DESIGN CHARTS

(3) $T = HW_i - (EL_{hd} - EL_{sf})$
T IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL

(6) $h_o$ = TW or $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = [1 + k_e + (K_u n^2 L) / R^{1.33}] v^2 / 2g$ WHERE Ku = 19.63 (29 IN ENGLISH UNITS)

(8) $EL_{ho} = EL_o + H + h_o$

**SUBSCRIPT DEFINITIONS:**

a. APPROXIMATE
f. CULVERT FACE
ha. ALLOWABLE HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

**COMMENTS / DISCUSSION:**

**CULVERT BARREL SELECTED:**

SIZE: ______

SHAPE: ______

MATERIAL: ______ n ______

ENTRANCE: ______

Figure 5.3-4 Culvert Design Calculation Form
(Source: HDS No. 5, 2012)

BACK TO TOC

## 5.3.6 Design Procedures for Beveled-Edged Inlets

### 5.3.6.1 INTRODUCTION

Improved inlets include inlet geometry refinements beyond those normally used in conventional culvert design practice. Several degrees of improvements are possible, including bevel-edged, side-tapered, and slope-tapered inlets. Those designers inter-ested in using side and slope tapered inlets should consult the detailed design criteria and example designs outlined in the U. S. Department of Transportation publication Hydraulic Engineering Circular No. 5 entitled, Hydraulic Design of Highway Culverts.

### 5.3.6.2 DESIGN FIGURES

Four inlet control figures for culverts with beveled edges are included in Appendix C.

| Chart | Page | Use for : |
|---|---|---|
| 3 | A-3 | circular pipe culverts with beveled rings |
| 10 | A-10 | 90° headwalls (same for 90° wingwalls) |
| 11 | A-11 | skewed headwalls |
| 12 | A-12 | wingwalls with flare angles of 18 to 45 degrees |

The following symbols are used in these figures:

- B - Width of culvert barrel or diameter of pipe culvert
- D - Height of box culvert or diameter of pipe culvert
- $H_f$ - Depth of pool or head, above the face section of invert
- N - Number of barrels
- Q - Design discharge

### 5.3.6.3 DESIGN PROCEDURE

The figures for bevel-edged inlets are used for design in the same manner as the con-ventional inlet design nomographs discussed earlier. *Note* that **Charts 10, 11,** and **12** in Appendix C apply only to bevels having either a 33° angle (1.5:1) or a 45° angle (1:1).

For box culverts the dimensions of the bevels to be used are based on the culvert dimensions. The top bevel dimension is determined by multiplying the height of the culvert by a factor. The side bevel dimensions are determined by multiplying the width of the culvert by a factor. For a 1:1 bevel, the factor is 0.5 inch/ft. For a 1.5:1 bevel the factor is 1 inch/ft. For example, the minimum bevel dimensions for a 8 ft x 6 ft box culvert with 1:1 bevels would be:

- **Top Bevel** = d = 6 ft x 0.5 inch/ft = 3 inches
- **Side Bevel** = b = 8 ft x 0.5 inch/ft = 4 inches

For a 1.5:1 bevel computations would result in d = 6 and b = 8 inches.

### 5.3.6.4 DESIGN FIGURE LIMITS

The improved inlet design figures are based on research results from culvert models with barrel width, B, to depth, D, ratios of from 0.5:1 to 2:1. For box culverts with more than one barrel, the figures are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size.

**For example**, in a double 8 ft by 8 ft box culvert:

- **Top Bevel** is proportioned based on the height of 8 feet which results in a bevel of 4 in. for the 1:1 bevel and 8 in. for the 1.5:1 bevel.
- **Side Bevel** is proportioned based on the clear width of 16 feet, which results in a bevel of 8 in. for the 1:1 bevel and 16 in. for the 1.5:1 bevel.

### 5.3.6.5 MULTIBARREL INSTALLATIONS

For multibarrel installations exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the **side bevel** be sized in proportion to the total clear width, B, or three times the height, whichever is smaller.

The **top bevel** dimension should always be based on the culvert height.

BACK TO TOC

The shape of the upstream edge of the intermediate walls of multibarrel installations is not as important to the hydraulic performance of a culvert as the edge condition of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to help direct debris through the culvert.

Multibarrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

#### 5.3.6.6 SKEWED INLETS

It is recommended that **Chart 11** (see Appendix C) for skewed inlets not be used for multiple barrel installations, as the intermediate wall could cause an extreme contraction in the downstream barrels. This would result in underdesign due to a greatly reduced capacity. Skewed inlets (at an angle with the centerline of the stream) should be avoided whenever possible and should not be used with side or slope tapered inlets. It is important to align culverts with streams in order to avoid erosion problems associated with changing the direction of the natural stream flow.

### 5.3.7 Flood Routing and Culvert Design

#### 5.3.7.1 INTRODUCTION

Flood routing through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. By not considering flood routing it is possible that the findings from culvert analyses will be conservative. If the selected allowable headwater is accepted without flood routing, then costly overdesign of both the culvert and outlet protection may result, depending on the amount of temporary storage involved. However, if storage is used in the design of culverts, consideration should be given to:

- The total area of flooding,
- The average time that bankfull stage is exceeded for the design flood up to 48 hours in rural areas or 6 hours in urban areas, and
- Ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement.

#### 5.3.7.2 DESIGN PROCEDURE

The design procedure for flood routing through a culvert is the same as for reservoir routing. The site data and roadway geometry are obtained and the hydrology analysis completed to include estimating a hydrograph. Once this essential information is available, the culvert can be designed. Flood routing through a culvert can be time consuming. It is recommended that a computer program be used to perform routing calculations; however, an engineer should be familiar with the culvert flood routing design process.

A multiple trial and error procedure is required for culvert flood routing. In general:

(Step 1) A trial culvert(s) is selected

(Step 2) A trial discharge for a particular hydrograph time increment (selected time increment to estimate discharge from the design hydrograph) is selected

(Step 3) Flood routing computations are made with successive trial discharges until the flood routing equation is satisfied

(Step 4) The hydraulic findings are compared to the selected site criteria

(Step 5) If the selected site criteria are satisfied, then a trial discharge for the next time increment is selected and this procedure is repeated; if not, a new trial culvert is selected and the entire procedure is repeated.

# 5.4 Open Channel Design

## 5.4.1 Overview

### 5.4.1.1 INTRODUCTION

Open channel systems and their design are an integral part of stormwater drainage design, particularly for development sites utilizing better site design practices and open channel structural controls. Open channels include drainage ditches, grass channels, dry and wet enhanced swales, riprap channels and concrete-lined channels.

The purpose of this section is to provide an overview of open channel design criteria and methods, including the use of channel design nomographs.

### 5.4.1.2 OPEN CHANNEL TYPES

The three main classifications of open channel types according to channel linings are vegetated, flexible and rigid. Vegetated linings include grass with mulch, sod and lapped sod, and wetland channels. Riprap and some forms of flexible man-made linings or gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block.

1. **Vegetative Linings** – Vegetation, where practical, is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, provides habitat, provides runoff reduction, and provides water quality benefits (see Chapter 4 for more details on using enhanced swales and grass channels for runoff reduction and water quality purposes).

   » Conditions under which vegetation may not be acceptable include but are not limited to:

   - High velocities
   - Standing or continuously flowing water
   - Lack of regular maintenance necessary to prevent growth of taller or woody vegetation
   - Lack of nutrients and inadequate topsoil
   - Excessive shade

   » Proper seeding, mulching and soil preparation are required during construction to assure establishment of healthy vegetation.

2. **Flexible Linings** – Rock riprap, including rubble, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance. However, they may require the use of a filter fabric depending on the underlying soils, and the growth of grass and weeds may present maintenance problems.

3. **Rigid Linings** – Rigid linings are generally constructed of concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and channel headcutting.

### 5.4.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in **Table 5.4-1** will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 5.4-1 Symbols and Definitions

| Symbol | Definition | Units |
|---|---|---|
| $\alpha$ | Energy coefficient | - |
| A | Cross-sectional area | $ft^2$ |
| b | Bottom width | ft |
| $C_g$ | Specific weight correction factor | - |
| D or d | Depth of flow | ft |
| d | Stone Diameter | ft |
| delta d | Superelevation of the water surface profile | ft |
| $d_x$ | Diamter of stone for which x percent, by weight, of the gradation is finer | ft |
| E | Specific energy | ft |
| Fr | Froude Number | - |
| g | Acceleration of gravity | 32.2 $ft/s^2$ |
| $h_{loss}$ | Head loss | ft |
| K | Channel conveyance | - |
| $k_e$ | Eddy head loss coefficient | ft |
| $K_T$ | Trapezoidal open channel conveyance factor | - |
| L | Length of channel | ft |
| $L_p$ | Length of downstream protection | ft |
| n | Manning's roughness coefficient | - |
| P | Wetted Perimeter | ft |
| Q | Discharge rate | cfs |
| R | Hydraulic radius of flow | ft |
| $R_C$ | Mean radius of the bend | ft |
| S | Slope | ft/ft |
| $SW_S$ | Specific weight of stone | $lbs/ft^3$ |
| T | Top width of water surface | ft |
| V or v | Velocity of flow | ft/s |
| w | Stone weight | lbs |
| $y_C$ | Critical depth | ft |
| $y_n$ | Normal depth | ft |
| z | Critical flow section factor | - |

BACK TO TOC

## 5.4.3 Design Criteria

### 5.4.3.1 GENERAL CRITERIA

The following criteria should be followed for open channel design:

- Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 12 to 1, or with compound cross sections.
- Channel side slopes shall be stable throughout the entire length and side slope shall depend on the channel material. A maximum of 2:1 should be used for channel side slopes, unless otherwise justified by calculations. Roadside ditches should have a maximum side slope of 3:1.
- Trapezoidal or parabolic cross sections are preferred over triangular shapes.
- For vegetative channels, design stability should be determined using low vegetative retardance conditions (Class D) and for design capacity higher vegetative retardance conditions (Class C) should be used.
- For vegetative channels, flow velocities within the channel should not exceed the maximum permissible velocities given in **Tables 5.4-2** and **5.4-3**.
- If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.
- Streambank stabilization should be provided, when appropriate, as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.
- Open channel drainage systems are sized to handle a 25-year design storm. The 100-year design storm should be routed through the channel system to determine if the 100-year plus applicable building elevation restrictions are exceeded, structures are flooded, or flood damages increased.

### 5.4.3.2 VELOCITY LIMITATIONS

The final design of artificial open channels should be consistent with the velocity limitations for the selected channel lining. Maximum velocity values for selected lining categories are presented in **Table 5.4-2**. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in **Table 5.4-3**. Vegetative lining calculations are presented in Section 5.4.7 and riprap procedures are presented in Section 5.4.8.

Table 5.4-2 Maximum Velocities for Comparing Lining Materials

| Material | Maximum Velocity (ft/s) |
|---|---|
| Sand | 2.0 |
| Silt | 3.5 |
| Firm Loam | 3.5 |
| Fine Gravel | 5.0 |
| Stiff Clay | 5.0 |
| Graded Loam or Silt to Cobbles | 5.0 |
| Coarse Gravel | 6.0 |
| Shales and Hard Pans | 6.0 |

Source: AASHTO Model Drainage Manual, 2005

Table 5.4-3 Maximum Velocities for Vegetative Channel Linings

| Vegetation Type | Slope Range (%)[1] | Maximum Velocity[2] (ft/s) |
|---|---|---|
| Bermuda Grass | 0->10 | 5 |
| Bahia | | 4 |
| Tall fescue grass mixtures[3] | 0-10 | 4 |
| Kentucky bluegrass | 0-5 | 6 |
| Buffalo Grass | 5-10 | 5 |
| | >10 | 4 |
| Grass Mixture | 0-5[1] | 4 |
| | 5-10 | 3 |
| Sericea lespedeza, Weeping lovegrass Alfalfa | 0-5[4] | 3 |
| Annuals[5] | 0-5 | 3 |
| Sod | | 4 |
| Lapped Sod | | 5 |

[1] Do not use on slopes steeper than 10%, except for side-slope in combination channel.
[2] Use velocities exceeding 5 ft/s only where good stands can be maintained.
[3] Mixures of Tall Fescue, Bahia, and/or Bermuda
[4] Do not use on slopes steeper than 5%, except for side-slope in combination channel.
[5] Annuals - used on mild slopes or as temporary protection until permanent covers are established.

Source: Manual for Erosion and Sediment Control in Georgia

## 5.4.4 Manning's Values

The Manning's n value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth, and velocity estimates. Since Manning's n values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process.

Recommended Manning's n values for artificial channels with rigid, unlined, temporary, and riprap linings are given in **Table 5.4-4**. Recommended values for vegetative linings should be determined using **Figure 5.4-1**, which provides a graphical relationship between Manning's n values and the product of velocity and hydraulic radius for several vegetative retardance classifications (see **Table 5.4-6**). **Figure 5.4-1** is used iteratively as described in Section 5.4.6. Recommended Manning's values for natural channels that are either excavated or dredged and natural are given in **Table 5.4-5.** For natural channels, Manning's n values should be estimated using experienced judgment and information presented in publications such as the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA-TS-84-204, 1984.

## 5.4.5 Uniform Flow Calculations

### 5.4.5.1 DESIGN CHARTS

Following is a discussion of the equations that can be used for the design and analysis of open channel flow. The Federal Highway Administration has prepared numerous design charts to aid in the design of rectangular, trapezoidal and triangular open channel cross sections. In addition, design charts for grass-lined channels have been developed. These charts and instructions for their use are given in Appendix C.

### 5.4.5.2 MANNING'S EQUATION

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$v = (1.49/n)\ R^{2/3}\ S^{1/2} \qquad (5.4.1)$$

$$Q = (1.49/n)\ A\ R^{2/3}\ S^{1/2} \qquad (5.4.2)$$

$$S = [Q_n/(1.49\ A\ R^{2/3})]^2 \qquad (5.4.3)$$

*Where*:

- **v** = average channel velocity (ft/s)
- **Q** = discharge rate for design conditions (cfs)
- **n** = Manning's roughness coefficient
- **A** = cross-sectional area ($ft^2$)
- **R** = hydraulic radius A/P (ft)
- **P** = wetted perimeter (ft)
- **S** = slope of the energy grade line (ft/ft)

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line, water surface and channel bottom are assumed to be equal.

### 5.4.5.3 GEOMETRIC RELATIONSHIPS

Area, wetted perimeter, hydraulic radius, and channel top width for standard channel cross sections can be calculated from geometric dimensions. Irregular channel cross sections (i.e., those with a narrow deep main channel and a wide shallow overbank channel) must be subdivided into segments so that the flow can be computed separately for the main channel and overbank portions. This same process of subdivision may be used when different parts of the channel cross section have different roughness coeff-icients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter.

### 5.4.5.4 DIRECT SOLUTIONS

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from **equation 5.4.2**. The slope can be calculated using **equation 5.4.3** when the discharge, roughness coefficient, area, and hydraulic radius are known.

Nomographs for obtaining direct solutions to Manning's Equation are presented in **Figures 5.4-2** and **5.4-3. Figure 5.4-2** provides a general solution for the velocity form of Manning's Equation, while **Figure 5.4-3** provides a solution of Manning's Equation for trapezoidal channels.

Table 5.4-4 Manning's Roughness Coefficients (n) for Artificial Channels

| Category | Lining Type | Depth Ranges 0-0.5 ft | Depth Ranges 0.5-2.0 ft | Depth Ranges >2.0 ft |
|---|---|---|---|---|
| Rigid | Concrete | 0.015 | 0.013 | 0.013 |
| | Grouted Riprap | 0.040 | 0.030 | 0.028 |
| | Stone Masonry | 0.042 | 0.032 | 0.030 |
| | Soil Cement | 0.025 | 0.022 | 0.020 |
| | Asphalt | 0.018 | 0.016 | 0.016 |
| Unlined | Bare Soil | 0.023 | 0.020 | 0.020 |
| | Rock Cut | 0.045 | 0.035 | 0.025 |
| Temporary* | Woven Paper Net | 0.016 | 0.015 | 0.015 |
| | Jute Net | 0.028 | 0.022 | 0.019 |
| | Fiberglass Roving | 0.028 | 0.022 | 0.019 |
| | Straw with Net | 0.065 | 0.033 | 0.025 |
| | Curled Wood Mat | 0.066 | 0.035 | 0.028 |
| | Synthetic Mat | 0.036 | 0.025 | 0.021 |
| Gravel Riprap | 1-inch $D_{50}$ | 0.044 | 0.033 | 0.030 |
| | 2-inch $D_{50}$ | 0.066 | 0.041 | 0.034 |
| Rock Riprap | 6-inch $D_{50}$ | 0.104 | 0.069 | 0.035 |
| | 12-inch $D_{50}$ | - | 0.078 | 0.040 |

Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth.

*Some "temporary" linings become permanent when buried.

Source: HEC-15, 2005.

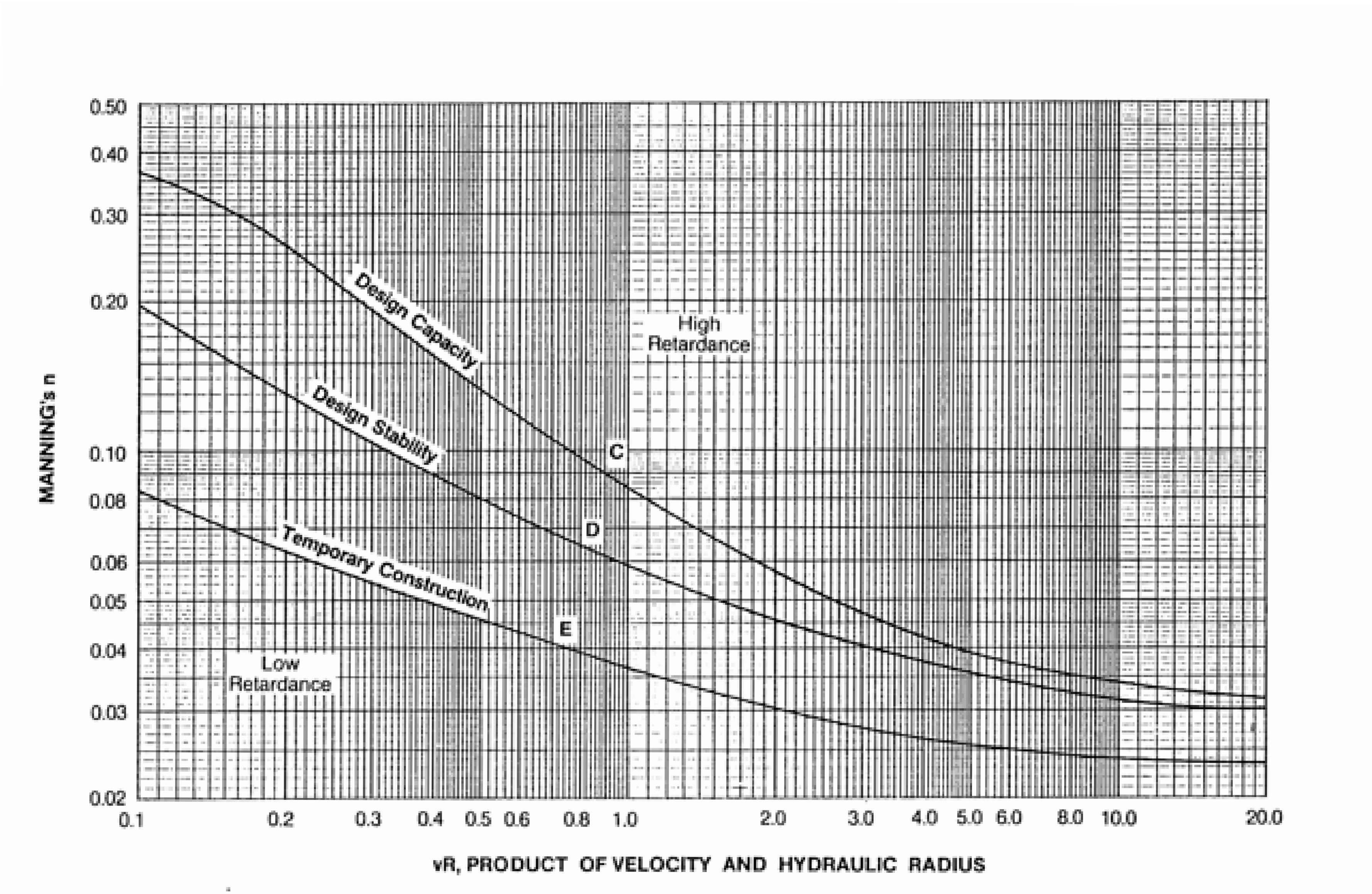



Figure 5.4-1 Manning's n Values for Vegetated Channels

(Source: USDA, TP-61, 1947)

Table 5.4-5 Uniform Flow Values of Roughness Coefficient n

| Type of Channel and Description | Minimum | Normal | Maximum |
|---|---|---|---|
| EXCAVATED OR DREDGED | | | |
| a. Earth, straight and uniform | 0.016 | 0.018 | 0.020 |
| 1. Clean, recently completed | 0.018 | 0.022 | 0.025 |
| 2. Clean, after weathering | 0.022 | 0.025 | 0.030 |
| 3. Gravel, uniform sector, clean | 0.022 | 0.027 | 0.033 |
| b. Earth, winding and sluggish | | | |
| 1. No vegetation | 0.023 | 0.025 | 0.030 |
| 2. Grass, some weeds | 0.025 | 0.030 | 0.033 |
| 3. Dense weeds/plants in deep channels | 0.030 | 0.035 | 0.040 |
| 4. Earth bottom and rubble sides | 0.025 | 0.030 | 0.035 |
| 5. Stony bottom and weedy sides | 0.025 | 0.035 | 0.045 |
| 6. Cobble bottom and clean sides | 0.030 | 0.040 | 0.050 |
| c. Dragline-excavated or dredged | | | |
| 1. No vegetation | 0.025 | 0.028 | 0.033 |
| 2. Light brush on banks | 0.035 | 0.050 | 0.060 |
| d. Rock cuts | | | |
| 1. Smooth and uniform | 0.025 | 0.035 | 0.040 |
| 2. Jagged and irregular | 0.035 | 0.040 | 0.050 |
| e. Channels not maintained, weeds and brush uncut | | | |
| 1. Dense weeds, high as flow depth | 0.050 | 0.080 | 0.120 |
| 2. Clean bottom, brush on sides | 0.040 | 0.050 | 0.080 |
| 3. Same, highest stage of flow | 0.045 | 0.070 | 0.110 |
| 4. Dense brush, high stage | 0.080 | 0.100 | 0.140 |
| NATURAL STREAMS | | | |
| Minor Streams (top width at flood stage < 100 ft) | | | |
| a. Steams on Plain | | | |
| 1. Clean, straight, full stage, no rifts or deep pools | 0.025 | 0.030 | 0.033 |
| 2. Same as above, but more stones and weeds | 0.030 | 0.035 | 0.040 |
| 3. Clean, winding, some pools and shoals | 0.033 | 0.040 | 0.045 |
| 4. Same as above, but some weeds and some stones | 0.035 | 0.045 | 0.050 |
| 5. Same as above, lower stages, more ineffective slopes and sections | 0.040 | 0.048 | 0.055 |
| 6. Same as 4, but more stones | 0.045 | 0.050 | 0.060 |
| 7. Sluggish reaches, weedy, deep pools | 0.050 | 0.070 | 0.080 |
| 8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush | 0.075 | 0.100 | 0.150 |

| Type of Channel and Description | Minimum | Normal | Maximum |
|---|---|---|---|
| b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages | | | |
| 1. Bottom: gravels, cobbles, few boulders | 0.030 | 0.040 | 0.050 |
| 2. Bottom: cobbles with large boulders | 0.040 | 0.050 | 0.070 |
| FLOODPLAINS | | | |
| a. Pasture, no brush | | | |
| 1. Short grass | 0.025 | 0.030 | 0.035 |
| 2. High grass | 0.030 | 0.035 | 0.050 |
| b. Cultivated area | | | |
| 1. No crop | 0.020 | 0.030 | 0.040 |
| 2. Mature row crops | 0.025 | 0.035 | 0.045 |
| 3. Mature field crops | 0.030 | 0.040 | 0.050 |
| c. Brush | | | |
| 1. Scattered brush, heavy weeds | 0.035 | 0.050 | 0.070 |
| 2. Light brush and trees, in winter | 0.035 | 0.050 | 0.060 |
| 3. Light brush and trees, in summer | 0.040 | 0.060 | 0.080 |
| 4. Medium to dense brush, in winter | 0.045 | 0.070 | 0.110 |
| 5. Medium to dense brush, in summer | 0.070 | 0.100 | 0.160 |
| d. Trees | | | |
| 1. Dense willows, summer, straight | 0.110 | 0.150 | 0.200 |
| 2. Cleared land, tree stumps, no sprouts | 0.030 | 0.040 | 0.050 |
| 3. Same as above, but with heavy growth of spouts | 0.050 | 0.060 | 0.080 |
| 4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches | 0.080 | 0.100 | 0.120 |
| 5. Same as above, but with flood stage reaching branches | 0.100 | 0.120 | 0.160 |
| Major streams (top width at flood stage > 100 ft). The n value is less than that for minor streams of similar description, because banks offers less effective resistance. | | | |
| a. Regular section with no boulders or brush | 0.025 | - | 0.060 |
| b. Irregular and rough section | 0.035 | - | 0.100 |

Source: HEC-15, 2005

Table 5.4-6 Classification of Vegetal Covers as to Degrees of Retardance

| Retardance | Cover | Condition |
|---|---|---|
| A | Weeping Lovegrass<br>Yellow Bluestem Ischaemum | Excellent stand, tall (average 30")<br>Excellent stand, tall (average 36") |
| B | Kudzu | Very dense growth, uncut |
| | Bermuda Grass | Good stand, tall (average 12") |
| | Native grass mixture:<br>little bluestem, bluestem, blue gamma, other short and long stem midwest lovegrass | Good stand, unmowed |
| | Weeping lovegrass | Good stand, tall (average 24") |
| | Laspedeza sericea | Good stand, not woody, tall (average 19") |
| | Alfalfa | Good stand, uncut (average 11") |
| | Weeping lovegrass | Good stand, unmowed (average 13") |
| | Kudzu | Dense growth, uncut |
| | Blue gamma | Good stand, uncut (average 13") |
| C | Crabgrass<br>Bermuda Grass | Fair stand, uncut (10 - 48")<br>Good stand, mowed (average 6") |
| | Common lespedeza | Good stand, uncut (average 11") |
| | Grass-legume mixture:<br>summer (orchard grass redtop, Italian ryegrass, and common lespedeza | Good stand, uncut (6 - 8") |
| | Centipede Grass | Very dense cover (average 6") |
| D | Bermuda grass<br>Common lespedeza | Good stand, cut to 2.5"<br>Excellent stand, uncut (average 4.5") |
| | Buffalo grass | Good stand, uncut (3 - 6") |
| | Grass-legume mixture:<br>fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza | Good stand, uncut (4 - 5") |
| | Lespedeza serices | After cutting to 2" (very good before cutting) |
| E | Bermuda grass<br>Bermuda grass | Good stand, cut to 1.5"<br>Burned stubble |

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

Source: HEC-15, 2005

### General Solution Nomograph

The following steps are used for the general solution nomograph in **Figure 5.4-2**:

(Step 1) Determine open channel data, including slope in ft/ft, hydraulic radius in ft, and Manning's n value.

(Step 2) Connect a line between the Manning's n scale and slope scale and note the point of intersection on the turning line.

(Step 3) Connect a line from the hydraulic radius to the point of intersection obtained in Step 2.

(Step 4) Extend the line from Step 3 to the velocity scale to obtain the velocity in ft/s.

### Trapezoidal Solution Nomograph

The trapezoidal channel nomograph solution to Manning's Equation in **Figure 5.4-3** can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known.

Determine input data, including slope in ft/ft, Manning's n value, bottom width in ft, and side slope in ft/ft.

1. **Given Q, find d.**
   a. Given the design discharge, find the product of Q times n, connect a line from the slope scale to the Qn scale, and find the point of intersection on the turning line.

   b. Connect a line from the turning point from Step 2a to the b scale and find the intersection with the z = 0 scale.

   c. Project horizontally from the point located in Step 2b to the appropriate z value and find the value of d/b.

   d. Multiply the value of d/b obtained in Step 2c by the bottom width b to find the depth of uniform flow, d.

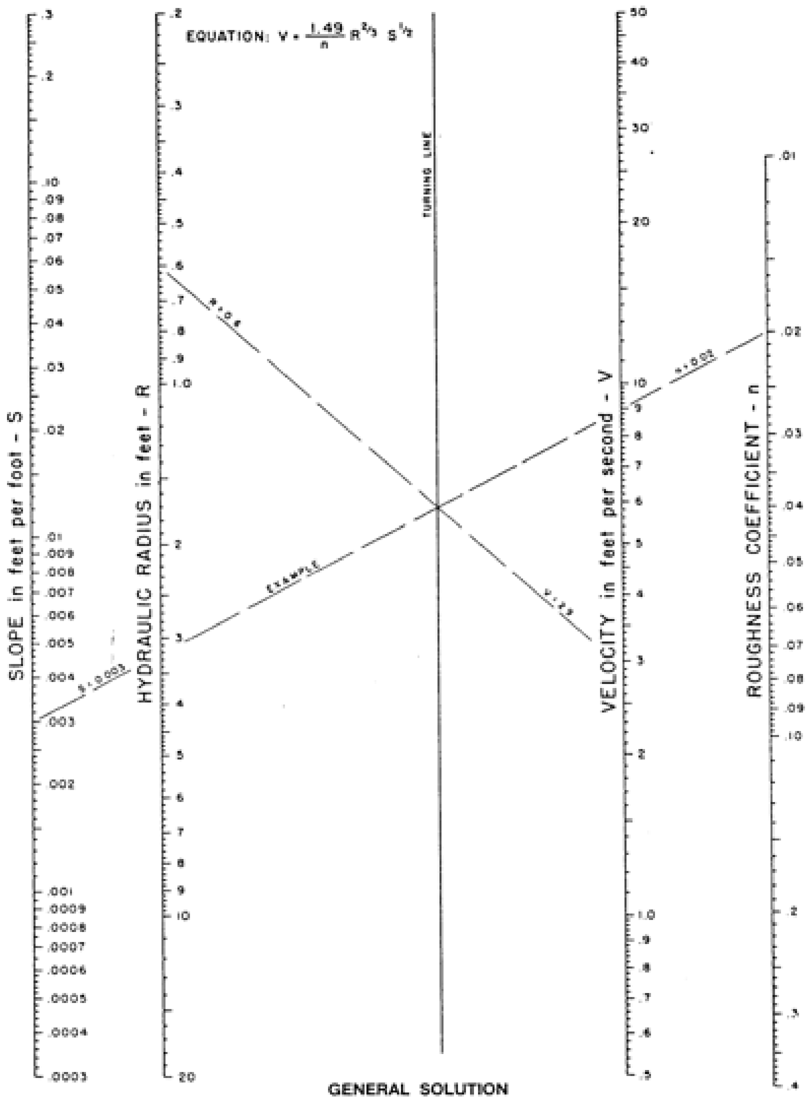
2. **Given d, find Q**
   a. Given the depth of flow, find the ratio d divided by b and project a horizontal line from the d/b ratio at the appropriate side slope, z, to the z = 0 scale.

   b. Connect a line from the point located in Step 3a to the b scale and find the intersection with the turning line.

   c. Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Qn scale.

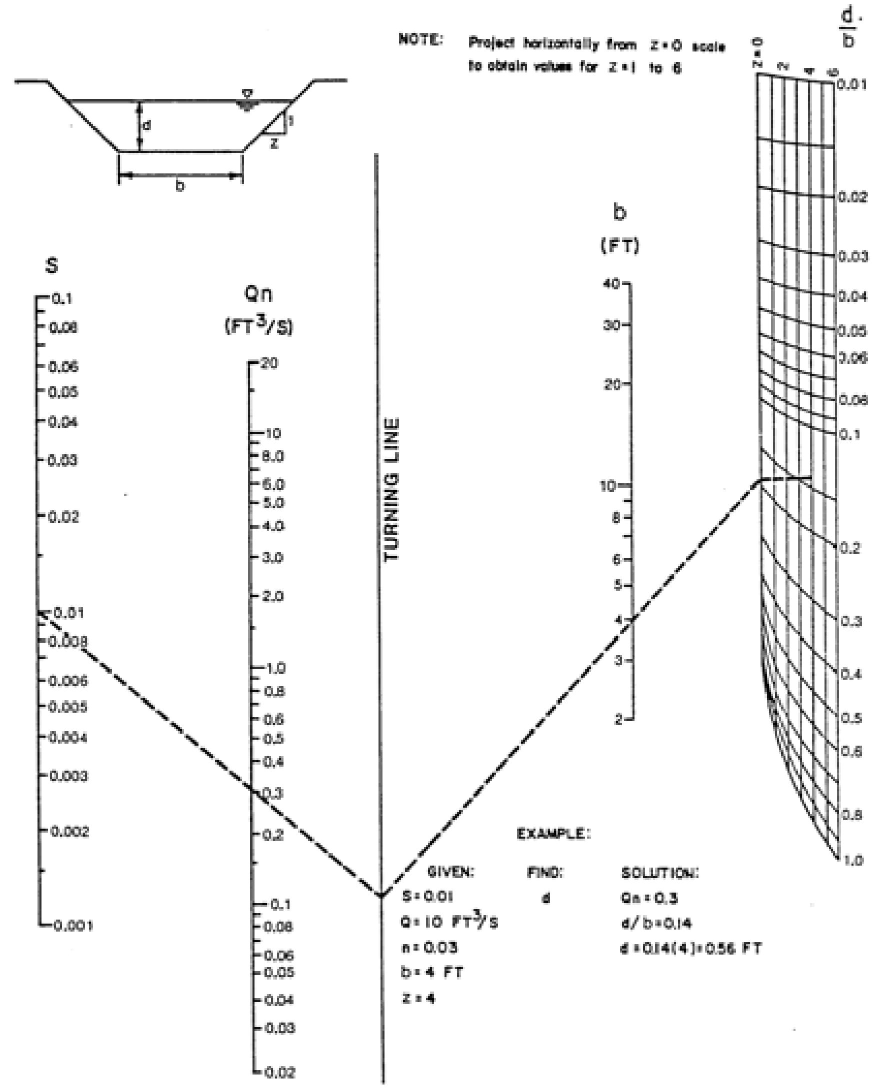
   d. Divide the value of Qn obtained in Step 3c by the n value to find the design discharge, Q.

BACK TO TOC




Reference: USDOT, FHWA, HDS-3 (1961).

Figure 5.4-2 Nomograph for the Solution of Manning's Equation




Reference: USDOT, FHWA, HEC-15 (1986).

Figure 5.4-3 Solution of Manning's Equation for Trapezoidal Channels

BACK TO TOC

#### 5.4.5.5 TRIAL AND ERROR SOLUTIONS

A trial and error procedure for solving Manning's Equation is used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness, and design discharge are known. For purposes of the trial and error process, Manning's Equation can be arranged as:

$$AR^{2/3} = (Q_n)/(1.49\ S^{1/2}) \qquad (5.4.4)$$

*Where*:

**A** = cross-sectional area (ft)
**R** = hydraulic radius (ft)
**Q** = discharge rate for design conditions (cfs)
**n** = Manning's roughness coefficient
**S** = slope of the energy grade line (ft/ft)

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of **equation 5.4.4** is satisfied such that the design flow is conveyed for the slope and selected channel cross section.

Graphical procedures for simplifying trial and error solutions are presented in **Figure 5.4-4** for trapezoidal channels.

(Step 1) Determine input data, including design discharge, Q, Manning's n value, channel bottom width, b, channel slope, S, and channel side slope, z.

(Step 2) Calculate the trapezoidal conveyance factor using the equation:

$$K_T = (Q_n)/(b^{8/3}S^{1/2}) \qquad (5.4.5)$$

*Where*:

**$K_T$** = trapezoidal open channel conveyance factor
**Q** = discharge rate for design conditions (cfs)
**n** = Manning's roughness coefficient
**b** = bottom width (ft)
**S** = slope of the energy grade line (ft/ft)

(Step 3) Enter the x-axis of **Figure 5.4-4** with the value of KT calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriate z value from Step 1.

(Step 4) From the point of intersection obtained in Step 3, draw a horizontal line to the y-axis and read the value of the normal depth of flow over the bottom width, d/b.

(Step 5) Multiply the d/b value from Step 4 by b to obtain the normal depth of flow.

Note: If bends are considered, refer to **equation 5.4.11**

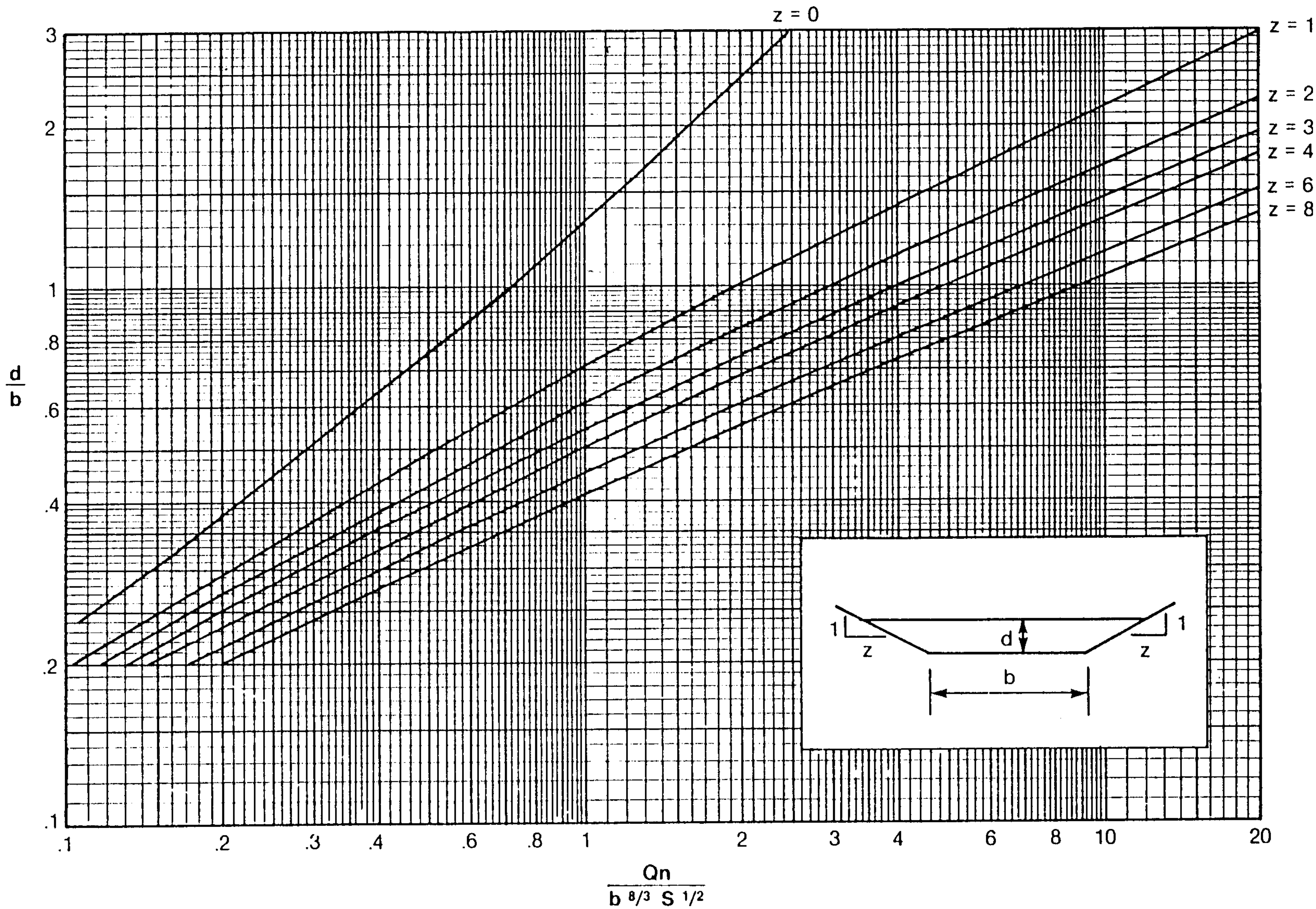



**Figure 5.4-4** Trapezoidal Channel Capacity Chart

(Source: Nashville Storm Water Management Manual, 1988)

BACK TO TOC

## 5.4.6 Critical Flow Calculations

### 5.4.6.1 BACKGROUND

In the design of open channels, it is important to calculate the critical depth in order to determine if the flow in the channel will be subcritical or supercritical. If the flow is subcritical it is relatively easy to handle the flow through channel transitions because the flows are tranquil and wave action is minimal. In subcritical flow, the depth at any point is influenced by a downstream control, which may be either the critical depth or the water surface elevation in a pond or larger downstream channel. In supercritical flow, the depth of flow at any point is influenced by a control upstream, usually critical depth. In addition, the flows have relatively shallow depths and high velocities.

Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth is expressed as:

$$Q^2/g = A^3/T \qquad (5.4.6)$$

*Where*:

**Q** = discharge rate for design conditions (cfs)

**g** = acceleration due to gravity (32.2 ft/sec$^2$)

**A** = cross-sectional area (ft$^2$)

**T** = top width of water surface (ft)

Note: A trial and error procedure is needed to solve **equation 5.4-6**.

### 5.4.6.2 SEMI-EMPIRICAL EQUATIONS

Semi-empirical equations (as presented in **Table 5.4-7**) or section factors (as presented in **Figure 5.4-5**) can be used to simplify trial and error critical depth calculations. The following equation is used to determine critical depth with the critical flow section factor, Z:

$$Z = Q/(g^{0.5}) \qquad (5.4.7)$$

*Where*:

**Z** = critical flow section factor

**Q** = discharge rate for design conditions (cfs)

**g** = acceleration due to gravity (32.2 ft/sec$^2$)

The following guidelines are given for evaluating critical flow conditions of open channel flow:

1. A normal depth of uniform flow within about 10% of critical depth is unstable and should be avoided in design, if possible.
2. If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.
3. If the velocity head is equal to one-half the mean depth of flow, the flow is critical.
4. If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.

*Note: The head is the height of water above any point, plane or datum of reference. The velocity head in flowing water is calculated as the velocity squared divided by 2 times the gravitational constant ($V^2/2g$).*

The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

$$Fr = v/(gA/T)^{0.5} \qquad (5.4.8)$$

*Where*:

**Fr** = Froude number (dimensionless)

**v** = velocity of flow (ft/s)

**g** = acceleration of gravity (32.2 ft/sec$^2$)

**A** = cross-sectional area of flow (ft$^2$)

**T** = top width of flow (ft)

If Fr is greater than 1.0, flow is supercritical; if it is under 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

Table 5.4-7 Critical Depth Equations for Uniform Flow in Selected Channel Cross Sections

| Channel Type[1] | Semi-Empirical Equations[2] for Estimating Critical Depth | Range of Applicability |
|---|---|---|
| 1. Rectangular[3] | $d_c = [Q^2/(gb^2)]^{1/3}$ | N/A |
| 2. Trapezoidal[3] | $d_c = 0.81[Q^2/gz^{0.75}b^{1.25})]^{0.27} - b/30z$ | $0.1 < 0.5522\ Q/b^{2.5} < 0.4$<br>For $0.5522\ Q/b^{2.5} < 0.1$, use rectangular channel |
| 3. Triangular[3] | $d_c = [(2Q^2)/(gz^2)]^{1/5}$ | N/A |
| 4. Circular[3] | $d_c = 0.325(Q/D)^{2/3} + 0.083D$ | $0.3 < d_c/D < 0.9$ |
| 5. General[3] | $(A^3/T) = (Q^2/g)$ | N/A |

*Where*:

$d_c$ = critical depth (ft)

Q = design discharge (cfs)

G = acceleration due to gravity (32.2 ft/s$^2$)

b = bottom width of channel (ft)

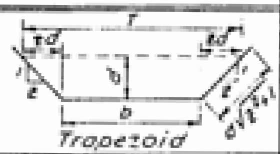
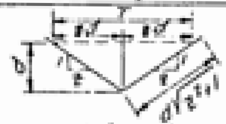
z = side slopes of a channel (horizontal to vertical)

D = diameter of a circular conduit (ft)

A = cross-sectional area of flow (ft$^2$)

T = top width of water surface (ft)

[1]See **Figure 5.4-5** for channel sketches
[2]Assumes uniform flow with the kinetic energy coefficient equal to 1.0
[3]Reference: French (1985)
[4]Reference: USDOT, FHWA, HDS-4 (1965)
[5]Reference: Brater and King (1976)

| Section | Area $A$ | Wetted Perimeter $P$ | Hydraulic Radius $R$ | Top Width $T$ | Critical Depth Factor, $Z$ |
|---|---|---|---|---|---|
|  Trapezoid | $bd+zd^2$ | $b+2d\sqrt{z^2+1}$ | $\frac{bd+zd^2}{b+2d\sqrt{z^2+1}}$ | $b+2zd$ | $\frac{[(b+zd)d]^{1.5}}{\sqrt{b+2zd}}$ |
|  Rectangle | $bd$ | $b+2d$ | $\frac{bd}{b+2d}$ | $b$ | $bd^{1.5}$ |
|  Triangle | $zd^2$ | $2d\sqrt{z^2+1}$ | $\frac{zd}{2\sqrt{z^2+1}}$ | $2zd$ | $\frac{\sqrt{2}}{2}zd^{2.5}$ |
|  Parabola | $\frac{2}{3}dT$ | $T+\frac{8d^2}{3T}$ [1] | $\frac{2dT^2}{3T^2+8d^2}$ [1] | $\frac{3a}{2d}$ | $\frac{2}{9}\sqrt{6}\,Td^{1.5}$ |
|  Circle - <½ full [2] | $\frac{D^2}{8}\left(\frac{\pi\theta}{180}-\sin\theta\right)$ | $\frac{\pi D\theta}{360}$ | $\frac{45D}{\pi\theta}\left(\frac{\pi\theta}{180}-\sin\theta\right)$ | $D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$ | $a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$ |
|  Circle - >½ full [3] | $\frac{D^2}{8}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$ | $\frac{\pi D(360-\theta)}{360}$ | $\frac{45D}{\pi(360-\theta)}\left(2\pi-\frac{\pi\theta}{180}+\sin\theta\right)$ | $D\sin\frac{\theta}{2}$ or $2\sqrt{d(D-d)}$ | $a\sqrt{\frac{a}{D\sin\frac{\theta}{2}}}$ |

[1] Satisfactory approximation for the interval $0 < \frac{d}{T} \leq 0.25$
When $d/T > 0.25$, use $p = \frac{1}{2}\sqrt{16d^2+T^2} + \frac{T^2}{8d}\sinh^{-1}\frac{4d}{T}$

[2] $\theta = 4\sin^{-1}\sqrt{d/D}$
[3] $\theta = 4\cos^{-1}\sqrt{d/D}$
Insert $\theta$ in degrees in above equations

Note: Small z = Side Slope Horizontal Distance
Large Z = Critical Depth Section Factor

Reference: USDA, SCS, NEH-5 (1956).

**Figure 5.4-5** Open Channel Geometric Relationships for Various Cross Sections

## 5.4.7 Vegetative Design

### 5.4.7.1 INTRODUCTION

A two-part procedure is recommended for final design of temporary and vegetative channel linings. Part 1, the design stability component, involves determining channel dimensions for low vegetative retardance conditions, using Class D as defined in **Table 5.4-6**. Part 2, the design capacity component, involves determining the depth increase necessary to maintain capacity for higher vegetative retardance conditions, using Class C as defined in **Table 5.4-6**. If temporary lining is to be used during construction, vegetative retardance Class E should be used for the design stability calculations.

If the channel slope exceeds 10%, or a combination of channel linings will be used, additional procedures can be found in HEC-15 (USDOT, FHWA, 2005) and HEC-14 (USDOT, FHWA, 2012).

### 5.4.7.2 DESIGN STABILITY

The following are the steps for design stability calculations:

(Step 1) Determine appropriate design variables, including discharge, Q, bottom slope, S, cross section parameters, and vegetation type.

(Step 2) Use **Table 5.4-3** to assign a maximum velocity, vm based on vegetation type and slope range.

(Step 3) Assume a value of n and determine the corresponding value of vR from the n versus vR curves in **Figure 5.4-1**. Use retardance Class D for permanent vege-tation and E for temporary construction.

(Step 4) Calculate the hydraulic radius using the equation:

$$\mathbf{R = (vR)/v_m} \quad \mathbf{(5.4.9)}$$

*Where*:

**R** = hydraulic radius of flow (ft)

**vR** = value obtained from **Figure 5.4-1** in Step 3

$\mathbf{v_m}$ = maximum velocity from Step 2 (ft/s)

(Step 5) Use the following form of Manning's Equation to calculate the value of vR:

$$\mathbf{vR = (1.49\ R^{5/3}\ S^{1/2})/n} \quad \mathbf{(5.4.10)}$$

*Where*:

**vR** = calculated value of vR product

**R** = hydraulic radius value from Step 4 (ft)

**S** = channel bottom slope (ft/ft)

**n** = Manning's n value assumed in Step 3

(Step 6) Compare the vR product value obtained in Step 5 to the value obtained from **Figure 5.4-1** for the assumed n value in Step 3. If the values are not rea-sonably close, return to Step 3 and repeat the calculations using a new assumed n value.

(Step 7) For trapezoidal channels, find the flow depth using **Figures 5.4-3** or **5.4-4**, as described in Section 5.4.4. The depth of flow for other channel shapes can be evaluated using the trial and error procedure described in Section 5.4.5.

(Step 8) If bends are considered, calculate the length of downstream protection, $L_p$, for the bend, using **Figure 5.4-6**. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, $L_p$.

### 5.4.7.3 DESIGN CAPACITY

The following are the steps for design capacity calculations:

(Step 1) Assume a depth of flow greater than the value from Step 7 above and compute the waterway area and hydraulic radius (see **Figure 5.4-5** for equations).

(Step 2) Divide the design flow rate, obtained using appropriate procedures from Chapter 2, by the waterway area from Step 1 to find the velocity.

(Step 3) Multiply the velocity from Step 2 by the hydraulic radius from Step 1 to find the value of vR.

(Step 4) Use **Figure 5.4-1** to find a Manning's n value for retardance Class C based on the vR value from Step 3.

(Step 5) Use Manning's Equation (**equation 5.4.1**) or **Figure 5.4-2** to find the velocity using the hydraulic radius from Step 1, Manning's n value from Step 4, and appropriate bottom slope.

(Step 6) Compare the velocity values from Steps 2 and 5. If the values are not reasonably close, return to Step 1 and repeat the calculations.

(Step 7) Add an appropriate freeboard to the final depth from Step 6. Generally, 20% is adequate.

(Step 8) If bends are considered, calculate superelevation of the water surface profile at the bend using the equation:

$$\Delta d = (v^2T)/(gR_c) \qquad (5.4.11)$$

*Where*:

**Δd** = superelevation of the water surface profile due to the bend (ft)

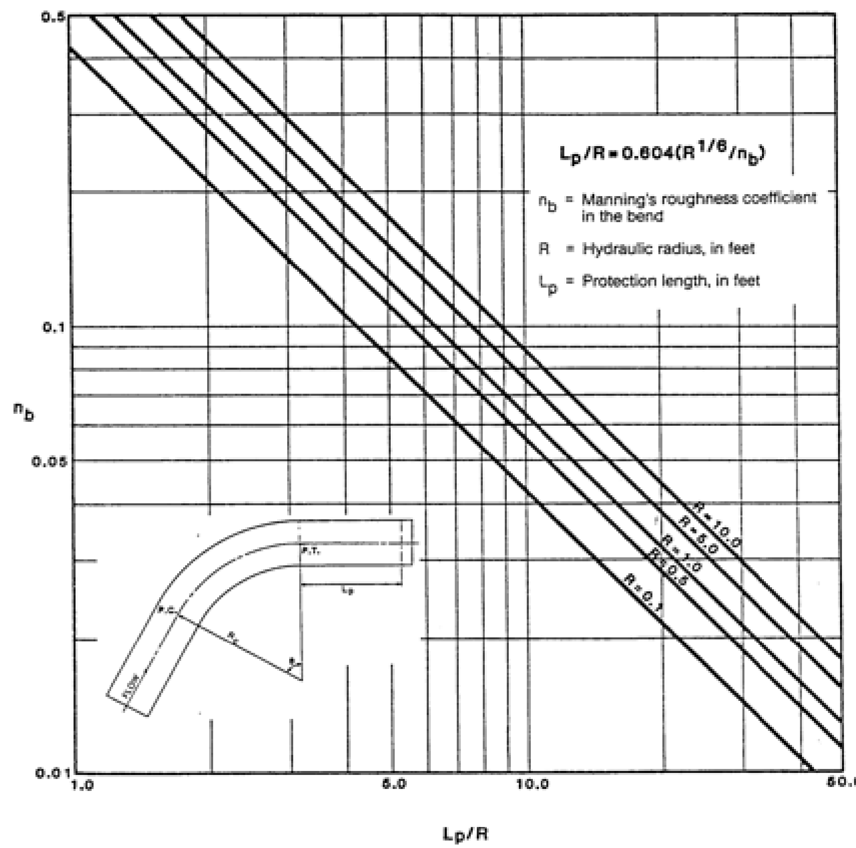
**v** = average velocity from Step 6 (ft/s)

**T** = top width of flow (ft)

**g** = acceleration of gravity (32.2 ft/sec$^2$)

**R$_c$** = mean radius of the bend (ft)

*Note: Add freeboard consistent with the calculated Δd.*




*Reference: USDOT, FHWA, HEC-15 (1986).*

**Figure 5.4-6** Protection Length, Lp, Downstream of Channel Bend

BACK TO TOC

## 5.4.8 Riprap Design

### 5.4.8.1 ASSUMPTIONS

The following procedure is based on results and analysis of laboratory and field data (Maynord, 1987; Reese, 1984; Reese, 1988). This procedure applies to riprap place-ment in both natural and prismatic channels and has the following assump-tions and limitations:

- Minimum riprap thickness equal to $d_{100}$
- The value of $d_{85}/d_{15}$ less than 4.6
- Froude number less than 1.2
- Side slopes up to 2:1
- A safety factor of 1.2
- Maximum velocity less than 18 feet per second

If significant turbulence is caused by boundary irregularities, such as obstructions or structures, this procedure is not applicable.

### 5.4.8.2 PROCEDURE

Following are the steps in the procedure for riprap design:

(Step 1) Determine the average velocity in the main channel for the design condition. Manning's n values for riprap can be calculated from the equation:

$$n = 0.0395\,(d_{50})^{1/6} \qquad (5.4.12)$$

*Where*:

**n** = Manning's roughness coefficient for stone riprap

**$d_{50}$** = diameter of stone for which 50%, by weight, of the gradation is finer (ft)

(Step 2) If rock is to be placed at the outside of a bend, multiply the velocity determined in Step 1 by the bend correction coefficient, $C_b$, given in **Figure 5.4-7** for either a natural or prismatic channel. This requires determining the channel top width, T, just upstream from the bend and the centerline bend radius, $R_b$.

(Step 3) If the specific weight of the stone varies significantly from 165 pounds per cubic foot, multiply the velocity from Step 1 or 2 (as appropriate) by the specific weight correction coefficient, $C_g$, from **Figure 5.4-8**.

(Step 4) Determine the required minimum $d_{30}$ value from **Figure 5.4-9**, or from the equation:

$$d_{30}/D = 0.193\,Fr^{2.5} \qquad (5.4.13)$$

*Where*:

**$d_{30}$** = diameter of stone for which 30%, by weight, of the gradation is finer (ft)

**D** = depth of flow above stone (ft)

**Fr** = Froude number (see equation 5.4.8), dimensionless

**v** = mean velocity above the stone (ft/s)

(Step 5) Determine available riprap gradations. A well graded riprap is preferable to uniform size or gap graded. The diameter of the largest stone, $d_{100}$, should not be more than 1.5 times the $d_{50}$ size. Blanket thickness should be greater than or equal to $d_{100}$ except as noted below. Sufficient fines (below $d_{15}$) should be available to fill the voids in the larger rock sizes. The stone weight for a selected stone size can be calculated from the equation:

$$W = 0.5236\,SW_s\,d^3 \qquad (5.4.14)$$

*Where*:

**W** = stone weight (lbs)

**d** = selected stone diameter (ft)

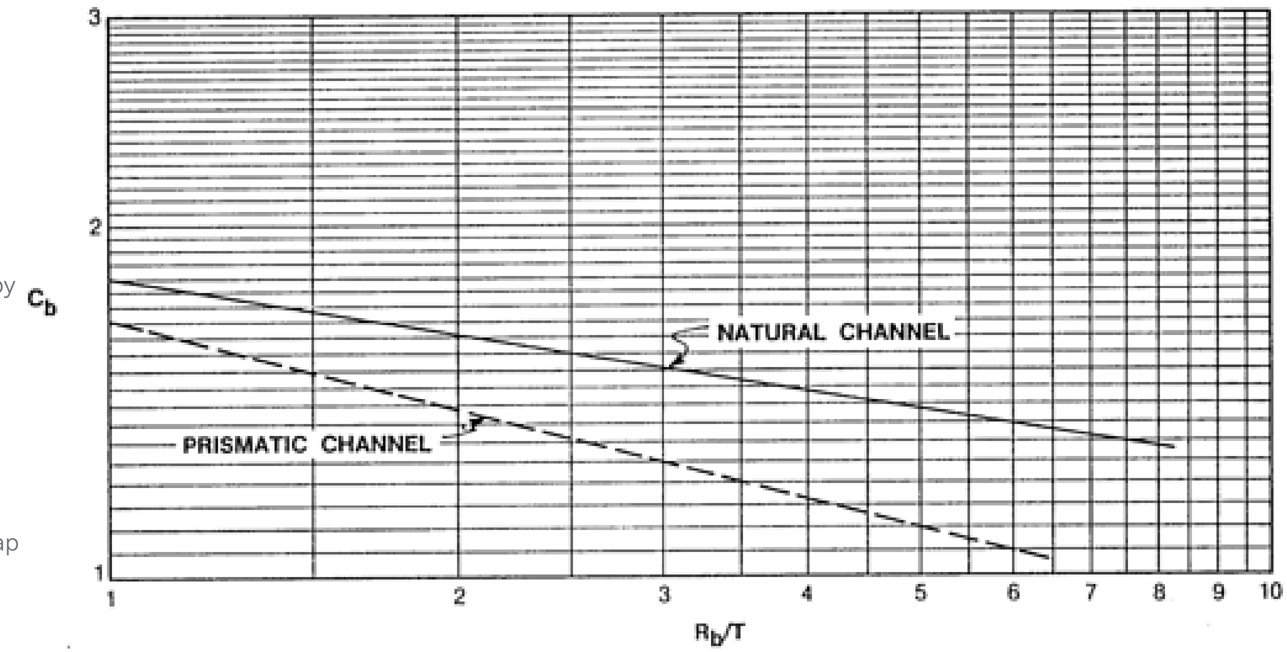
**$SW_s$** = specific weight of stone ($lbs/ft^3$)

Filter fabric or a filter stone layer should be used to prevent turbulence or groundwater seepage from removing bank material through the stone or to serve as a foundation for unconsolidated material. Layer thickness should be increased by 50% for underwater placement.

(Step 6) If $d_{85}/d_{15}$ is between 2.0 and 2.3 and a smaller $d_{30}$ size is desired, a thickness greater than $d_{100}$ can be used to offset the smaller $d_{30}$ size. **Figure 5.4-10** can be used to make an approximate adjustment using the ratio of $d_{30}$ sizes. Enter the y-axis with the ratio of the desired $d_{30}$ size to the standard $d_{30}$ size and find the thickness ratio increase on the x-axis. Other minor gradation deficiencies may be compensated for by increasing the stone blanket thickness.

(Step 7) Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.

**Figure 5.4-7** Riprap Lining Bend Correction Coefficient




To obtain effective velocity, multiply known velocity by $C_b$.

T = Channel Top Width
$R_b$ = Centerline Bend Radius
$C_b$ = Correction Coefficient

*Reference: Maynord (1987).*

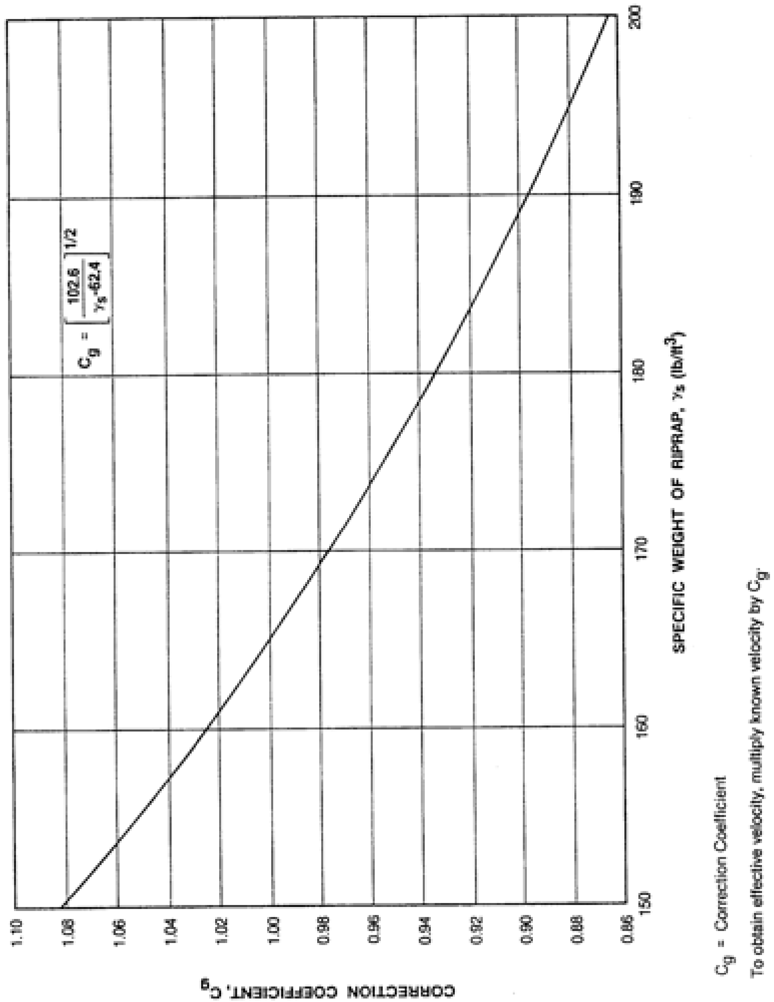
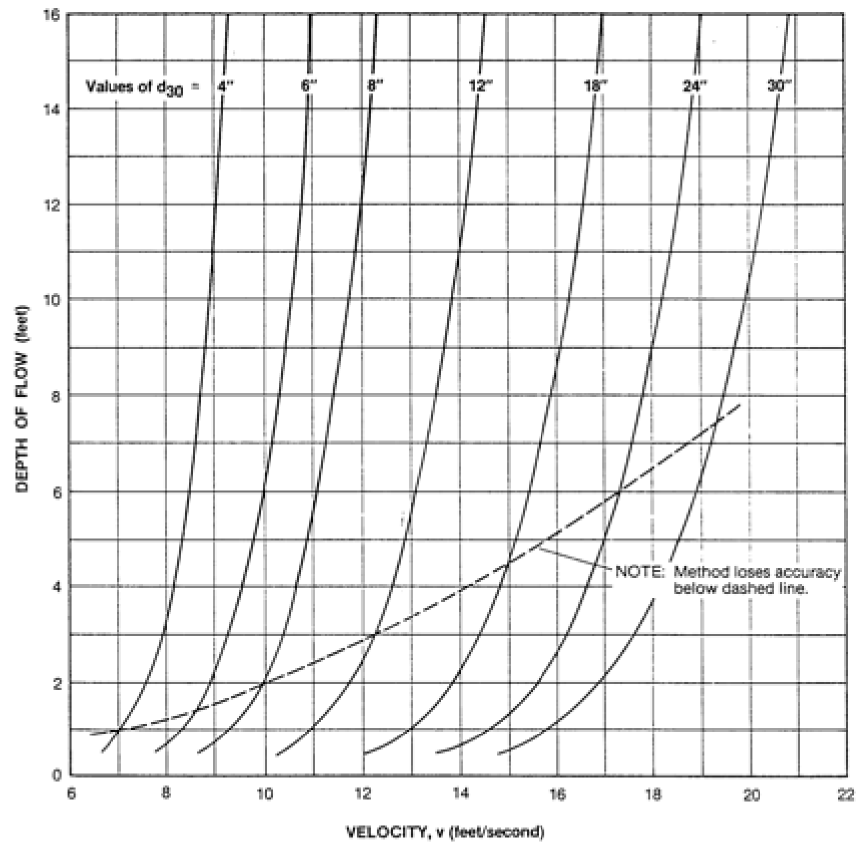



**Figure 5.4-8** Riprap Lining Specific Weight Correction Coefficient

(Source: Nashville Storm Water Management Manual, 1988)




*Reference: Reese (1988).*

**Figure 5.4-9** Riprap Lining $d^{30}$ Stone Size – Function of Mean Velocity and Depth

BACK TO TOC

## 5.4.9 Uniform Flow - Example Problems

**Example 1 -- Direct Solution of Manning's Equation**

Use Manning's Equation to find the velocity, v, for an open channel with a hydraulic radius value of 0.6 ft, an n value of 0.020, and slope of 0.003 ft/ft. Solve using **Figure 5.4-2**:

1. Connect a line between the slope scale at 0.003 and the roughness scale at 0.020 and note the intersection point on the turning line.

2. Connect a line between that intersection point and the hydraulic radius scale at 0.6 ft and read the velocity of 2.9 ft/s from the velocity scale.

**Example 2 -- Grassed Channel Design Stability**

A trapezoidal channel is required to carry 50 cfs at a bottom slope of 0.015 ft/ft. Find the channel dimensions required for design stability criteria (retardance Class D) for a grass mixture.

1. From **Table 5.4-3**, the maximum velocity, vm, for a grass mixture with a bottom slope less than 5% is 4 ft/s.

2. Assume an n value of 0.035 and find the value of vR from **Figure 5.4-1**, vR = 5.4

3. Use **equation 5.4.9** to calculate the value of R: R = 5.4/4 = 1.35 ft

4. Use **equation 5.4.10** to calculate the value of vR:

   $vR = [1.49\ (1.35)^{5/3}\ (0.015)^{1/2}]/0.035 = 8.60$

5. Since the vR value calculated in Step 4 is higher than the value obtained from Step 2, a higher n value is required and calculations are repeated. The results from each trial of calculations are presented below:

| Assumed n Value | vR (Fig. 5.4-1) | R (eq. 5.4.9) | vR (eq. 5.4.10) |
|---|---|---|---|
| 0.035 | 5.40 | 1.35 | 8.60 |
| 0.038 | 3.8 | 0.95 | 4.41 |
| 0.039 | 3.4 | 0.85 | 3.57 |
| 0.040 | 3.2 | 0.80 | 3.15 |

Select n = 0.040 for stability criteria.

6. Use **Figure 5.4-3** to select channel dimensions for a trapezoidal shape with 3:1 side slopes.

   - Qn = (50) (0.040) = 2.0 S = 0.015
   - For b = 10 ft, d = (10) (0.098) = 0.98 ft
     b = 8 ft, d = (8) (0.14) = 1.12 ft

*Select*:

- b = 10 ft, such that R is approximately 0.80 ft
- z = 3
- d = 1 ft
- v = 3.9 ft/s (**equation 5.4.1**)
- Fr = 0.76 (**equation 5.4.8**)
- Flow is subcritical

Design capacity calculations for this channel are presented in Example 3 below.

**Example 3 -- Grassed Channel Design Capacity**

Use a 10-ft bottom width and 3:1 side-slopes for the trapezoidal channel sized in Example 2 and find the depth of flow for retardance Class C.

1. Assume a depth of 1.0 ft and calculate the following (see Figure 5.4-5):
   - $A = (b + zd)\ d = [10 + (3)\ (1)\ ]\ (1) = 13.0$ sq ft
   - $R = \{[b + zd]\ d\}/\{b + [2d(1 + z^2)^{0.5}]\} = \{[10+(3)(1)](1)\}/\{10+[(2)(1)(1+3^2)^{0.5}]\}$
   - R = 0.796 ft

2. Find the velocity: v = Q/A = 50/13.0 = 3.85 ft/s

3. Find the value of vR: vR = (3.85) (0.796) = 3.06

4. Using the vR product from Step 3, find Manning's n from **Figure 5.4-1** for retardance Class C (n = 0.047)

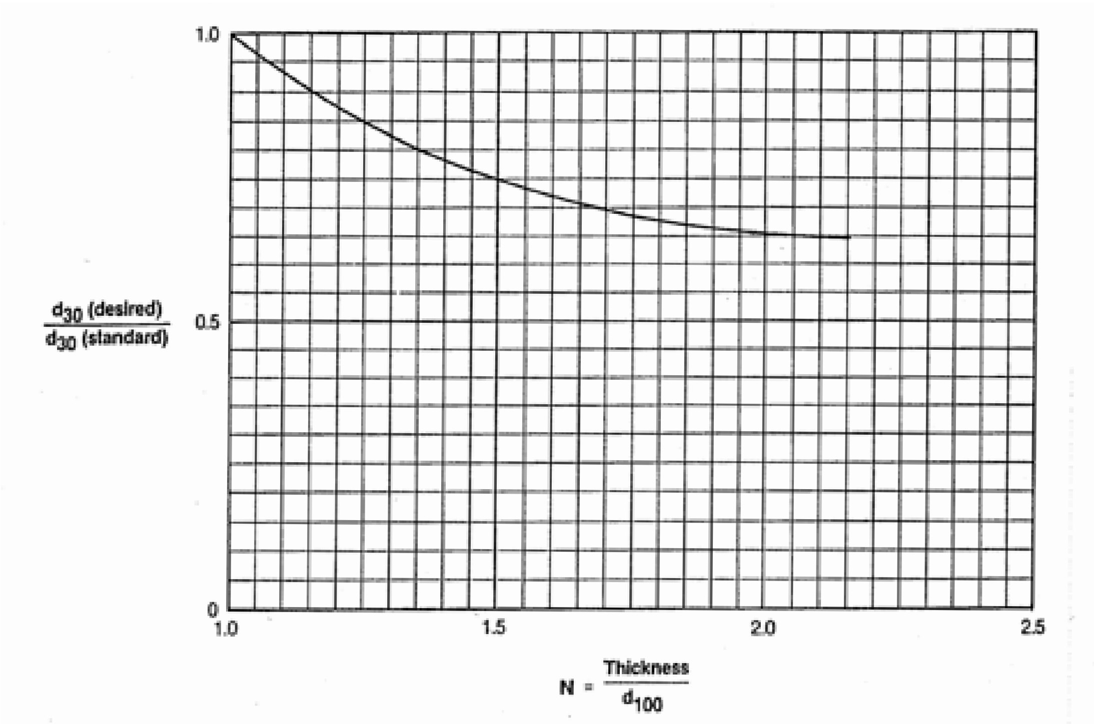



**Figure 5.4-10** Riprap Lining Thickness Adjustment for $d_{85}/d_{15}$ = 1.0 to 2.3

(Source: Maynord, 1987)

5. Use **Figure 5.4-2** or **equation 5.4.1** to find the velocity for S = 0.015, R = 0.796, and n = 0.047: v = 3.34 ft/s

6. Since 3.34 ft/s is less than 3.85 ft/s, a higher depth is required and calculations are repeated. Results from each trial of calculations are presented below:

| Assumed Depth (ft) | Area (ft) | R (ft) | Velocity Q/A (ft/sec) | vR | Manning's n **(Fig. 5.4-3)** | Velocity **(Eq. 5.4.11)** |
|---|---|---|---|---|---|---|
| 1.0 | 13.00 | 0.796 | 3.85 | 3.06 | 0.047 | 3.34 |
| 1.05 | 13.81 | 0.830 | 3.62 | 3.00 | 0.047 | 3.39 |
| 1.1 | 14.63 | 0.863 | 3.42 | 2.95 | 0.048 | 3.45 |
| 1.2 | 16.32 | 0.928 | 3.06 | 2.84 | 0.049 | 3.54 |

7. Select a depth of 1.1 with an n value of 0.048 for design capacity requirements. Add at least 0.2 ft for freeboard to give a design depth of 1.3 ft. Design data for the trapezoidal channel are summarized as follows:

   - Vegetation lining = grass mixture, $v_m$ = 4 ft/s
   - Q = 50 cfs
   - b = 10 ft, d = 1.3 ft, z = 3, S = 0.015
   - Top width = (10) + (2) (3) (1.3) = 17.8 ft
   - n (stability) = 0.040, d = 1.0 ft, v = 3.9 ft/s, Froude number = 0.76 (**equation 5.4.8**)
   - n (capacity) = 0.048, d = 1.1 ft, v = 3.45 ft/s, Froude number = 0.64 (**equation 5.4.8**)

**Example 4 -- Riprap Design**

A natural channel has an average bankfull channel velocity of 8 ft per second with a top width of 20 ft and a bend radius of 50 ft. The depth over the toe of the outer bank is 5 ft. Available stone weight is 170 lbs/ft$^3$. Stone placement is on a side slope of 2:1 (horizontal:vertical). Determine riprap size at the outside of the bend.

BACK TO TOC

1. Use 8 ft/s as the design velocity, because the reach is short and the bend is not protected.

2. Determine the bend correction coefficient for the ratio of $R_b/T = 50/20 = 2.5$. From **Figure 5.4-7**, $C_b = 1.55$. The adjusted effective velocity is (8) (1.55) = 12.4 ft/s.

3. Determine the correction coefficient for the specific weight of 170 lbs from **Figure 5.4-8** as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 ft/s.

4. Determine minimum $d_{30}$ from **Figure 5.4-9** or **equation 5.4.13** as about 10 inches.

5. Use a gradation with a minimum $d_{30}$ size of 10 inches.

6. (Optional) Another gradation is available with a $d_{30}$ of 8 inches. The ratio of desired to standard stone size is 8/10 or 0.8. From **Figure 5.4-10**, this gradation would be acceptable if the blanket thickness was increased from the original $d_{100}$ (diameter of the largest stone) thickness by 35% (a ratio of 1.35 on the horizontal axis).

7. Perform preliminary design. Make sure that the stone is carried up and downstream far enough to ensure stability of the channel and that the toe will not be undermined. The downstream length of protection for channel bends can be determined using **Figure 5.4-6**.

## 5.4.10 Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile should be computed using backwater techniques.

Many computer programs are available for computation of backwater curves. The most general and widely used programs are, HEC-RAS, developed by the U.S. Army Corps of Engineers and Bridge Waterways Analysis Model (WSPRO) developed for the Federal Highway Administration. These programs can be used to compute water surface profiles for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the direct step method. For an irregular nonuniform channel, the standard step method is recommended, although it is a more tedious and iterative process. The use of HEC-RAS is recommended for standard step calculations.

Cross sections for water surface profile calculations should be normal to the direction of flood flow. The number of sections required will depend on the irregularity of the stream and flood plain. In general, a cross section should be obtained at each location where there are significant changes in stream width, shape, or vegetal patterns. Sections should usually be no more than 4 to 5 channel widths apart or 100 feet apart for ditches or streams and 500 feet apart for floodplains, unless the channel is very regular.

## 5.4.11 Rectangular, Triangular, and Trapezoidal Open Channel Design Figures

### 5.4.11.1 INTRODUCTION

The Federal Highway Administration has prepared numerous design figures to aid in the design of open channels. Copies of these figures, a brief description of their use, and several example design problems are presented. For design conditions not covered by the figures, a trial and error solution of Manning's Equation must be used.

### 5.4.11.2 DESCRIPTION OF FIGURES

Figures given in Appendix C are for the direct solution of the Manning's Equation for various sized open channels with rectangular, triangular, and trapezoidal cross sections. Each figure (except for the triangular cross section) is prepared for a channel of given bottom width and a particular value of Manning's n.

The figures for rectangular and trapezoidal cross section channels (Appendix C) are used the same way. The abscissa scale of discharge in cubic feet per second (cfs), and the ordinate scale is velocity in feet per second (ft/s). Both scales are logarithmic. Superimposed on the logarithmic grid are steeply inclined lines representing depth (ft), and slightly inclined lines representing channel slope (ft/ft).

BACK TO TOC

A heavy dashed line on each figure shows critical flow conditions. Auxiliary abscissa and ordinate scales are provided for use with other values of n and are explained in the example problems. In the figures, interpolations may be made not only on the ordinate and abscissa scales but between the inclined lines representing depth and slope.

The chart for a triangular cross section (Appendix C) is in nomograph form. It may be used for street sections with a vertical (or nearly vertical) curb face. The nomograph also may be used for shallow V shaped sections by following the instructions on the chart.

### 5.4.11.3 INSTRUCTIONS FOR RECTANGULAR AND TRAPEZOIDAL FIGURES

Figures in Appendix C provide a solution of the Manning equation for flow in open channels of uniform slope, cross section, and roughness, provided the flow is not affected by backwater and the channel has a length sufficient to establish uniform flow.

For a given slope and channel cross section, when n is 0.015 for rectangular channels or 0.03 for trapezoidal channels, the depth and velocity of uniform flow may be read directly from the figure for that size channel. The initial step is to locate the intersection of a vertical line through the discharge (abscissa) and the appropriate slope line. At this intersection, the depth of flow is read from the depth lines, and the mean velocity is read on the ordinate scale.

The procedure is reversed to determine the discharge at a given depth of flow. Critical depth, slope, and velocity for a given discharge can be read on the appropriate scale at the intersection of the critical curve and a vertical line through the discharge.

Auxiliary scales, labeled Qn (abscissa) and Vn (ordinate), are provided so the figures can be used for values of n other than those for which the charts were basically prepared. To use these scales, multiply the discharge by the value of n and use the Qn and Vn scales instead of the Q and V scales, except for computation of critical depth or critical velocity. To obtain normal velocity V from a value on the Vn scale, divide the value by n. The following examples will illustrate these points.

**Example Design Problem 1**

*Given*:
A rectangular concrete channel 5 ft wide with n = 0.015, .06 percent slope (S = .0006), discharging 60 cfs.

*Find*:
Depth, velocity, and type of flow

*Procedure*:

1. From Appendix C select the rectangular figure for a 5-ft width (**Figure 5.4-11**).
2. From 60 cfs on the Q scale, move vertically to intersect the slope line S = .0006, and from the depth lines read dn = 3.7 ft.
3. Move horizontally from the same intersection and read the normal velocity, V = 3.2 ft/s, on the ordinate scale.
4. The intersection lies below the critical curve, and the flow is therefore in the subcritical range.

**Example Design Problem 2**

*Given*:
A trapezoidal channel with 2:1 side slopes and a 4 ft bottom width, with n = 0.030, 0.2% slope (S = 0.002), discharging 50 cfs.

*Find*:
Depth, velocity, type flow.

*Procedure*:

1. Select the trapezoidal figure for b = 4 ft (see **Figure 5.4-12**).
2. From 50 cfs on the Q scale, move vertically to intersect the slope line S = 0.002 and from the depth lines read dn = 2.2 ft.
3. Move horizontally from the same intersection and read the normal velocity, V = 2.75 ft/s, on the ordinate scale. The intersection lies below the critical curve,\; the flow is therefore subcritical.

**Example Design Problem 3**

*Given*:

A rectangular cement rubble masonry channel 5 ft wide, with n = 0.025, 0.5% slope (S = 0.005), discharging 80 cfs.

*Find*:

Depth velocity and type of flow

*Procedure*:

1. Select the rectangular figure for a 5 ft width (**Figure 5.4-13**).

2. Multiply Q by n to obtain Qn: 80 x 0.025 = 2.0.

3. From 2.0 on the Qn scale, move vertically to intersect the slope line, S = 0.005, and at the intersection read $d_n$ = 3.1 ft.

4. Move horizontally from the intersection and read Vn = .13, then Vn/n = 0.13/0.025 = 5.2 ft/s.

5. Critical depth and critical velocity are independent of the value of n so their values can be read at the intersection of the critical curve with a vertical line through the discharge. For 80 cfs, on **Figure 5.4-13**, $d_c$ = 2.0 ft and $V_c$ = 7.9 ft/s. The normal velocity, 5.2 ft/s (from step 4), is less than the critical velocity, and the flow is therefore subcritical. It will also be noted that the normal depth, 3.0 ft, is greater than the critical depth, 2.0 ft, which also indicates subcritical flow.

6. To determine the critical slope for Q = 80 cfs and n = 0.025, start at the intersection of the critical curve and a vertical line through the discharge, Q = 80 cfs, finding $d_c$ (2.0 ft) at this point. Follow along this $d_c$ line to its intersection with a vertical line through Qn = 2.0 (step 2), at this intersection read the slope value $S_c$ = 0.015.

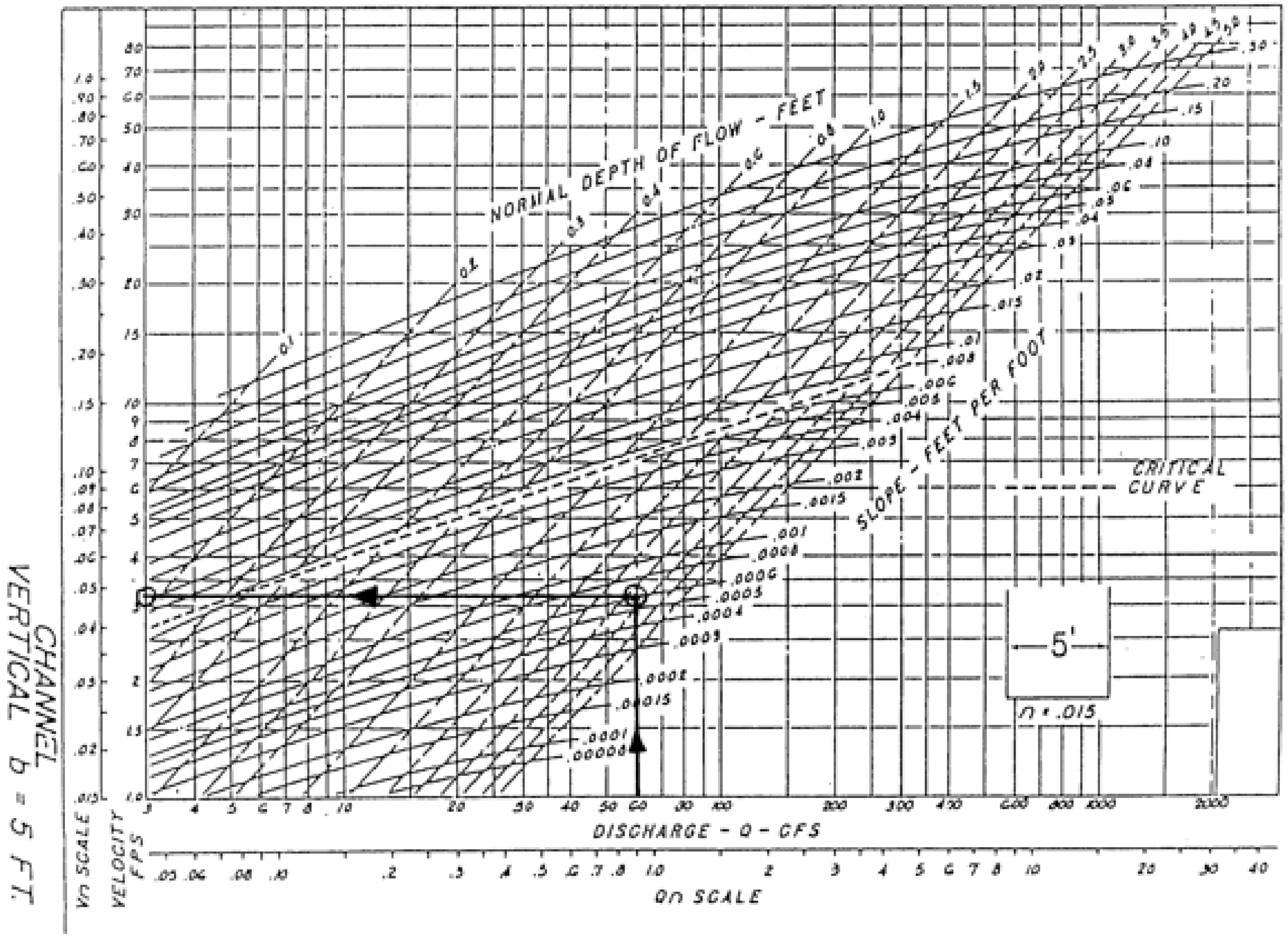



**Figure 5.4-11** Example Nomograph #1

(Source: FHWA)

BACK TO TOC

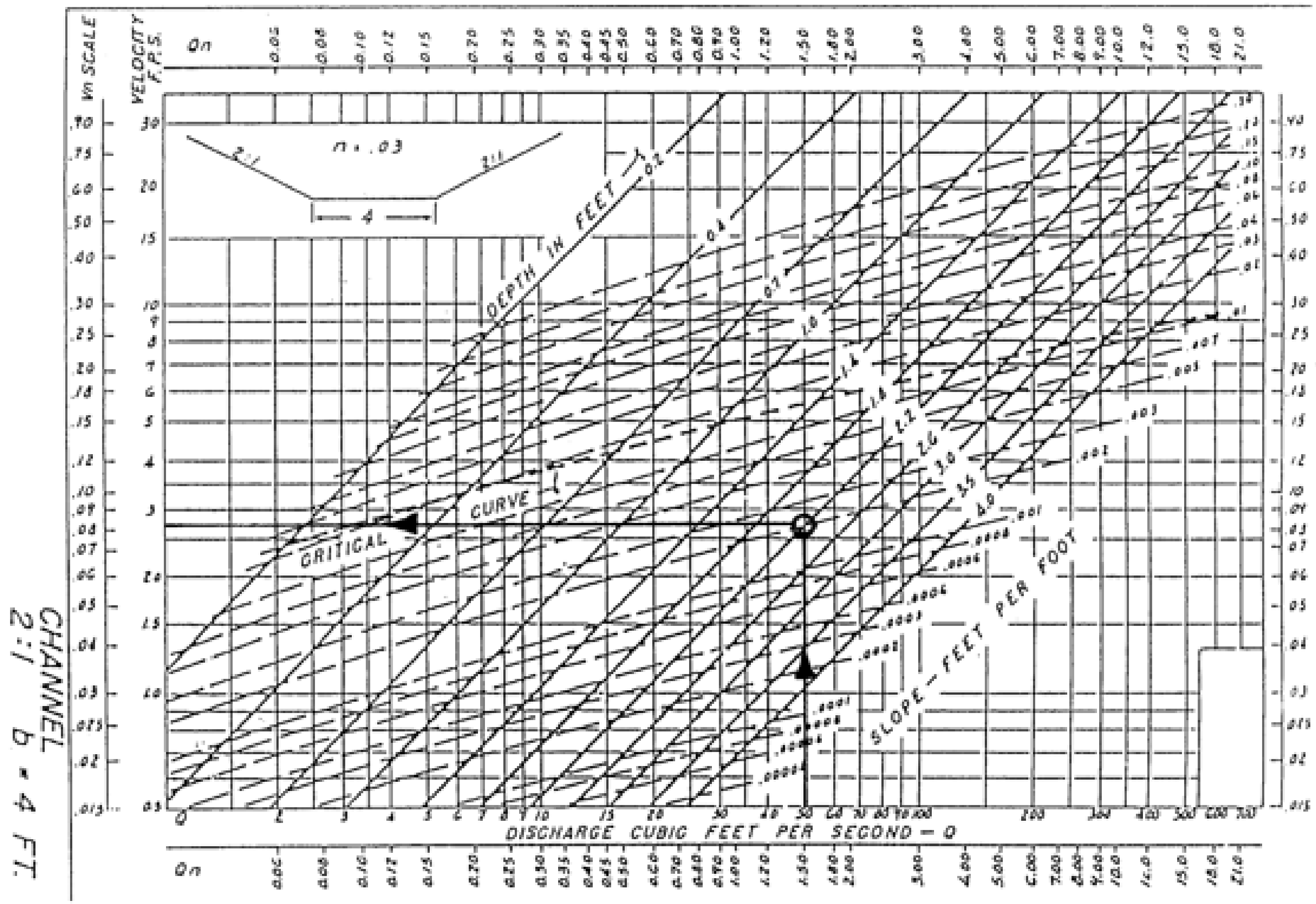



**Figure 5.4-12** Example Nomograph #2

(Source: FHWA)

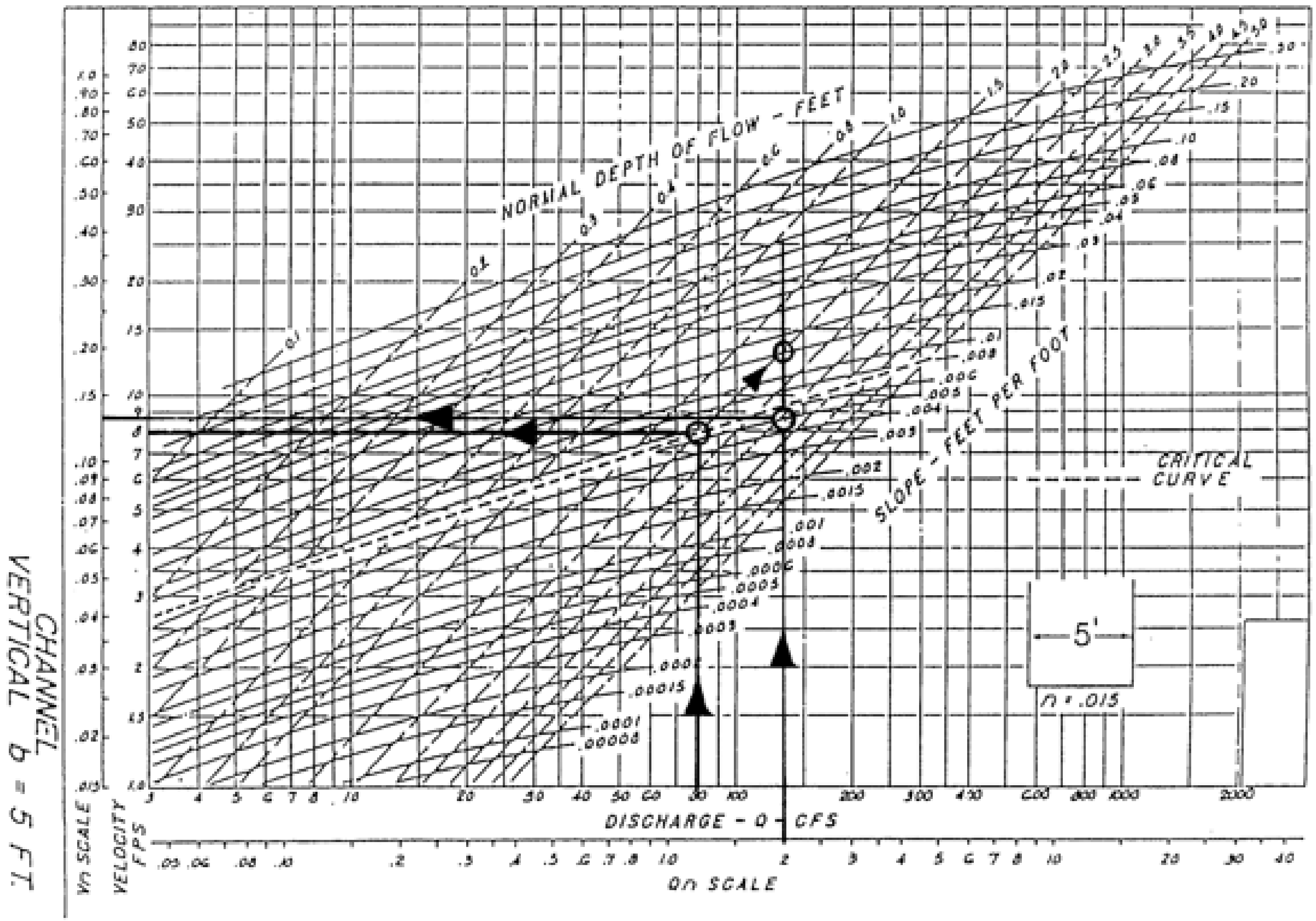



Figure 5.4-13 Example Nomograph #3

(Source: FHWA)

#### 5.4.11.4 GRASSED CHANNEL FIGURES

The Manning equation can be used to determine the capacity of a grass lined channel, but the value of n varies with the type of grass, development of the grass cover, depth, and velocity of flow. The variable value of n complicates the solution of the Manning equation. The depth and velocity of flow must be estimated and the Manning equation solved using the n value that corresponds to the estimated depth and velocity. The trial solution provides better estimates of the depth and velocity for a new value of n and the equation is again solved. The procedure is repeated until a depth is found that carries the design discharge.

To prevent excessive erosion, the velocity of flow in a grass lined channel must be kept below some maximum value (referred to as permissible velocity). The permissible velocity in a grass lined channel depends upon the type of grass, condition of the grass cover, texture of the soil comprising the channel bed, channel slope, and to some extent the size and shape of the drainage channel. To guard against overtopping, the channel capacity should be computed for taller grass than is expected to be maintained, while the velocity used to check the adequacy of the protection should be computed assuming a lower grass height than will likely be maintained.

To aid in the design of grassed channels, the Federal Highway Administration has prepared numerous design figures. Copies of these figures are in Appendix C. Following is a brief description of general design criteria, instructions on how to use the figures, and several example design problems. For design conditions not covered by the figures, a trial and error solution of the Manning equation must be used.

#### 5.4.11.5 DESCRIPTION OF FIGURES

The figures in Appendix C are designed for use in the direct solution of the Manning equation for various channel sections lined with grass. The figures are similar in appearance and use to those for trapezoidal cross sections described earlier. However, their construction is much more difficult because the roughness coefficient (n) changes as higher velocities and/or greater depths change the condition of the grass. The effect of velocity and depth of flow on n is evaluated by the product of velocity and hydraulic radius V times R. The variation of Manning's n with the retardance (**Table 5.4-6**) and the product V times R is shown in **Figure 5.4-1**. As indicated in **Table 5.4-6**, retardance varies with the height of the grass and the condition of the stand. Both of these factors depend upon the type of grass, planting conditions, and maintenance practices. **Table 5.4-6** is used to determine retardance classification.

The grassed channel figures each have two graphs, the upper graph for retardance Class D and the lower graph for retardance Class C. The figures are plotted with discharge in cubic feet per second on the abscissa and slope in feet per foot on the ordinate. Both scales are logarithmic. Superimposed on the logarithmic grid are lines for velocity in feet per second and lines for depth in feet. A dashed line shows the position of critical flow.

#### 5.4.11.6 INSTRUCTIONS FOR GRASSED CHANNEL FIGURES

The grassed channel figures provide a solution of the Manning equation for flow in open grassed channels of uniform slope and cross section. The flow should not be affected by backwater and the channel should have length sufficient to establish uniform flow. The figures are sufficiently accurate for design of drainage channels of fairly uniform cross section and slope, but are not appropriate for irregular natural channels.

The design of grassed channels requires two operations:

1. Selecting a section that has the capacity to carry the design discharge on the available slope
2. Checking the velocity in the channel to ensure that the grass lining will not be eroded.

Because the retardance of the channel is largely beyond the control of the designer, it is good practice to compute the channel capacity using retardance Class C and the velocity using retardance Class D. The calculated velocity should then be checked against the permissible velocities listed in **Tables 5.4-2** and **5.4-3**.

BACK TO TOC

The use of the figures is explained in the following steps:

(Step 1) Select the channel cross section to be used and find the appropriate figure.

(Step 2) Enter the lower graph (for retardance Class C) on the figure with the design discharge value on the abscissa and move vertically to the value of the slope on the ordinate scale. As this intersection, read the normal velocity and normal depth and note the position of the critical curve. If the intersection point is below the critical curve, the flow is subcritical; if it is above, the flow is supercritical.

(Step 3) To check the velocity developed against the permissible velocities (**Tables 5.4-2** and **5.4-3**), enter the upper graph on the same figure and repeat Step 2. Then compare the computed velocity with the velocity permissible for the type of grass, channel slope, and erosion resistance of the soil. If the computed velocity is less, the design is acceptable. If not, a different channel section must be selected and the process repeated.

**Example Design Problem 1**

*Given*:
A trapezoidal channel in easily eroded soil, lined with a grass mixture with 4:1 side slopes, and a 4 ft bottom width on slope of 0.02 ft per foot (S=.02), discharging 20 cfs.

*Find*:
Depth, velocity, type of flow, and adequacy of grass to prevent erosion

*Procedure*:

1. From Appendix C-4, select the FHWA channel design figure for 4:1 side slopes (see **Figure 5.4-15**).

2. Enter the lower graph with Q = 20 cfs, and move vertically to the line for S=0.02. At this intersection read $d_n$ = 1.0 ft, and normal velocity $V_n$ 2.6 ft/s.

3. The velocity for checking the adequacy of the grass cover should be obtained from the upper graph, for retardance Class D. Using the same procedure as in step 2, the velocity is found to be 3.0 ft/s. This is about three-quarters of that listed as permissible, 4.0 ft/s in **Table 5.4-3**.

**Example Design Problem 2**

*Given*:
The channel and discharge of Example 1.

*Find*:
The maximum grade on which the 20 cfs could safely be carried

*Procedure*:
With an increase in slope (but still less than 5%), the allowable velocity is estimated to be 4 ft/s (see **Table 5.4-3**). On the upper graph of **Figure 5.4-15** for short grass, the intersection of the 20 cfs line and the 4 ft/s line indicates a slope of 3.7% and a depth of 0.73 ft.

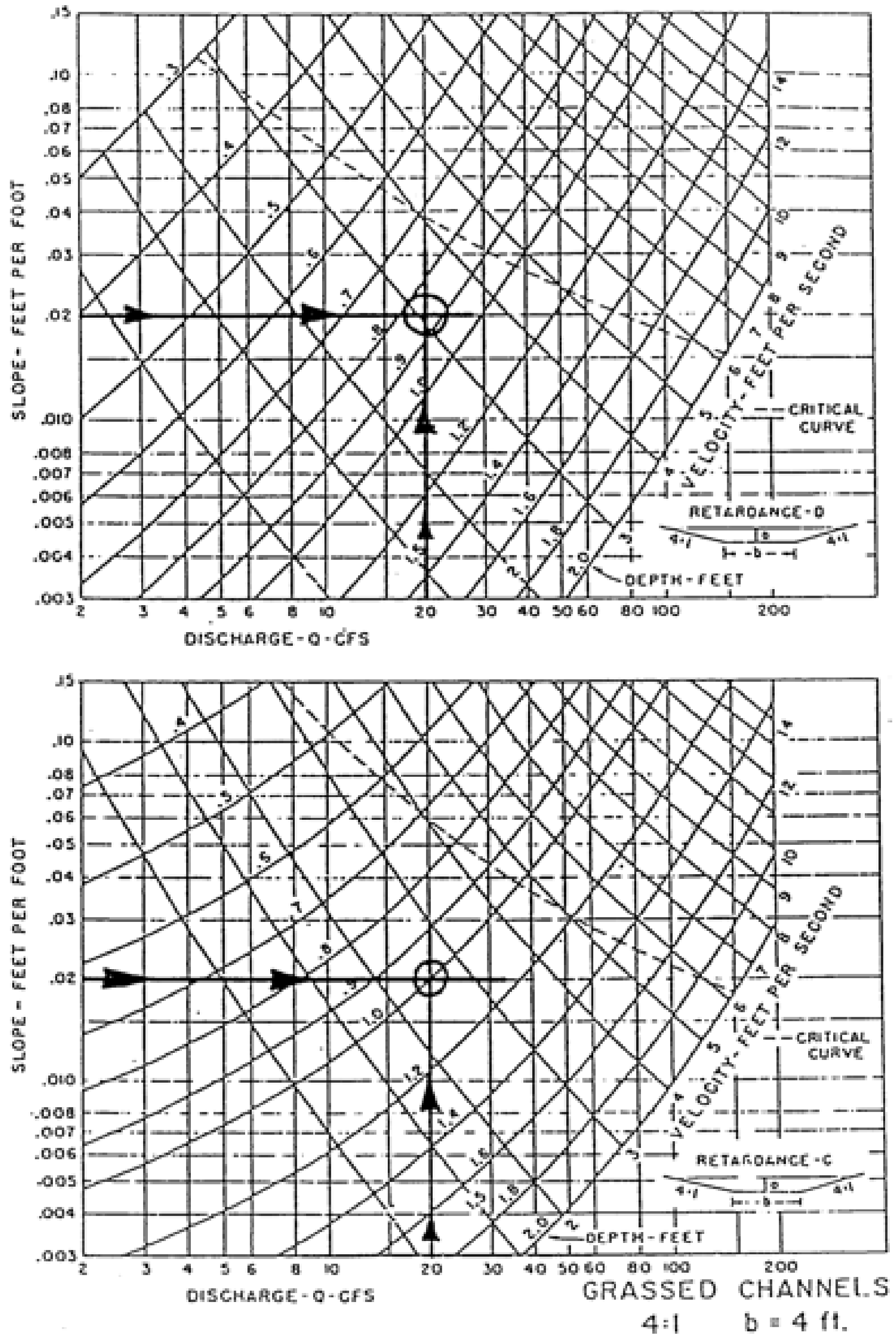



**Figure 5.4-14** Example Nomograph #4

(Source: FHWA)

BACK TO TOC

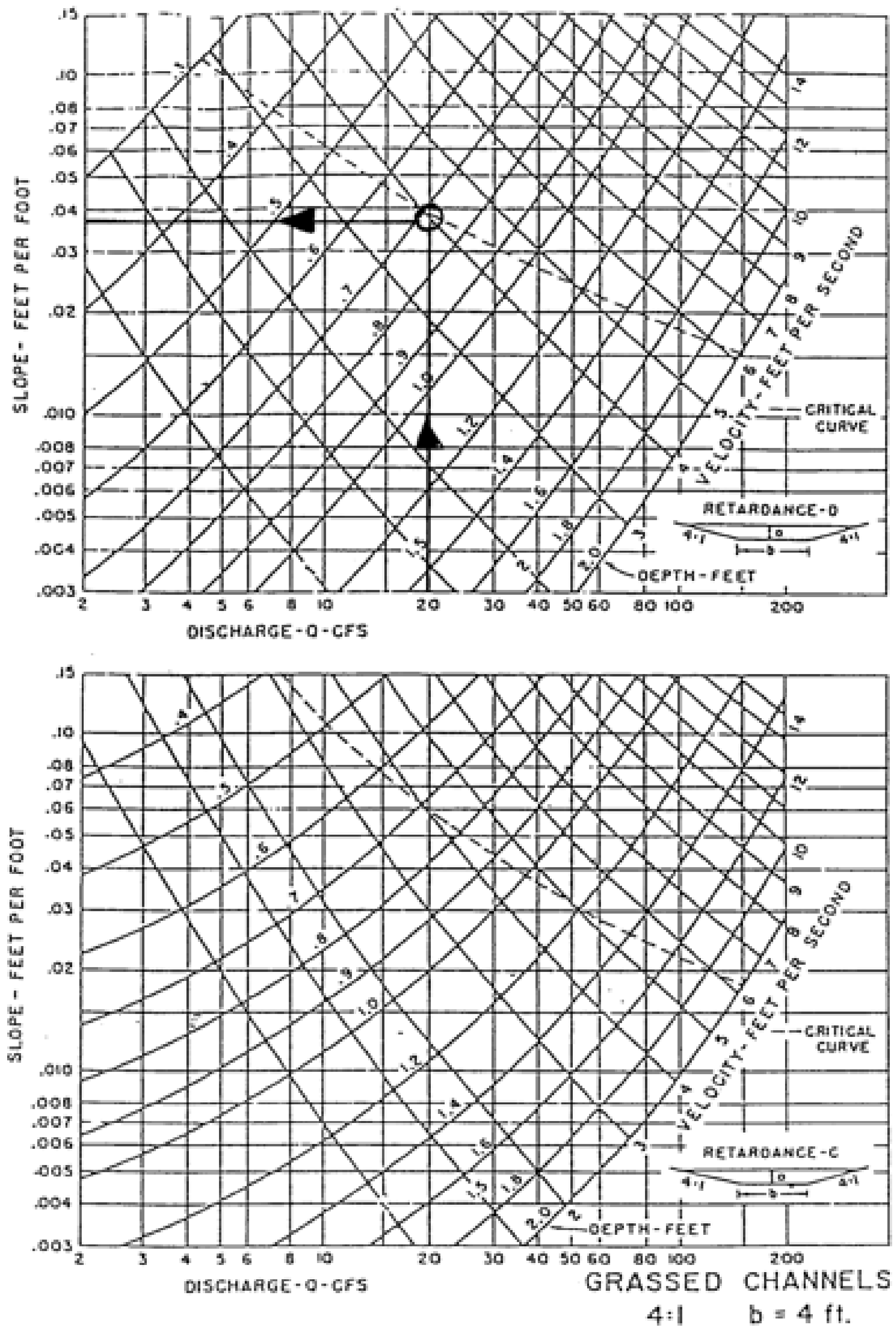



**Figure 5.4-15** Example Nomograph #5

(Source: FHWA)

BACK TO TOC

# 5.5 Energy Dissipation Design

## 5.5.1 Overview

### 5.5.1.1 INTRODUCTION

The outlets of pipes and lined channels are points of critical erosion potential. Stormwater that is transported through man-made conveyance systems at design capacity generally reaches a velocity that exceeds the capacity of the receiving channel or area to resist erosion. To prevent scour at stormwater outlets, protect the outlet structure and minimize the potential for downstream erosion, a flow transition structure is needed to absorb the initial impact of flow and reduce the speed of the flow to a non-erosive velocity.

*Energy dissipators* are engineered devices such as rip-rap aprons or concrete baffles placed at the outlet of stormwater conveyances for the purpose of reducing the velocity, energy and turbulence of the discharged flow.

### 5.5.1.2 GENERAL CRITERIA

- Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs.
- Energy dissipators shall be employed whenever the velocity of flows leaving a stormwater management facility exceeds the erosion velocity of the downstream area channel system.
- Energy dissipator designs will vary based on discharge specifics and tailwater conditions.
- Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence.

### 5.5.1.3 RECOMMENDED ENERGY DISSIPATORS

For many designs, the following outlet protection devices and energy dissipators provide sufficient protection at a reasonable cost:

- Riprap apron
- Riprap outlet basins
- Baffled outlets

This section focuses on the design on these measures. The reader is referred to the Federal Highway Administration Hydraulic Engineering Circular No. 14 entitled, *Hydraulic Design of Energy Dissipators for Culverts and Channels*, for the design procedures of other energy dissipators.

Table 5.5-1 Symbols and Definitions

| Symbol | Definition | Units |
|---|---|---|
| A | Cross-sectional area | $ft^2$ |
| D | Height of box culvert | ft |
| $d_{50}$ | Size of riprap | ft |
| $d_W$ | Culvert width | ft |
| Fr | Froude Number | - |
| g | Acceleration of gravity | $ft/s^2$ |
| $h_S$ | Depth of dissipator pool | ft |
| L | Length | ft |
| $L_a$ | Riprap apron length | ft |
| $L_B$ | Overall length of basin | ft |
| $L_S$ | Length of dissipator pool | ft |
| PI | Plasticity index | - |
| Q | Rate of discharge | cfs |
| $S_V$ | Saturated shear strength | $lbs/in^2$ |
| t | Time of scour | min. |
| $t_C$ | Critical tractive shear stress | $lbs/in^2$ |
| TW | Tailwater depth | ft |
| $V_L$ | Velocity L feet from brink | ft/s |
| $V_O$ | Normal velocity at brink | ft/s |
| $V_O$ | Outlet mean velocity | ft/s |
| $V_S$ | Volume of dissipator pool | $ft^3$ |
| $W_O$ | Diameter or width of culvert | ft |
| $W_S$ | Width of dissipator pool | ft |
| $y_e$ | Hydraulic depth at brink | ft |
| $y_O$ | Normal flow depth at brink | ft |

## 5.5.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in **Table 5.5-1** will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

## 5.5.3 Design Guidelines

1. If outlet protection is required, choose an appropriate type. Suggested outlet protection facilities and applicable flow conditions (based on Froude number and dissipation velocity) are described below:

    » **Riprap aprons** may be used when the outlet Froude number (Fr) is less than or equal to 2.5. In general, riprap aprons prove economical for transitions from culverts to overland sheet flow at terminal outlets, but may also be used for transitions from culvert sections to stable channel sections. Stability of the surface at the termination of the apron should be considered.

    » **Riprap outlet basins** may also be used when the outlet Fr is less than or equal to 2.5. They are generally used for transitions from culverts to stable channels. Since riprap outlet basins function by creating a hydraulic jump to dissipate energy, performance is impacted by tailwater conditions.

    » **Baffled outlets** have been used with outlet velocities up to 50 feet per second. Practical application typically requires an outlet Fr between 1 and 9. Baffled outlets may be used at both terminal outlet and channel outlet transitions. They function by dissipating energy through impact and turbulence and are not significantly affected by tailwater conditions.

2. When outlet protection facilities are selected, appropriate design flow conditions and site-specific factors affecting erosion and scour potential, construction cost, and long-term durability should be considered.

3. If outlet protection is not provided, energy dissipation will occur through formation of a local scourhole. A cutoff wall will be needed at the discharge outlet to prevent structural undermining. The wall depth should be slightly greater than the computed scourhole depth, $h_s$. The scourhole should then be stabilized. If the scourhole is of such size that it will present maintenance, safety, or aesthetic problems, other outlet protection will be needed.

4. Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is expected to occur. **Figure 5.5-1** provides the riprap size recommended for use downstream of energy dissipators.

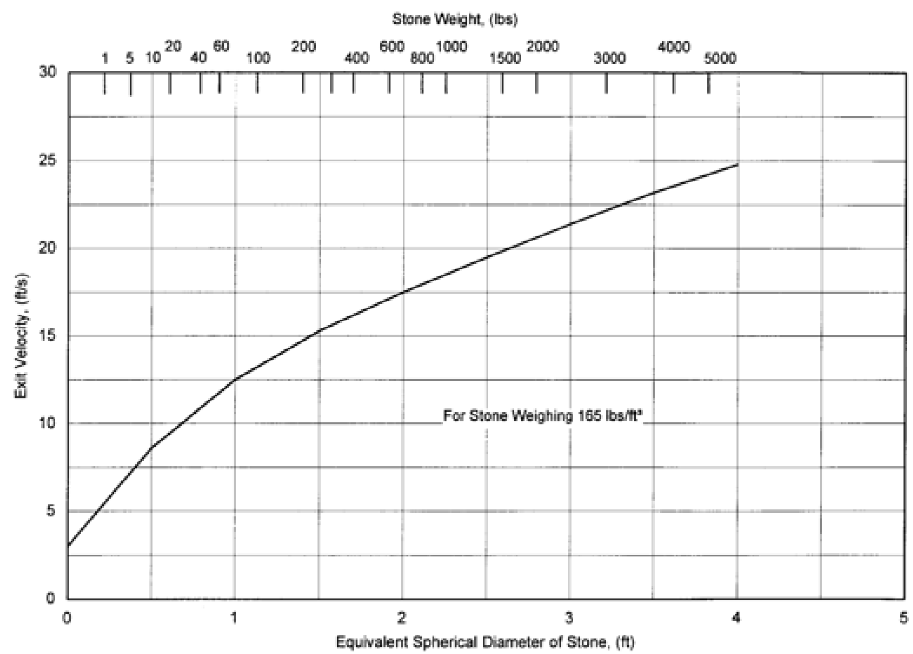



**Figure 5.5-1** Riprap Size for Use Downstream of Energy Dissipator
(Source: Searcy, 1967)

## 5.5.4 Riprap Aprons

### 5.5.4.1 DESCRIPTION

A riprap-lined apron is a commonly used practice for energy dissipation because of its relatively low cost and ease of installation. A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow. Riprap aprons are appropriate when the culvert outlet Fr is less than or equal to 2.5.

### 5.5.4.2 DESIGN PROCEDURE

The procedure presented in this section is taken from USDA, SCS (1975). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter, $d_{50}$. If tailwater conditions are unknown, or if both minimum and maximum conditions may occur, the apron should be designed to meet criteria for both. Although the design curves are based on round pipes flowing full, they can be used for partially full pipes and box culverts. The design procedure consists of the following steps:

(Step 1) If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in **Figure 5.5-2** apply. Otherwise, maximum tailwater conditions exist and the curves in **Figure 5.5-3** should be used.

(Step 2) Determine the correct apron length and median riprap diameter, $d_{50}$, using the appropriate curves from **Figures 5.5-2** and **5.5-3**. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in **Figure 5.5-4**.

» For pipes flowing full:

Use the depth of flow, d, which equals the pipe diameter, in feet, and design discharge, in cfs, to obtain the apron length, $L_a$, and median riprap diameter, $d_{50}$, from the appropriate curves.

» For pipes flowing partially full:

Use the depth of flow, d, in feet, and velocity, v, in ft/s. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, $d_{50}$, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, $L_a$, from the scale on the left.

» For box culverts:

Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, $d_{50}$, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth, d. Find the minimum apron length, $L_a$, using the scale on the left.

(Step 3) If tailwater conditions are uncertain, the median riprap diameter should be the larger of the values for minimum and maximum conditions. The dimensions of the apron will be as shown in **Figure 5.5-4**. This will provide protection under either of the tailwater conditions.

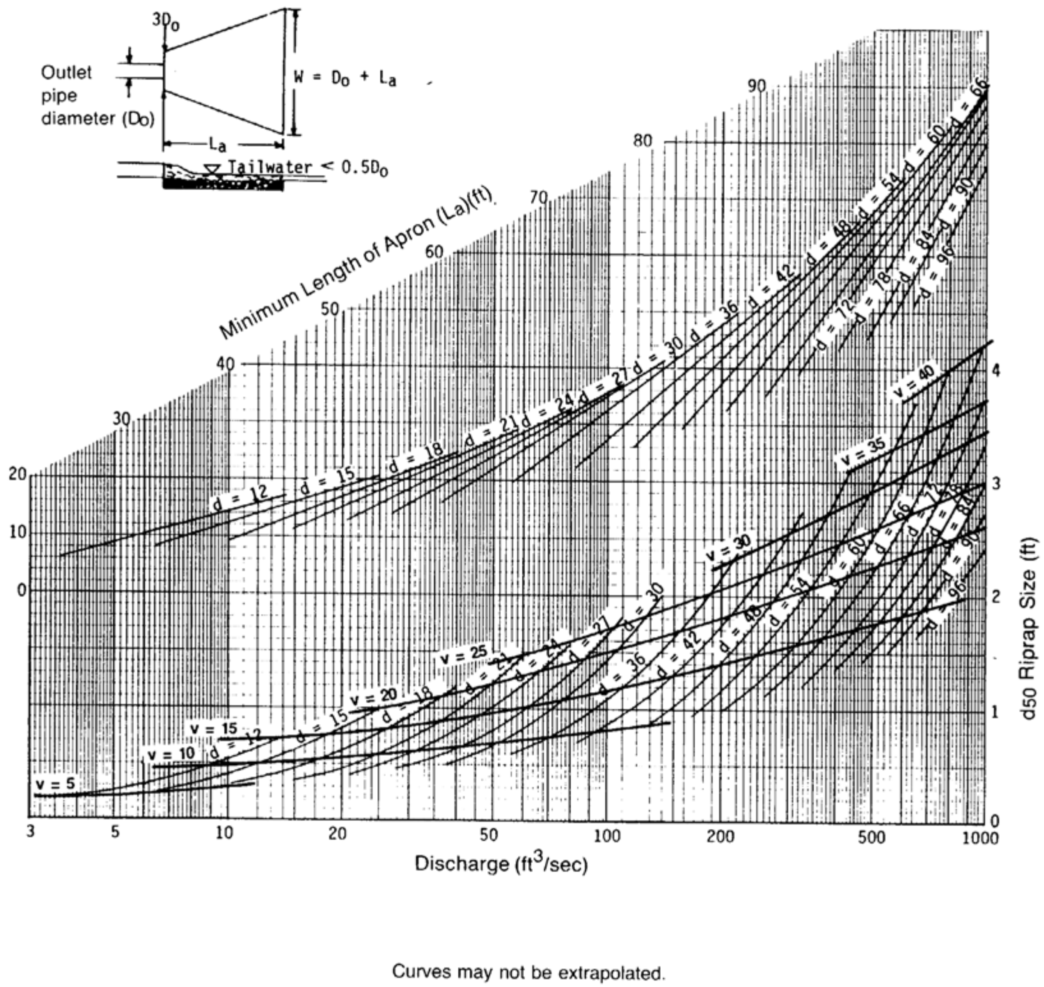
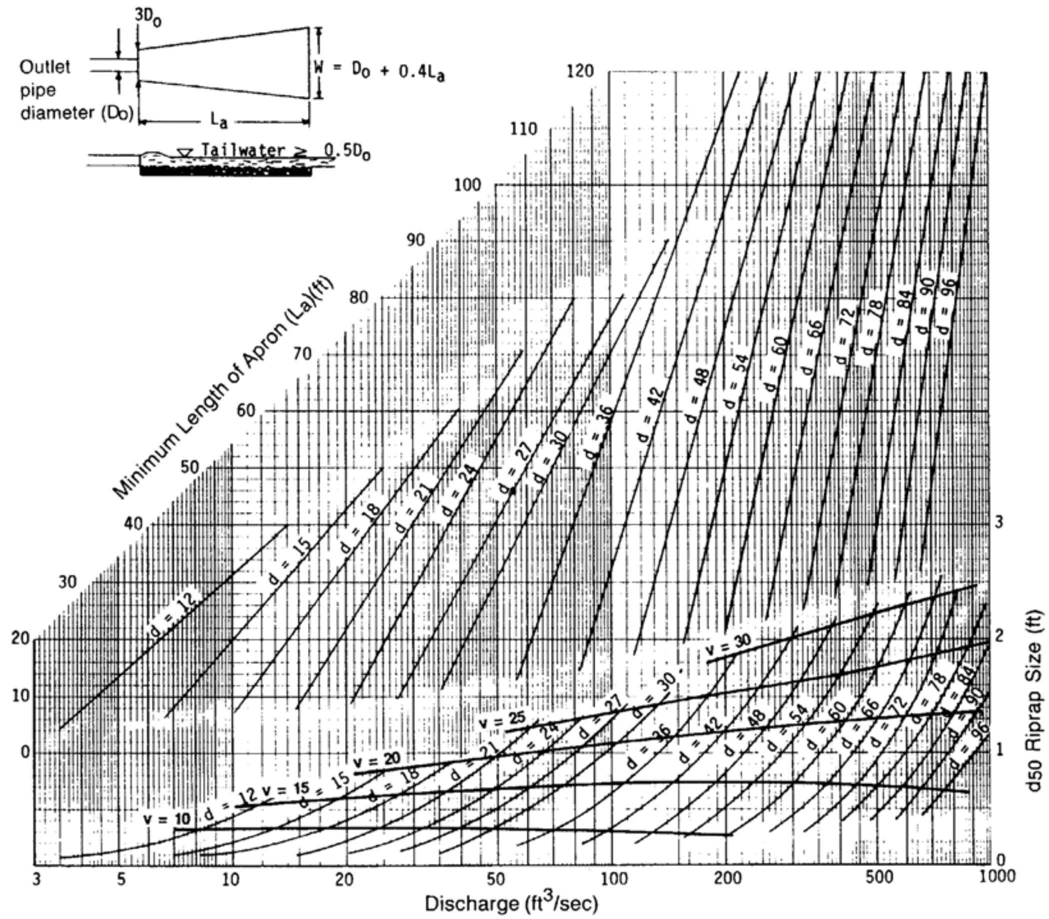



**Figure 5.5-2** Design of Riprap Apron under Minimum Tailwater Conditions

(Source: USDA, SCS, 1975)




Curves may not be extrapolated.

**Figure 5.5-3** Design of Riprap Apron under Maximum Tailwater Conditions

(Source: USDA, SCS, 1975)

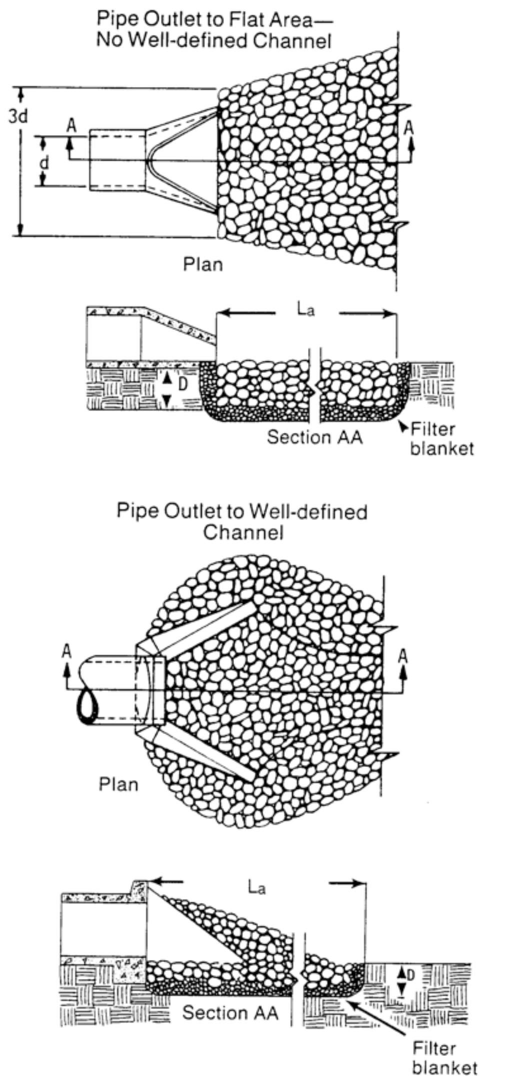



Figure 5.5-4 Riprap Apron

(Source: Manual for Erosion and Sediment Control in Georgia, 2014)

*Notes on* **Figure 5.5-4**:

1. $L_a$ is the length of the riprap apron.
2. D = 1.5 times the maximum stone diameter, but not less than 6".
3. In a well-defined channel, extend the apron up to the channel banks to an elevation of 6" above the maximum tailwater depth or to the top of the bank, whichever is less.
4. A filter blanket or filter fabric should be installed between the riprap and soil foundation.

### 5.5.4.3 DESIGN CONSIDERATIONS

The following items should be considered during riprap apron design:

- The maximum stone diameter should be 1.5 times the median riprap diameter. $d_{max}$ = 1.5 x $d_{50}$ , $d_{50}$ = the median stone size in a well-graded riprap apron.
- The riprap thickness should be 1.5 times the maximum stone diameter or 6 inches, whichever is greater. Apron thickness = 1.5 x $d_{max}$ (Apron thickness may be reduced to 1.5 x $d_{50}$ when an appropriate filter fabric is used under the apron.)
- The apron width at the discharge outlet should be at least equal to the pipe diameter or culvert width, $d_w$. Riprap should extend up both sides of the apron and around the end of the pipe or culvert at the discharge outlet at a maximum slope of 2:1 and a height not less than the pipe diameter or culvert height, and should taper to the flat surface at the end of the apron.
- If there is a well-defined channel, the apron length should be extended as necessary so that the downstream apron width is equal to the channel width. The sidewalls of the channel should not be steeper than 2:1.
- If the ground slope downstream of the apron is steep, channel erosion may occur. The apron should be extended as necessary until the slope is gentle enough to prevent further erosion.
- The potential for vandalism should be considered if the rock is easy to carry. If vandalism is a possibility, the rock size must be increased or the rocks held in place using concrete or grout.

### 5.5.4.4 EXAMPLE DESIGNS

**Example 1 Riprap Apron Design for Minimum Tailwater Conditions**

A flow of 280 cfs discharges from a 66-in pipe with a tailwater of 2 ft above the pipe invert. Find the required design dimensions for a riprap apron.

1. Minimum tailwater conditions = 0.5 $d_o$, $d_o$ = 66 in = 5.5 ft; therefore, 0.5 $d_o$ = 2.75 ft.
2. Since TW = 2 ft, use **Figure 5.5-2** for minimum tailwater conditions.
3. **Figure 5.5-2**, the apron length, $L_a$, and median stone size, $d_{50}$, are 38 ft and 1.2 ft, respectively.

4. The downstream apron width equals the apron length plus the pipe diameter:
   - $W = d + L_a = 5.5 + 38 = 43.5$ ft

5. Maximum riprap diameter is 1.5 times the median stone size:
   - $1.5 (d_{50}) = 1.5 (1.2) = 1.8$ ft

6. (Riprap depth = $1.5 (d_{max}) = 1.5 (1.8) = 2.7$ ft.

**Example 2 Riprap Apron Design for Maximum Tailwater Conditions**

A concrete box culvert 5.5 ft high and 10 ft wide conveys a flow of 600 cfs at a depth of 5.0 ft. Tailwater depth is 5.0 ft above the culvert outlet invert. Find the design dimensions for a riprap apron.

1. Compute $0.5\ d_o = 0.5 (5.0) = 2.5$ ft.
2. Since TW = 5.0 ft is greater than 2.5 ft, use **Figure 5.5-3** for maximum tailwater conditions.
   - $v = Q/A = [600/(5)(10)] = 12$ ft/s
3. On **Figure 5.5-3**, at the intersection of the curve, do = 60 in and v = 12 ft/s, $d_{50} = 0.4$ ft. Reading up to the intersection with d = 60 in, find $L_a = 40$ ft.
4. Apron width downstream = $d_w + 0.4\ L_a = 10 + 0.4 (40) = 26$ ft.
5. Maximum stone diameter = $1.5\ d_{50} = 1.5 (0.4) = 0.6$ ft.
6. Riprap depth = 1.5 dmax = 1.5 (0.6) = 0.9 ft.

## 5.5.5 Riprap Basins

### 5.5.5.1 DESCRIPTION

Another method to reduce the exit velocities from stormwater outlets is through the use of a riprap basin. A riprap outlet basin is a preshaped scourhole lined with riprap that functions as an energy dissipator by forming a hydraulic jump.

### 5.5.5.2 BASIN FEATURES

General details of the basin recommended in this section are shown in **Figure 5.5 5**. Principal features of the basin are:

- The basin is preshaped and lined with riprap of median size ($d_{50}$).
- The floor of the riprap basin is constructed at an elevation of hs below the culvert invert. The dimension $h_s$ is the approximate depth of scour that would occur in a thick pad of riprap of size $d_{50}$ if subjected to design discharge. The ratio of $h_s$ to $d_{50}$ of the material should be between 2 and 4.
- The length of the energy dissipating pool is 10 x $h_s$ or 3 x $W_o$, whichever is larger. The overall length of the basin is 15 x $h_s$ or 4 x $W_o$, whichever is larger.

### 5.5.5.3 DESIGN PROCEDURE

The following procedure should be used for the design of riprap basins.

(Step 1) Estimate the flow properties at the brink (outlet) of the culvert. Establish the outlet invert elevation such that $TW/y_o \leq 0.75$ for the design discharge.

(Step 2) For subcritical flow conditions (culvert set on mild or horizontal slope) use **Figure 5.5-6** or **Figure 5.5-7** to obtain $y_o/D$, then obtain $V_o$ by dividing Q by the wetted area associated with $y_o$. D is the height of a box culvert. If the culvert is on a steep slope, $V_o$ will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.

(Step 3) For channel protection, compute the Froude number for brink conditions with $y_e = (A/2)^{1.5}$. Select $d_{50}/y_e$ appropriate for locally available riprap (usually the most satisfactory results will be obtained if $0.25 < d_{50}/y_e < 0.45$). Obtain $h_s/y_e$ from **Figure 5.5-8**, and check to see that $2 < h_s/d_{50} < 4$. Recycle computations if $hs/d_{50}$ falls out of this range.

(Step 4) Size basin as shown in **Figure 5.5-5**.

(Step 5) Where allowable dissipator exit velocity is specified:

» Determine the average normal flow depth in the natural channel for the design discharge.

» Extend the length of the energy basin (if necessary) so that the width of the energy basin at section A A, **Figure 5.5-5**, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

(Step 6) In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so that the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be avoided to minimize separation zones and resultant eddies.

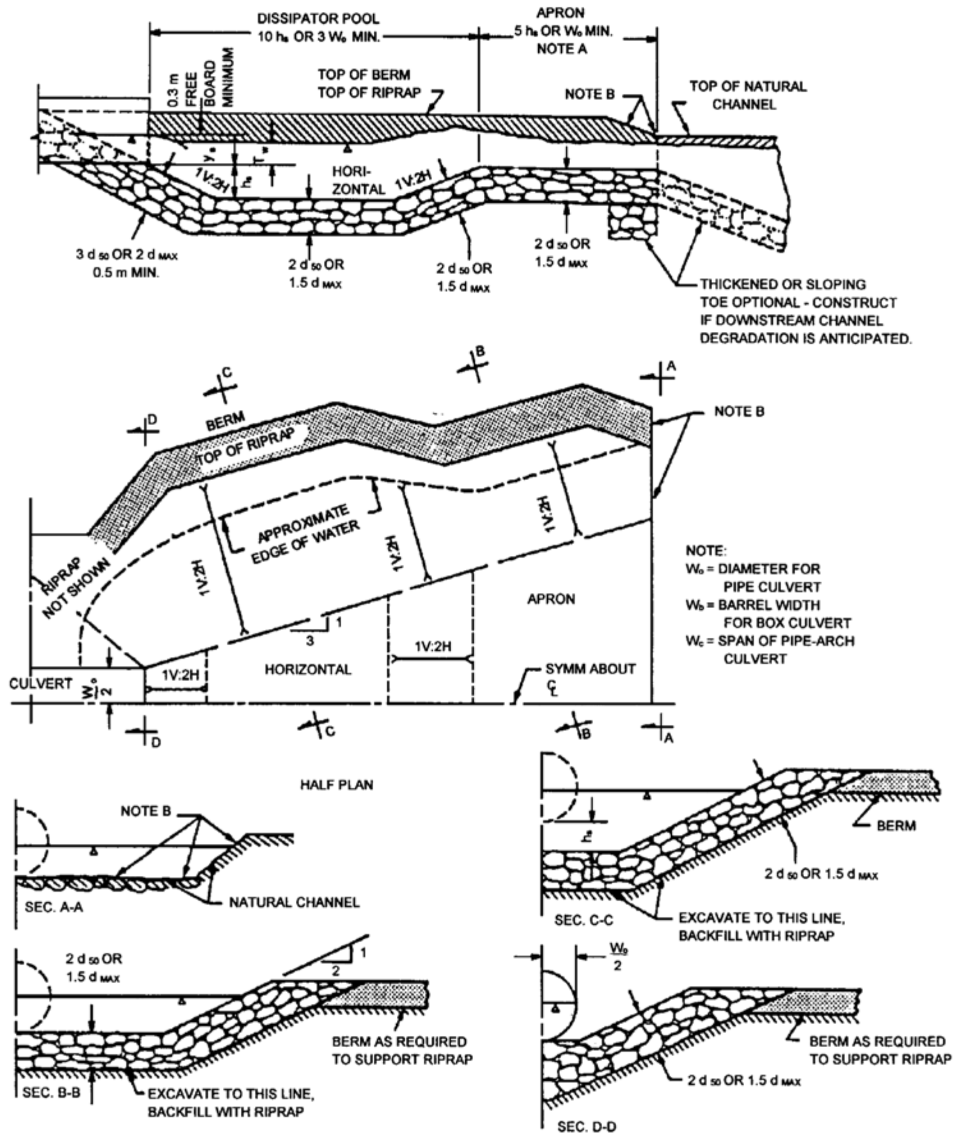
(Step 7) If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:

» Design a conventional basin for low tailwater conditions in accordance with the instructions above.

» Estimate centerline velocity at a series of downstream cross sections using the information shown in **Figure 5.5-9**.

» Shape downstream channel and size riprap using **Figure 5.5-1** and the stream velocities obtained above.

Material, construction techniques, and design details for riprap should be in accordance with specifications in the Federal Highway publication HEC No. 11 entitled *Use of Riprap For Bank Protection*.




NOTE A - IF EXIT VELOCITY OF BASIN IS SPECIFIED, EXTEND BASIN AS REQUIRED TO OBTAIN SUFFICIENT CROSS-SECTIONAL AREA AT SECTION A-A SUCH THAT Q/(CROSS SECTION AREA AT SEC. A-A) = SPECIFIED EXIT VELOCITY.

NOTE B - WARP BASIN TO CONFORM TO NATURAL STREAM CHANNEL. TOP OF RIPRAP IN FLOOR OF BASIN SHOULD BE AT THE SAME ELEVATION OR LOWER THAN NATURAL CHANNEL BOTTOM AT SEC. A-A.

**Figure 5.5-5** Details of Riprap Outlet Basin

(Source: HEC-14, 2012)

BACK TO TOC

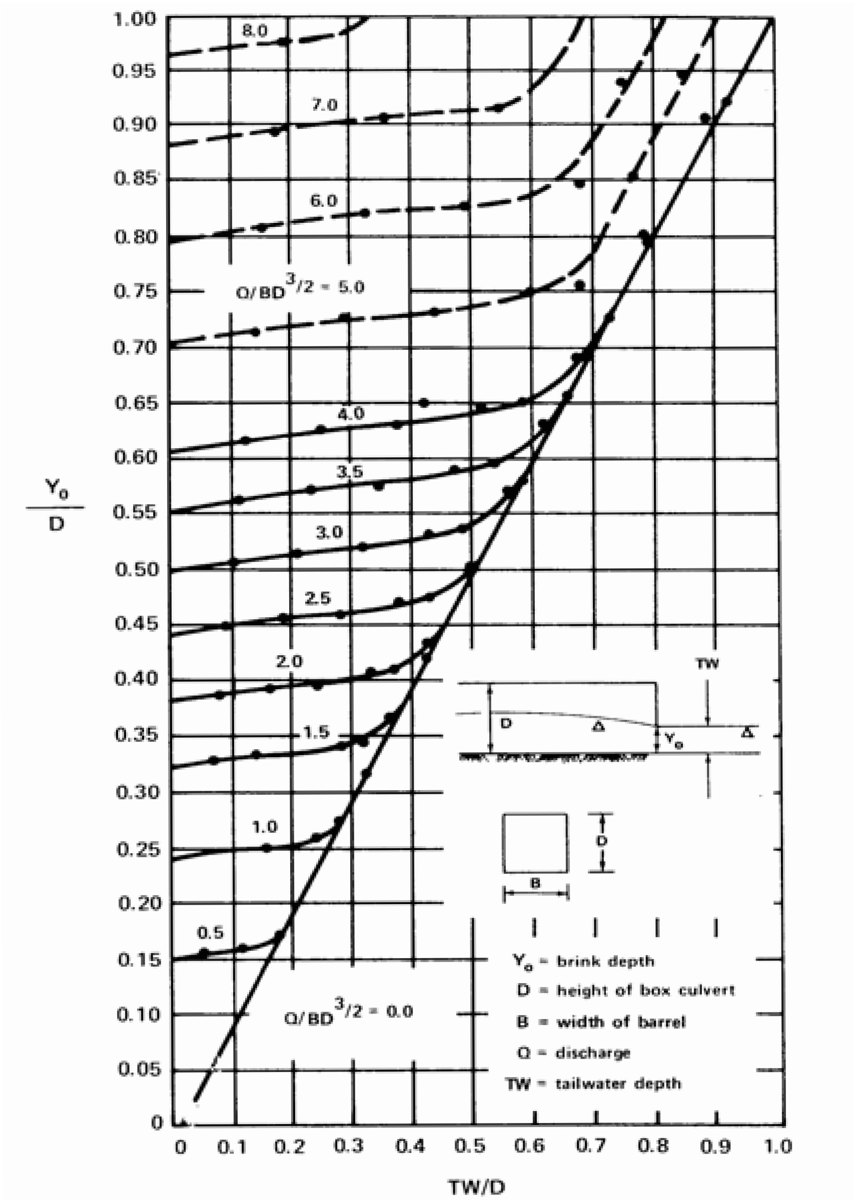



Figure 5.5-6 Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 2012)

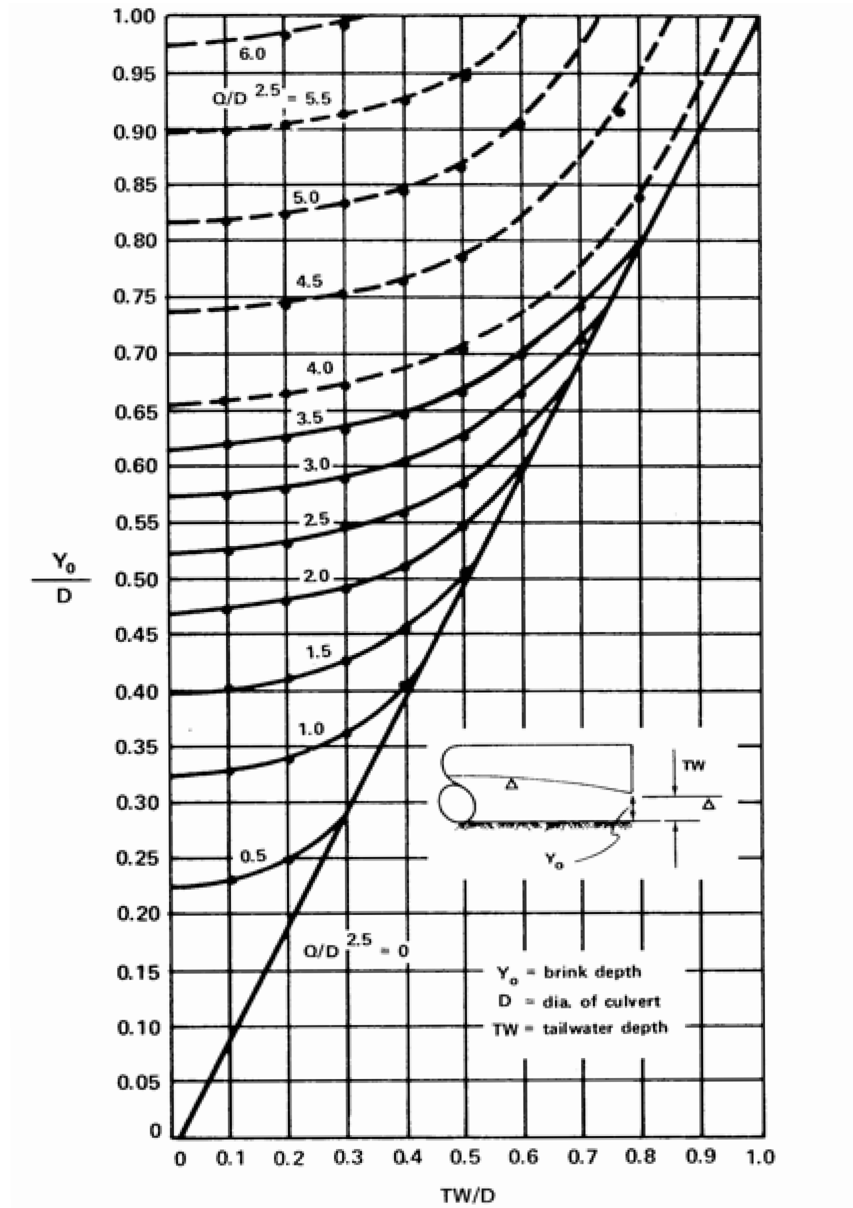



Figure 5.5-7 Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 2012)

BACK TO TOC

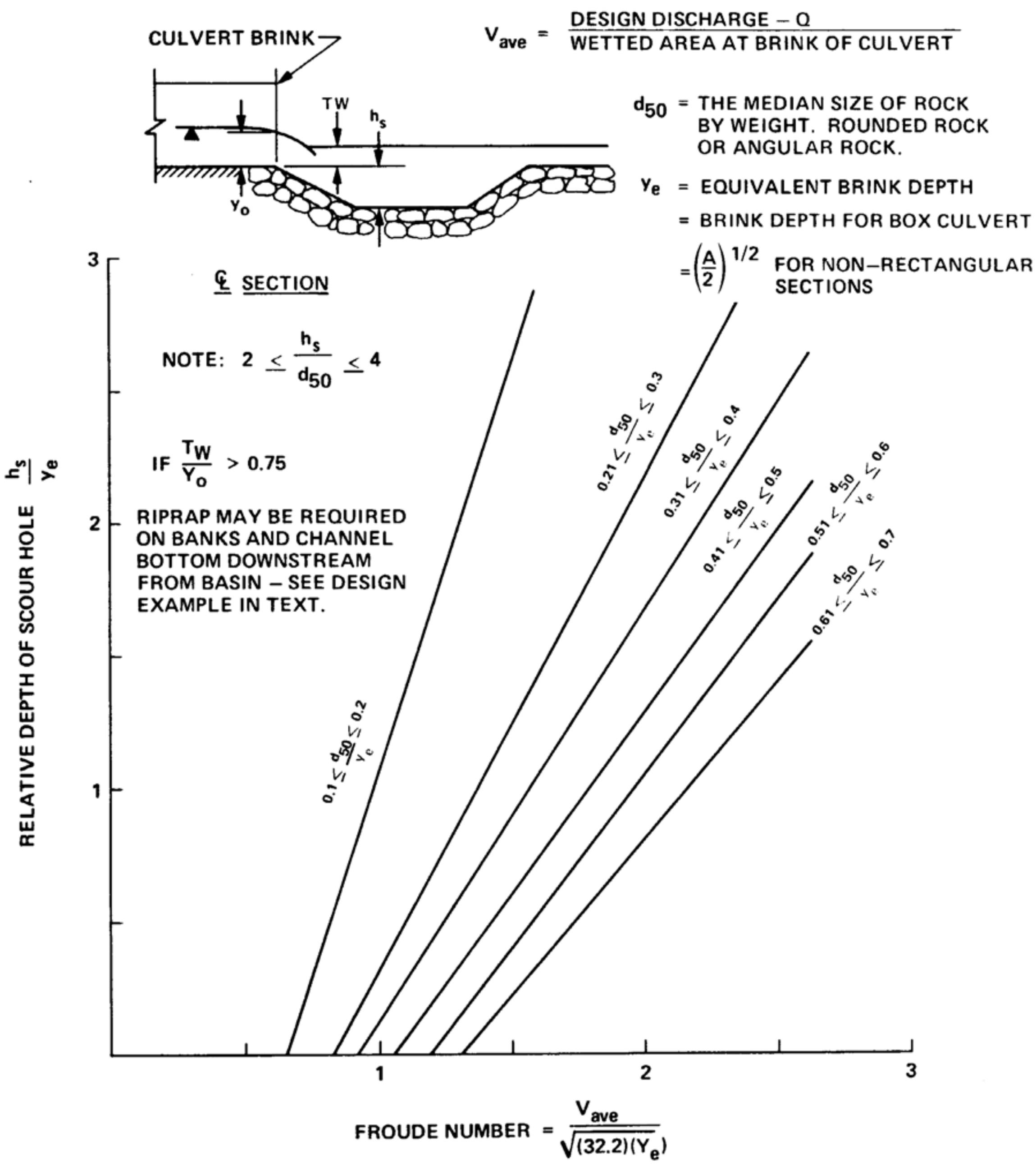
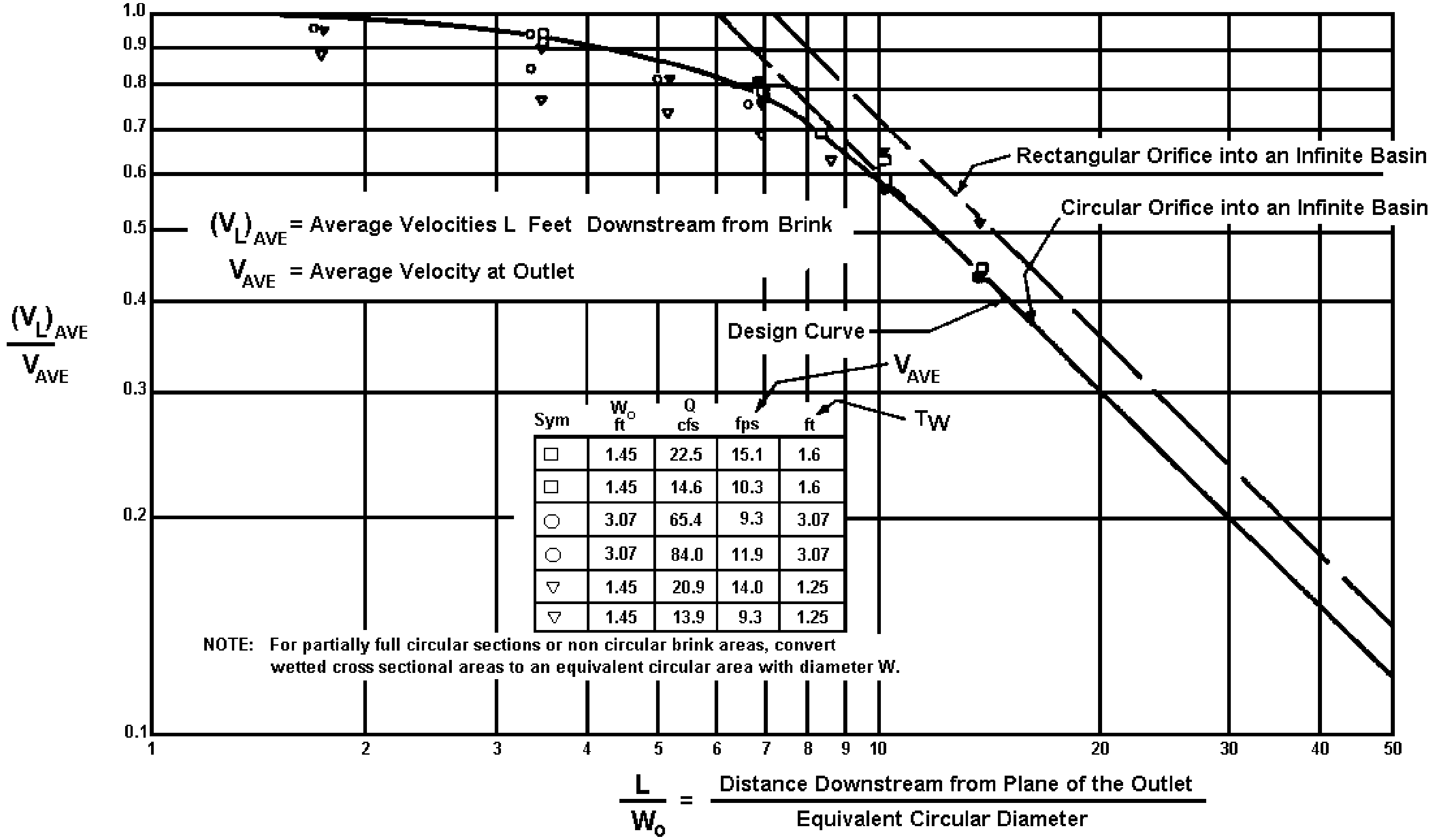



**Figure 5.5-8** Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable

(Source: USDOT, FHWA, HEC-14, 2012)




| Sym | $W_o$ ft | Q cfs | fps | ft |
|---|---|---|---|---|
| □ | 1.45 | 22.5 | 15.1 | 1.6 |
| □ | 1.45 | 14.6 | 10.3 | 1.6 |
| ○ | 3.07 | 65.4 | 9.3 | 3.07 |
| ○ | 3.07 | 84.0 | 11.9 | 3.07 |
| ▽ | 1.45 | 20.9 | 14.0 | 1.25 |
| ▽ | 1.45 | 13.9 | 9.3 | 1.25 |

NOTE: For partially full circular sections or non circular brink areas, convert wetted cross sectional areas to an equivalent circular area with diameter W.

**Figure 5.5-9** Distribution of Centerline Velocity for Flow from Submerged Outlets to Be Used for Predicting Channel Velocities Downstream from Culvert Outlet Where High Tailwater Prevails

(Source: USDOT, FHWA, HEC-14, 2012)

BACK TO TOC

### 5.5.5.4 DESIGN CONSIDERATIONS

Riprap basin design should include consideration of the following:

- The dimensions of a scourhole in a basin constructed with angular rock can be approximately the same as the dimensions of a scourhole in a basin constructed of rounded material when rock size and other variables are similar.
- When the ratio of tailwater depth to brink depth, $TW/y_o$, is less than 0.75 and the ratio of scour depth to size of riprap, $h_s/d_{50}$, is greater than 2.0, the scourhole should function very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunges into the hole, a jump forms against the downstream extremity of the scourhole, and flow is generally well dispersed leaving the basin.
- The mound of material formed on the bed downstream of the scourhole contributes to the dissipation of energy and reduces the size of the scourhole; that is, if the mound from a stable scoured basin is removed and the basin is again subjected to design flow, the scourhole will enlarge.
- For high tailwater basins ($TW/y_o$ greater than 0.75), the high velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin and diffuses similarly to a concentrated jet diffusing in a large body of water. As a result, the scourhole is much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rock-lined basin.
- It should be recognized that there is a potential for limited degradation to the floor of the dissipator pool for rare event discharges. With the protection afforded by the $2(d_{50})$ thickness of riprap, the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, such damage should be superficial.
- See Standards in the in FHWA HEC No. 11 for details on riprap materials and use of filter fabric.
- Stability of the surface at the outlet of a basin should be considered using the methods for open channel flow as outlined in Section 5.4, *Open Channel Design*.

### 5.5.5.5 EXAMPLE DESIGNS

Following are some example problems to illustrate the design procedures outlined.

**Example 1**

*Given*:

| | |
|---|---|
| Box culvert- 8 ft by 6 ft | Design Discharge Q = 800 cfs |
| Supercritical flow in culvert | Normal flow depth = brink depth |
| $Y_o$ = 4 ft Tailwater depth | TW = 2.8 ft |

*Find*:

Riprap basin dimensions for these conditions

*Solution*:

Definition of terms in Steps 1 through 5 can be found in **Figures 5.5-5** and **5.5-8**.

1. $y_o = y_e$ for rectangular section; therefore, with $y_o$ given as 4 ft, $y_e$ = 4 ft.
2. $V_o = Q/A = 800/(4 \times 8) = 25$ ft/s
3. Froude Number = $Fr = V/(g \times y_e)^{0.5}$ ($g = 32.3$ ft/s$^2$)

   $Fr = 25/(32.2 \times 4)^{0.5} = 2.20 < 2.5$ O.K.
4. $TW/y_e = 2.8/4.0 = 0.7$ Therefore, $TW/y_e < 0.75$ OK
5. Try $d_{50}/y_e = 0.45$, $d_{50} = 0.45 \times 4 = 1.80$ ft

   From **Figure 5.5-8**, $h_s/y_e = 1.6$, $h_s = 4 \times 1.6 = 6.4$ ft

   $h_s/d_{50} = 6.4/1.8 = 3.6$ ft, $2 < h_s/d_{50} < 4$ OK
6. $L_s = 10 \times h_s = 10 \times 6.4 = 64$ ft ($L_s$ = length of energy dissipator pool); $L_s$ min = $3 \times W_o = 3 \times 8 = 24$ ft; therefore, use $L_s = 64$ ft

   . $L_B = 15 \times h_s = 15 \times 6.4 = 96$ ft ($L_B$ = overall length of riprap basin); $L_B$ min = $4 \times W_o = 4 \times 8 = 32$ ft; therefore, use $L_B = 96$ ft
7. Thickness of riprap: On the approach = $3 \times d_{50} = 3 \times 1.8 = 5.4$ ft

   Remainder = $2 \times d_{50} = 2 \times 1.8 = 3.6$ ft

   Other basin dimensions designed according to details shown in **Figure 5.5-5**.

BACK TO TOC

**Example 2**

*Given*:

Same design data as Example 1 except:

- Tailwater depth TW = 4.2 ft
- Downstream channel can tolerate only 7 ft/s discharge

*Find*:

Riprap basin dimensions for these conditions

*Solutions*:

**Note**- High tailwater depth, $TW/y_o = 4.2/4 = 1.05 > 0.75$

1. From Example 1: $d_{50} = 1.8$ ft, $h_s = 6.4$ ft, $L_s = 64$ ft, $L_B = 96$ ft.
2. Design riprap for downstream channel. Use **Figure 5.5-9** for estimating average velocity along the channel. Compute equivalent circular diameter $D_e$ for brink area from:
   - $A = 3.14D_e^2/4 = y_o \times W_o = 4 \times 8 = 32\ ft^2$
   - $D_e = ((32 \times 4)/3.14)^{0.5} = 6.4$ ft
   - $V_o = 25$ ft/s (From Example 1)
3. Set up the following table:

*$L/W_o$ is on a logarithmic scale so interpolations must be done logarithmically.

Riprap should be at least the size shown but can be larger. As a practical

| $L/D_e$ | L (ft) | $V_L/V_O$ | $v_1$ (ft/s) | Rock Size $d_{50}$ (ft) |
|---|---|---|---|---|
| (Assume) $D_e = W_o$ | (Compute) | (**Fig. 5.5-9**) | | (**Fig. 5.5-1**) |
| 10 | 64 | 0.59 | 14.7 | 1.4 |
| 15* | 96 | 0.37 | 9.0 | 0.6 |
| 20 | 128 | 0.30 | 7.5 | 0.4 |
| 21 | 135 | 0.28 | 7.0 | 0.4 |

consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 ft downstream from the culvert brink. Channel should be shaped and riprap should be installed in accordance with details shown in the HEC No. 11 publication.

**Example 3**

*Given*:

| | |
|---|---|
| 6-ft diameter CMC | Design discharge Q = 135 cfs |
| Slope channel So = 0.004 | Manning's n = 0.024 |
| Normal velocity is 5.9 ft/s | Flow is subcritical |
| Tailwater depth TW = 2.0 ft | |
| Normal depth in pipe for Q = 135 cfs is 4.5 ft | |

*Find*:

Riprap basin dimensions for these conditions.

*Solution*:

1. Determine $y_o$ and $V_o$

From Figure 5.5-7, $y_o/D = 0.45$

. $Q/D^{2.5} = 135/6^{2.5} = 1.53$

$TW/D = 2.0/6 = 0.33$

$y_o = .45 \times 6 = 2.7$ ft

$TW/y_o = 2.0/2.7 = 0.74$   $TW/y_o < 0.75$ O.K.

Determine Brink Area (A) for $y_o/D = 0.45$

From Uniform Flow in Circular Sections Table (from Section 5.3)

For $y_o/D = d/D = 0.45$

$A/D^2 = 0.3428$; therefore, $A = 0.3428 \times 6^2 = 12.3\ f^{t2}$

$V_o = Q/A = 135/12.3 = 11.0$ ft/s

BACK TO TOC

2. For Froude number calculations at brink conditions,

   $y_e = (A/2)^{1/2} = (12.3/2)^{1/2} = 2.48$ ft

3. Froude number = $Fr = V_o/(32.2 \times y_e)^{1/2} = 11/(32.2 \times 2.48)^{1/2} = 1.23 < 2.5$ OK

4. For most satisfactory results $0.25 < d_{50}/y_e < 0.45$

   Try $d_{50}/y_e = 0.25$

   $d_{50} = 0.25 \times 2.48 = 0.62$ ft

   From **Figure 5.5-8**, $h_s/y_e = 0.75$; therefore, $h_s = 0.75 \times 2.48 = 1.86$ ft

   Uniform Flow in Circular Sections Flowing Partly Full (From Section 5.3)

   Check: $h^s/d^{50} = 1.86/0.62 = 3$, $2 < h_s/d_{50} < 4$ OK

5. $L_s = 10 \times hs = 10 \times 1.86 = 18.6$ ft or $L_s = 3 \times W_o = 3 \times 6 = 18$ ft; therefore, use $L_s = 18.6$ ft

   $L_B = 15 \times hs = 15 \times 1.86 = 27.9$ ft or $L_B = 4 \times W_o = 4 \times 6 = 24$ ft; therefore, use $L_B = 27.9$ ft

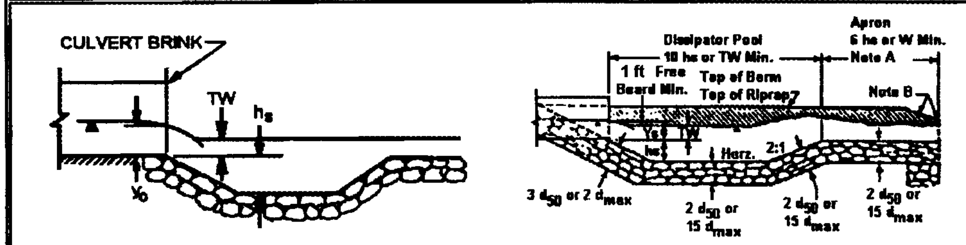
   $d_{50} = 0.62$ ft or use $d_{50} = 8$ in

Other basin dimensions should be designed in accordance with details shown on **Figure 5.5-5**. **Figure 5.5-10** is provided as a convenient form to organize and present the results of riprap basin designs.

*Note*: When using the design procedure outlined in this section, it is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the 3 x $d_{50}$ thickness of riprap on the approach and the 2 x $d_{50}$ thickness of riprap on the basin floor and the apron in the downstream portion of the basin, the damage should be superficial.

**RIPRAP BASIN**

Project No. ______
Designer ______ Date ______
Reviewer ______ Date ______




| DESIGN VALUES | TRIAL 1 | FINAL TRIAL |
|---|---|---|
| Equi. Depth, $d_E$ | | |
| $D_{50}/d_E$ | | |
| $D_{50}$ | | |
| Froude No., Fr | | |
| $h_s/d_E$ | | |
| $h_s$ | | |
| $h_s/D_{50}$ | | |
| $2 < h_s/D_{50} < 4$ | | |

| BASIN DIMENSIONS | | FEET |
|---|---|---|
| Pool length is the larger of: | $10h_s$ | |
| | $3W_o$ | |
| Basin length is the larger of: | $15h_s$ | |
| | $4W_o$ | |
| Approach Thickness | $3D_{50}$ | |
| Basin Thickness | $2D_{50}$ | |

| TAILWATER CHECK | |
|---|---|
| Tailwater, TW | |
| Equivalent depth, $d_E$ | |
| $TW/d_E$ | |
| IF $TW/d_E > 0.75$, calculate riprap downstream | |
| $D_E=(4A_c/\pi)^{0.5}$ | |

| DOWNSTREAM RIPRAP | | | | |
|---|---|---|---|---|
| $L/D_E$ | L | $V_L/V_o$ | $V_L$ | $D_{50}$ |
| | | | | |
| | | | | |
| | | | | |
| | | | | |
| | | | | |

**Figure 5.5-10** Riprap Basin Design Form

(Source: USDOT, FHWA, HEC-14, 2012)

BACK TO TOC

## 5.5.6 Baffled Outlets

### 5.5.6.1 DESCRIPTION

The baffled outlet (also known as the Impact Basin - USBR Type VI) is a boxlike structure with a vertical hanging baffle and an end sill, as shown in **Figure 5.5-11**. Energy is dissipated primarily through the impact of the water striking the baffle and, to a lesser extent, through the resulting turbulence. This type of outlet protection has been used with outlet velocities up to 50 feet per second and with Froude numbers from 1 to 9. Tailwater depth is not required for adequate energy dissipation, but a tailwater will help smooth the outlet flow.

### 5.5.6.2 DESIGN PROCEDURE

The following design procedure is based on physical modeling studies summarized from the U.S. Department of Interior (1978). The dimensions of a baffled outlet as shown in **Figure 5.5-11** should be calculated as follows:

(Step 1) Determine input parameters, including:

- **h** = Energy head to be dissipated, in ft (can be approximated as the difference between channel invert elevations at the inlet and outlet)
- **Q** = Design discharge (cfs)
- **v** = Theoretical velocity (ft/s = 2gh)
- **A** = Q/v = Flow area (ft$^2$)
- **d** = $A^{0.5}$= Representative flow depth entering the basin (ft) *assumes square jet*
- **Fr** = $v/(gd)^{0.5}$ = Froude number, dimensionless

(Step 2) Calculate the minimum basin width, W, in ft, using the following equation.

$$W/d = 2.88Fr^{0.566} \text{ or } W = 2.88dFr^{0.566} \quad (5.5.2)$$

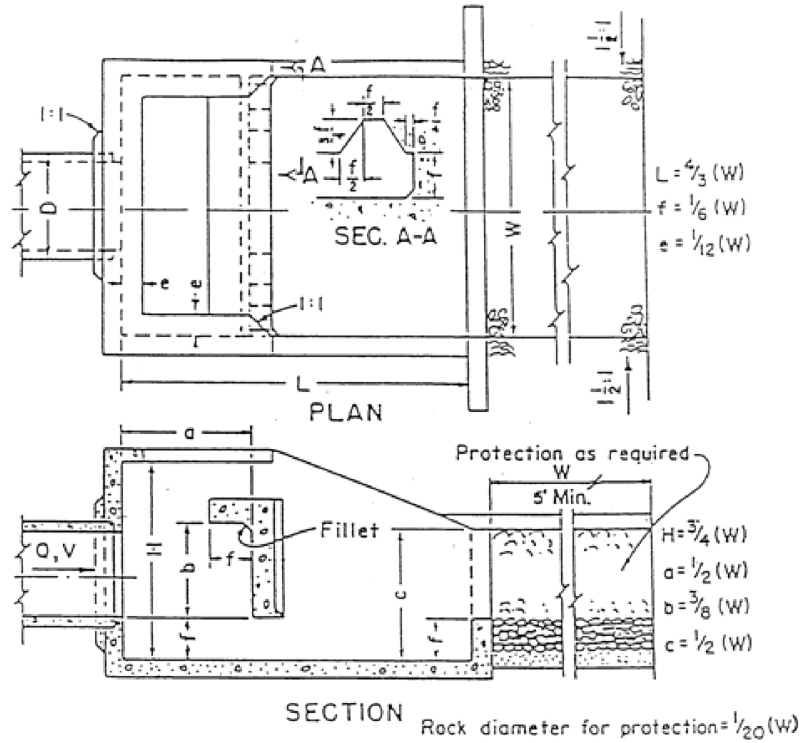



**Figure 5.5-11** Schematic of Baffled Outlet

(Source: U.S. Dept. of the Interior, 1978)

*Where*:

**W** = minimum basin width (ft)

**d** = depth of incoming flow (ft)

**Fr** = $v/(gd)^{0.5}$ = Froude number, dimensionless

(Step 3) Calculate the other basin dimensions as shown in **Figure 5.5-11**, as a function of W. Construction drawings for selected widths are available from the U.S. Department of the Interior (1978).

(Step 4) Calculate required protection for the transition from the baffled outlet to the natural channel based on the outlet width. A riprap apron should be added of width W, length W (or a 5-foot minimum), and depth f (W/6). The side slopes should be 1.5:1, and median rock diameter should be at least W/20.

(Step 5) Calculate the baffled outlet invert elevation based on expected tailwater. The maximum distance between expected tailwater elevation and the invert should be b + f or some flow will go over the baffle with no energy dissipation. If the tailwater is known and fairly controlled, the baffled outlet invert should be a distance, b/2 + f, below the calculated tailwater elevation. If tailwater is uncontrolled, the baffled outlet invert should be a distance, f, below the down-stream channel invert.

(Step 6) Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 ft/s flowing full.

(Step 7) If the entrance pipe slopes steeply downward, the outlet pipe should be turned horizontal for at least 3 ft before entering the baffled outlet.

(Step 8) If it is possible that both the upstream and downstream ends of the pipe will be submerged, provide an air vent approximately 1/6 the pipe diameter near the upstream end to prevent pressure fluctuations and possible surging flow conditions.

#### 5.5.6.3 EXAMPLE DESIGN

A cross-drainage pipe structure has a design flow rate of 150 cfs, a head, h, of 15 ft from invert of pipe, and a tailwater depth, TW, of 3 ft above ground surface. Find the baffled outlet basin dimensions and inlet pipe requirements.

1. Compute the theoretical velocity from

   . $v = (2gh)^{0.5} = [2(32.2 \text{ ft/sec}^2)(15 \text{ ft})]^{0.5} = 31.1$ ft/s

   This is less than 50 ft/s, so a baffled outlet is suitable.

2. Determine the flow area using the theoretical velocity as follows:

   $A = Q/v = 150 \text{ cfs}/31.1 \text{ ft/sec} = 4.8 \text{ ft}^2$

3. Compute the flow depth using the area from Step 2.

   $d = (A)^{0.5} = (4.8 \text{ ft}^2)^{0.5} = 2.12$ ft

4. Compute the Froude number using the results from Steps 1 and 3.

   $Fr = v/(gd)^{0.5} = 31.1 \text{ ft/sec}/[(32.2 \text{ ft/sec}^2)(2.12 \text{ ft})]^{0.5} = 3.8$

5. Determine the basin width using **equation 5.5.2** with the Froude number from Step 4.

   $W = 2.88\, dFr^{0.566} = 2.88\,(2.12)\,(3.8)^{0.566} = 13.0$ ft (minimum)

   Use 13 ft as the design width.

6. Compute the remaining basin dimensions (as shown in **Figure 5.5-11**):
   - L = 4/3 (W) = 17.3 ft, use L = 17 ft, 4 in
   - f = 1/6 (W) = 2.17 ft, use f = 2 ft, 2 in
   - e = 1/12 (W) = 1.08 ft, use e = 1 ft, 1 in
   - H = 3/4 (W) = 9.75 ft, use H = 9 ft, 9 in
   - a = 1/2 (W) = 6.5 ft, use a = 6 ft, 6 in
   - b = 3/8 (W) = 4.88 ft, use b = 4 ft, 11 in
   - c = 1/2 (W) = 6.5 ft, use c = 6 ft, 6 in

   Baffle opening dimensions would be calculated as shown in **Figure 5.5-11**.

7. Basin invert should be at b/2 + f below tailwater, or

   (4 ft, 11 in)/2 + 2 ft, 2 in = 4.73 ft

   Use 4 ft 8 in; therefore, invert should be 2 ft, 8 in below ground surface.

8. The riprap transition from the baffled outlet to the natural channel should be 13 ft long by 13 ft wide by 2 ft, 2 in deep (W x W x f). Median rock diameter should be of diameter W/20, or about 8 in.

9. Inlet pipe diameter should be sized for an inlet velocity of about 12 ft/s.

   $(3.14d)^2/4 = Q/v;\ d = [(4Q)/3.14v)]^{0.5} = [(4(150 \text{ cfs})/3.14(12 \text{ ft/sec})]^{0.5} = 3.99$ ft

   Use 48-in pipe. If a vent is required, it should be about 1/6 of the pipe diameter or 8 in.

BACK TO TOC

## References

American Association of State Highway and Transportation Officials, 1982. *Highway Drainage Guidelines*.

American Association of State Highway and Transportation Officials, 1981 and 1998. *Model Drainage Manual*.

Chow, V. T., ed., 1959. *Open Channel Hydraulics*. McGraw Hill Book Co. New York.

Debo, Thomas N. and Andrew J. Reese, 1995. *Municipal Storm Water Management*. Lewis Publishers.

Federal Highway Administration, 1989. *Bridge Waterways Analysis Model (WSPRO), Users Manual*, FHWA IP-89-027.

Federal Highway Administration, 1971. *Debris-Control Structures*. Hydraulic Engineering Circular No. 9.

Federal Highway Administration, 1987. *HY8 Culvert Analysis Microcomputer Program Applications Guide*. Hydraulic Microcomputer Program HY8.

Federal Highway Administration, 1983. *Hydraulic Design of Energy Dissipators for Culverts and Channels*. Hydraulic Engineering Circular No. 14.

Federal Highway Administration, 2012. *Hydraulic Design of Highway Culverts*. Hydraulic Design Series No. 5.

Federal Highway Administration, 1978. *Hydraulics of Bridge Waterways*. Hydraulic Design Series No. 1.

Federal Highway Administration, 1996. *Urban Drainage Design Manual*. Hydraulic Engineering Circular No. 22.

Federal Highway Administration, 1967. *Use of Riprap for Bank Protection*. Hydraulic Engineering Circular No. 11.

French, R. H., 1985. *Open Channel Hydraulics*. McGraw Hill Book Co. New York.

Harza Engineering Company, 1972. *Storm Drainage Design Manual*. Prepared for the Erie and Niagara Counties Regional Planning Bd. Harza Engineering Company, Grand Island, NY.

HYDRAIN Culvert Computer Program (HY8). Available from McTrans Software, University of Florida, 512 Weil Hall, Gainesville, Florida 32611.

Maynord, S. T., 1987. *Stable Riprap Size for Open Channel Flows*. Ph.D. Dissertation. Colorado State University, Fort Collins, CO.

Morris, J. R., 1984. *A Method of Estimating Floodway Setback Limits in Areas of Approximate Study*. In Proceedings of 1984 International Symposium on Urban Hydrology, Hydraulics and Sediment Control. Lexington, Kentucky: University of Kentucky.

Peterska, A. J., 1978. *Hydraulic Design of Stilling Basins and Energy Dissipators*. Engineering Monograph No. 25. U. S. Department of Interior, Bureau of Reclamation. Washington, DC.

Prince George's County, MD, 1999. *Low-Impact Development Design Strategies, An Integrated Design Approach*.

Reese, A. J., 1984. *Riprap Sizing, Four Methods*. In Proceedings of ASCE Conference on Water for Resource Development, Hydraulics Division, ASCE. David L. Schreiber, ed.

Reese, A. J., 1988. *Nomographic Riprap Design*. Miscellaneous Paper HL 88-2. Vicksburg, Mississippi: U. S. Army Engineers, Waterways Experiment Station.

Searcy, James K., 1967. *Use of Riprap for Bank Protection*. Federal Highway Administration.

U. S. Department of Interior, 1983. *Design of Small Canal Structures*.

U.S. Department of Interior, Bureau of Reclamation, 1978. *Design of Small Canal Structures*.

BACK TO TOC

U. S. Department of Transportation, Federal Highway Administration, 1973. *Design Charts For Open Channel Flow*. Hydraulic Design Series No. 3. Washington, DC.

U. S. Department of Transportation, Federal Highway Administration, 1986. *Design of Stable Channels with Flexible Linings*. Hydraulic Engineering Circular No. 15. Washington, DC.

U.S. Department of Transportation, Federal Highway Administration, 1984. *Drainage of Highway Pavements*. Hydraulic Engineering Circular No. 12.

U. S. Department of Transportation, Federal Highway Administration, 1984. *Guide for Selecting Manning's Roughness Coefficients For Natural Channels and Flood Plains*. FHWA-TS-84-204. Washington, DC.

U. S. Department of Transportation, Federal Highway Administration, 1983. *Hydraulic Design of Energy Dissipators for Culverts and Channels*. Hydraulic Engineering Circular No. 14. Washington, DC.

Wright-McLaughlin Engineers, 1969. *Urban Storm Drainage Criteria Manual, Vol. 2*. Prepared for the Denver Regional Council of Governments. Wright-McLaughlin Engineers, Denver, CO

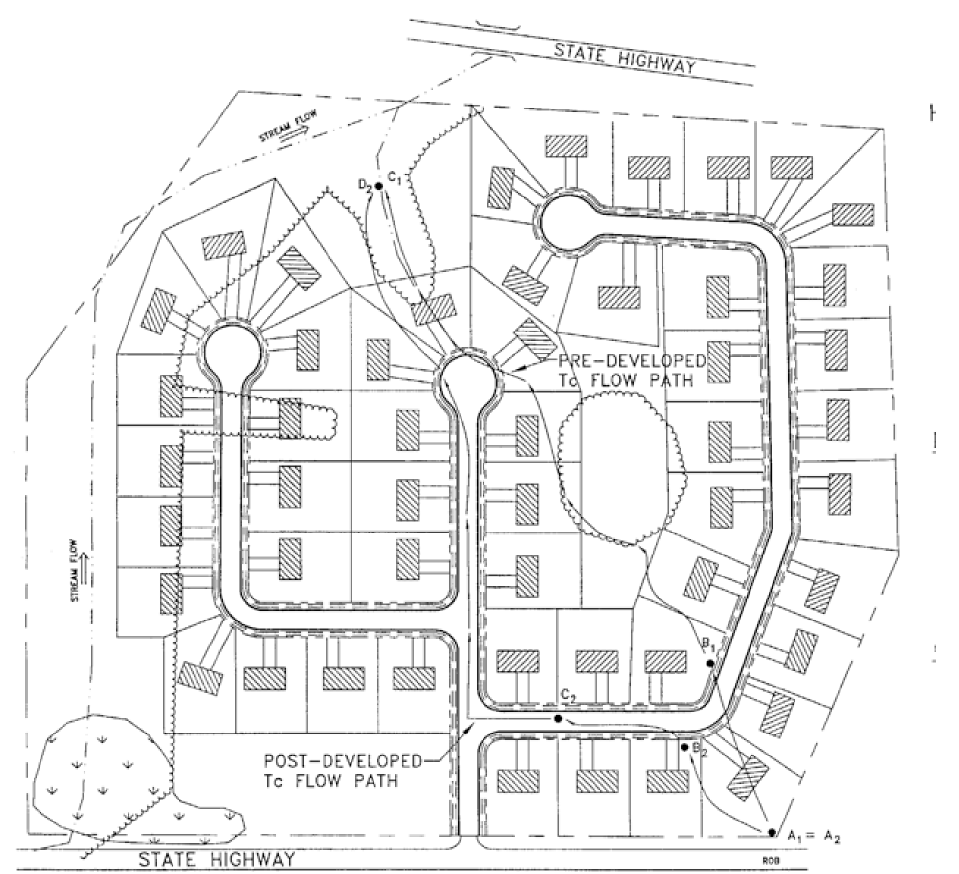
# Appendix A: Rainfall Tables for Georgia

The National Oceanic and Atmoshpheric Administration provides rainfall tables for the State of Georgia on their website:

http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ga

# Appendix B-1: Stormwater Pond Design Example

The following design example is for a wet extended detention (ED) stormwater pond




**Base Data**

Site Area = Total Drainage Area (A) = 38.0 ac (1,655,280 ft$^2$)
Measured Impervious Area=13.8 ac; or I=13.8/38=36.3%
Soils Types: 60% "C", 40% "B"
Zoning: Residential (½ acre lots)

**Hydrologic Data**

| | Pre | Post |
|---|---|---|
| CN | 65 | 78 |
| $t_c$ | 0.32 hr | 0.17 hr |

Figure 1. Peachtree Meadows Site Plan

BACK TO TOC

This example focuses on the design of a wet stormwater facility to meet the water quality requirements of the site through the water quality treatment (method 4) requirement. In general, the primary function of stormwater ponds is to provide water quality treatment (TSS removal) and detention. Stormwater ponds do not contribute to a runoff reduction goal.

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

The layout of the Peachtree Meadows subdivision is shown on the previous page. This example assumes that the local community has adopted the unified stormwater sizing criteria requirements.

**STEP 1 - CONFIRM LOCAL DESIGN CRITERIA AND APPLICABILITY**

There are no additional requirements for this site.

**STEP 2 - DETERMINE IF THE DEVELOPMENT SITE AND CONDITIONS ARE APPROPRIATE FOR THE USE OF A STORMWATER POND**

Site Specific Data:

The site area and drainage area to the pond is 38.0 acres (38 ac > min. 25 ac required). Existing ground at the pond outlet is 919 MSL. Soil boring observations reveal that the seasonally high water table is at elevation 918. The underlying soils are SC (sandy clay) and are suitable for earthen embankments and to support a wet pond without a liner. The stream invert at the adjacent stream is at elevation 916.

Other site screening aspects listed in Section 4.25 were assessed and a pond was found to be suitable.

**STEP 3 - COMPUTE RUNOFF CONTROL VOLUMES FROM THE UNIFIED STORMWATER SIZING CRITERIA**

More detailed hydrologic calculations will be required during the design step – these numbers are preliminary.

Compute Water Quality Volume ($WQ_v$)

- Compute Runoff Coefficient, $R_v$

$R_v$ = 0.05 + (I) (0.009)
= 0.05 + (36.3) (0.009) = **0.38**

- Compute $WQ_v$

$WQ_v$ = (1.2") ($R_v$) (A) / 12
= (1.2") (0.38) (1,655,280 ft$^2$) (1ft/12in)
= <u>62,900</u> ft$^3$ (1.44 ac-ft)

Develop Site Hydrologic Input Parameters

Per Figures 2 and 3. Note that any hydrologic models using NRCS TR-55 procedures, such as TR-20, HEC-HMS, or other software platforms, can be used to perform preliminary hydrologic calculations.

| Condition | Area *ac* | CN | TC *hrs* |
|---|---|---|---|
| Pre-developed | 38 | 65 | 0.32 |
| Post-developed | 38 | 78 | 0.17 |

Perform Preliminary Hydrologic Calculations

| Condition Runoff | $Q_{1\text{-yr}}$ *inches* | $Q_{5\text{-yr}}$ *cfs* | $Q_{25\text{-yr}}$ *cfs* | $Q_{100\text{-yr}}$ *cfs* |
|---|---|---|---|---|
| Pre-developed | 0.70 | 22 | 104 | 148 |
| Post-developed | 1.42 | 67 | 192 | 251 |

**COMPUTE CHANNEL PROTECTION VOLUME, ($CP_v$)**

For stream channel protection, provide 24 hours of extended detention for the 1-year event.

In order to determine a preliminary estimate of storage volume for channel protection and overbank flood control, it will be necessary to perform hydrologic calculations using approved methodologies. This example uses the NRCS TR-55 methodology presented in Section 3.1.5 to determine pre- and post-development peak discharges for the 1-yr, 25-yr, and 100-yr 24-hour return frequency storms.

Utilize NRCS TR-55 approach to Compute Channel Protection Storage Volume

See Section 3.1.5

Initial abstraction (Ia) for CN of 78 is 0.564: [Ia = (200/CN - 2)]

Ia/P = (0.564)/ 3.4 inches = 0.17
$T_c$ = 0.17 hours
$q_u$ = 800 csm/in (From Figure 3.1.5-6)

Knowing $q_u$ and T (extended detention time), find $q_o/q_i$. For a Type II rainfall distribution.

Peak outflow discharge/peak inflow discharge ($q_o/q_i$) = 0.022 (From Figure 3.3.5-1)

$$Vs/Vr = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$$ (Equation 3.3.9)

Where Vs equals channel protection storage ($CP_v$) and Vr equals the volume of runoff in inches.

Vs/Vr = 0.65 Using Equation 3.3.10, calculate $V_S$...

Therefore, Vs = $CP_v$ = 0.65(1.42")(1/12)(38ac) = 2.9 ac-ft (126,324 cubic feet)

**Define the average CPv-ED Release Rate**

The above volume, 2.9 ac-ft (126,324 ft$^3$), is to be released over 24 hours. (126,324 ft$^3$) / (24 hrs × 3,600 sec/hr) = 1.46 cfs

## DETERMINE OVERBANK FLOOD PROTECTION VOLUME, ($Q_{P25}$)

- For a $Q_{in}$ of 192 cfs, and an allowable $Q_{out}$ of 104 cfs, and a runoff volume of 520,364 cubic feet (11.95 ac-ft) the Vs necessary for 25-year control is 3.33 ac-ft, under a developed CN of 78. Note that 6.5 inches of rain fall during this event.
- While the TR-55 short-cut method reports to incorporate multiple stage structures, experience has shown that an additional 10-15% storage is required when multiple levels of extended detention are provided inclusive with the 25-year storm. So, for preliminary sizing purposes add 15% to the required volume for the 25-year storm. $Q_{p-25}$ = 3.33 × 1.15 = 3.8 ac-ft.

## ANALYZE SAFE PASSAGE OF 100 YEAR DESIGN STORM ($Q_f$)

At final design, provide safe passage for the 100-year event, or detain it, depending on downstream conditions and local policy. Based on field observation and review of local requirements no control of the 100-year storm is necessary. If it were storage estimates would have been made similar to the $Q_p$ Volume in the previous sub-step.

**Table 1** Summary of General Storage Requirements for Peachtree Meadows

| Symbol | Control Volume | Volume Required (ac-ft) | Notes |
|---|---|---|---|
| $WQ_v$ | Water Quality | 1.44 | |
| $CP_v$ | Channel Protection | 2.9 | Average ED release rate is 1.46 cfs over 24 hours |
| $Q_{p25}$ | Overbank Flood Protection | 3.33 | 3.8 ac-ft will be provided |
| $Q_f$ | Extreme Flood Protection | NA | Provide safe passage for the 100-year event in final design |

BACK TO TOC

**Figure 2** Peachtree Meadows Pre-Development Conditions

## Peak Discharge Summary

| | | |
|---|---|---|
| Job: | P'tree Meadows | EWB |
| Drainage Area Name: | Pre-Developed Conditions | 3-Jan-00 |

| Cover Description | Soil Name | Group A,B,C,D? | CN from TABLE 2.1-5.1 | Area (in acres) |
|---|---|---|---|---|
| meadow (good cond.) | | C | 71 | 22.80 Ac. |
| meadow (good cond.) | | B | 58 | 9.20 Ac. |
| wood (good cond.) | | B | 55 | 6.00 Ac. |
| | | | Area Subtotals: | 38.00 Ac. |

| Time of Concentration<br>2-Yr 24 Hr Rainfall = 4.1 In | Surface Cover/<br>Cross Section | Manning 'n'/<br>Wetted Per | Flow Length/<br>Avg Velocity | Slope/<br>Tt (Hrs) |
|---|---|---|---|---|
| Sheet Flow | dense grass | 'n' = 0.24 | 150 Ft. | 2.50%<br>0.27 Hrs |
| Shallow Flow | unpaved | | 500 Ft.<br>3.23 F.P.S. | 4.00%<br>0.04 Hrs. |
| Channel Flow | | | | |

| | | | | |
|---|---|---|---|---|
| Total Area in Acres = | 38.00 Ac. | Total Sheet Flow = | Total Shallow Flow = | Total Channel Flow = |
| Weighted CN = | 65 | 0.27 Hrs. | 0.04 Hrs. | 0.00 Hrs. |
| Time of Concentration = | 0.31 Hrs. | | | |
| Pond Factor = | 1 | | RAINFALL TYPE II | |

| Storm | Precipitation (P) inches | Runoff (Q) | Qp, Peak Discharge | Total Storm Volumes |
|---|---|---|---|---|
| 1 Year | 3.4 In. | 0.7 In. | 21.9 CFS | 93,771 Cu. Ft. |
| 2 Year | 4.1 In. | 1.1 In. | 37.2 CFS | 148,313 Cu. Ft |
| 5 Year | 4.8 In. | 1.5 In. | 54 CFS | 209,936 Cu. Ft. |
| 10 Year | 5.5 In. | 2.0 In. | 74 CFS | 277,081 Cu. Ft. |
| 25 Year | 6.5 In. | 2.7 In. | 101 CFS | 373,288 Cu. Ft. |
| 50 Year | 7.2 In. | 3.3 In. | 124 CFS | 449,409 Cu. Ft. |
| 100 Year | 7.9 In. | 3.8 In. | 147 CFS | 528,261 Cu. Ft. |



BACK TO TOC

**Figure 3** Peachtree Meadows Post-Development Conditions

| Peak Discharge Summary | | | | |
|---|---|---|---|---|
| Job: | P'tree Meadows | | | EWB |
| Drainage Area Name: | Post-Developed Conditions | | | 3-Jan-00 |
| **Cover Description** | **Soil Name** | **Group A,B,C,D?** | **CN from TABLE 2.1-5.1** | **Area (in acres)** |
| open space | | C | 74 | 13.00 Ac. |
| open space | | B | 61 | 5.20 Ac. |
| woods (good cond.) | | B | 55 | 6.00 Ac. |
| impervious area | | C | 98 | 7.90 Ac. |
| impervious area | | B | 98 | 5.90 Ac. |
| | | | Area: | 38.00 Ac. |
| **Time of Concentration 2-Yr 24 Hr Rainfall = 4.1 In** | **Surface Cover/ Cross Section** | **Manning 'n'/ Wetted Per** | **Flow Length/ Avg Velocity** | **Slope/ Tt (Hrs)** |
| Sheet Flow | short grass | 'n' = 0.15 | 100 Ft. | 2.50%<br>0.13 Hrs. |
| Shallow | paved | | 300 Ft.<br>2.87 F.P.S. | 2.00%<br>0.03 Hrs. |
| Channel<br>Hydraulic Radius | X-S estimated | 'n' = 0.013<br>WP estimated | 600 Ft.<br>16.21 F.P.S. | 2.00%<br>0.01 Hrs. |
| Total Area in Acres = | 38.00 Ac. | Total Sheet Flow = 0.13 Hrs. | Total Shallow Flow = 0.03 Hrs. | Total Channel Flow = 0.01 Hrs. |
| Weighted CN = | 78 | | | |
| Time of Concentration = | 0.17 Hrs. | | | |
| Pond Factor = | 1 | | RAINFALL TYPE | |
| **Storm** | **Precipitation (P) inches** | **Runoff (Q)** | **Qp, Peak Discharge** | **Total Storm Volumes** |
| 1 Year | 3.4 In. | 1.4 In. | 67.1 CFS | 198,988 Cu. Ft. |
| 2 Year | 4.1 In. | 2.0 In. | 95.8 CFS | 269,103 Cu. Ft. |
| 5 Year | 4.8 In. | 2.5 In. | 127 CFS | 350,750 Cu. Ft. |
| 10 Year | 5.5 In. | 3.2 In. | 159 CFS | 435,668 Cu. Ft. |
| 25 Year | 6.5 In. | 4.0 In. | 202 CFS | 552,584 Cu. Ft. |
| 50 Year | 7.2 In. | 4.7 In. | 234 CFS | 642,337 Cu. Ft. |
| 100 Year | 7.9 In. | 5.3 In. | 267 CFS | 733,444 Cu. Ft. |

BACK TO TOC

### STEP 4 - DETERMINE PRETREATMENT VOLUME

Size wet forebay to treat 0.1"/impervious acre. (13.8 ac) (0.1") (1'/12") = **0.12 ac-ft** (forebay volume is included in $WQ_v$ as part of permanent pool volume)

### STEP 5 - DETERMINE PERMANENT POOL VOLUME (AND WATER QUALITY ED VOLUME)

Size permanent pool volume to contain 50% of $WQ_v$:
0.5 x (1.44 ac-ft) = **0.72 ac-ft.** (includes 0.12 ac-ft of forebay volume)

Size ED volume to contain 50% of $WQ_v$: 0.5 x (1.44 ac-ft) = **0.72 ac-ft**

Note: This design approach assumes that all of the ED volume will be in the pond at once. While this will not be the case, since there is a discharge during the early stages of storms, this conservative approach allows for ED control over a wider range of storms, not just the target rainfall.

### STEP 6 - DETERMINE POND LOCATION AND PRELIMINARY GEOMETRY. CONDUCT POND GRADING AND DETERMINE STORAGE AVAILABLE FOR PERMANENT POOL AND WATER QUALITY EXTENDED DETENTION

This step involves initially grading the pond (establishing contours) and determining the elevation-storage relationship for the pond. Storage must be provided for the permanent pool (including sediment forebays), extended detention ($WQ_v$-ED), $CP_v$-ED, and 25-year storm, plus sufficient additional storage to pass the 100-year storm with minimum freeboard. An elevation-storage table and curve is prepared using the average area method for computing volumes. See Figure 4 for pond location on site, Figure 5 grading and Figure 6 for Elevation-Storage Data.

BACK TO TOC

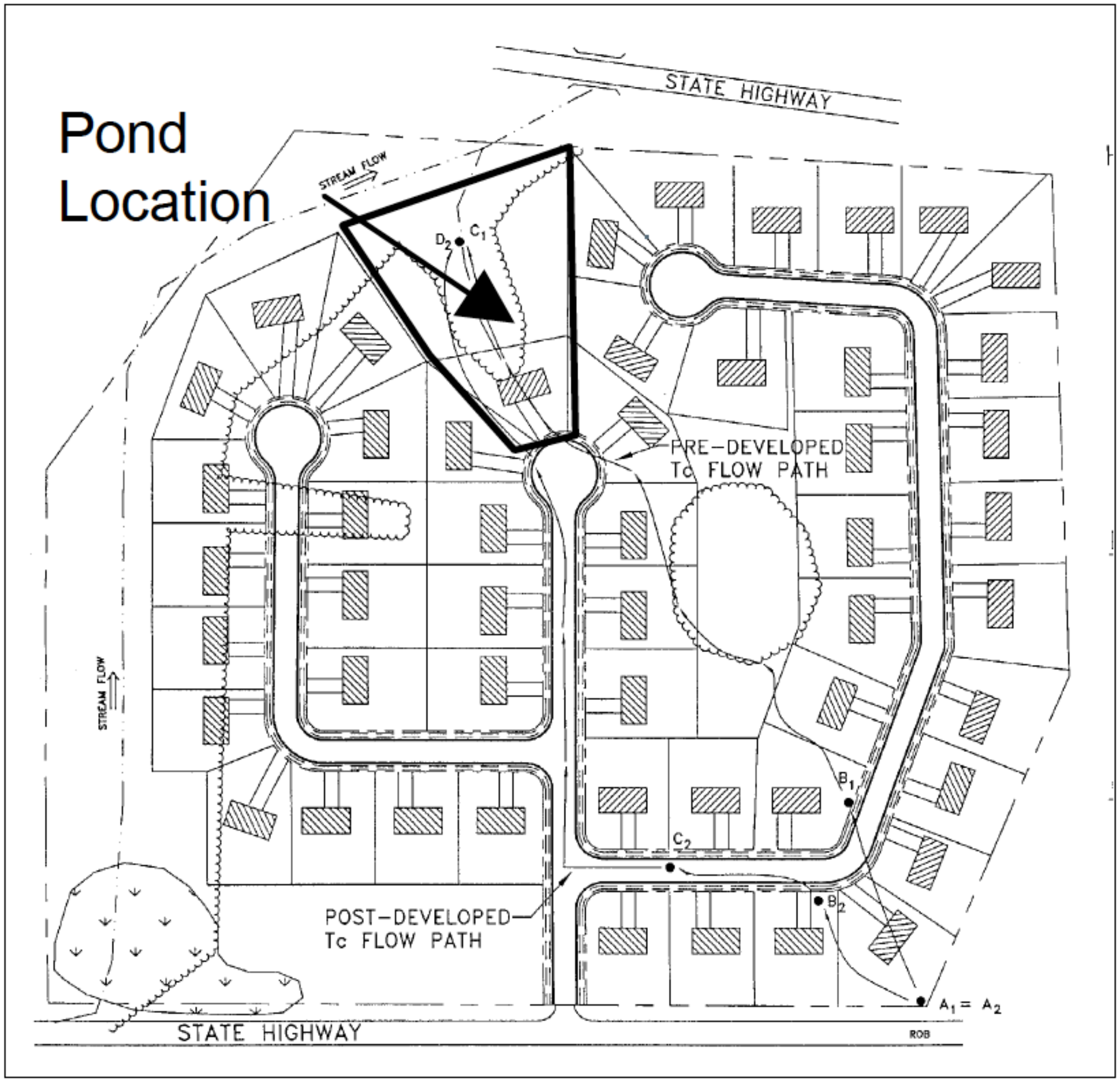



Figure 4. Pond Location on Site

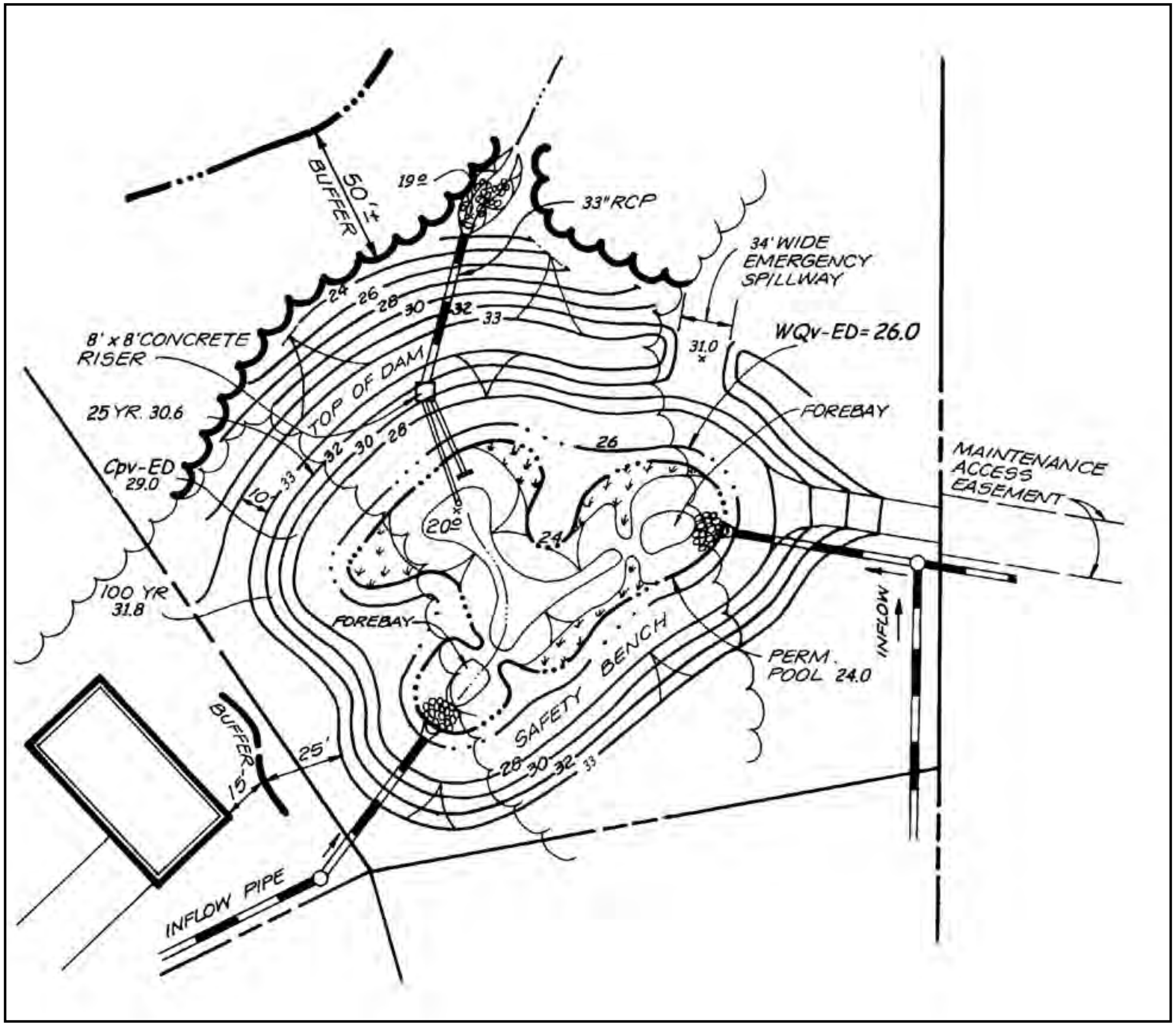



Figure 5. Plan View of Pond Grading (Not to Scale)

| Elevation (MSL) | Average Area (ft²) | Depth (ft) | Volume (ft³) | Cumulative Volume (ft³) | Cumulative Volume (ac-ft) | Volume Above Permanent Pool (ac-ft) |
|---|---|---|---|---|---|---|
| 920.0 | | | | | | |
| 921.0 | 7838 | 1 | 7838 | 7838 | 0.18 | |
| 923.0 | 11450 | 2 | 22900 | 30738 | 0.71 | |
| 924.0 | 14538 | 1 | 14538 | 45275 | 1.04 | 0 |
| 925.0 | 15075 | 1 | 15075 | 60350 | 1.39 | 0.35 |
| 925.5 | 16655 | 0.5 | 8328 | 68678 | 1.58 | 0.54 |
| 926.0 | 17118 | 0.5 | 8559 | 77236 | 1.77 | 0.73 |
| 926.5 | 21000 | 0.5 | 10500 | 87736 | 2.01 | 0.97 |
| 927.0 | 25000 | 0.5 | 12500 | 100236 | 2.30 | 1.26 |
| 927.5 | 30000 | 0.5 | 15000 | 115236 | 2.65 | 1.61 |
| 928.0 | 36000 | 0.5 | 18000 | 133236 | 3.06 | 2.02 |
| 928.5 | 38000 | 0.5 | 19000 | 152236 | 3.49 | 2.46 |
| 929.0 | 41000 | 0.5 | 20500 | 172736 | 3.97 | 2.93 |
| 929.5 | 43000 | 0.5 | 21500 | 194236 | 4.46 | 3.42 |
| 930.0 | 45000 | 0.5 | 22500 | 216736 | 4.98 | 3.94 |
| 930.5 | 47000 | 0.5 | 23500 | 240236 | 5.52 | 4.48 |
| 931.0 | 49000 | 0.5 | 24500 | 264736 | 6.08 | 5.04 |
| 931.5 | 52000 | 0.5 | 26000 | 290736 | 6.67 | 5.64 |
| 932.0 | 55000 | 0.5 | 27500 | 318236 | 7.31 | 6.27 |
| 932.5 | 58000 | 0.5 | 29000 | 347236 | 7.97 | 6.93 |
| 933.0 | 61000 | 0.5 | 30500 | 377736 | 8.67 | 7.63 |
| 933.5 | 65000 | 0.5 | 32500 | 410236 | 9.42 | 8.38 |
| 934.0 | 69000 | 0.5 | 34500 | 444736 | 10.21 | 9.17 |
| 935.0 | 74000 | 1 | 74000 | 518736 | 11.91 | 10.87 |

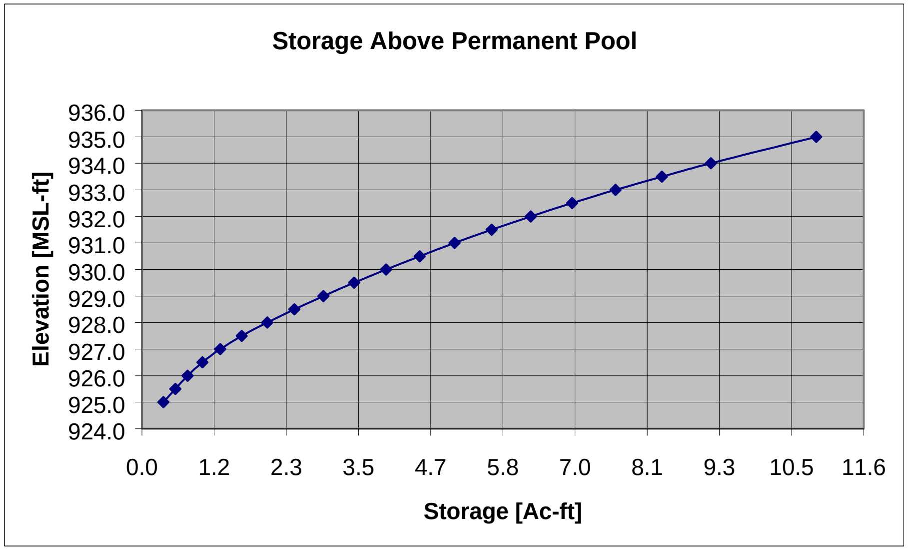



Figure 6. Storage-Elevation Table/ Curve

Set basic elevations for pond structures

- The pond bottom is set at elevation 920.0.
- Provide gravity flow to allow for pond drain, set riser invert at 919.5
- Set barrel outlet elevation at 919.0.

Set water surface and other elevations

- Required permanent pool volume = 50% of WQv = 0.72 ac-ft. From the elevation-storage table, read elevation 924.0 (1.04 ac-ft > 0.72 ac-ft) site can accommodate it and it allows a small safety factor for fine sediment accumulation - OK
- Forebay volume provided in two pools with avg. vol. = 0.08 ac-ft each (0.16 ac-ft > 0.12 ac-ft) OK
- Required extended detention volume (WQv-ED)= 0.72 ac-ft. From the elevation-storage table (volume above permanent pool), read elevation 926.0 (0.73 ac-ft > 0.72 ac-ft) OK. Set ED wsel = 926.0

Note: Total storage at elevation 926.0 = 1.77 ac-ft (greater than required WQv of 1.44 ac-ft)

Compute the required WQv-ED orifice diameter to release 0.72 ac-ft over 24 hours

- Avg. ED release rate = (0.72 ac-ft)(43,560 ft2/ac)/(24 hr)(3600 sec/hr) = 0.36 cfs
- Average head = (926.0 - 924.0)/ 2 = 1.0'
- Use orifice equation to compute cross-sectional area and diameter
  - » $Q = CA(2gh)^{0.5}$, for Q=0.36 cfs h = 1.0 ft; C = 0.6 = discharge coefficient solve for A
  - » $A = 0.36$ cfs / $[(0.6)((2)32.2\ ft/s2)(1.0\ ft))^{0.5}]$ $A = 0.075\ ft^2$, $A = \pi d^2 / 4$; dia. = 0.31 ft = 3.7"
  - » Use 4" pipe with 4" gate valve to achieve equivalent diameter

Compute the stage-discharge equation for the 3.7" dia. WQv orifice

- $Q_{WQv\text{-}ED} = CA(2gh)^{0.5} = (0.6)\ (0.075\ ft^2)\ [((2)(32.2\ ft/s2))^{0.5}]\ (h^{0.5})$,
- $Q_{WQv\text{-}ED} = (0.36)\ h^{0.5}$, where: h = wsel - 924.16

(Note: account for one half of orifice diameter when calculating head)

**STEP 7 - COMPUTE EXTENDED DETENTION ORIFICE RELEASE RATE(S) AND SIZE(S), AND ESTABLISH $CP_v$ ELEVATION**

Set the $CP_v$ pool elevation

- Required $CP_v$ storage = 2.9 ac-ft (see Table 1).
- From the elevation-storage table, read elevation 929 (this includes the $WQ_v$).
- Set $CP_v$ wsel = 929

Size $CP_v$ orifice

- Size to release average of 1.46 cfs.
  - » Average $WQ_v$-ED orifice release rate is 0.66 cfs, based on average head of 3.34' (926 – 924.16 + (929 – 926)/2)
  - » $CP_v$-ED orifice release = 1.46 -0.66 = 0.80 cfs
- Head = (929 - 926.0)/2 = 1.5'

Use orifice equation to compute cross-sectional area and diameter

- $Q = CA(2gh)^{0.5}$, for h = 1.5'
  - » $A = 0.80$ cfs / $[(0.6)((2)(32.2'/s^2)(1.5'))^{0.5}]$
  - » $A = 0.14\ ft^2$, $A = \pi d^2 / 4$;
  - » dia. = 0.42 ft = 5.0"
- Use 6" pipe with 6" gate valve to achieve equivalent diameter

Compute the stage-discharge equation for the 5.0" dia. $CP_v$ orifice

- $Q_{CPv\text{-}ED} = CA(2gh)^{0.5} = (0.6)\ (0.14\ ft^2)\ [((2)\ (32.2'/s^2))^{0.5}]\ (h^{0.5})$,
- $Q_{CPv\text{-}ED} = (0.67)\ (h^{0.5})$, where: h = wsel - 926.21

(Note: account for one half of orifice diameter when calculating head)

BACK TO TOC

## STEP 8 - CALCULATE $Q_{P25}$ (25-YEAR STORM) RELEASE RATE AND WATER SURFACE ELEVATION

In order to calculate the 25 year release rate and water surface elevation, the designer must set up a stage-storage-discharge relationship for the control structure for each of the low flow release pipes ($WQ_v$-ED and $CP_v$-ED) plus the 25 year storm.

Develop basic data and information

- The 25 year pre-developed peak discharge = 104 cfs,
- The post developed inflow = 192 cfs, from Table 1,
- From previous estimate $Q_{p\text{-}25}$ = 3.33 ac-ft. Adding 15% to account for ED storage yields a preliminary volume of 3.8 ac-ft.
- From elevation-storage table (Figure 6), read elevation 930.1.

Size 25 year slot to release 104 cfs at elevation 930.1.

- @ wsel 930.1:
  - » $WQ_v$-ED orifice releases 0.88 cfs,
  - » $CP_v$-ED orifice releases 1.32 cfs, therefore;
  - » Allowable $Q_{p\text{-}25}$ = 104 cfs - (.88 + 1.32) = 101.8 cfs, say 102 cfs.
- Max head = (930.1 – 929) = 1.1'
- Use weir equation to compute slot length
  - » $Q = CLH^{3/2}$
  - » L = 102 cfs / (3.1) (1.13/2) = 28 ft
- Use four 7ft x 1.5 ft slots for 25-year release (opening should be slightly larger than needed so as to have the barrel control before slot goes from weir flow to orifice flow).

Check orifice equation using cross-sectional area of opening

- $Q = CA(2gh)^{0.5}$, for h = 0.75' (For orifice equation, h is from midpoint of slot)
- A = 4 (7.0') (1.5') = 42.0ft$^2$
- Q = 0.6 (42.0ft$^2$) [(64.4)(0.75)]$^{0.5}$ = 175 cfs > 104 cfs, so use weir equation

$Q_{25}$ = (3.1) (28') $H^{3/2}$ $Q_{25}$ = (86.8) $H^{3/2}$, where H = wsel – 929.0

Size barrel to release approximately 104 cfs at elevation 930.1

- Check inlet condition: (use Section 5.3 culvert charts)
  - » $H_w$ = 930.1-919.5 = 10.6 ft
  - » Try 33" diameter RCP, Using Figure 5.3-1a
  - » $H_w$ / D = 10.6/2.75 = 3.85, Discharge = 88 cfs
- Check outlet condition:
  - $Q = a\,[(2gH)/(1+k_m+k_pL)]^{0.5}$

  where:Q = discharge in cfs

  a = pipe cross sectional area in ft$^2$

  g = acceleration of gravity in ft/sec$^2$

  H = head differential (wsel - downstream centerline of pipe or tailwater elev.)

  $k_m$ = coefficient of minor losses (use 1.0)

  $k_p$ = pipe friction loss coef. (= $5087n^2/d^{4/3}$, d in ", n is Manning's n)

  L = pipe length in ft
  - » H = 930.1 - (919.0 + 1.38) = 9.72'
  - » for 33" RCP, 70 feet long:
  - » Q = 7.1 [(64.4) (9.72) / 1+1+(.007) (70)]$^{0.5}$ = 112.6 cfs
  - » 88 cfs < 112.6 cfs, so barrel is inlet controlled.

Note: pipe will control flow before high stage inlet reaches max head.

Complete stage-storage-discharge summary (Figure 7) up to preliminary 25-year wsel (930.1) and route 25 year post-developed condition inflow using computer software. Pond routing computes 25-year wsel at 930.8 with discharge = 92.4 cfs.

| Elevation (MSL) | Storage (ac-ft) | Low Flow $WQ_v$-ED 3.7" eq dia | | Riser | | | | | | Barrel | | | | Emergency Spillway | | Total Discharge |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | $CP_v$-ED 5.0" eq. dia | | High Stage Slot | | | | Inlet | | Pipe | | | | |
| | | | | | | Orifice | | Weir | | | | | | | | |
| | | *H* *ft* | *Q* *cfs* | *H* *ft* | *Q* *cfs* | *H* *ft* | *Q* *cfs* | *H* *ft* | *Q* *cfs* | *H* *ft* | *Q* *cfs* | *H* *ft* | *Q* *cfs* | *H* *ft* | *Q* *cfs* | *Q* *cfs* |
| 924.0 | 0.00 | 0 | 0 | | | | | | | | | | | | | 0.00 |
| 925.0 | 0.35 | 0.8 | 0.33 | | | | | | | | | | | | | 0.33 |
| 925.5 | 0.54 | 1.3 | 0.42 | | | | | | | | | | | | | 0.42 |
| 926.0 | 0.73 | 1.8 | 0.49 | 0 | 0 | | | | | | | | | | | 0.49 |
| 926.5 | 0.97 | 2.3 | 0.55 | 0.3 | 0.36 | | | | | | | | | | | 0.91 |
| 927.0 | 1.26 | 2.8 | 0.61 | 0.8 | 0.60 | | | | | | | | | | | 1.20 |
| 927.5 | 1.61 | 3.3 | 0.66 | 1.3 | 0.76 | | | | | | | | | | | 1.42 |
| 928.0 | 2.02 | 3.8 | 0.71 | 1.8 | 0.90 | | | | | | | | | | | 1.60 |
| 928.5 | 2.46 | 4.3 | 0.75 | 2.3 | 1.01 | | | | | | | | | | | 1.76 |
| 929.0 | 2.93 | 4.8 | 0.79 | 2.8 | 1.12 | N/A | - | 0.0 | 0.0 | | | | | | | 1.91 |
| 929.5 | 3.42 | 5.3 | 0.83 | 3.3 | 1.22 | | | 0.5 | 30.7 | | | | | | | 32.7 |
| 930.0 | 3.94 | 5.8 | 0.87 | 3.8 | 1.30 | | | 1.0 | 86.8 | | | | | | | 89.0 |
| 930.1 | 4.10 | 5.9 | 0.88 | 3.9 | 1.32 | | | 1.1 | 100.1 | 10.6 | 88 | 9.7 | 112.6 | | | 90.2 |
| 930.5 | 4.48 | 6.3 | 0.91 | 4.3 | 1.39 | 0.75 | 175 | 1.5 | 159.5 | 11.0 | 90 | 10.1 | 114.9 | | | 92.3 |
| 931.0 | 5.04 | - | - | - | - | - | - | - | - | 11.5 | 92.5 | 10.6 | 117.7 | 0.0 | 0.0 | 92.5 |
| 931.5 | 5.64 | - | - | - | - | - | - | - | - | 12.0 | 95 | 11.1 | 120.4 | 0.5 | 24.0 | 119.0 |
| 932.0 | 6.27 | - | - | - | - | - | - | - | - | 12.5 | 97 | 11.6 | 123.1 | 1.0 | 79.0 | 176.0 |
| 932.5 | 6.93 | - | - | - | - | - | - | - | - | 13.0 | 100 | 12.1 | 125.7 | 1.5 | 154.0 | 253.5 |
| 933.0 | 7.63 | - | - | - | - | - | - | - | - | 13.5 | 101.7 | 12.6 | 128.3 | 2.0 | 252.0 | 353.7 |

Figure 7 Stage-Storage-Discharge Summary

Note: Adequate outfall protection must be provided in the form of a riprap channel, plunge pool, or combination to ensure non-erosive velocities.

### STEP 9 - DESIGN EMBANKMENT(S) AND SPILLWAY(S)

The 25-year wsel is at 930.8. Set the emergency spillway at elevation 931.0 and use design information and criteria Earth Spillways (not included in this manual)

- $Q_{100}$ inflow = 251 cfs.
- Try 34' wide vegetated emergency spillway with 3:1 side slopes.
- @ elevation 932.6, H = 1.5', Emergency spillway, $Q_{ES}$ = 172 cfs. Primary spillway, $Q_{PS}$ 100 cfs
- $Q_{ES}$ + $Q_{PS}$ = 272 cfs, will be able to safely convey $Q_f$ = 251. (use computer routing for exact elevations and discharges).
- 100 year wsel = 931.7, say 932, so set top of embankment with 1 foot of freeboard at elevation 933.

### STEP 10 - INVESTIGATE POTENTIAL POND HAZARD CLASSIFICATION

Refer to Georgia Department of Natural Resources Rules for Dam Safety in Appendix H to establish preliminary classification of embankment and whether special design criteria need to be met.

Per Chapter 391-3-8.04, Dam safety rules do not apply to artificial barriers that are:

- Classified as a Category II Dam – dams where improper operation or dam failure would not expect to result in probable loss of human life
- Not in excess of 6 feet in height regardless of storage capacity, or which has a storage capacity at maximum water storage elevation not in excess of 15 acre-feet, regardless of height.

Check pond classification: Height = 931 -919 = 12', equals assumed embank-ment height, Pond will remain Category II or lower.

As reported in Table 1, the preliminary maximum storage volume required is about 3.33 acre-feet, which is substantially less than the 15 acre-feet exempt limit. Therefore, for initial design considerations, no additional dam safety requirements will apply. Once final design elevations and storage volumes have been determined, a final check for dam rules exemption should be made by the designer.

### STEP 11 - DESIGN INLETS, SEDIMENT FOREBAY(S), OUTLET STRUCTURES, MAINTENANCE ACCESS, AND SAFETY FEATURES.

**Table 2** Summary of Controls Provided

| Control Element | Type/Size of Control | Storage Provided | Elevation | Discharge | Remarks |
|---|---|---|---|---|---|
| *Units* | | *Acre-feet* | *MSL* | *cfs* | |
| Permanent Pool | | 0.86 | 924.0 | 0 | part of $WQ_v$ |
| Forebay | submerged berm | 0.12 | 924.0 | 0 | included in permanent pool volume |
| Water Quality Extended Detention ($WQ_v$-ED) | 4" pipe, sized to 3.7" equivalent diameter | 0.72 | 926.0 | 0.36 | part of $WQ_v$ above perm. pool, discharge is average release rate over 24 hours |
| Channel Protection ($CP_v$-ED) | 6" pipe sized to 5.0" equivalent diameter | 2.9 | 929.0 | 1.46 | volume above perm. pool, discharge is average release rate over 24 hours |
| Overbank Flood Protection ($Q_{p25}$) | Four 7' x 1.5' slots on a 8' x 8' riser, 36"barrel. | 3.8 | 930.8 | 92.4 | volume above perm. pool |
| Extreme Flood Protection ($Q_{f-100}$) | 34' wide earth spillway | 6.3 | 931.7 | 141 | volume above perm. pool |

See Figure 8 for profile through principal spillway of the facility.
See Figure 9 for a schematic of the riser.

BACK TO TOC

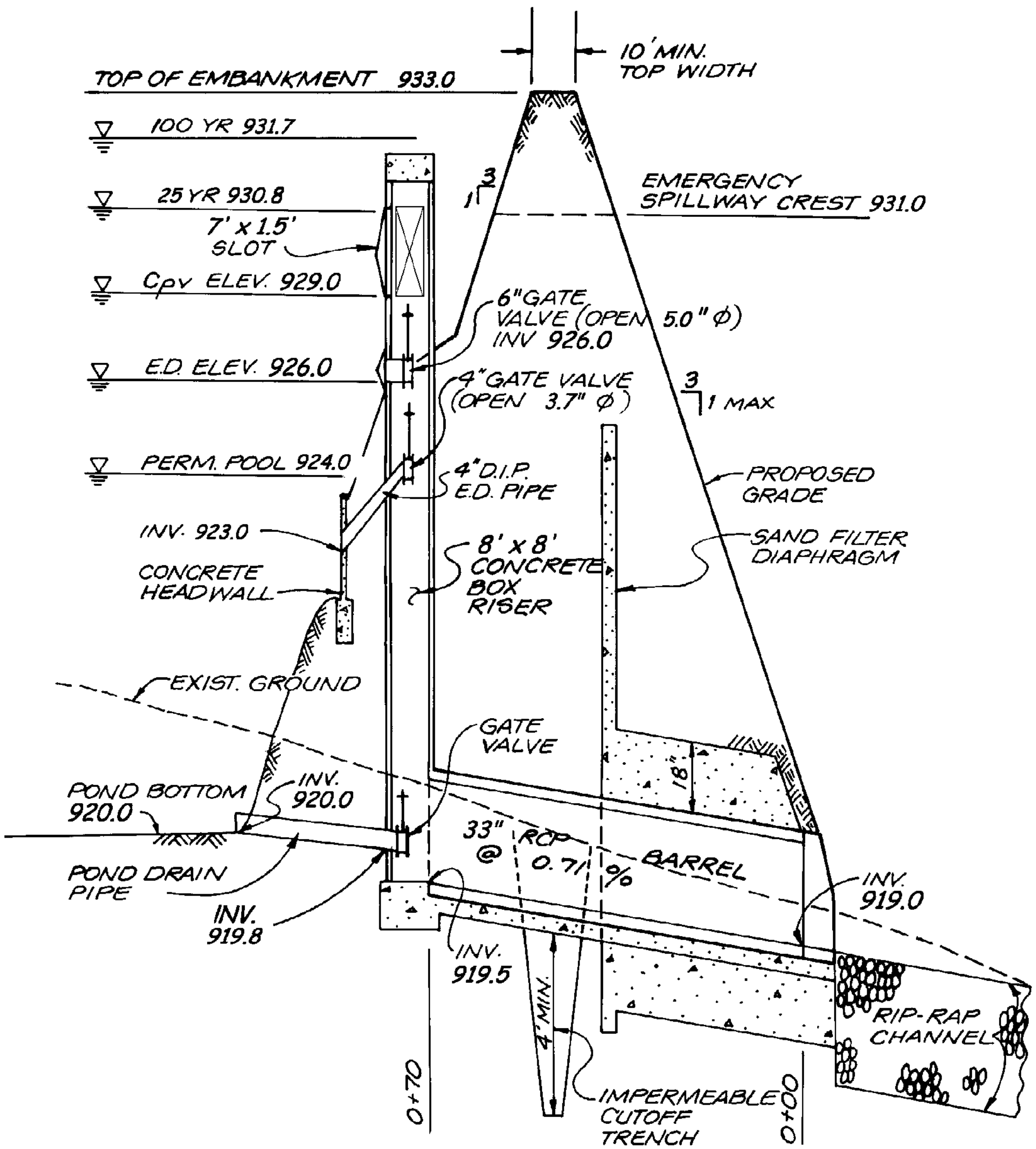



Figure 8. Profile of Principle Spillway

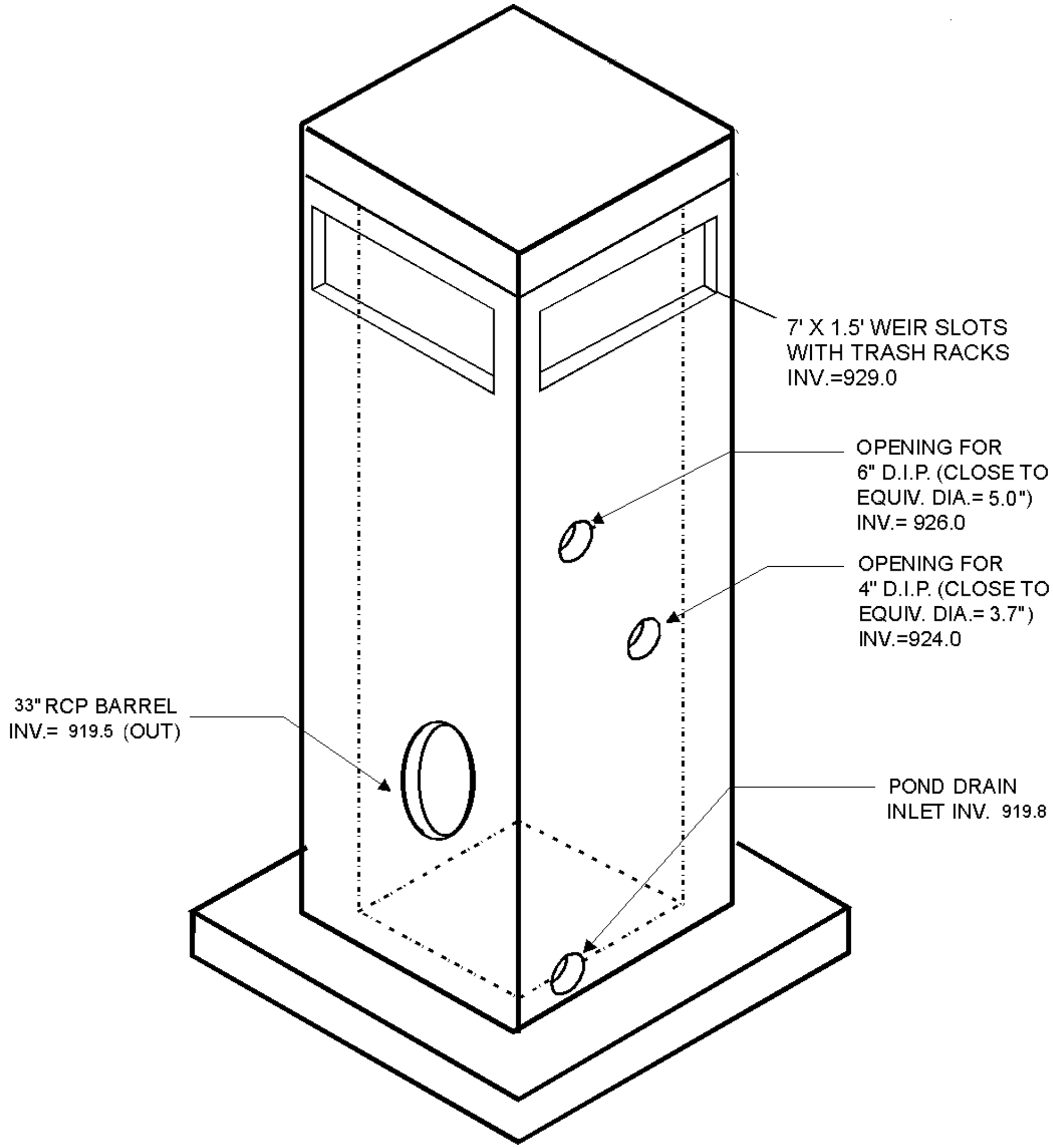
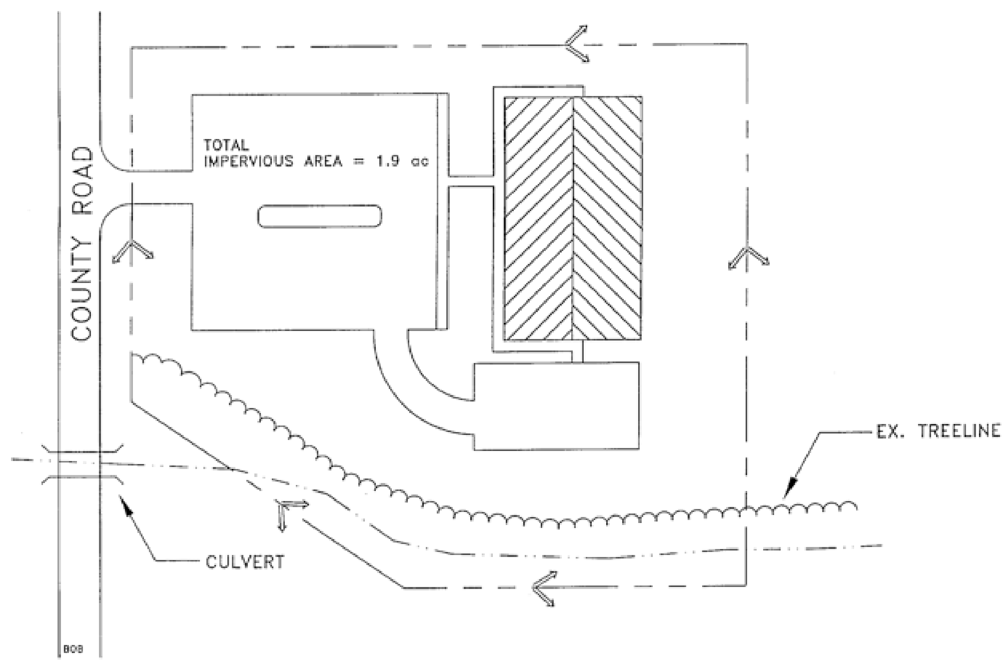



Figure 9. Schematic of Riser Detail

# Appendix B-2: Bioretention Area Design Example




**Base Data**

Site Area = Total Drainage Area (A) = 3.0 ac (130,680 $ft^2$)
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%
Soils Type "C"
1-yr, 24-hour Rainfall "P" = 3.4 inches
25-yr, 24-hour Rainfall "P" = 6.5 inches
100-yr, 24-hour Rainfall "P" = 7.9 inches

**Hydrologic Data**

| | **Pre** | **Post** |
|---|---|---|
| CN | 70 | 88 |
| $t_c$ | .39 | .20 |

Figure 1. Etowah Recreation Center Site Plan

This example focuses on the design of a bioretention facility to meet the water quality requirements of the site. Channel protection and overbank flood control are not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of bioretention is to provide water quality through runoff reduction or treatment (TSS removal) and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility or pass through the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). Under some conditions, channel protection storage can be provided by bioretention facilities. Due to the Class C soils identified on-site, an underdrain will be provided in this practice that will provide a runoff reduction volume credit of 50%.

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges
The layout of the Etowah Recreation Center is shown in Figure 1. This example assumes that the local community has adopted the unified stormwater sizing criteria requirements.

### STEP 1 – CONFIRM LOCAL DESIGN REQUIREMENTS AND DETERMINE THE GOAL AND FUNCTION OF THE BMP

It was determined by the local municipality that this best management practice would be designed by the runoff reduction volume calculation approach (described as Step 3 in Section 4.2)

The total designed volume of the practice must be provided to retain or remove the stormwater volume associated with the 1.0 inch storm event (this examples target runoff reduction). Runoff reduction credit will then be utilized by the designer through an adjusted curve number calculations.

### STEP 2 - DETERMINE IF THE DEVELOPMENT SITE AND CONDITIONS ARE APPROPRIATE FOR THE USE OF A BIORETENTION AREA.

Existing ground elevation at the facility location is 922.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 913.0 feet and underlying soil is silt loam (ML). Adjacent creek invert is at 912.0 feet.

### STEP 3 - COMPUTE RUNOFF CONTROL TARGET VOLUMES FROM THE UNIFIED STORMWATER SIZING CRITERIA (EITHER METHOD 3 OR 4 AS DESCRIBED IN THE DESIGN STEPS)

Method 3) Compute Runoff Reduction Volume ($RR_V$):

- Compute Runoff Coefficient, Rv

$$R_V = 0.05 + (63.3)(0.009) = 0.62$$

- Compute the target Runoff Reduction Volume ($RR_V$):

$$\begin{aligned} RR_V &= (1.0")(R_V)(A) / 12 \\ &= (1.0")(0.62)(130{,}680\ f^{t2})(1ft/12in) \\ &= \underline{6{,}752}\ ft^3 \end{aligned}$$

- Using the $RR_V$ calculated above, Compute the minimum volume of the practice ($VP_{MIN}$):

$$\begin{aligned} (VP_{MIN}) &\geq RR_V\,(target) / (RR\%) \\ &= (6{,}752\ ft^3) / (50\%) \\ &= \underline{13{,}504}\ ft^3 \end{aligned}$$

Note: The Volume Provided (VP) of this practice must be a minimum of 13,504 ft³.

- Calculate the amount of Runoff Reduction ($RR_V$ provided) by the bioretention area with an underdrain. This information will be needed in the adjusted curve number calculation, Compute the $RR_V$ provided:

$$\begin{aligned} RR_V\,(provided) &= (RR\%)(VP) \\ &= (50\%)(13{,}504\ ft^3)\ * \\ &= 6{,}752\ ft^3\ * \end{aligned}$$

BACK TO TOC

**Compute Stream Channel Protection Volume ($CP_v$):**
For stream channel protection, provide 24 hours of extended detention for the 1-year event.

In order to determine a preliminary estimate of storage volume for channel protection and overbank flood control, it will be necessary to perform hydrologic calculations using approved methodologies. This example uses the NRCS TR-55 methodology presented in Section 3.1.5 to determine pre- and post-development peak discharges for the 1-yr, 25-yr, and 100-yr 24-hour return frequency storms.

- Hydrologic output based on the given information provided

| Condition | CN | | $Q_{1-year}$ inches | $Q_{1-year}$ cfs | $Q_{25-year}$ inches | $Q_{25-year}$ cfs | $Q_{100-year}$ inches | $Q_{100-year}$ cfs |
|---|---|---|---|---|---|---|---|---|
| Pre-Developed | 70 | | 0.95 | 2.6 | 3.21 | 9.8 | 4.38 | 13.5 |
| Post-Developed | 88 | | 2.18 | 8.2 | 5.12 | 18.5 | 6.48 | 23.2 |
| * Post-Developed (Adjusted CN for $RR_v$) | 1-yr<br>25-yr<br>100-yr | 80.1<br>82.4<br>82.8 | 1.56 | 5.9 | 4.50 | 16.7 | 5.86 | 21.5 |

- *Adjusted Curve Number Procedure for Peak Flow Reduction of $CP_v$ (Section 3.1.7.5)

Given Q = 2.18 in. and P = 3.4 in., Find "R" and "S" to back calculate an adjusted CN

$$Q - R = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$ Modified Equation 3.1.5)

Retention storage (expressed in inches) for this basin is calculated by the following formula:

R = $RR_v$ (provided) / Basin Area
= (6,752 $ft^3$) / A
= 6,752 $ft^3$ / 130,680 $ft^2$ (12 in / 1 ft) = **0.62 inches**

$$1.56 = \frac{(3.4 - 0.2S)^2}{(3.4 + 0.8S)}$$

Solve for "S" to back calculate CN: S = 2.49

S = 1000/CN-10: therefore, CN = 80.1

- Utilize modified TR-55 approach to compute channel protection storage volume (based on adjusted CN value calculated above)

Initial abstraction (Ia) for CN of 80.1 is 0.5: [Ia = (200/CN - 2)]

Ia/P = (0.50)/ 3.4 inches = 0.15
$T_c$ = 0.20 hours
$q_u$ = 770 csm/in (From Figure 3.1.5-6)

Knowing $q_u$ and T (extended detention time), find $q_o/q_i$ for a Type II rainfall distribution.

Peak outflow discharge/peak inflow discharge ($q_o/q_i$) = 0.023
(From Figure 3.3.5-1)

For a Type II rainfall distribution,
$Vs/Vr = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$
(Equation 3.3.9)

Where Vs equals channel protection storage ($CP_v$) and Vr equals the volume of runoff in inches.

Vs/Vr = 0.65 Using Equation 3.3.10, calculate $V_S$...

Therefore, Vs = $CP_v$ = 0.65 (1.56 in) (3 ac) (43,560 $ft^2$ / ac) (1 ft / 12 in)
= 11,043 $ft^3$

Determine Overbank Flood Protection Volume (Qp25) based on the adjusted CN:
For a $Q_{in}$ of 16.7 cfs, and an allowable $Q_{out}$ of 9.8 cfs, the Vs necessary for 25-year control is 10,942 $ft^3$, under an adjusted post-developed CN of 82.4. Note that 6.5 inches of rainfall occurs during this event.

Analyze for Safe Passage of 100 Year Design Storm ($Q_f$):
At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure, typically the same one used for the overbank flood protection control.

**Table 1** Summary of General Design Information for Etowah Recreation Center

| Symbol | Control Volume | Volume Required (cubic feet) | Notes |
|---|---|---|---|
| $RR_V$ | Runoff Reduction | 6,752 | 6,752 cf provided (100%) |
| $WQ_V$ | Water Quality | 8,102 | Not used in this example |
| $CP_V$ | Channel Protection | 11,043 | |
| $Q_{p25}$ | Overbank Flood Protection | 10,942 | |
| $Q_f$ | Extreme Flood Protection | N/A | Provide safe passage for the 100-year event in final design |

### STEP 4 - COMPUTE $WQ_V$ PEAK DISCHARGE ($Q_{WQ}$)

### STEP 5 - SIZE FLOW DIVERSION STRUCTURE, IF NEEDED

Bioretention areas can be either on or off-line. On-line facilities are generally sized to receive, but not necessarily treat, the 25-year event. Off-line facilities are designed to receive a more or less exact flow rate through a weir, channel, manhole, "flow splitter", etc. This facility is situated to receive direct runoff from grass areas and parking lot curb openings and piping for the 25-year event (16.7 cfs – adjusted flow rate), and *no special flow diversion structure is incorporated.*

### STEP 6 - DETERMINE SIZE OF BIORETENTION PONDING / FILTER AREA BASED ON $VP_{MIN}$

$$VP_{MIN} = PV + (VES)(N)$$

Where: $VP_{MIN}$ = Volume Provided
(Calculated above, 13,504 $ft^3$)
PV = Ponding Volume (Ponding depth typically 9 inches)
VES = Volume of Engineered Soils
(Media depth typically 36 inches)
N = Porosity of engineered soils, typically 0.25

$$13{,}504\ ft^3 = (\text{Surface Area} \times 0.75\ ft) + (\text{Surface Area} \times 3\ ft \times 0.25)$$

$$13{,}504\ ft^3 = (\text{Surface Area} \times 1.5\ ft)$$

$$9{,}003\ ft^2 = \text{Surface Area}$$

### STEP 7 - SET DIMENSIONS OF FACILITY

Assume a roughly 2 to 1 rectangular shape. Given a filter area requirement of 9,003 sq ft, say facility is roughly 67' by 135' for constructability.

### STEP 8 - DESIGN CONVEYANCE TO FACILITY (OFF-LINE SYSTEMS)

This facility is not designed as an off-line system.

BACK TO TOC

**STEP 9 - SIZE UNDERDRAIN AREA**

Base underdrain design on 10% of the Surface Area or 900 sq ft. Using 6" perforated plastic pipes surrounded by a three-foot-wide gravel bed, 10' on center (o.c.). See Figure 2.

(900 sq ft)/3' per foot of underdrain = <u>300' of perforated underdrain</u>

**STEP 10 - DESIGN EMERGENCY OVERFLOW**

The parking area, curb and gutter is sized to convey the 25-year event to the facility. Should filtering rates become reduced due to facility age or poor maintenance, an overflow weir is provided to pass the 25-year event. Size this weir with 6" of head, using the weir equation.

$Q = CLH^{3/2}$

Where C = 2.65 (smooth crested grass weir)

Q = 16.7 cfs

H = 6"

Solve for L: $L = Q / [(C)(H^{3/2})]$ or $(16.7\text{ cfs}) / [(2.65)(.5^{1.5})] = 17.8'$ (say 18')

Outlet protection in the form of riprap or a plunge pool/stilling basin should be provided to ensure non-erosive velocities.

**STEP 11 - PREPARE VEGETATION AND LANDSCAPING PLAN**

Choose plants based on factors such as whether native or not, resistance to drought and inundation, cost, aesthetics, maintenance, etc. Select species locations (i.e., on center planting distances) so species will not "shade out" one another. Do not plant trees and shrubs with extensive root systems near pipe work. A potential plant list is presented in Appendix D.

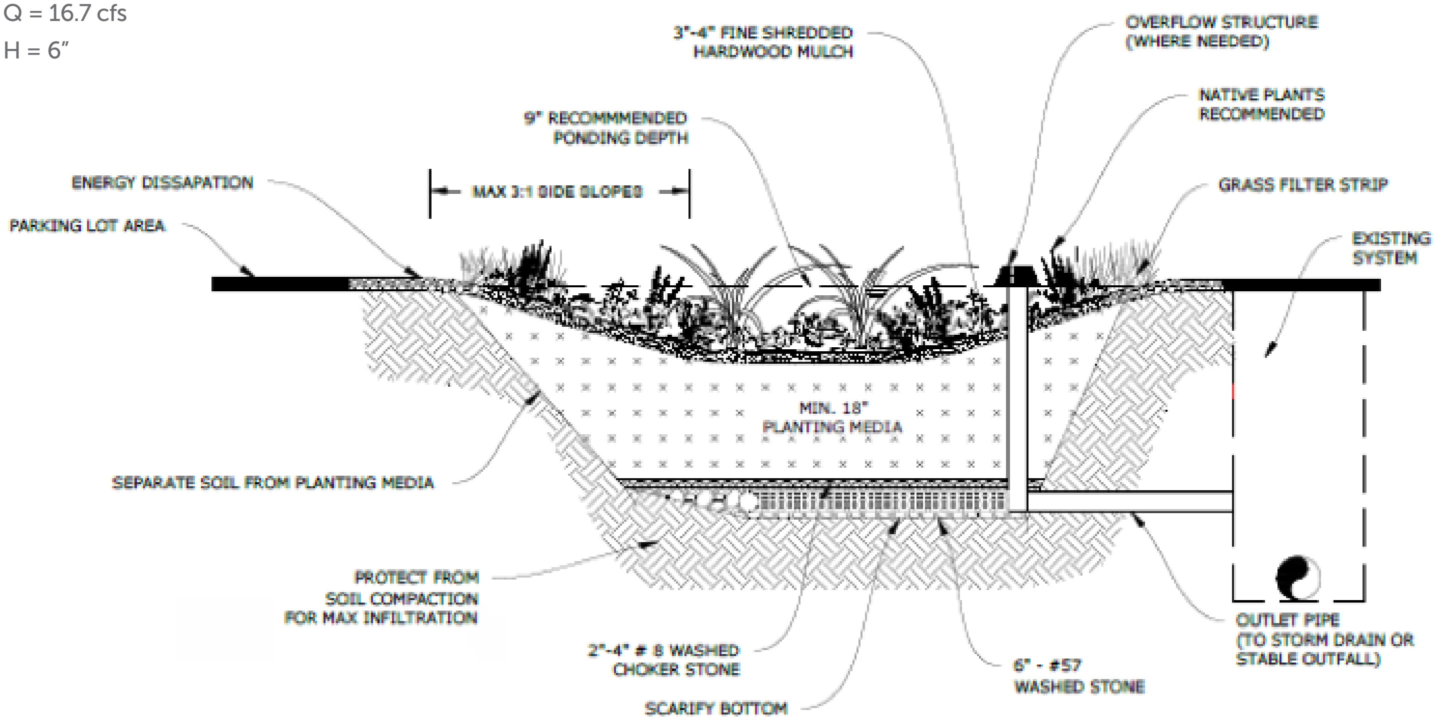
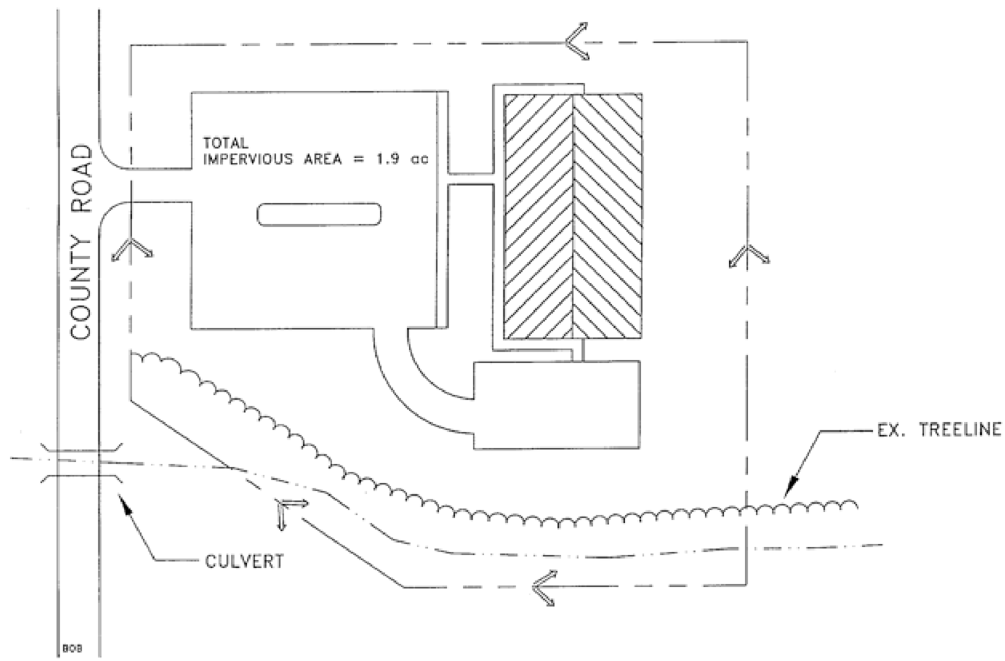



Figure 2. Typical Section of Bioretention Facility

BACK TO TOC

# Appendix B-3: Sand Filter Design Example




**Base Data**

Site Area = Total Drainage Area (A) = 3.0 ac (130,680 $ft^2$)
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%
Soils Type “B”
1-yr, 24-hour Rainfall “P” = 3.6 inches
25-yr, 24-hour Rainfall “P” = 7.9 inches
100-yr, 24-hour Rainfall “P” = 9.8 inches

**Hydrologic Data**

| | Pre | Post |
|---|---|---|
| CN | 57 | 83 |
| $t_c$ | .36 | .15 |

Type III Rainfall Dist.

Figure 1. Georgia Pines Community Center Site Plan

BACK TO TOC

This example focuses on the design of a surface sand filter to meet the water quality treatment (TSS removal) requirements of the site. Channel protection and overbank flood control is not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of sand filters is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). Sand filters do not contribute to a runoff reduction goal.

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

The layout of the Georgia Pines Community Center is shown in Figure 1. This example assumes that the local community has adopted the unified stormwater sizing criteria requirements.

**STEP 1 - CONFIRM LOCAL DESIGN CRITERIA AND APPLICABILITY.**

There are no additional requirements for this site.

**STEP 2 - DETERMINE IF THE DEVELOPMENT SITE AND CONDITIONS ARE APPROPRIATE FOR THE USE OF A SURFACE SAND FILTER.**

Site Specific Data:

The site area and drainage area to the sand filter is 3.0 acres (3 ac < 10 ac max.) Existing ground elevation at the facility location is 22.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 13.0 feet. Adjacent creek invert is at 12.0.

**STEP 3 - COMPUTE RUNOFF CONTROL VOLUMES FROM THE UNIFIED STORMWATER SIZING CRITERIA**

Method 4) Compute Water Quality Volume ($WQ_v$):

- Compute Runoff Coefficient, Rv

$$R_v = 0.05 + (63.3)(0.009) = 0.62$$

- Compute $WQ_v$

$$\begin{aligned} WQ_v &= (1.2")(Rv)(A) / 12 \\ &= (1.2")(0.62)(130{,}680\ ft^2)(1ft/12in) \\ &= \underline{8{,}102}\ ft^3 \end{aligned}$$

Compute Stream Channel Protection Volume, ($CP_v$):

For stream channel protection, provide 24 hours of extended detention for the 1-year event.

In order to determine a preliminary estimate of storage volume for channel protection and overbank flood control, it will be necessary to perform hydrologic calculations using approved methodologies. This example uses the NRCS TR-55 methodology presented in Section 3.1.5 to determine pre- and post-development peak discharges for the 1-yr, 25-yr, and 100-yr 24-hour return frequency storms.

- Hydrologic output based on the given information provided

Note that any hydrologic models using NRCS TR-55 procedures, such as TR-20, HEC-HMS, or other software platforms, can be used to perform preliminary hydrologic calculations.

| Condition | CN | $Q_{1\text{-year}}$ *inches* | $Q_{1\text{-year}}$ *cfs* | $Q_{25\text{-year}}$ *cfs* | $Q_{100\text{-year}}$ *cfs* |
|---|---|---|---|---|---|
| Pre-Developed | 57 | 0.45 | 0.7 | 6.4 | 9.7 |
| Post-Developed | 83 | 1.94 | 5.0 | 14.6 | 18.9 |

- Utilize modified TR-55 approach to compute channel protection storage volume

    Initial abstraction (Ia) for CN of 83 is 0.41: (TR-55) [Ia = (200/CN - 2)]

    Ia/P = (0.41)/ 3.6 inches = 0.11
    $T_c$ = 0.15 hours
    $q_u$ = 605 csm/in (From Figure 3.1.5-7)

Knowing $q_u$ and T (extended detention time), find $q_o/q_i$ for a Type III rainfall distribution.

Peak outflow discharge/peak inflow discharge ($q_o/q_i$) = 0.03 (From Figure 3.3.5-1)

$Vs/Vr = 0.683 - 1.43(q_o/q_i) + 1.64(qo/qi)^2 - 0.804(q_o/q_i)^3$ (Equation 3.3.9)

Where Vs equals channel protection storage ($CP_v$) and Vr equals the volume of runoff in inches.

Vs/Vr = 0.64 Using Equation 3.3.10, calculate $V_S$...

Therefore, Vs = $CP_v$ = 0.64(1.94")(1/12)(3 ac) (43,560 ft$^2$ / ac) = 13,521 ft$^3$

- Define the average ED Release Rate

    The above volume, 13,521 ft$^3$, is to be released over 24 hours.
    (13,521 ft$^3$) / (24 hrs x 3,600 sec/hr) = 0.16 cfs

Determine Overbank Flood Protection Volume ($Q_{p25}$):
For a $Q_{in}$ of 14.6 cfs, and an allowable $Q_{out}$ of 6.4 cfs, the Vs necessary for 25-year control is 40,981 ft$^3$, under a developed CN of 83. Note that 7.9 inches of rain fall during this event.

Analyze for Safe Passage of 100 Year Design Storm ($Q_f$):
At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure, typically the same one used for the overbank flood protection control.

**Table 1** Summary of General Design Information for Georgia Pines Community Center

| Symbol | Control Volume | Volume Required (cubic feet) | Notes |
|---|---|---|---|
| $WQ_V$ | Water Quality | 8,102 | |
| $CP_V$ | Channel Protection | 13,521 | |
| $Q_{p25}$ | Overbank Flood Protection | 40,981 | |
| $Q_f$ | Extreme Flood Protection | N/A | Provide safe passage for the 100-year event in final design |

**STEP 4 - COMPUTE $WQ_V$ PEAK DISCHARGE ($Q_{WQ}$) & HEAD**

- Water Quality Volume:

    $WQ_v$ previously determined to be 8,102 cubic feet.

- Determine available head (See Figure 3)

Low point at parking lot is 23.5. Subtract 2' to pass Q25 discharge (21.5) and a half foot for channel to facility (21.0). Low point at stream invert is 12.0. Set outfall underdrain pipe 2' above stream invert and add 0.5' to this value for drain (14.5). Add to this value 8" for the gravel blanket over the underdrains, and 18" for the sand bed (16.67). The total available head is 21.0 - 16.67 or 4.33 feet. Therefore, the average depth, hf, is (hf) = 4.33' / 2, and hf = 2.17'.

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as sand filters and grass channels. Conventional NRCS TR-55 methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2". This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the filtering treatment practice due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

The following equation can be used to estimate peak discharges for small storm events.

- Using the water quality volume ($WQ_v$), a corresponding Curve Number (CN) is computed utilizing the following equation (See Section 3.1.7.2):

$$CN = 1000/[10 + 5P + 10Q_{WV} - 10(Q_{WV}^{\ 2} + 1.25\ Q_{WV}\ P)^{½}]$$

where P = rainfall, in inches (use 1.2" for the Water Quality Storm)
and QWV = runoff volume, in inches (equal to 1.2 x $R_V$ = 0.744)

- Once a CN is computed (CN = 95), the time of concentration ($t_c$) is computed
- Using the computed CN, $t_c$ and drainage area (A), in acres; the peak discharge ($Q_{wq}$) for the Water Quality Storm is computed (based on the procedures identified in Section 3.1.5 (Type III for this example).

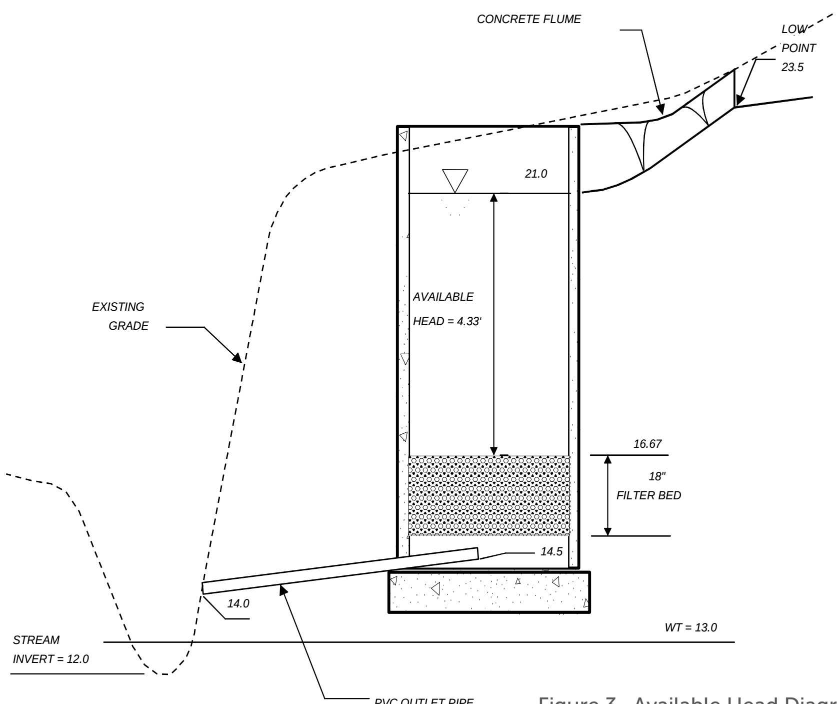



Figure 3. Available Head Diagram

- Read initial abstraction ($I_a$), compute $I_a/P$ ($I_a/P$ = 0.09)
- Read the unit peak discharge ($q_u$) for appropriate $t_c$ ($q_u$ = 625)
- Using the water quality volume ($WQ_v$), compute the water quality peak discharge ($Q_{wq}$)

$$Q_{wq} = q_u * A * WQ_v$$

where $Q_{wq}$ = the peak discharge, in cfs
$q_u$ = the unit peak discharge, in cfs/mi²/inch
A = drainage area, in square miles
$WQ_v$ = Water Quality Volume, in watershed inches

For this example, the steps are as follows:
Compute modified CN for 1.2" rainfall

P = 1.2"
Q = 1.2 $R_v$ = 1.2 * 0.62 = 0.74"
CN = $1000/[10+5P+10Q-10(Q^2+1.25*Q*P)^{½}]$
= $1000/[10+5*1.2+10*0.74-10(0.74^2+1.25*0.74*1.2)^{½}]$
= 95.01
<u>Use CN = 95</u>

For CN = 95 and the $T_c$ = 0.15 hours, compute the $Q_p$ for a 1.2" storm. With the CN = 95, a 1.2" storm will produce 0.74" of runoff. $I_a$ = 0.105, therefore Ia/P = 0.105/1.2 = 0.088. From Section 3.1.5.7 $q_u$ = 625 csm/in, and therefore $Q_{wq}$ = (625 csm/in) (3.0 ac/640ac/sq mi.) (0.74") = 2.2 cfs.

**STEP 5 - SIZE FLOW DIVERSION STRUCTURE (SEE FIGURE 4):**

Size a low flow orifice to pass 2.2 cfs with 1.5' of head using the Orifice equation.

$Q = CA(2gh)^{1/2}$ ; $2.2 \text{ cfs} = (0.6)(A)[(2)(32.2 \text{ ft/s}^2)(1.5')]^{1/2}$

$A = 0.37$ sq ft $= \pi d^2/4$: d = 0.7' or 8.5"; use 9 inches

Size the 25-year overflow as follows: the 25-year wsel is set at 23.0. Use a concrete weir to pass the 25-year flow (14.6 cfs) into a grassed overflow channel using the Weir equation. Assume 2' of head to pass this event. Overflow channel should be designed to provide sufficient energy dissipation (e.g., riprap, plunge pool, etc.) so that there will be non-erosive velocities.

$Q = CLH^{3/2}$

$14.6 = 3.1\,(L)\,(2')^{1.5}$

L = 1.66'; use L = 2'-0" which sets flow diversion chamber dimension.

Weir wall elev. = 21.0. Set low flow invert at 21.0 - [1.5' + (0.5*9"*1ft/12")] = 19.13.

**STEP 6 - SIZE FILTRATION BED CHAMBER (SEE FIGURE 6):**

From Darcy's Law: $A_f = WQ_v\,(d_f) / [k\,(h_f + d_f)\,(t_f)]$

where $d_f$ = 18"

k = 3.5 ft/day

$h_f$ = 2.17'

$t_f$ = 40 hours

$A_f$ = (8,102 cubic feet) (1.5') / [3.5 (2.17' + 1.5') (40hr/24hr/day)]

$A_f$ = 567.7 sq ft; using a 2:1 ratio, say filter is 17' by 34' (= 578 sq ft)

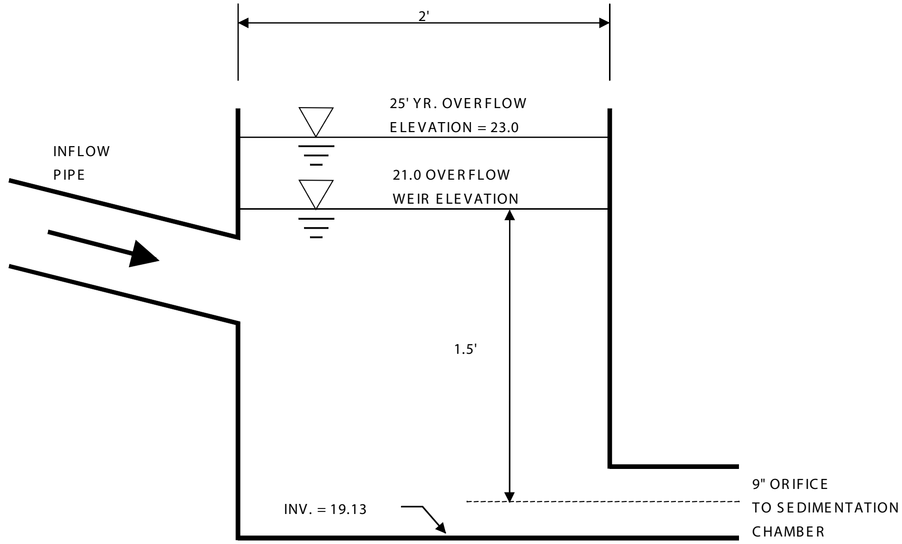



Figure 4. Flow Diversion Structure

**STEP 7- SIZE SEDIMENTATION CHAMBER**

From Camp-Hazen equation, for I < 75%: $A_s = 0.066\,(WQ_v)$

$A_s$ = 0.066 (8,102 cubic ft) or 535 sq ft

given a width of 17 feet, the length will be 535'/17' or 31.5 feet (use 17'x32')

**STEP 8 - COMPUTE $V_{MIN}$**

$V_{min}$ = 0.75 ($WQ_v$) or 0.75 (8,102 cubic feet) = 6,077 cubic feet

**STEP 9. COMPUTE STORAGE VOLUMES WITHIN ENTIRE FACILITY AND SEDIMENTATION CHAMBER ORIFICE SIZE:**

Volume within filter bed ($V_f$): $V_f = A_f\,(d_f)\,(n)$; n = 0.4 for sand

$V_f$ = (578 sq ft) (1.5') (0.4) = 347 cubic feet

Temporary storage above filter bed (Vf-temp): $V_{f\text{-temp}} = 2h_fA_f$

$V_{f\text{-temp}}$ = 2 (2.17') (578 sq ft) = 2,509 cubic feet

Compute remaining volume for sedimentation chamber ($V_s$):

$V_s = V_{min} - [V_f + V_{f\text{-temp}}]$ or 6,077 - [347 + 2,509] = 3,221 cubic feet

BACK TO TOC

Compute height in sedimentation chamber ($h_s$): $h_s = V_s/A_s$

(3,221 cubic ft)/(17' x 32') = 5.9' which is larger than the head available (4.33'); increase the size of the settling chamber, using 4.33 as the design height;

(3,221 cubic ft)/4.33' = 744 sq ft; 744'/17' yields a length of 43.8 feet (say 44')

New sedimentation chamber dimensions are 17' by 44'
With adequate preparation of the bottom of the settling chamber (rototil earth, place gravel, then surge stone), the bottom can infiltrate water into the substrate. The runoff will enter the groundwater directly without treatment. The stone will eventually clog without protection from settling solids, so use a removable geotextile to facilitate maintenance. Note that there is 2.17' of freeboard between bottom of recharge filter and water table.

Provide perforated standpipe with orifice sized to release volume (within sedimentation basin) over a 24 hr period (see Figure 6). Average release rate equals 3,221 $ft^3$/24 hr = 0.04 cfs

Equivalent orifice size can be calculated using orifice equation:

$Q = CA(2gh)^{1/2}$ , where h is average head, or 4.33'/2 = 2.17'.
0.04 cfs = $0.6*A*(2*32.2 \text{ ft/s}^2*2.17 \text{ ft})^{1/2}$
A = 0.005 $ft^2$ = $\pi D^2/4$: therefore equivalent orifice diameter equals 1".

Recommended design is to cap stand pipe with low flow orifice sized for 24 hr detention. Over-perforate pipe by a safety factor of 10 to account for clogging. Note that the size and number of perforations will depend on the release rate needed to achieve 24 hr detention. A multiple orifice stage-discharge relation needs to be developed for the proposed perforation configuration. Stand pipe should discharge into a flow distribution chamber prior to filter bed. Distribution chamber should be between 2 and 4 feet in length and same width as filter bed. Flow distribution to the filter bed can be achieved either with a weir or multiple orifices at constant elevation. See Figure 7 for stand pipe details.

**STEP 10 -DESIGN INLETS, PRETREATMENT FACILITIES, UNDERDRAIN SYSTEM, AND OUTLET STRUCTURES**

**STEP 11 - COMPUTE OVERFLOW WEIR SIZES**

Assume overflow that needs to be handled is equivalent to the 9" orifice discharge under a head of 3.5 ft (i.e., the head in the diversion chamber associated with the 25-year peak discharge).

$Q = CA(2gh)^{1/2}$
$Q = 0.6(0.44 \text{ ft}^2)[(2)(32.2 \text{ ft/s}^2)(3.5 \text{ ft})]^{1/2}$
Q = 3.96 cfs, say 4.0 cfs

For the overflow from the sediment chamber to the filter bed, size to pass 4 cfs.

Weir equation: $Q = CLh^{3/2}$, assume a maximum allowable head of 0.5'
$4.0 = 3.1 * L * (0.5 \text{ ft})^{3/2}$
L = 3.65 ft, Use L = 3.75 ft.

Similarly, for the overflow from the filtration chamber to the outlet of the facility, size to pass 4.0 cfs.

Weir equation: $Q = CLh^{3/2}$, assume a maximum allowable head of 0.5'
$4.0 = 3.1 * L * (0.5 \text{ ft})^{3/2}$
L = 3.65 ft, Use L = 3.75 ft.

Adequate outlet protection and energy dissipation (e.g., riprap, plunge pool, etc.) should be provided for the downstream overflow channel.

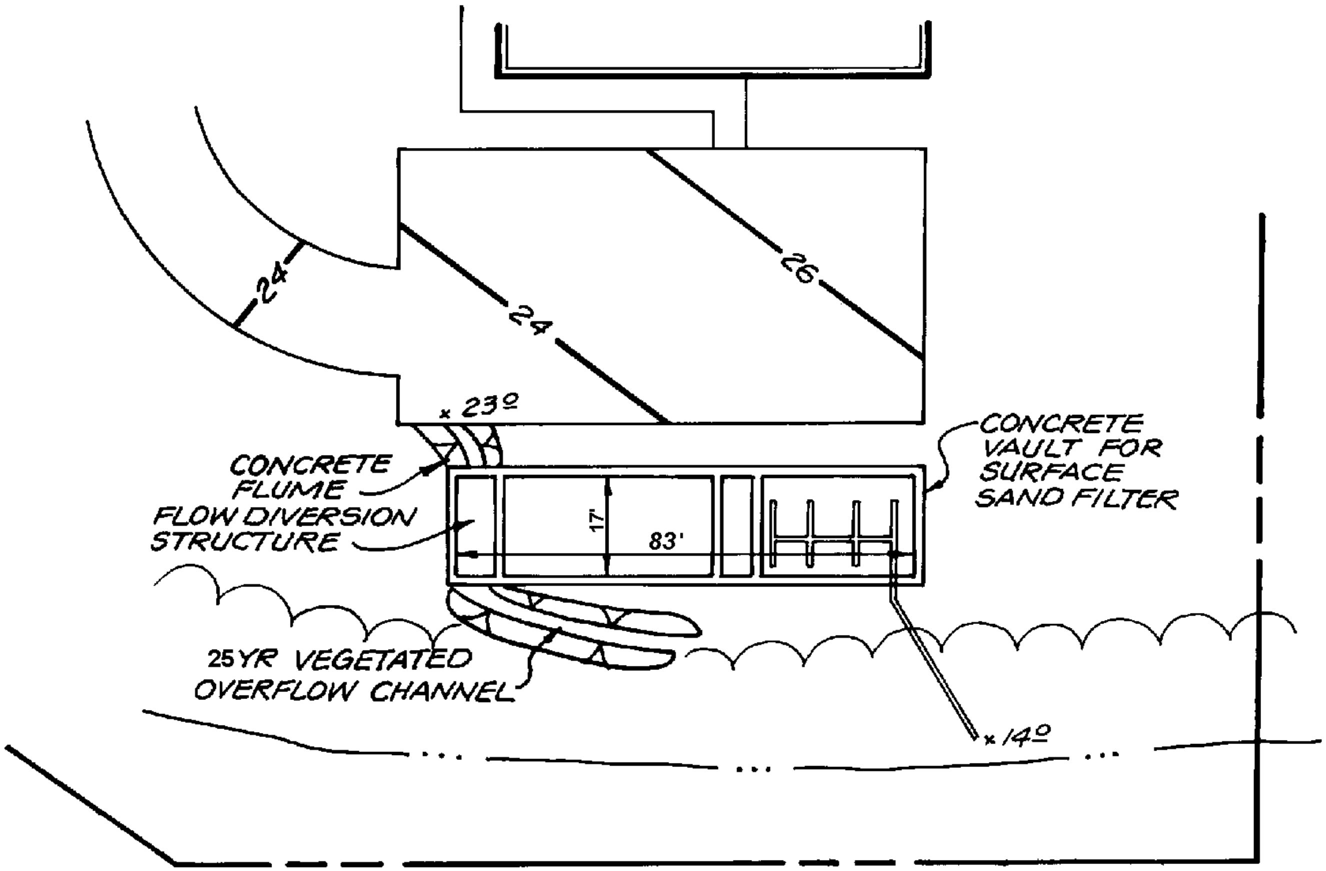
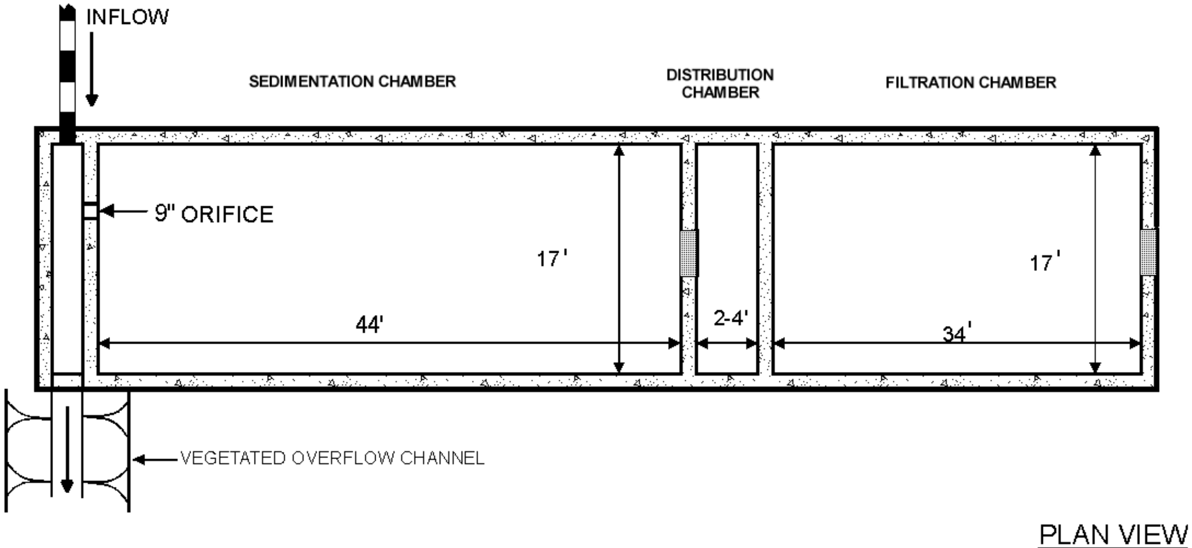



Figure 5. Surface Sand Filter Site Plan

BACK TO TOC



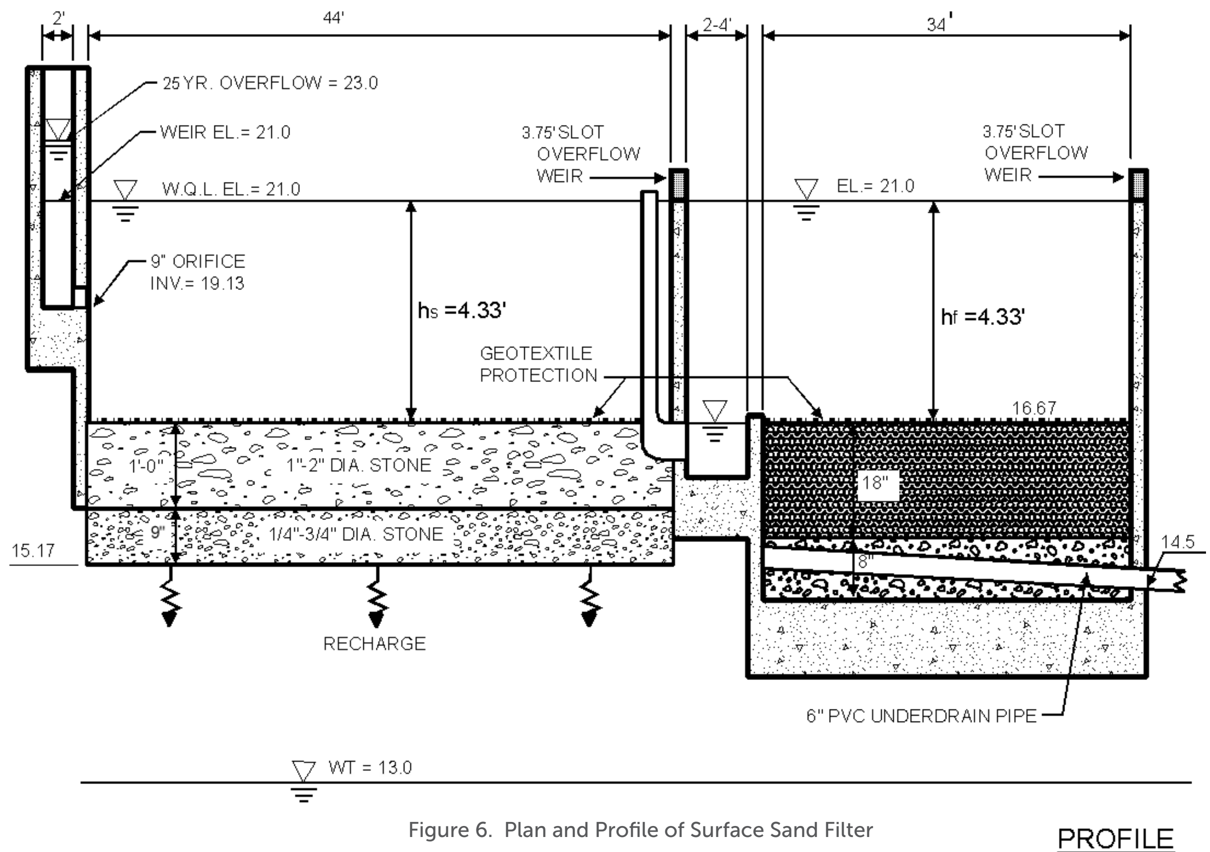
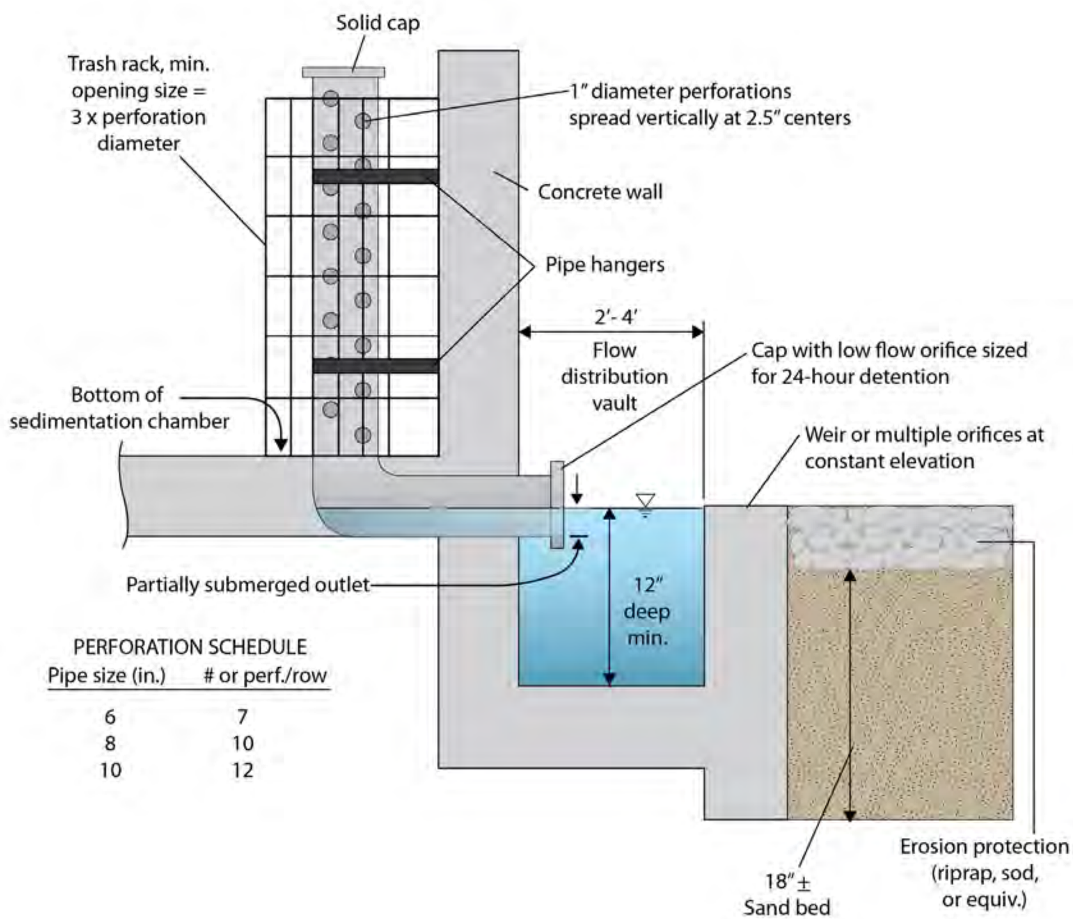




Figure 6. Plan and Profile of Surface Sand Filter



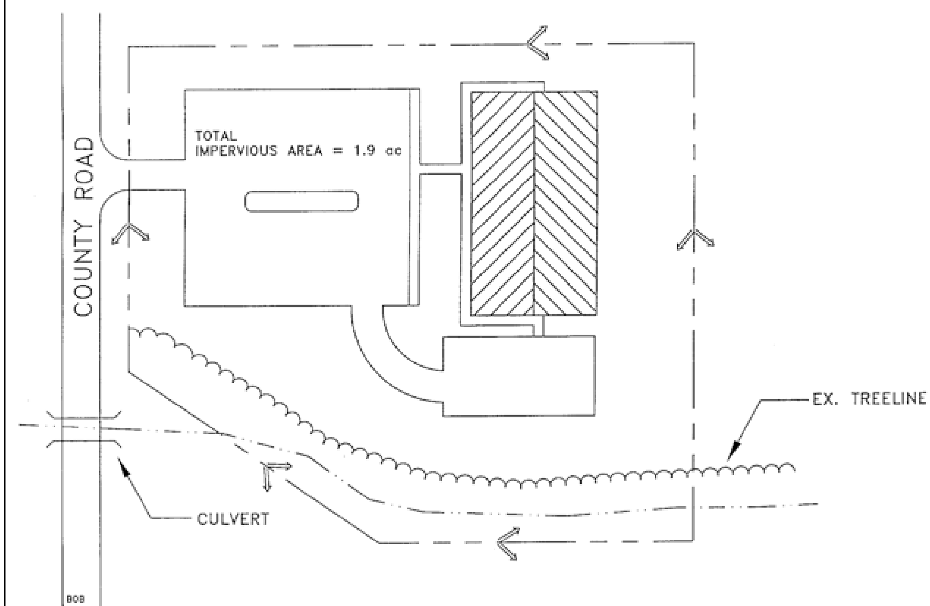

PERFORATION SCHEDULE

| Pipe size (in.) | # or perf./row |
|---|---|
| 6 | 7 |
| 8 | 10 |
| 10 | 12 |

Figure 7. Perforated Stand Pipe Detail

BACK TO TOC

# Appendix B-4: Infiltration Trench Design Example




Base Data

Site Area = Total Drainage Area (A) = 3.0 ac (130,680 ft$^2$)
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%
Soils Type “B”
1-yr, 24-hour Rainfall “P” = 3.6 inches
25-yr, 24-hour Rainfall “P” = 7.9 inches
100-yr, 24-hour Rainfall “P” = 9.8 inches

Hydrologic Data

| | **Pre** | **Post** |
|---|---|---|
| CN | 57 | 83 |
| $t_c$ | .36 | .15 |

Type III Rainfall Dist.

Figure 1. Georgia Pines Community Center Site Plan

BACK TO TOC

This example focuses on the design of an infiltration trench to meet the runoff reduction goals and water quality requirements of the site. Channel protection and overbank flood control is not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of infiltration trenches is to provide water quality treatment and/or runoff reduction and groundwater recharge, not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). The layout of the Georgia Pines Community Center is shown in Figure 1. Due to the Class B soils identified on-site, the infiltration trench will be designed for runoff reduction. The runoff reduction credit given for infiltration trenches is 100%.

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges
The layout of the Georgia Pines Community Center is shown in Figure 1.

### STEP 1 - CONFIRM LOCAL DESIGN CRITERIA AND APPLICABILITY

It was determined by the local municipality that this best management practice would be designed by the runoff reduction volume calculation approach (described as Step 3 in Section 4.12)

The total designed volume of the practice must be provided to retain or remove the stormwater volume associated with the 1.0 inch storm event (this examples target runoff reduction). Runoff reduction credit will then be utilized by the designer through an adjusted curve number calculations.

Table 1, below, summarizes the requirements that need to be met to successfully implement infiltration practices. On this site, infiltration is feasible, with restrictions on the depth and width of the trench.

**Table 1** Infiltration Feasibility

| Criteria | Status |
|---|---|
| Infiltration rate ($f_c$) greater than or equal to 0.5 inches/hour. | Infiltration rate is 1.0 inches/hour. OK. |
| Soils have a clay content of less than 20% and a silt/clay content of less than 40%. | Sandy Loam meets both criteria. |
| Infiltration cannot be located on slopes greater than 6% or in fill soils. | Slope is <1%; not fill soils. OK. |
| Hotspot runoff should not be infiltrated. | Not a hotspot land use. OK. |
| Infiltration is prohibited in karst topography. | Not in karst. OK. |
| The bottom of the infiltration facility must be separated by at least two feet vertically from the seasonally high water table. | Elevation of seasonally high water table: 13′<br>Elevation of BMP location: 20′.<br>The difference is 7′.<br>Thus, the trench can be up to 5′ deep. OK. |
| Infiltration facilities must be located 100 feet horizontally from any water supply well. | No water supply wells nearby. OK. |
| Maximum contributing area generally less than 2 or 5 acres. (Optional) | 3 acres. OK. |
| Setback 25 feet down-gradient from structures. | Fifty feet straight-line distance between the parking lot and the tree line. OK if the trench is 25′ wide or narrower. |

### STEP 2 - DETERMINE IF THE DEVELOPMENT SITE AND CONDITIONS ARE APPROPRIATE FOR THE USE OF AN INFILTRATION TRENCH

Site Specific Data:
Table 2 presents site-specific data, such as soil type, percolation rate, and slope, for consideration in the design of the infiltration trench.

**Table 2** Site Specific Data

| Criteria | Value |
|---|---|
| Soil | Sandy Loam |
| Percolation Rate | 1″/hour |
| Ground Elevation at BMP | 20′ |
| Seasonally High Water Table | 13′ |
| Stream Invert | 12′ |
| Soil slopes | <1% |

BACK TO TOC

**STEP 3 - COMPUTE RUNOFF CONTROL TARGET VOLUMES FROM THE UNIFIED STORMWATER SIZING CRITERIA (EITHER METHOD 3 OR 4 AS DESCRIBED IN THE DESIGN STEPS)**

Method 3) Compute Runoff Reduction Volume ($RR_V$):

- Compute Runoff Coefficient, $R_v$

  $R_V = 0.05 + (63.3)(0.009) = 0.62$

- Compute the target Runoff Reduction Volume ($RR_V$):

  $RR_V$ = (1.0") ($R_V$) (A) / 12
  = (1.0") (0.62) (130,680 $ft^2$) (1ft/12in)
  = 6,752 $ft^3$

- Using the $RR_V$ calculated above, Compute the minimum volume of the practice ($VP_{MIN}$):

  ($VP_{MIN}$) ≥ $RR_V$ (target) / (RR%)
  = (6,752 $ft^3$) / (100%)
  = 6,752 $ft^3$

- Calculate the amount of Runoff Reduction ($RR_V$ provided) by the infiltration practice. This information will be needed in the adjusted curve number calculation, Compute the $RR_V$ provided:

  $RR_V$ (provided) = (RR%) (VP)
  = (100%) (6,752 $ft^3$)*
  = 6,752 $ft^3$ *

Compute Stream Channel Protection Volume ($CP_v$):

For stream channel protection, provide 24 hours of extended detention for the 1-year event.

In order to determine a preliminary estimate of storage volume for channel protection and overbank flood control, it will be necessary to perform hydrologic calculations using approved methodologies. This example uses the NRCS TR-55 methodology presented in Section 3.1.5 to determine pre- and post-development peak discharges for the 1-yr, 25-yr, and 100-yr 24-hour return frequency storms.

- Hydrologic output based on the given information provided

| Condition | CN | $Q_{1\text{-year}}$ inches | $Q_{1\text{-year}}$ cfs | $Q_{25\text{-year}}$ inches | $Q_{25\text{-year}}$ cfs | $Q_{100\text{-year}}$ inches | $Q_{100\text{-year}}$ cfs |
|---|---|---|---|---|---|---|---|
| Pre-Developed | 57 | 0.45 | 0.7 | 2.93 | 6.4 | 4.34 | 9.7 |
| Post-Developed | 83 | 1.94 | 5.0 | 5.88 | 14.6 | 7.71 | 18.9 |
| * Post-Developed (Adjusted CN for $RR_V$) | 1-yr 74.2<br>25-yr 78<br>100-yr 78 | 1.32 | 3.3 | 5.26 | 13.4 | 7.09 | 17.7 |

- *Adjusted Curve Number Procedure for Peak Flow Reduction of $CP_v$ (Section 3.1.7.5)

  Given Q = 1.94 in. and P = 3.6 in., Find "R" and "S" to back calculate an adjusted CN

$$Q - R = \frac{(P - 0.2S)^2}{(P + 0.8S)} \quad \text{(Modified Equation 3.1.5)}$$

Retention storage (expressed in inches) for this basin is calculated by the following formula:

R = $RR_V$ (provided) / Basin Area
= (6,752 $ft^3$) / A
= 6,752 $ft^3$ / 130,680 $ft^2$ (12 in / 1 ft) = **0.62 inches**

$$1.32 = \frac{(3.6 - 0.2S)^2}{(3.6 + 0.8S)}$$

Solve for "S" to back calculate CN: S = 3.48

S = 1000/CN-10: therefore, CN = 74.2

- Utilize modified TR-55 approach to compute channel protection storage volume (based on adjusted CN value calculated above)

Initial abstraction (Ia) for CN of 74.2 is 0.7: [Ia = (200/CN - 2)]

Ia/P = (0.70)/ 3.6 inches = 0.19
$T_c$ = 0.15 hours
$q_u$ = 575 csm/in (From Figure 3.1.5-7)

Knowing $q_u$ and T (extended detention time), find $q_o/q_i$ for a Type II rainfall distribution.

Peak outflow discharge/peak inflow discharge ($q_o/q_i$) = 0.035 (From Figure 3.3.5-1)

For a Type III rainfall distribution,

$Vs/Vr = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$ (Equation 3.3.9)

Where Vs equals channel protection storage ($CP_v$) and Vr equals the volume of runoff in inches.

Vs/Vr = 0.63 Using Equation 3.3.10, calculate $V_S$...

Therefore, Vs = $CP_v$ = 0.63 (1.32 in) (3 ac) (43,560 ft² / ac) (1 ft / 12 in = 9,056 ft³

**Determine Overbank Flood Protection Volume ($Q_{p25}$) based on the adjusted CN:**
For a $Q_{in}$ of 13.4 cfs, and an allowable $Q_{out}$ of 6.4 cfs, the Vs necessary for 25-year control is 22,175 ft³, under an adjusted developed CN of 78. Note that 7.9 inches of rain fall during this event.

**Analyze for Safe Passage of 100 Year Design Storm ($Q_f$):**
At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure, typically the same one used for the overbank flood protection control.

**Table 3** Summary of General Design Information for Georgia Pines Community Center

| Symbol | Control Volume | Volume Required (cubic feet) | Notes |
|---|---|---|---|
| $RR_V$ | Runoff Reduction | 6,752 | 100% provided |
| $WQ_V$ | Water Quality | 8,102 | Not required in this example |
| $CP_V$ | Channel Protection | 9,056 | |
| $Q_{p25}$ | Overbank Flood Protection | 22,175 | |
| $Q_f$ | Extreme Flood Protection | N/A | Provide safe passage for the 100-year event in final design |

### STEP 4 - COMPUTE $WQ_V$ PEAK DISCHARGE ($Q_{WQ}$)

### STEP 5 - SIZE FLOW DIVERSION STRUCTURE, IF NEEDED

Infiltration areas can be either on or off-line. On-line facilities are generally sized to receive, but not necessarily treat, the 25-year event. Off-line facilities are designed to receive a more or less exact flow rate through a weir, channel, man-hole, "flow splitter", etc. This facility is situated to receive direct runoff from grass areas and parking lot curb openings and piping for the 25-year event (14.6 cfs – unadjusted flow rate), and *no special flow diversion structure is incorporated.*

### STEP 6 - SIZE THE INFILTRATION TRENCH BASED ON $V_{PMIN}$

$VP_{MIN} = PV + (V_G)(N)$

Where: $VP_{MIN}$ = Volume Provided (Calculated above, 6,752 ft³)
PV = Ponding Volume (Ponding depth typically 9 inches)
$V_G$ = Volume of Gravel (depth typically 2-10 feet)
N = Porosity of gravel, typically 0.4

Note: Assume a maximum depth value of 5 feet, and work through iterations as needed to fit the practice to the available space within the site. Recall that Table 1 indicates the practice cannot exceed 5 feet due to the proximity of the water table.

6,752 ft³ = (Surface Area x 0.75 ft) + (Surface Area x 5 ft x 0.4)
6,752 ft³ = (Surface Area x 2.75 ft)
2,456 ft² = Surface Area

BACK TO TOC

### STEP 7 - SET DIMENSIONS OF FACILITY

Since the width can be no greater than 25′ (see Table 1; feasibility), determine the length:

$L = 2{,}456\ ft^2 / 25\ ft$

$L = 98.25$ feet

Say this facility is roughly 25′ wide x 98.25′ long x 5′ deep

### STEP 8 - SIZE PRETREATMENT VOLUME AND DESIGN PRETREATMENT MEASURES

As rule of thumb, size pretreatment to treat 10% of the $WQ_v$. Therefore, treat 8,102 x 0.10 = 810.2 $ft^3$.

For pretreatment, use a pea gravel filter layer with filter fabric, a plunge pool and a grass channel.

Pea Gravel Filter

The pea gravel filter layer covers the entire trench with 2" (see Figure 2). Assuming a porosity of 0.32, the water quality treatment in the pea gravel filter layer is:

$WQ_{filter} = (0.32)(2")(1\ ft/12\ inches)(2{,}456\ ft^2) = 131\ ft^3$

Plunge Pools

Use a 10′X10′ plunge pool with average depths of 2′.

Total $WQ_{pool} = (10' \times 10')(2') = 200\ ft^3$

Grass Channel

Thus, the grass channel needs to treat at least (810.2 – 131 - 200) $ft^3$ = 479.2 $ft^3$

Use a Manning's equation nomograph or software to size the swale.

The channel should treat 479.2 $ft^3$

- Assume a trapezoidal channel with 10′ channel bottom, 5H:1V side slopes, and a Manning's n value of 0.30. Use a nomograph to size the swale; assume a 0.5% slope.
- Use a peak discharge of 1.71 cfs (Peak flow for WQv, or 8102 ft3)
- Compute velocity: V=0.99 ft/s
- To retain the RRv (6,752 $ft^3$ ) for 5 minutes, the length would be 300 feet.
- Since the swale needs to treat the 10% of the water quality volume minus the treatment provided by the plunge pool and the gravel layer, or 479.2 $ft^3$, the length should be pro-rated to reflect this reduction.

Therefore, adjust length:

$L = (300\ ft)(479.2\ ft^3/8{,}102\ ft^3) = 17.7$ feet. Use 20 feet.

### STEP 8 - DESIGN SPILLWAY(S)

Adequate stormwater outfalls should be provided for the overflow associated with the 25-year and larger design storm events, ensuring non-erosive velocities on the down slope.

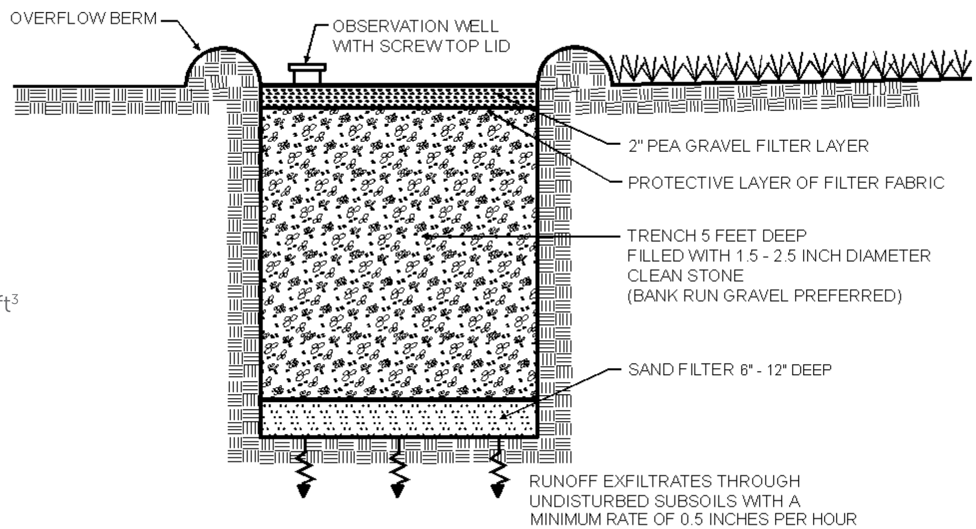
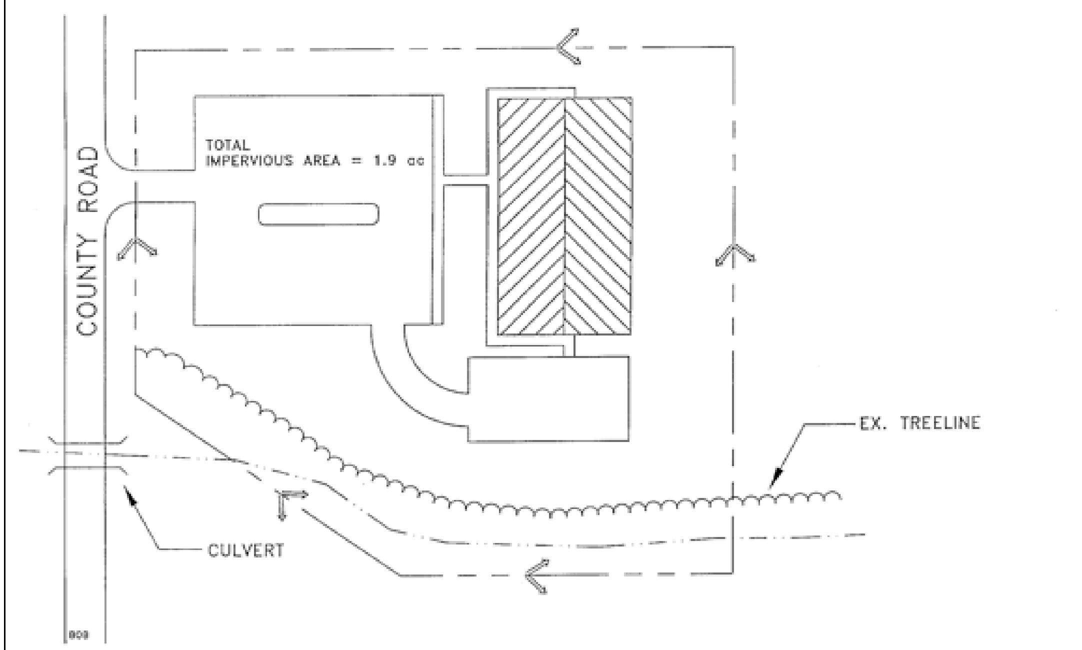



Figure 2. Infiltration Trench Cross Section

# Appendix B-5: Enhanced Swale Design Example




| Base Data | Hydrologic Data | | |
|---|---|---|---|
| Site Area = Total Drainage Area (A) = 3.0 ac (130,680ft$^2$) | | Pre | Post |
| Impervious Area = 1.9 ac; or 1 = 1.9/3.0 = 63.3% | CN | 70 | 88 |
| Soils Type "C" | $t_c$ | .39 | .20 |
| 1-yr, 24-hour Rainfall "P" = 3.4 inches | | | |
| 25-yr, 24-hour Rainfall "P" = 6.5 inches | | | |
| 100-yr, 24-hour Rainfall "P" = 7.9 inches | | | |

Figure 1. Etowah Recreation Center Site Plan

BACK TO TOC

This example focuses on the design of a dry enhanced swale to meet the water quality treatment requirements of the site. Channel protection and overbank flood control is not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of dry swales is to provide water quality treatment and groundwater recharge and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults). Due to the Class C soils identified on-site, an underdrain will be provided in this practice that will provide a runoff reduction volume credit of 50%.

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

The layout of the Etowah Recreation Center is shown in Figure 1. This example assumes that the local community has adopted the unified stormwater sizing criteria requirements.

**STEP 1 - CONFIRM LOCAL DESIGN REQUIREMENTS AND DETERMINE THE GOAL AND FUNCTION OF THE BMP**

It was determined by the local municipality that this best management practices would be designed by the water quality volume calculation approach (described as Step 4 in Section 4.8)

The total designed volume of the practice must be provided to remove 80% TSS from the 1.2 inch storm event. Runoff reduction credit may still be utilized by the designer through an adjusted curve number calculations.

There is also a local requirement that the 25-year storm is contained within the top of banks of all channels, including these enhanced swale controls.

**STEP 2 - DETERMINE IF THE DEVELOPMENT SITE AND CONDITIONS ARE APPROPRIATE FOR THE USE OF AN ENHANCED DRY SWALE SYSTEM**

Existing ground elevation at the facility location is 922.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 913.0 feet and underlying soils are silt loams (ML). Adjacent creek invert is at 912.0 feet.

Two swales will be designed to carry flow to the existing stream, one around each side of the development.

.

**STEP 3 - COMPUTE RUNOFF CONTROL TARGET VOLUMES FROM THE UNIFIED STORMWATER SIZING CRITERIA (EITHER METHOD 3 OR 4 AS DESCRIBED IN THE DESIGN STEPS)**

Method 4) Compute Water Quality Volume ($WQ_V$):

- Compute Runoff Coefficient, $R_V$

$$R_V = 0.05 + (63.3)(0.009) = 0.62$$

- Compute Water Quality Volume ($WQ_V$):

$$\begin{aligned} WQ_V &= (1.2")(R_V)(A) / 12 \\ &= (1.2")(0.62)(130{,}680\ ft^2)(1ft/12in) \\ &= \underline{8{,}102}\ ft^3 \end{aligned}$$

Note: The Volume Provided (VP) of this practice must be a minimum of 8,102 $ft^3$.

- Even though the water quality volume calculation approach was used in this design, some amount of Runoff Reduction ($RR_V$ provided) is provided by the enhanced dry swale with an underdrain. This information will be needed in the adjusted curve number calculation, Compute the $RR_V$ provided:

$$\begin{aligned} RR_V\ (provided) &= (RR\%)(VP) \\ &= (50\%)(8{,}102\ ft^3)\ * \\ &= \underline{4{,}051}\ ft^3\ * \end{aligned}$$



BACK TO TOC

**Compute Stream Channel Protection Volume ($CP_v$):**
For stream channel protection, provide 24 hours of extended detention for the 1-year event.

In order to determine a preliminary estimate of storage volume for channel protection and overbank flood control, it will be necessary to perform hydrologic calculations using approved methodologies. This example uses the NRCS TR-55 methodology presented in Section 3.1.5 to determine pre- and post-development peak discharges for the 1-yr, 25-yr, and 100-yr 24-hour return frequency storms.

- Hydrologic output based on the given information provided

| Condition | CN | | $Q_{1\text{-year}}$ inches | $Q_{1\text{-year}}$ cfs | $Q_{25\text{-year}}$ inches | $Q_{25\text{-year}}$ cfs | $Q_{100\text{-year}}$ inches | $Q_{100\text{-year}}$ cfs |
|---|---|---|---|---|---|---|---|---|
| Pre-Developed | 70 | | 0.95 | 2.6 | 3.21 | 9.8 | 4.38 | 13.5 |
| Post-Developed | 88 | | 2.18 | 8.2 | 5.12 | 18.5 | 6.48 | 23.2 |
| * Post-Developed (Adjusted CN for $RR_V$) | 1-yr<br>25-yr<br>100-yr | 83.5<br>84.7<br>84.9 | 1.81 | 6.9 | 4.75 | 17.5 | 6.11 | 22.2 |

- *Adjusted Curve Number Procedure for Peak Flow Reduction of $CP_V$ (Section 3.1.7.5)

Given Q = 2.18 in. and P = 3.4 in., Find "R" and "S" to back calculate an adjusted CN

$$Q - R = \frac{(P - 0.2S)^2}{(P + 0.8S)} \quad \text{(Modified Equation 3.1.5)}$$

Retention storage (expressed in inches) for this basin is calculated by the following formula:

R = $RR_V$ (provided) / Basin Area
= [(RR%)(VP)] / A
= 4,051 $ft^3$/130,680 $ft^2$ (12 in / 1 ft) = **0.37 inches**

$$1.81 = \frac{(3.4 - 0.2S)^2}{(3.4 + 0.8S)}$$

Solve for "S" to back calculate CN: S = 1.98
S = 1000/CN-10: therefore, CN = 83.5

- Utilize modified TR-55 approach to compute channel protection storage volume (based on adjusted CN value calculated above)

Initial abstraction (Ia) for CN of 83.5 is 0.4: [Ia = (200/CN - 2)]

Ia/P = (0.40)/ 3.4 inches = 0.12
$T_c$ = 0.20 hours
$q_u$ = 800 csm/in (From Figure 3.1.5-6)

Knowing $q_u$ and T (extended detention time), find $q_o/q_I$ for a Type II rainfall distribution.

Peak outflow discharge/peak inflow discharge ($q_o/q_i$) = 0.023
(From Figure 3.3.5-1)

For a Type II rainfall distribution,

Vs/Vr = 0.683 - 1.43(qo/qi) +1.64$(q_o/q_i)^2$ - 0.804$(qo/qi)^3$
(Equation 3.3.9)
Where Vs equals channel protection storage ($CP_v$) and Vr equals the volume of runoff in inches.

Vs/Vr = 0.65 Using Equation 3.3.10, calculate $V_S$...

Therefore, Vs = $CP_v$ = 0.65 (1.81 in) (3 ac) (43,560 $ft^2$ / ac) (1 ft / 12 in) = 12,812 $ft^3$

BACK TO TOC

Determine Overbank Flood Protection Volume ($Qp_{25}$) based on the adjusted CN:
For a $Q_{in}$ of 17.5 cfs, and an allowable $Q_{out}$ of 9.8 cfs, the Vs necessary for 25-year control is 13,498 $ft^3$, under an adjusted post-developed CN of 84.7. Note that 6.5 inches of rainfall occurs during this event.

Analyze for Safe Passage of 100 Year Design Storm ($Q_f$):
At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure, typically the same one used for the overbank flood protection control.

**Table 1** Summary of General Design Information for Etowah Recreation Center

| Symbol | Control Volume | Volume Required (cubic feet) | Notes |
|---|---|---|---|
| $RR_V$ | Runoff Reduction | 6,752 | 4,051 cf provided (60%) |
| $WQ_V$ | Water Quality | 8,102 | Swale sized for this volume |
| $CP_V$ | Channel Protection | 12,812 | |
| $Q_{p25}$ | Overbank Flood Protection | 13,498 | |
| $Q_f$ | Extreme Flood Protection | N/A | Provide safe passage for the 100-year event in final design |

**STEP 4 - DETERMINE PRETREATMENT VOLUME AND STORAGE VOLUME**

Size two shallow forebays at the head of the swales equal to 0.05" per impervious acre of drainage (each) (Note, total recommended pretreatment requirement is 0.1"/imp acre). (1.9 ac) (0.05") (1ft/12") (43,560 sq ft/ac) = 344.9 $ft^3$ for each forebay.

Use a 2' deep pea gravel drain at the head of the swale to provide erosion protection and to assist in the distribution of the inflow. There will be no side inflow nor need for pea gravel diaphragm along the sides.

Required: bottom width, depth, length, and slope necessary to store required volume with less than 18" of ponding (see Figure 2 for representative site plan).

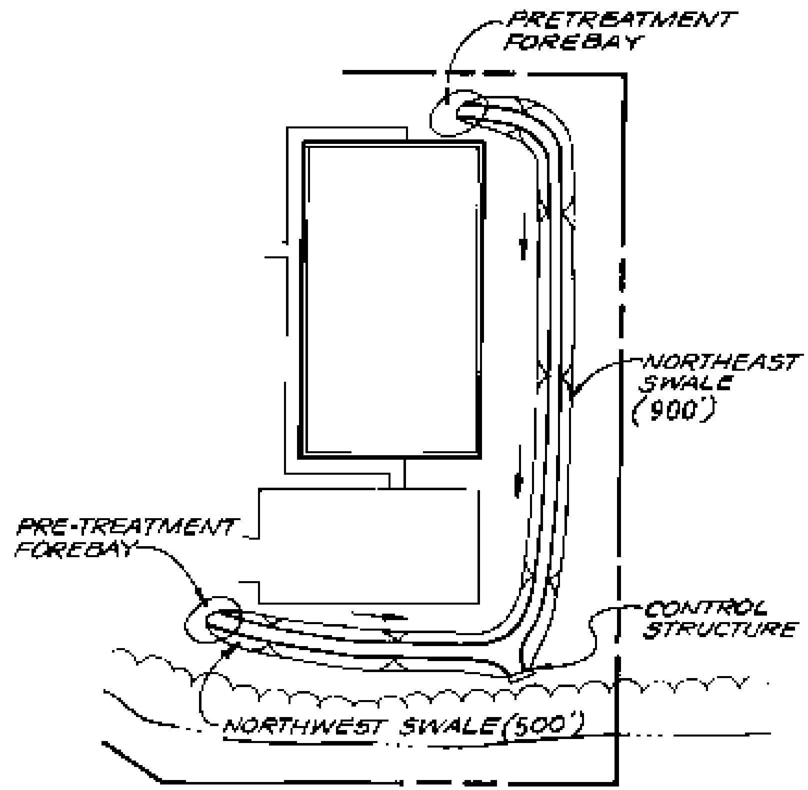



Figure 2. Enhanced Dry Swale Site Plan

Assume a trapezoidal channel with an average $WQ_V$ depth of 9" and a maximum $WQ_V$ depth of 18". Control for this swale will be a shallow concrete wall with a low flow orifice, trash rack located per Figures 2 and 3. Per the site plan, we have about 1,400' of swale available, if the swale is put in with two tails. The outlet control will be set at the existing invert minus three feet (922.0 - 3.0 = 919.0). The existing uphill invert for the northwest fork is 924.0 (length of 500'), the invert for the northeast fork is 928.0 (at a length of 900').

Slope of northwest fork is (924 - 919)/500' = 0.01 or 1.0%
Slope of northeast fork is (928 - 919)/900' = 0.01 or 1.0%

Minimum slope is 1.0 % [okay]

For an enhanced swale, use the following equation to calculate volume provided (VP):

VP = (PV + VES (N))

Where:

- VP = Volume provided (temporary storage)
- PV = Ponding Volume
- VES = Volume of Engineered Soils
- N = Porosity (typically 0.25 for soil media)

To calculate the ponding volume (PV), use the trapezoidal section with a bottom width of 6', an average depth of 9", 3:1 side slopes, compute a cross-sectional area of (6') (0.75') + (0.75') (2.25') = 6.2 $ft^2$ (see Figure 4). Using this cross-sectional area, multiply by the provided length to find the total ponding volume: (6.2 $ft^2$) x (1,400 ft) = **8,680 $ft^3$**

To calculate the volume of engineered soil media (VES), use the base width and depth of media and compute a cross-sectional area of (6') (2.5') = 15 $ft^2$ (See Figure 4). Using this cross-sectional area, multiply by the provided length to find the total volume of engineered soils: (15 ft2)(1,400 ft) = **21,000 $f^{t3}$**

Using a engineered soil porosity of 0.25, the maximum volume able to be provided of the practice can be calculated as follows:

$VP_{MAX}$ = 8,680 $ft^3$ + (21,000 $ft^3$ x 0.25) = **13,930 $f^{t3}$**

**13,930** cubic feet > Minimum required volume of 8,102 $ft^3$; **OK**

To reduce the footprint of the enhanced swale area, or media portion of the swale, the new minimum required length could be back-calculated by using the minimum required volume above:

(Min. required volume) / (sum of the cross-sectional areas) = Min. length of BMP

8,102 $ft^3$ / (6.2 $ft^2$ + (15 $ft^2$ x 0.25)) = 815 Linear Feet (Revision to Figure 2)

New lengths of the swales would be the following:
Northwest fork: 290 feet
Northeast fork: 525 feet

VP = 8,102 $ft^3$ $RR_V$(Provided) = (RR%)(VP) = (50%)(8,102 $ft^3$) = 4,051 $ft^3$

**STEP 5 - COMPUTE NUMBER OF CHECK DAMS (OR SIMILAR STRUCTURE) REQUIRED TO DETAIN $WQ_V$ (SEE FIGURE 4)**

For the northwest fork, 290 ft @ 1.0% slope, and maximum depth at 18", place checkdams at: 1.5'/0.01 = 150' place at 150', 1 required

For the northeast fork, 525 ft @ 1.0% slope, and maximum 18" depth, place checkdams at 1.5'/0.01 = 150' place at 150', 3 required

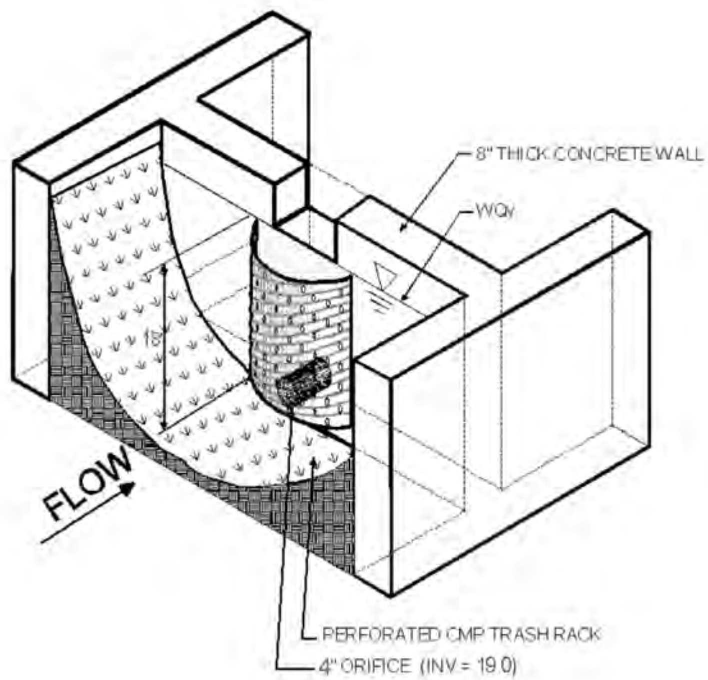



Figure 3 Control Structure at End of Swale

### STEP 6 - CALCULATE DRAW-DOWN TIME

In order to ensure that the swale will draw down within 24 hours, the planting soil will need to pass a maximum rate of 1.5' in 24 hours (k = 1.5' per day). Provide 6" perforated underdrain pipe and gravel system below soil bed (see Figure 4)

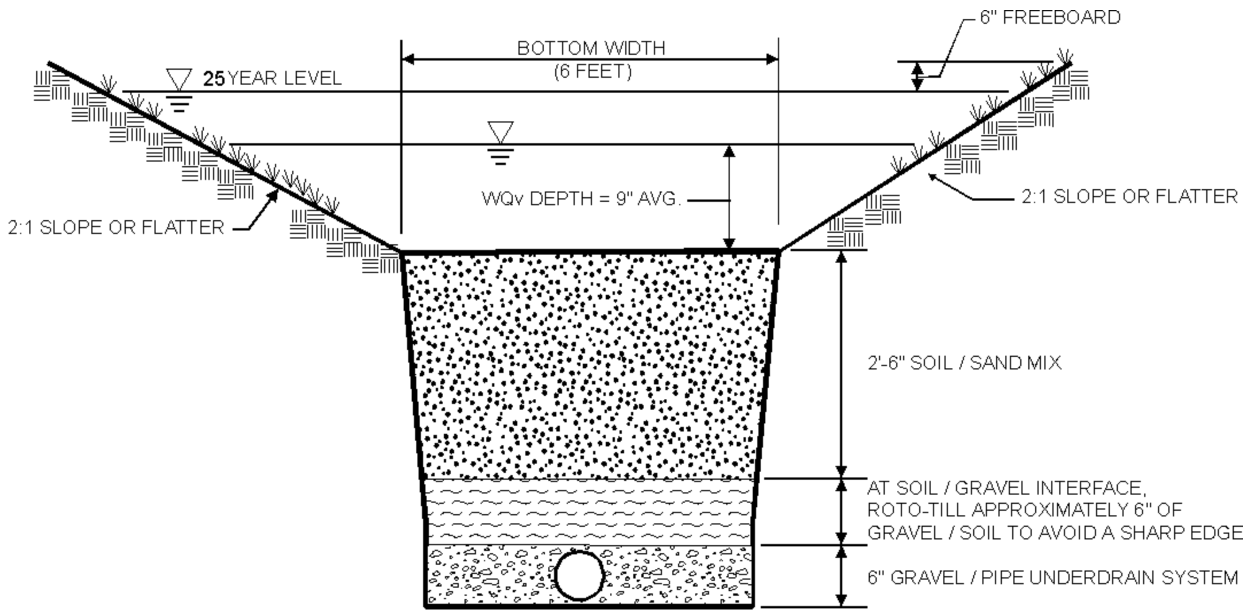



Figure 4 Trapezoidal Dry Swale Section

**STEP 7 - CHECK 2-YEAR AND 25-YEAR FLOWS FOR VELOCITY EROSION POTENTIAL AND FREEBOARD**

Given the local requirements to contain the 25-year flow within banks with freeboard. In this example only the 25-year flow will be checked assuming that lower flows will be handled. The 25-year flow is 17.1 cfs, assume that 30% goes through northwestern swale (5.1 cfs) and 70% goes through the northeastern swale (12.0 cfs). Design for the larger amount (12.0 cfs). From a separate computer analysis using an open channel flow calculator, with a slope of 1.0%, the 25-year velocity will be 2.8 feet-per-second at a depth of 0.60 feet, provide an additional 0.5' of freeboard above top of checkdams or about 1.1' (total channel depth = 2.6').

Find 25-year overflow weir length required: (weir eq. $Q= CLH^{3/2}$),
where C = 3.1, $Q_{25}$ = 17.1 cfs, H =1.1;
Rearranging the equation yields:

L = 17.1 cfs/ ($3.1*1.1^{1.5}$) = 4.8' Use 5 ft

**STEP 8 - DESIGN LOW FLOW ORIFICE AT DOWNSTREAM HEADWALL AND CHECKDAMS (SEE FIGURE 3)**

Design orifice to pass 8,102 cubic feet in 6 hours.

8,102 cubic feet/ [(6 hours) (3600 sec/hour)] = 0.38 cfs
Use Orifice equation: $Q = CA(2gh)^{1/2}$

Assume h = 1.5'

A = (0.38 cfs) / [(0.6) ((2) (32.2 ft/$s^2$) (1.5'))$^{1/2}$]
A = 0.06 sq ft, dia = 0.29 feet or 3.4" use 3" - 3.5" orifice

Provide 3" v-notch slot in each check dam

**STEP 9 - PREPARE VEGETATION AND LANDSCAPING PLAN**

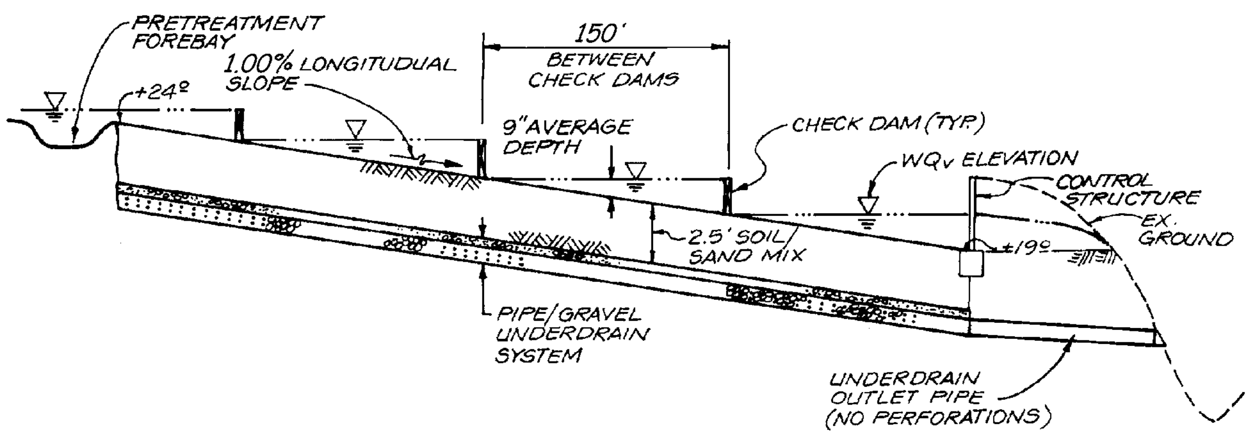
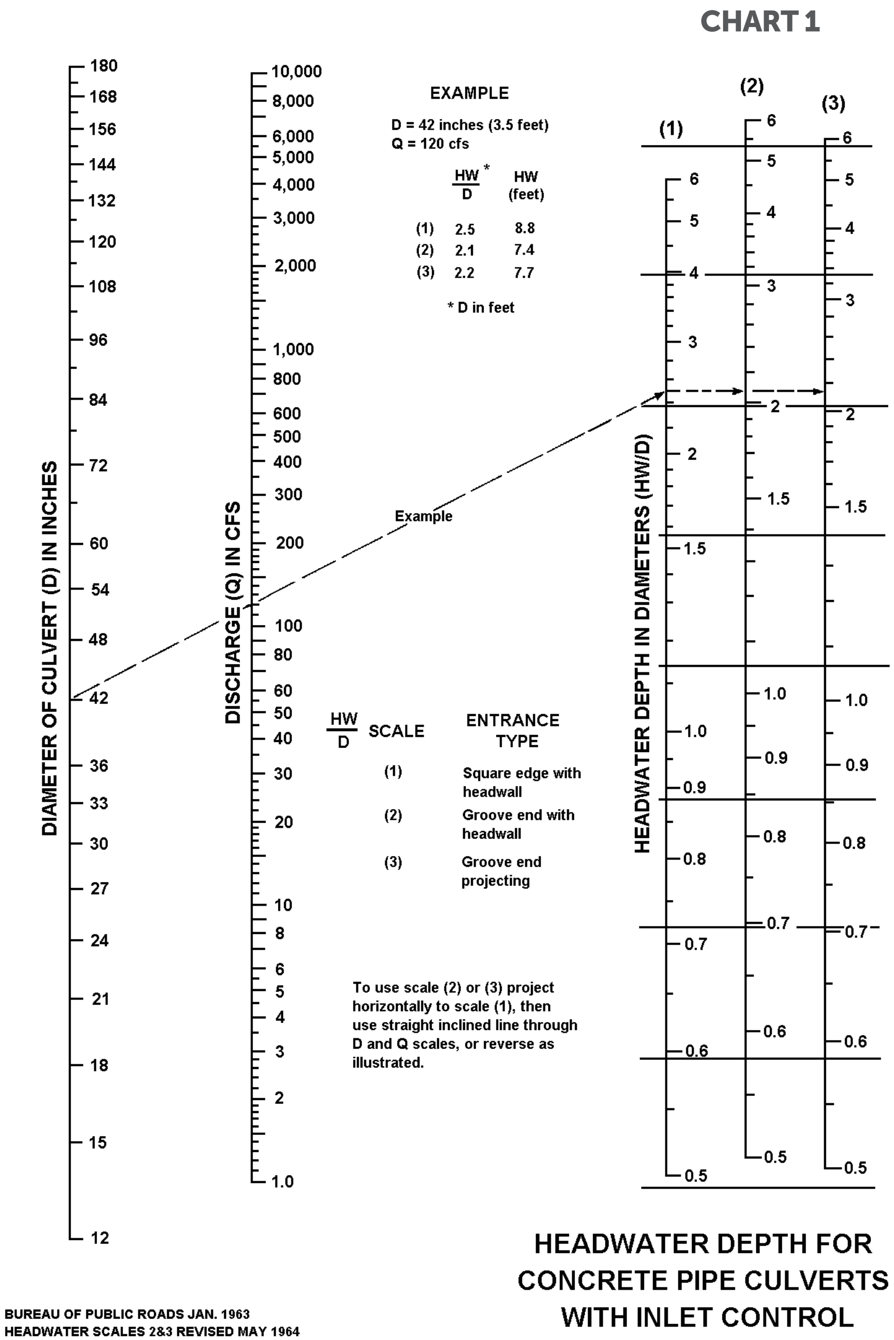



Figure 5 Profile of Northwest Fork Dry Swale

BACK TO TOC

# C-1 Culvert Design Charts and Nomographs

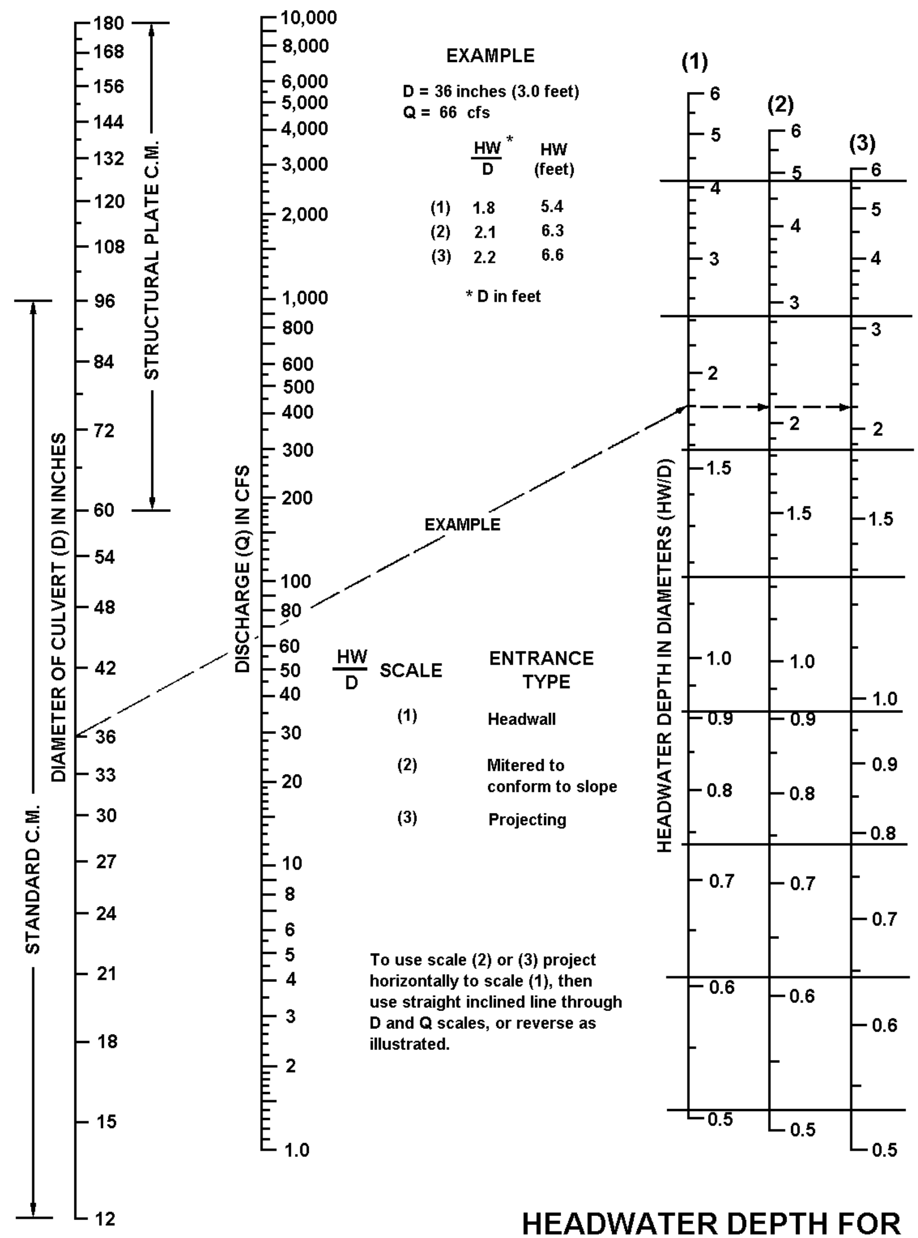
All of the figures in this section are from the AASHTO *Model Drainage* 1991.






BACK TO TOC

## CHART 2

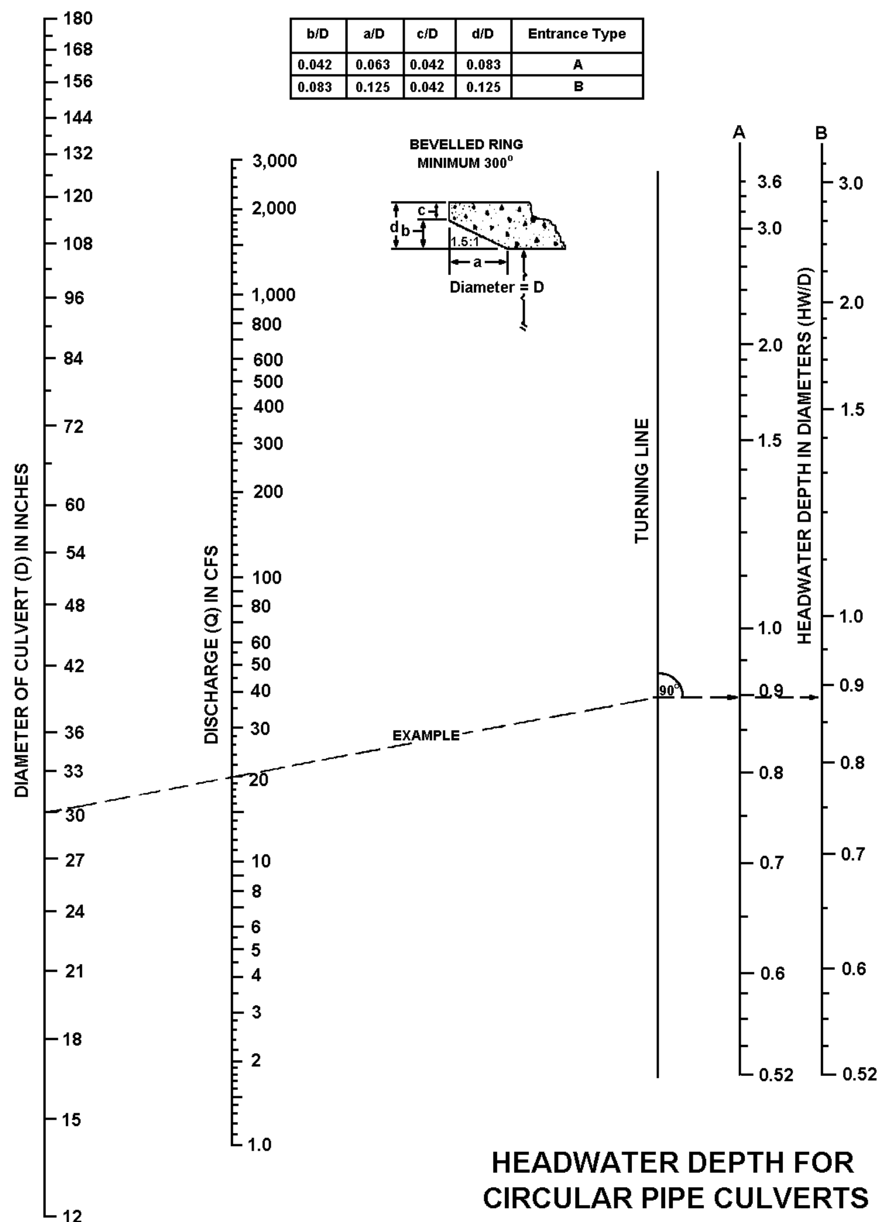



**HEADWATER DEPTH FOR C.M. PIPE CULVERTS WITH INLET CONTROL**

BUREAU OF PUBLIC ROADS JAN. 1963

## CHART 3

| b/D | a/D | c/D | d/D | Entrance Type |
|---|---|---|---|---|
| 0.042 | 0.063 | 0.042 | 0.083 | A |
| 0.083 | 0.125 | 0.042 | 0.125 | B |

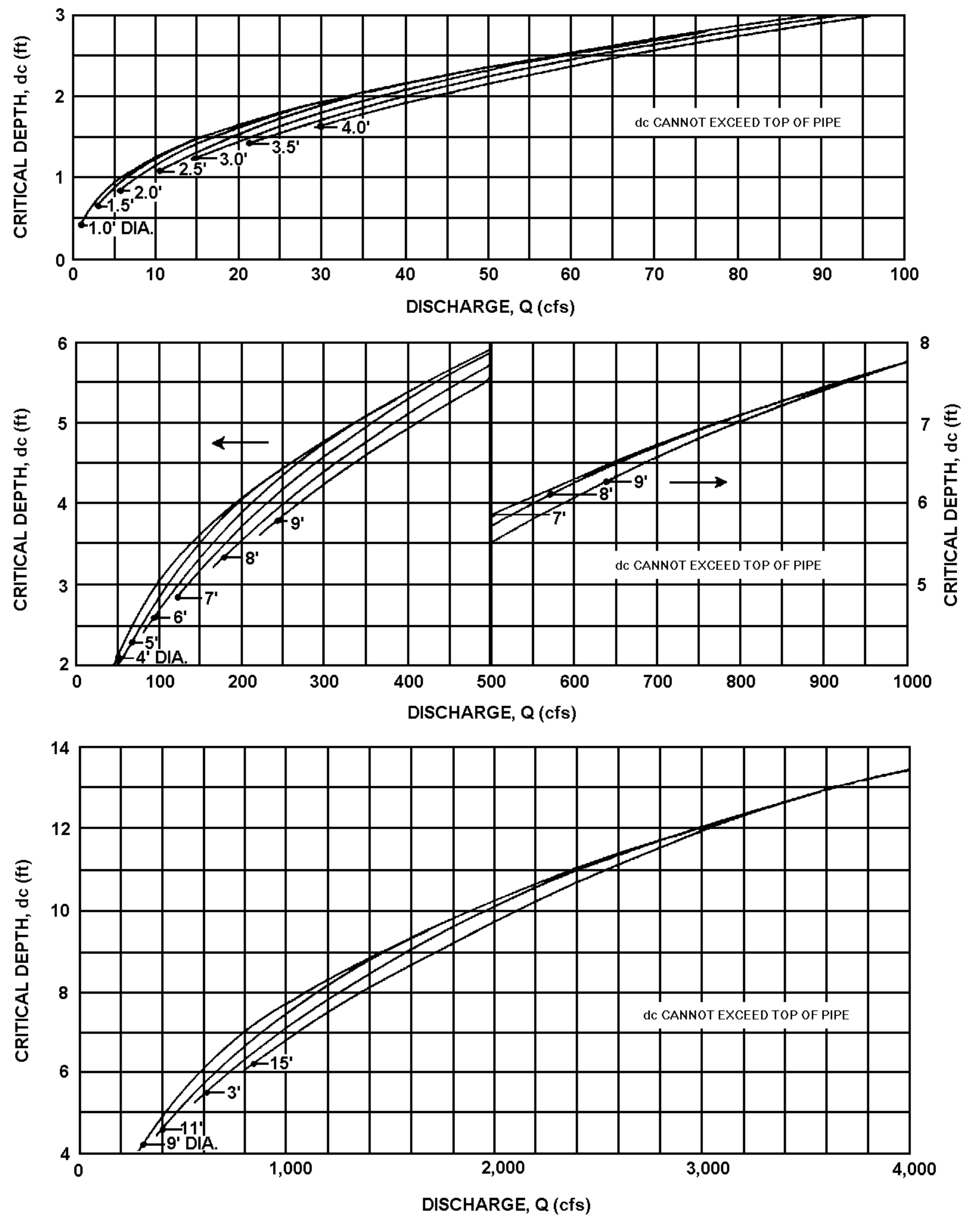



FEDERAL HIGHWAY ADMINISTRATION MAY 1973

**HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL**

BACK TO TOC

## CHART 4

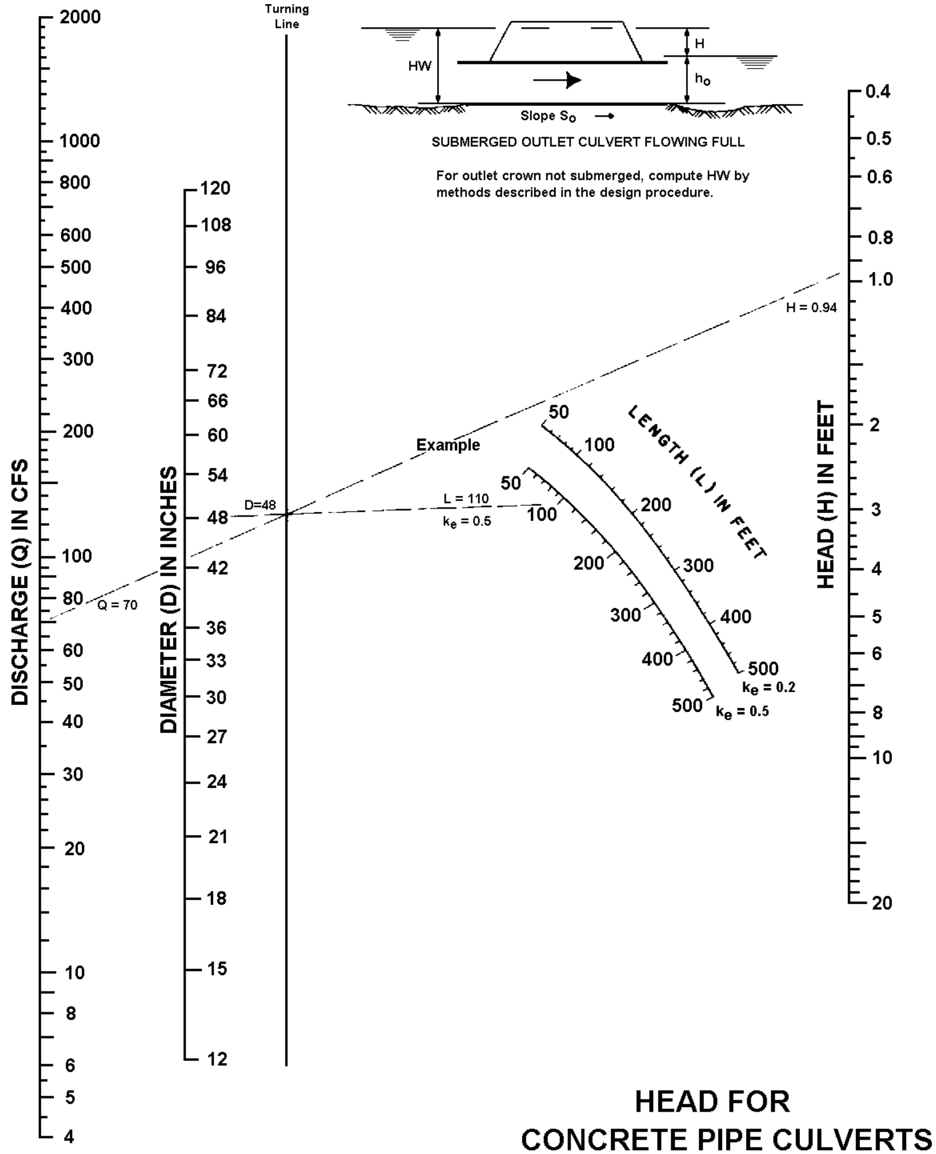



BUREAU OF PUBLIC ROADS JAN. 1964

**CRITICAL DEPTH**
**CIRCULAR PIPE**

BACK TO TOC

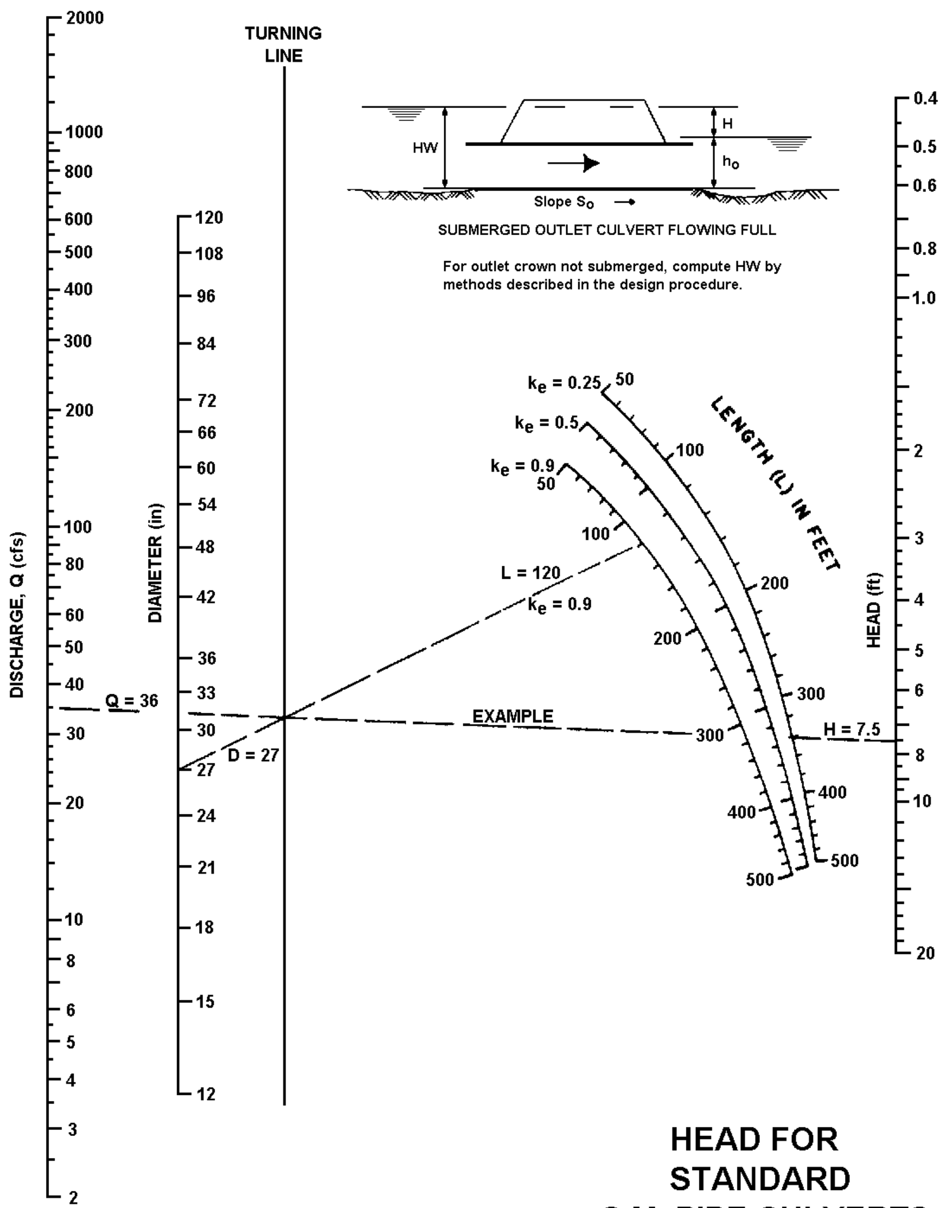
CHART 5




HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

## CHART 6



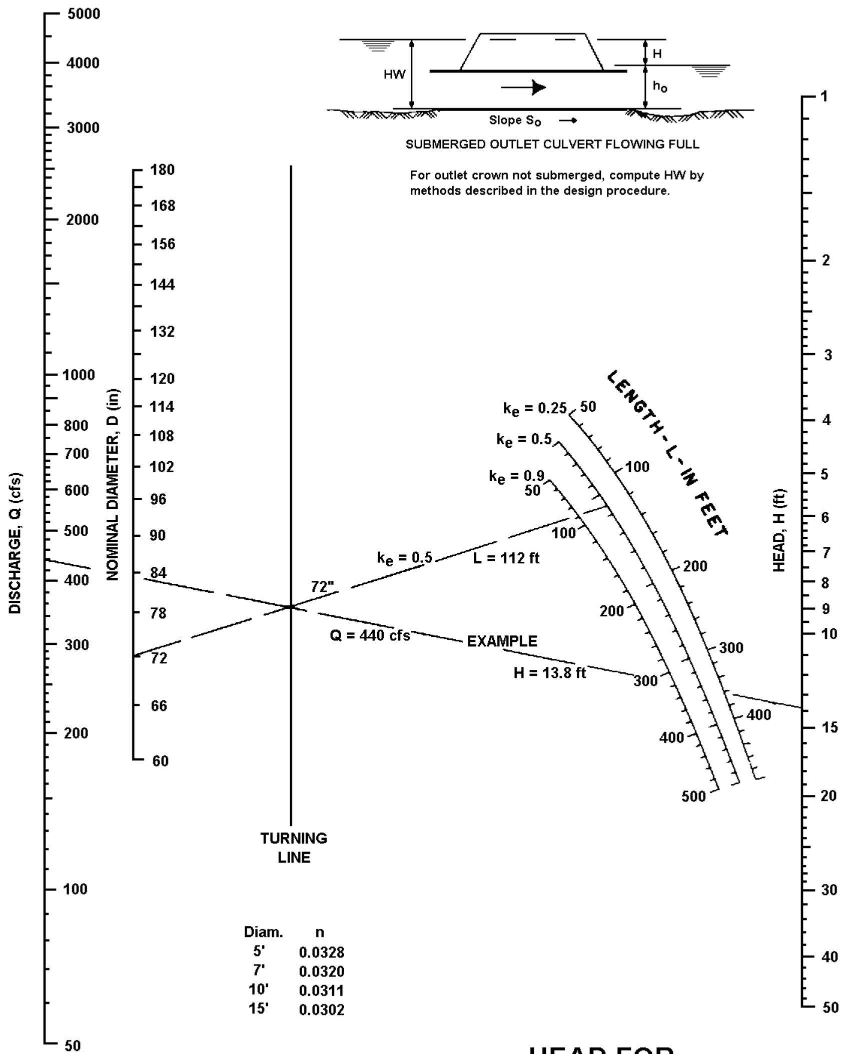

HEAD FOR STANDARD C.M. PIPE CULVERTS FLOWING FULL
n = 0.024

BUREAU OF PUBLIC ROADS JAN. 1963



BACK TO TOC

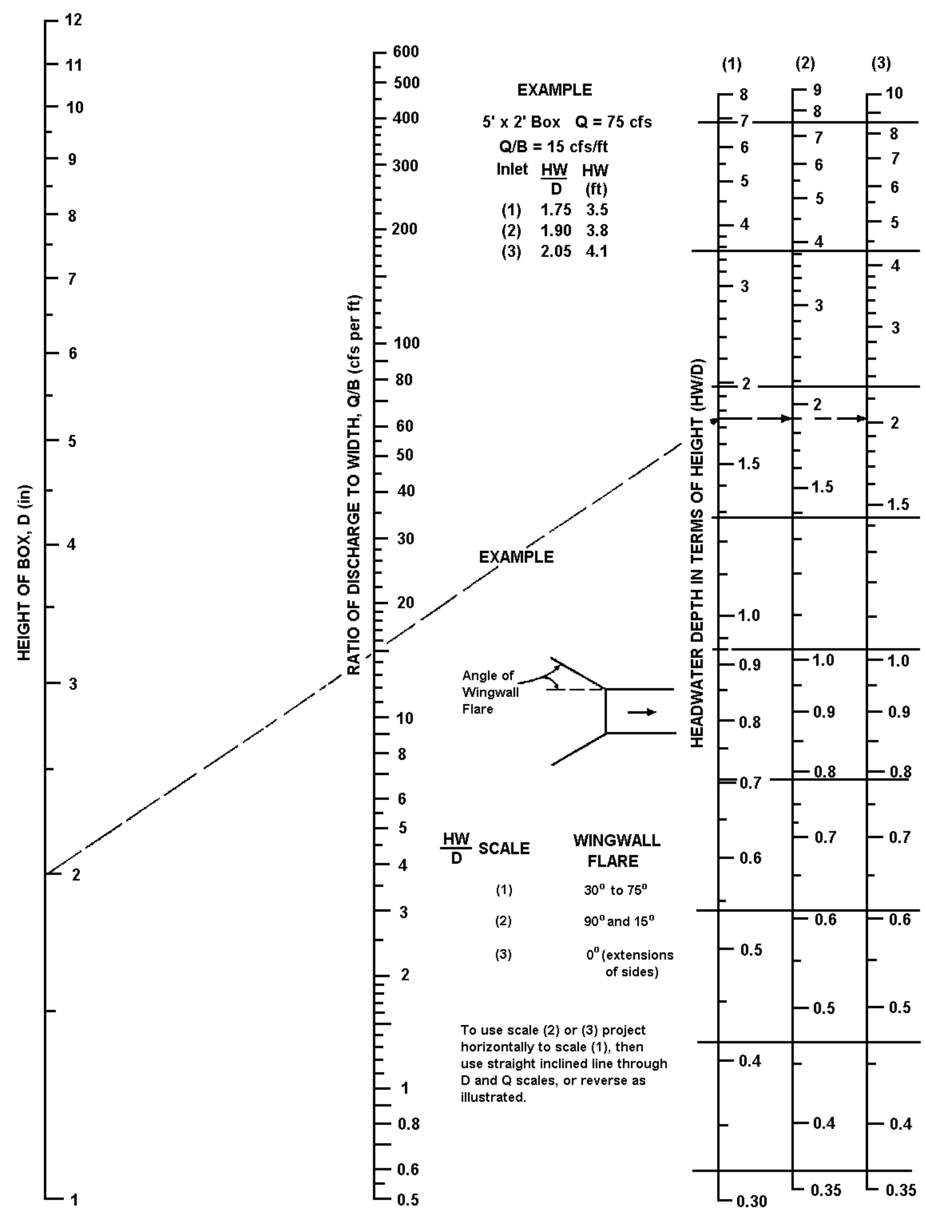
## CHART 7




BUREAU OF PUBLIC ROADS JAN. 1963

**HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL**
**n = 0.0328 TO 0.0302**

## CHART 8

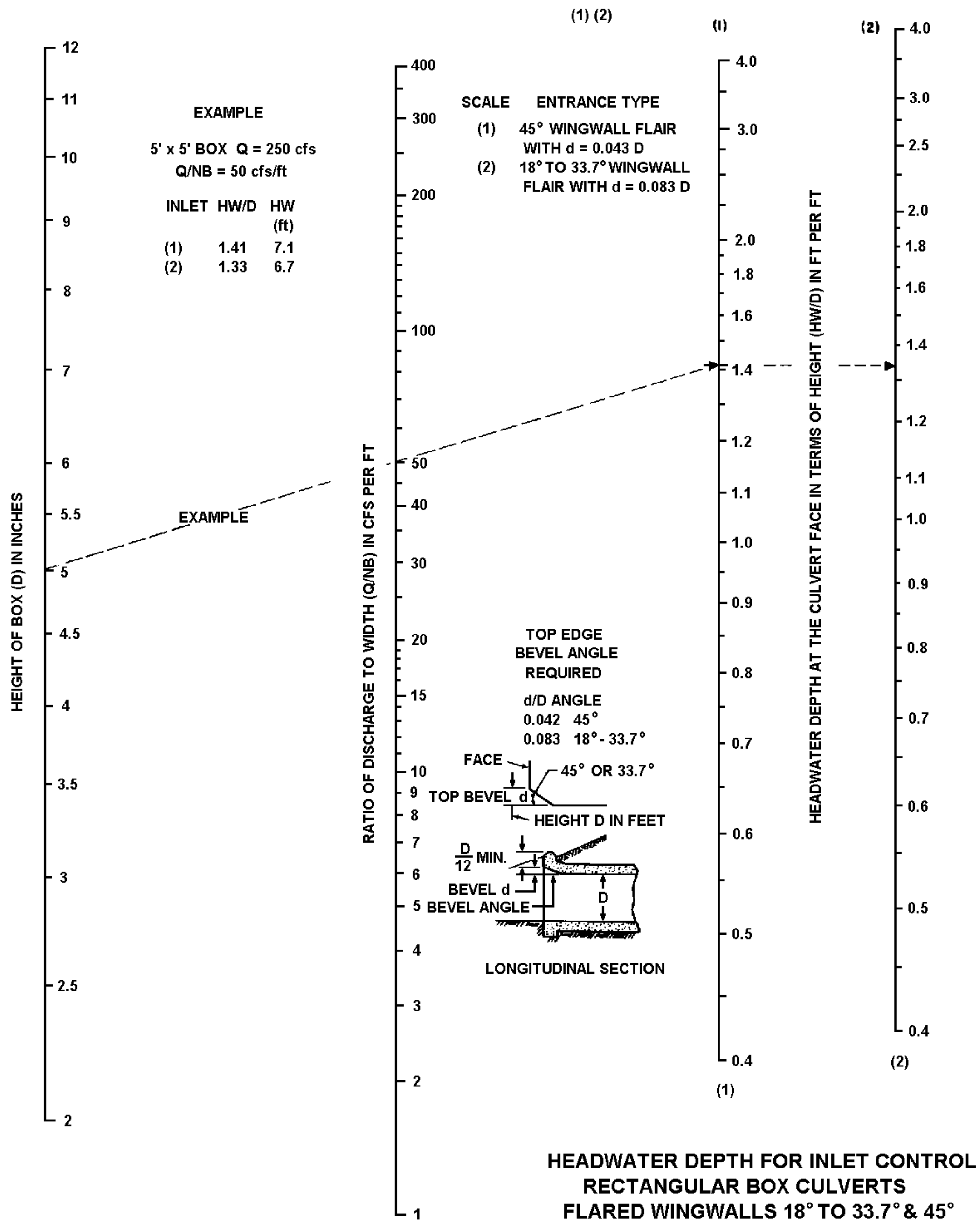



BUREAU OF PUBLIC ROADS JAN. 1963

**HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL**

BACK TO TOC

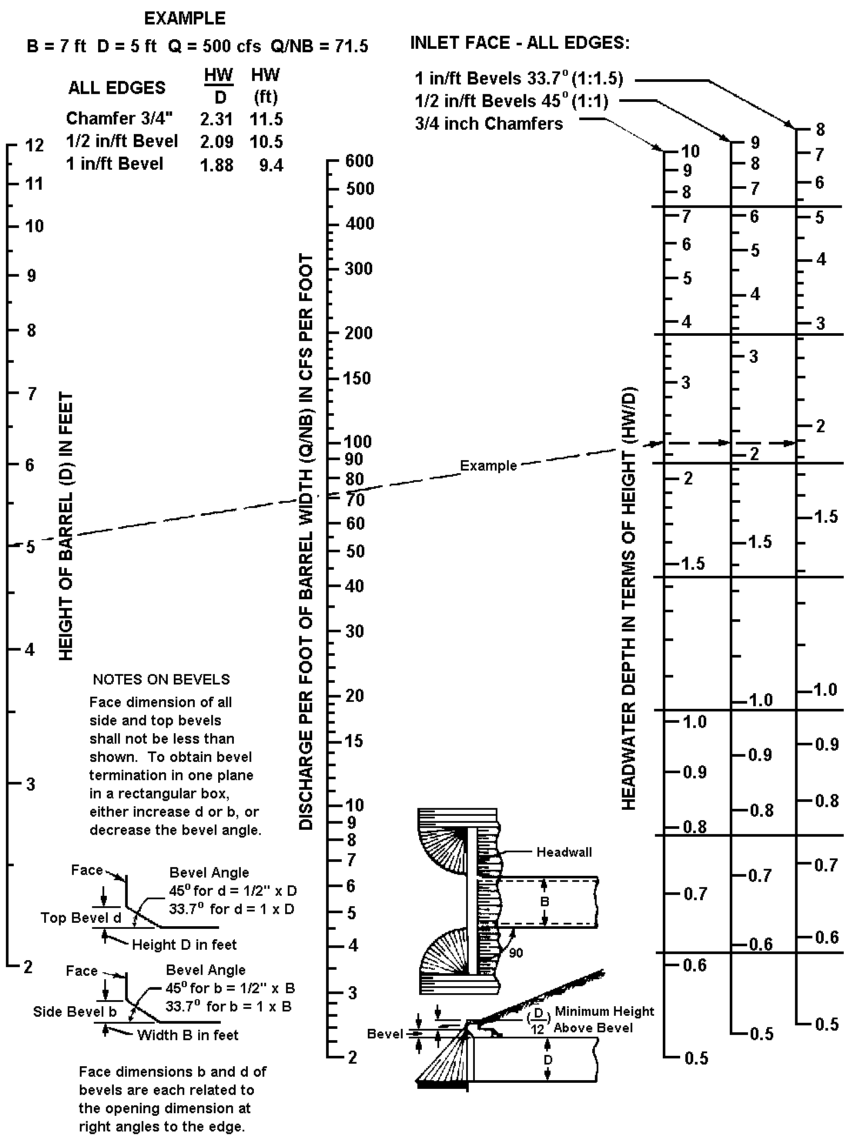
## CHART 9




**HEADWATER DEPTH FOR INLET CONTROL RECTANGULAR BOX CULVERTS FLARED WINGWALLS 18° TO 33.7° & 45° WITH BEVELED EDGE AT TOP OF INLET**

BACK TO TOC

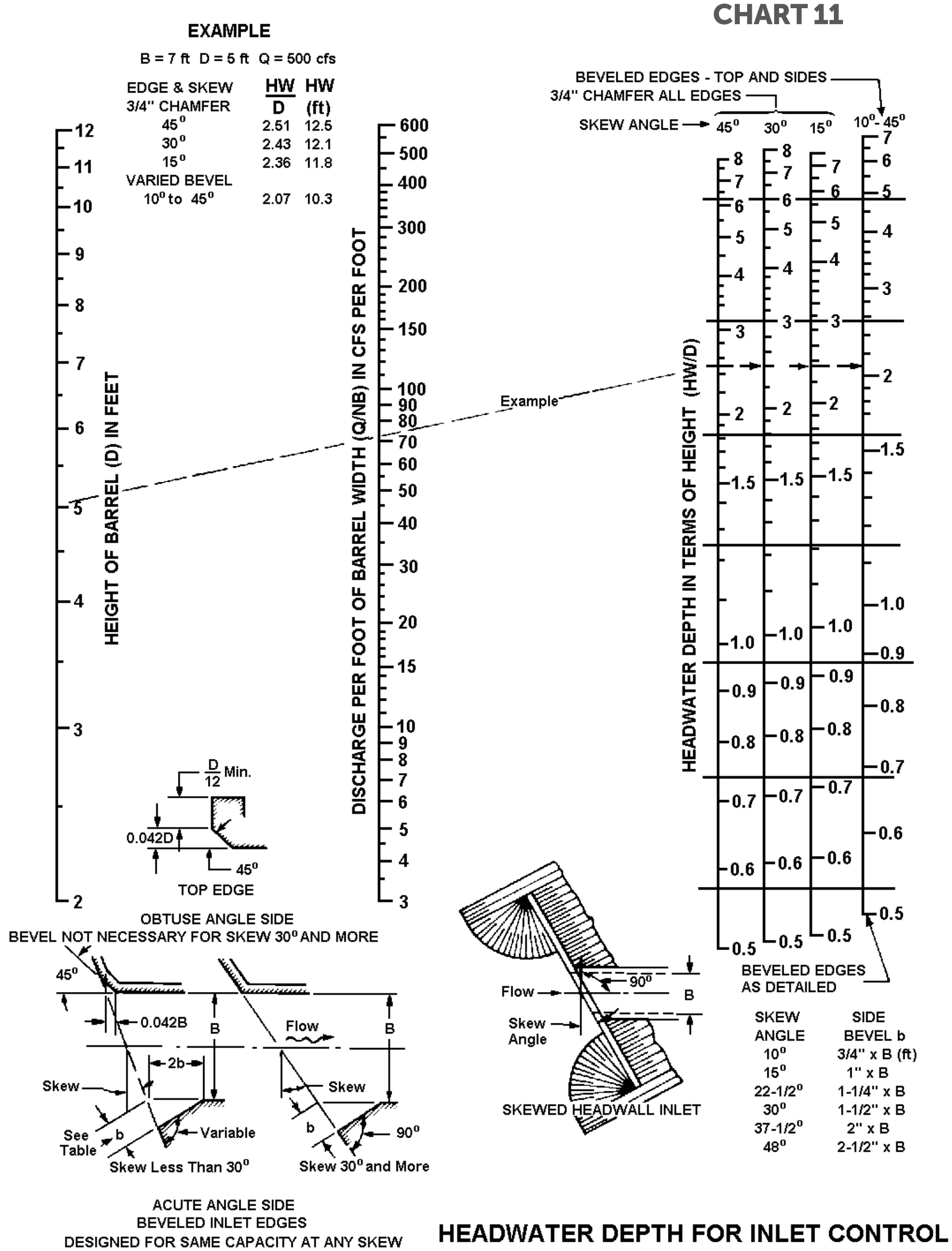
## CHART 10




FEDERAL HIGHWAY ADMINISTRATION MAY 1973

**HEADWATER DEPTH FOR INLET CONTROL**
**RECTANGULAR BOX CULVERTS**
**90° HEADWALL**
**CHAMFERED OR BEVELED INLET EDGES**

## CHART 11



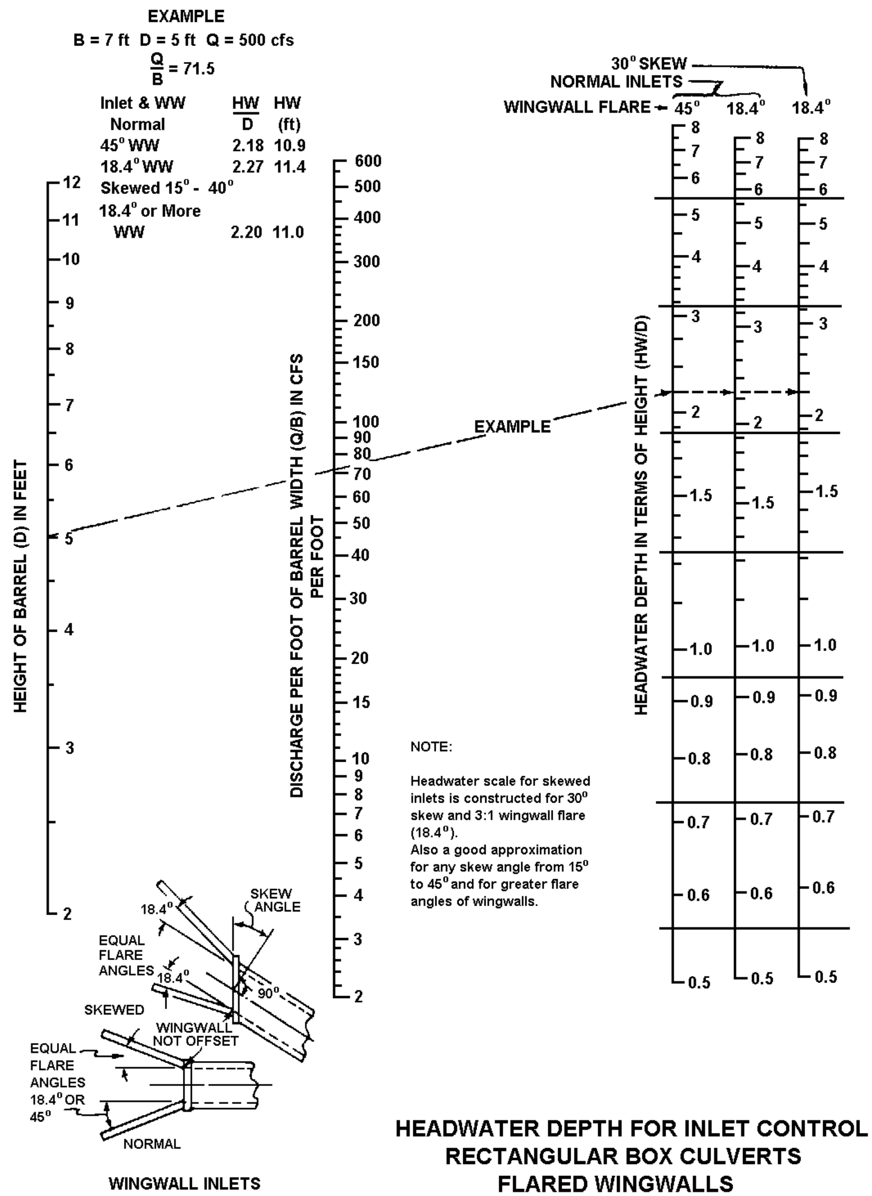

| SKEW ANGLE | SIDE BEVEL b |
|---|---|
| 10° | 3/4" x B (ft) |
| 15° | 1" x B |
| 22-1/2° | 1-1/4" x B |
| 30° | 1-1/2" x B |
| 37-1/2° | 2" x B |
| 48° | 2-1/2" x B |

FEDERAL HIGHWAY ADMINISTRATION MAY 1973

**HEADWATER DEPTH FOR INLET CONTROL**
**SINGLE BARREL BOX CULVERTS**
**SKEWED HEADWALLS**
**CHAMFERED OR BEVELED INLET EDGES**

BACK TO TOC

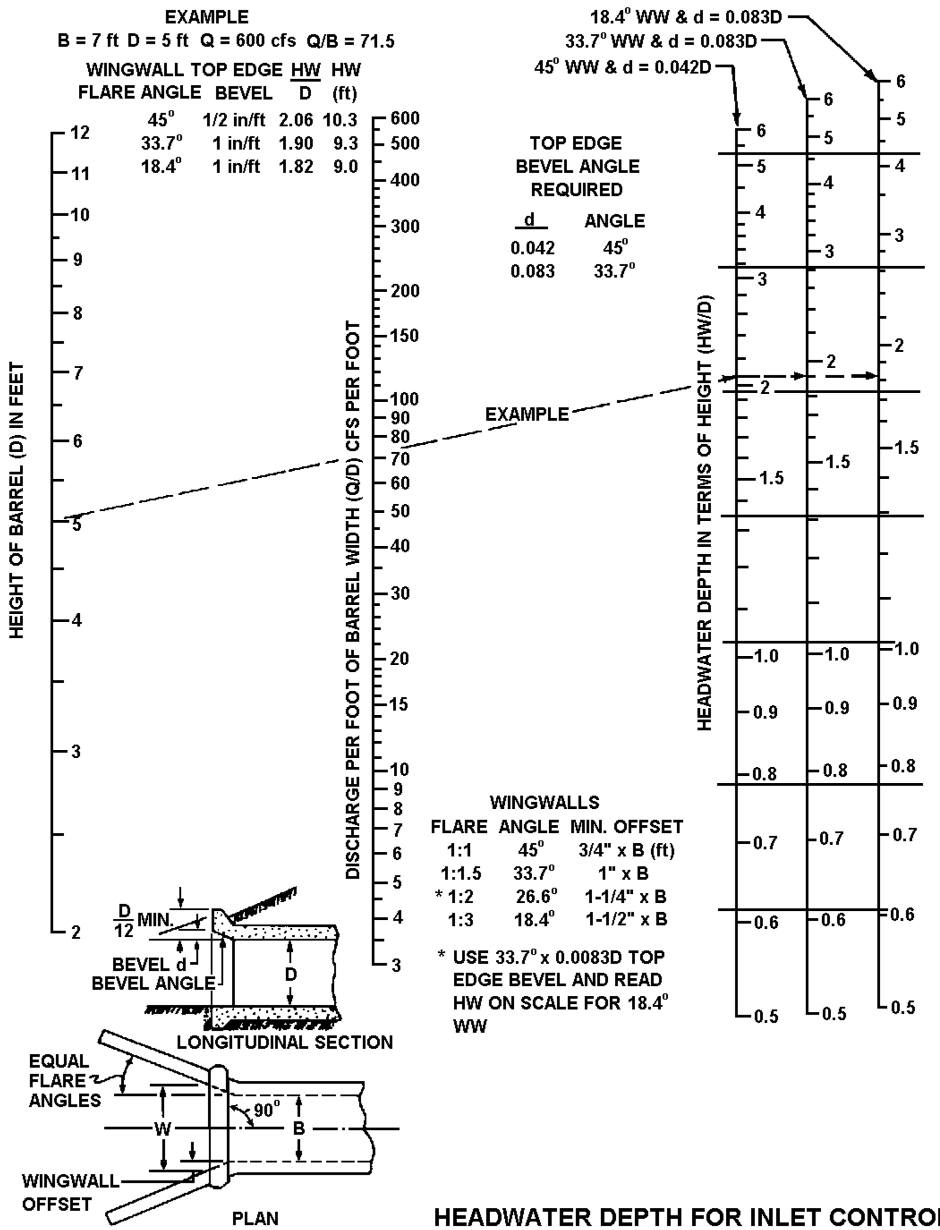
## CHART 12




**HEADWATER DEPTH FOR INLET CONTROL**
**RECTANGULAR BOX CULVERTS**
**FLARED WINGWALLS**
**NORMAL AND SKEWED INLETS**
**3/4" CHAMFER AT TOP OF OPENING**

BUREAU OF PUBLIC ROADS
OFFICE OF R&D AUGUST 1968

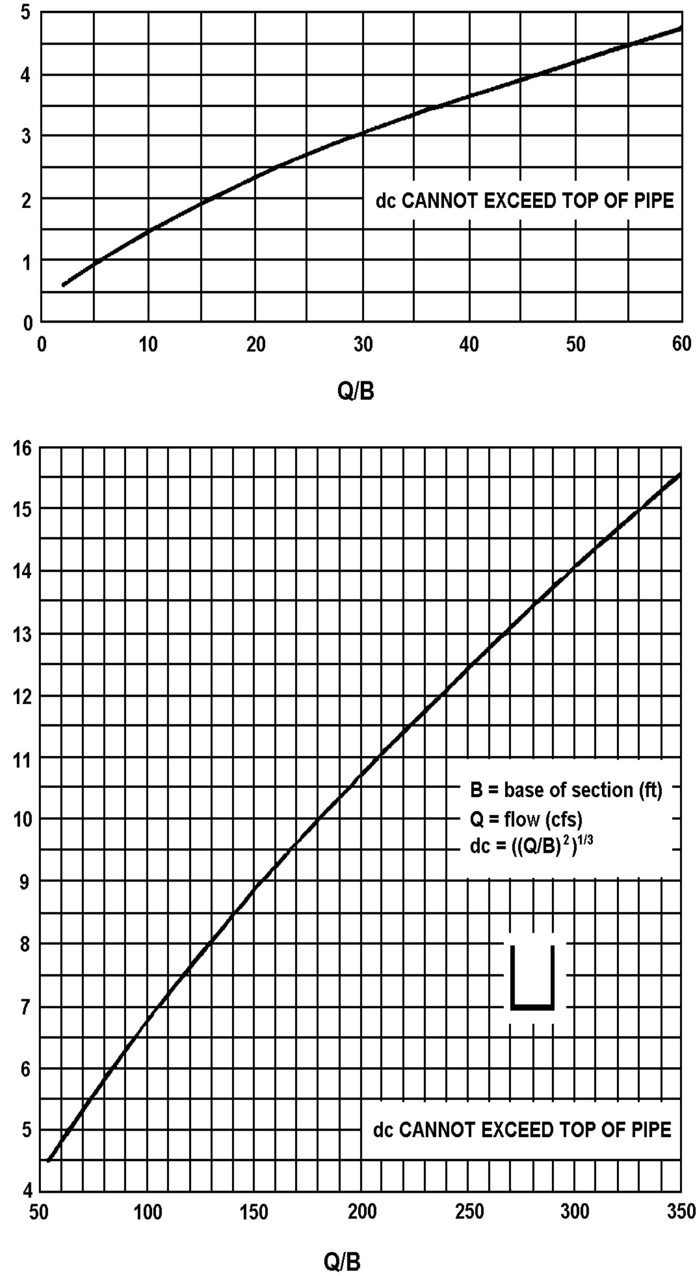
## CHART 13




**HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
OFFSET FLARED WINGWALLS
AND BEVELED EDGE AT TOP OF INLET**

BUREAU OF PUBLIC ROADS
OFFICE OF R&D AUGUST 1968

## CHART 14



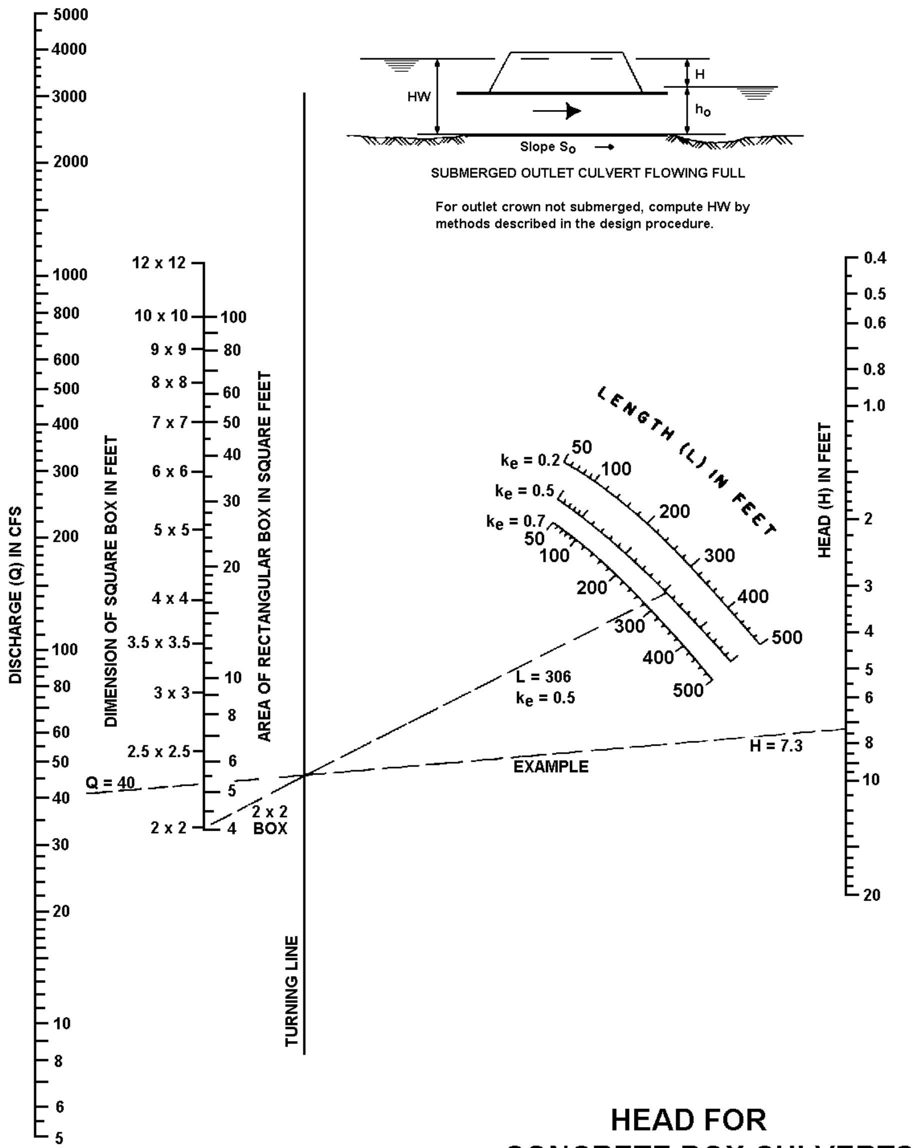

BUREAU OF PUBLIC ROADS JAN. 1963

CRITICAL DEPTH
RECTANGULAR SECTION



BACK TO TOC

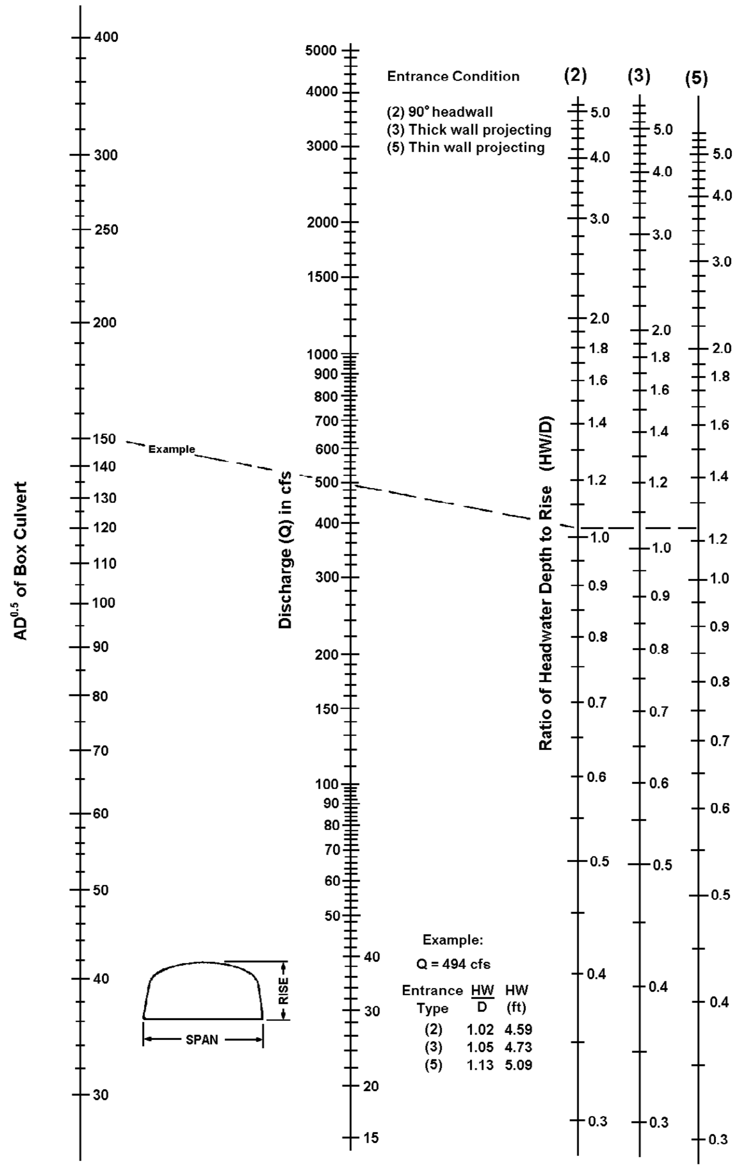
## CHART 15




# HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL
# n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

## CHART 16

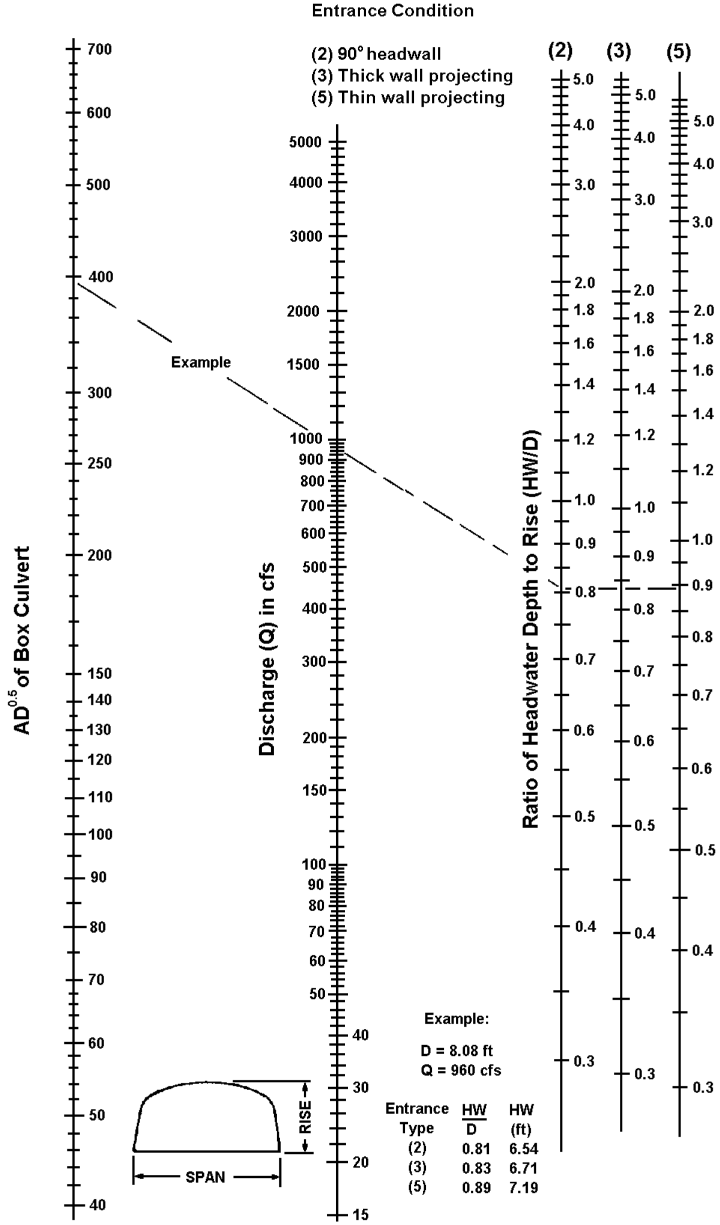



HEADWATER DEPTH
FOR C.M. BOX CULVERTS
RISE/SPAN < 0.3
WITH INLET CONTROL

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.

BACK TO TOC

# CHART 17

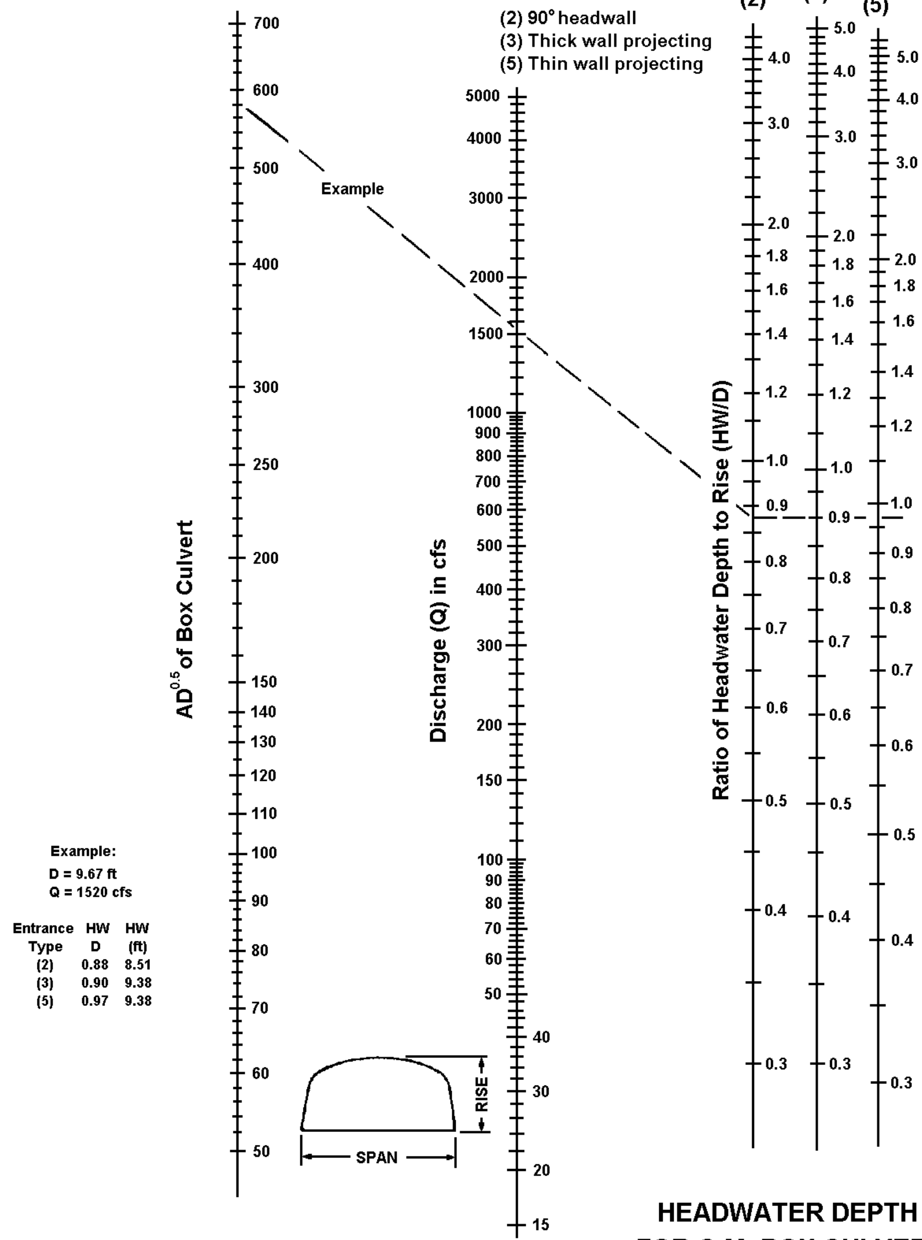



| Entrance Type | HW/D | HW (ft) |
|---|---|---|
| (2) | 0.81 | 6.54 |
| (3) | 0.83 | 6.71 |
| (5) | 0.89 | 7.19 |

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

**HEADWATER DEPTH FOR C.M. BOX CULVERTS 0.3≤ RISE/SPAN < 0.4 WITH INLET CONTROL**

## CHART 18

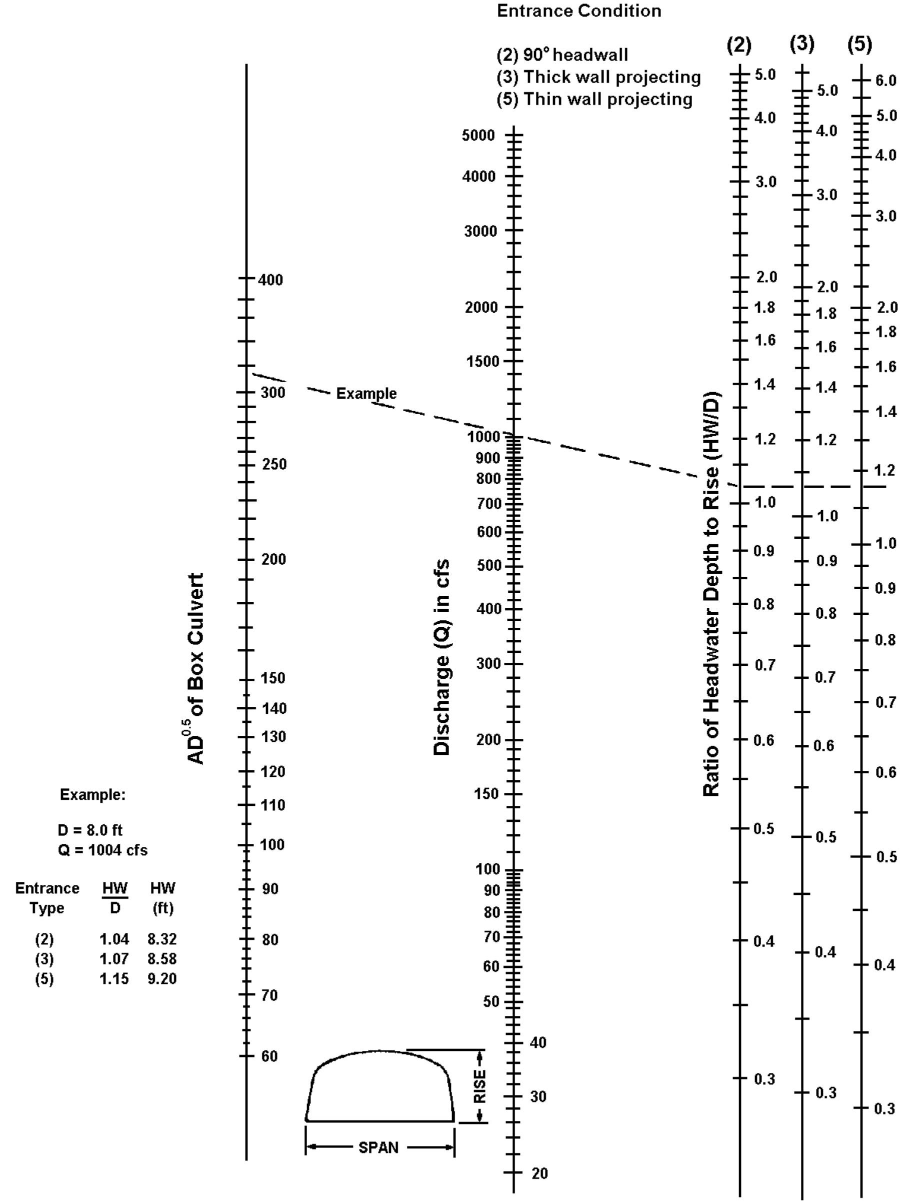



**HEADWATER DEPTH FOR C.M. BOX CULVERTS $0.4 \leq$ RISE/SPAN $< 0.5$ WITH INLET CONTROL**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.
Duplication of this nomograph may distort scale.

BACK TO TOC

## CHART 19

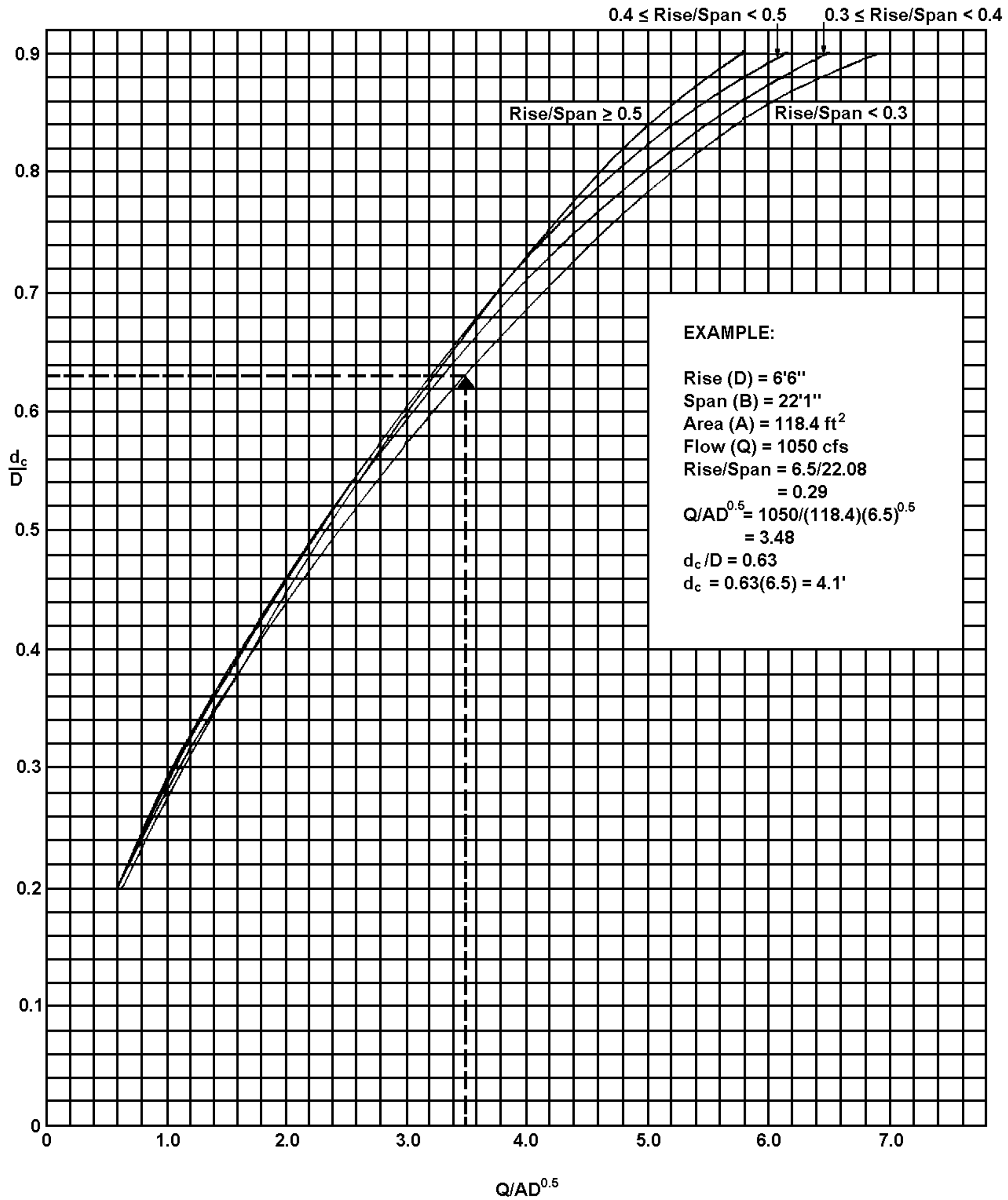



Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.

**HEADWATER DEPTH FOR C.M. BOX CULVERTS 0.5 ≤ RISE/SPAN WITH INLET CONTROL**

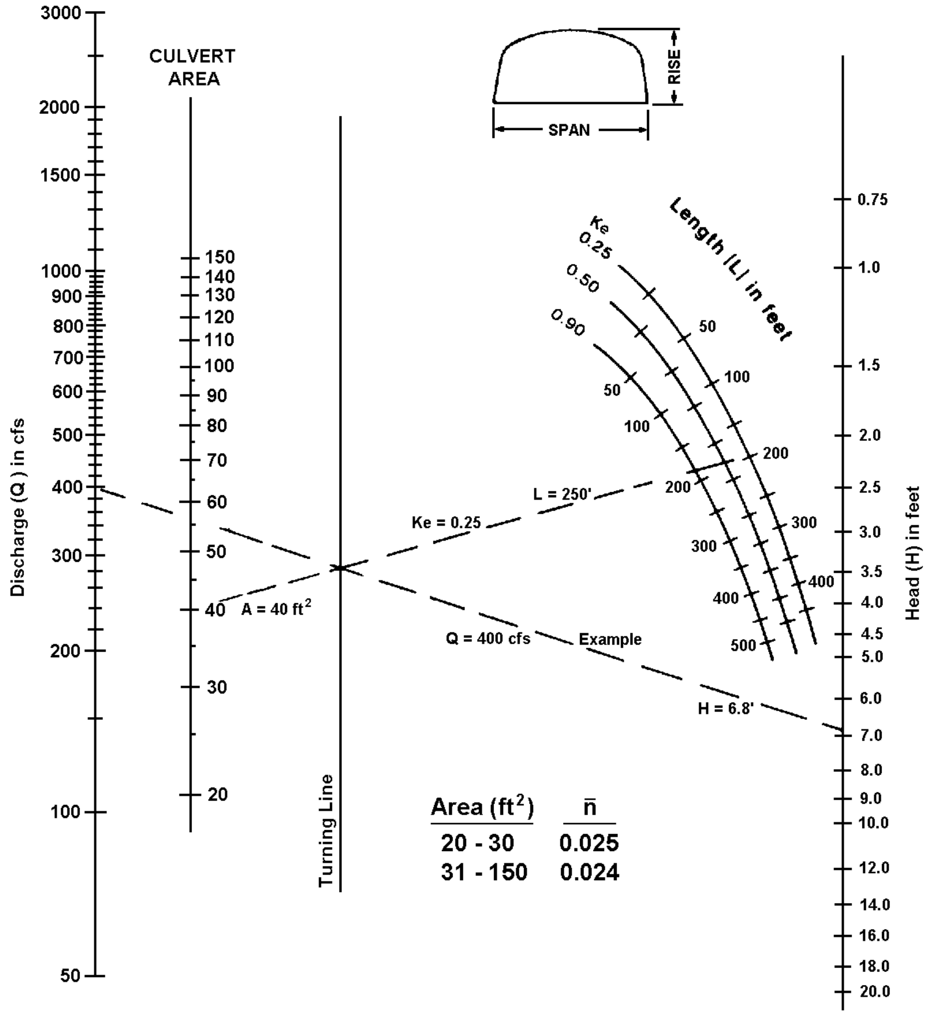
BACK TO TOC

## CHART 20

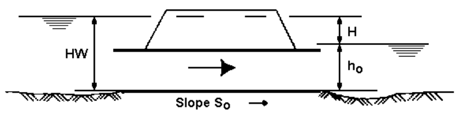


0.4 ≤ Rise/Span < 0.5
0.3 ≤ Rise/Span < 0.4
Rise/Span ≥ 0.5
Rise/Span < 0.3
EXAMPLE:
Rise (D) = 6'6"
Span (B) = 22'1"
Area (A) = 118.4 $ft^2$
Flow (Q) = 1050 cfs
Rise/Span = 6.5/22.08
= 0.29
$Q/AD^{0.5}$ = $1050/(118.4)(6.5)^{0.5}$
= 3.48
$d_c/D$ = 0.63
$d_c$ = 0.63(6.5) = 4.1'
$\frac{d_c}{D}$
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
$Q/AD^{0.5}$

## CHART 21




| Area (ft²) | $\bar{n}$ |
|---|---|
| 20 - 30 | 0.025 |
| 31 - 150 | 0.024 |




SUBMERGED OUTLET CULVERT FLOWING FULL

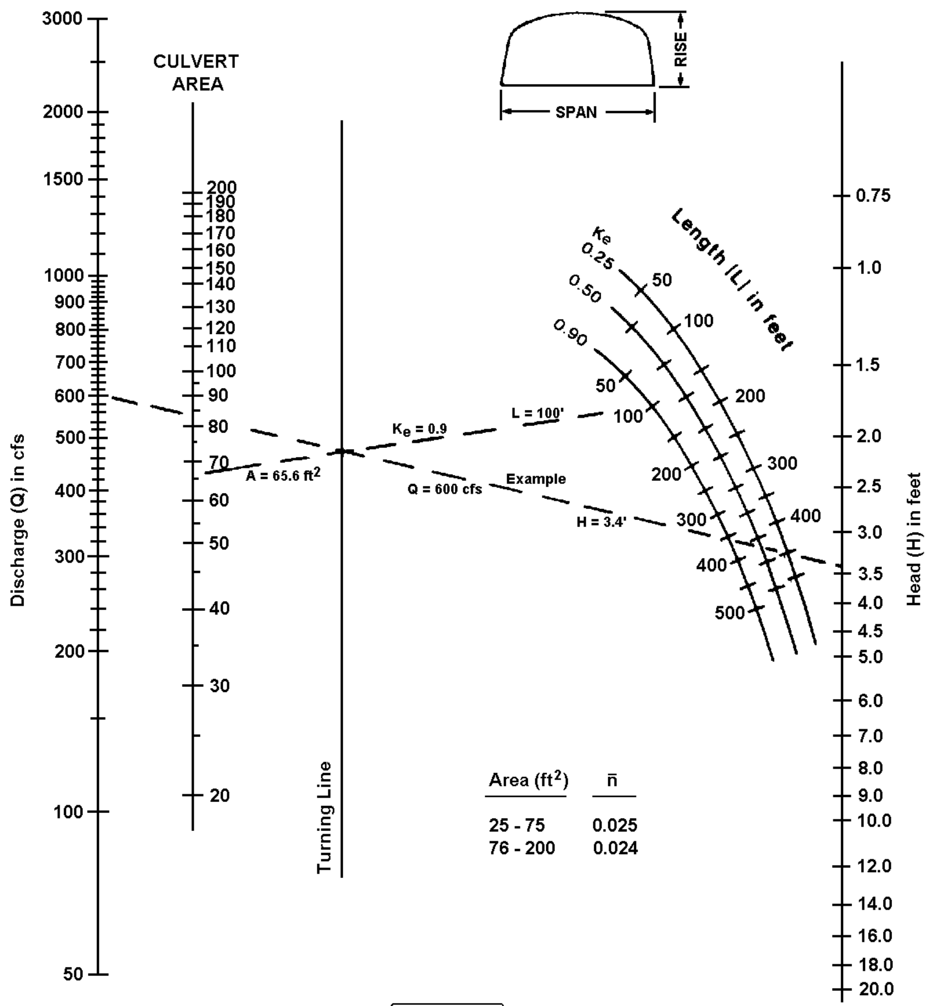
For outlet crown not submerged, compute HW by methods described in the design procedure.

**HEAD FOR C.M. BOX CULVERTS FLOWING FULL CONCRETE BOTTOM RISE/SPAN < 0.3**

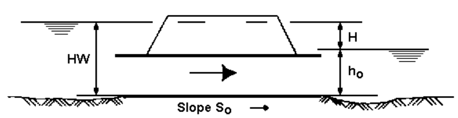
Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

BACK TO TOC

## CHART 22




| Area (ft²) | n̄ |
|---|---|
| 25 - 75 | 0.025 |
| 76 - 200 | 0.024 |




SUBMERGED OUTLET CULVERT FLOWING FULL

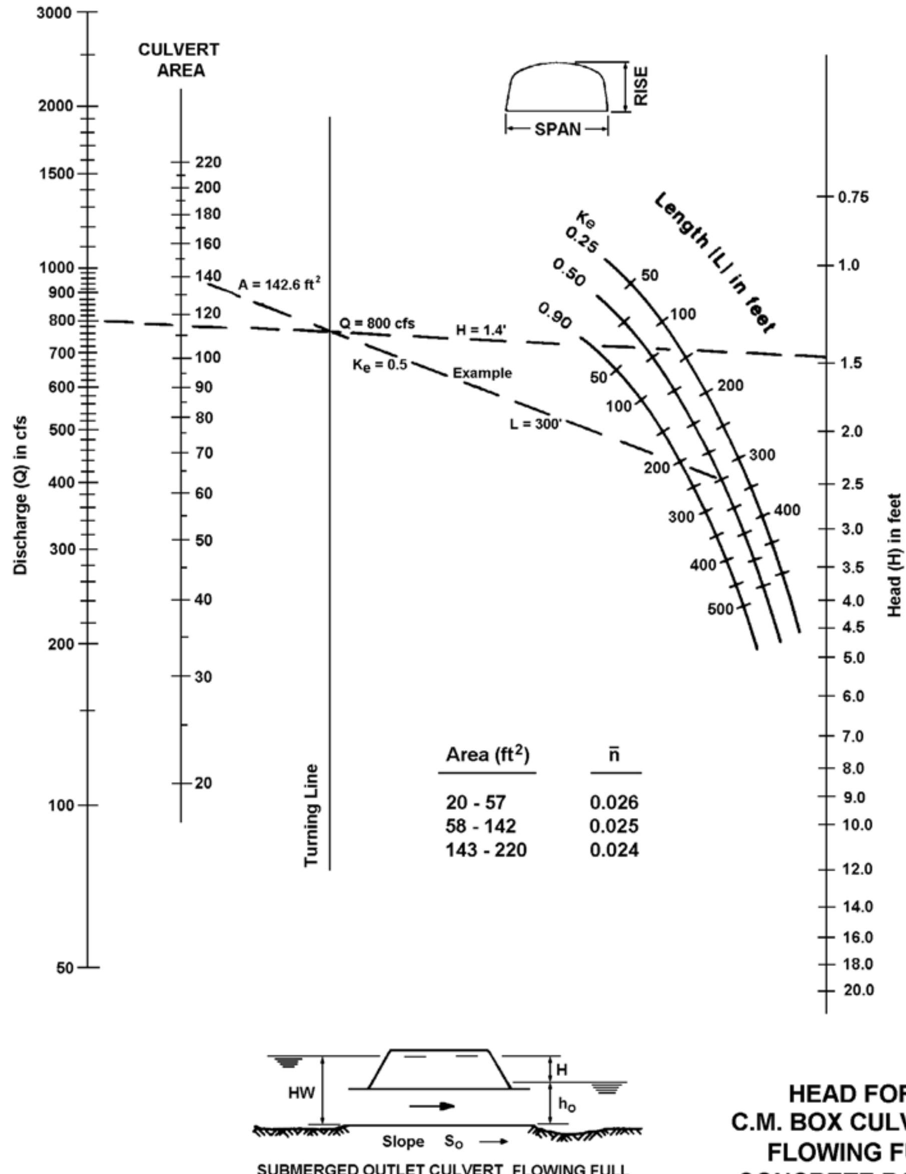
For outlet crown not submerged, compute HW by methods described in the design procedure.

**HEAD FOR C.M. BOX CULVERTS FLOWING FULL CONCRETE BOTTOM $0.3 \le$ RISE/SPAN < 0.4**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

BACK TO TOC

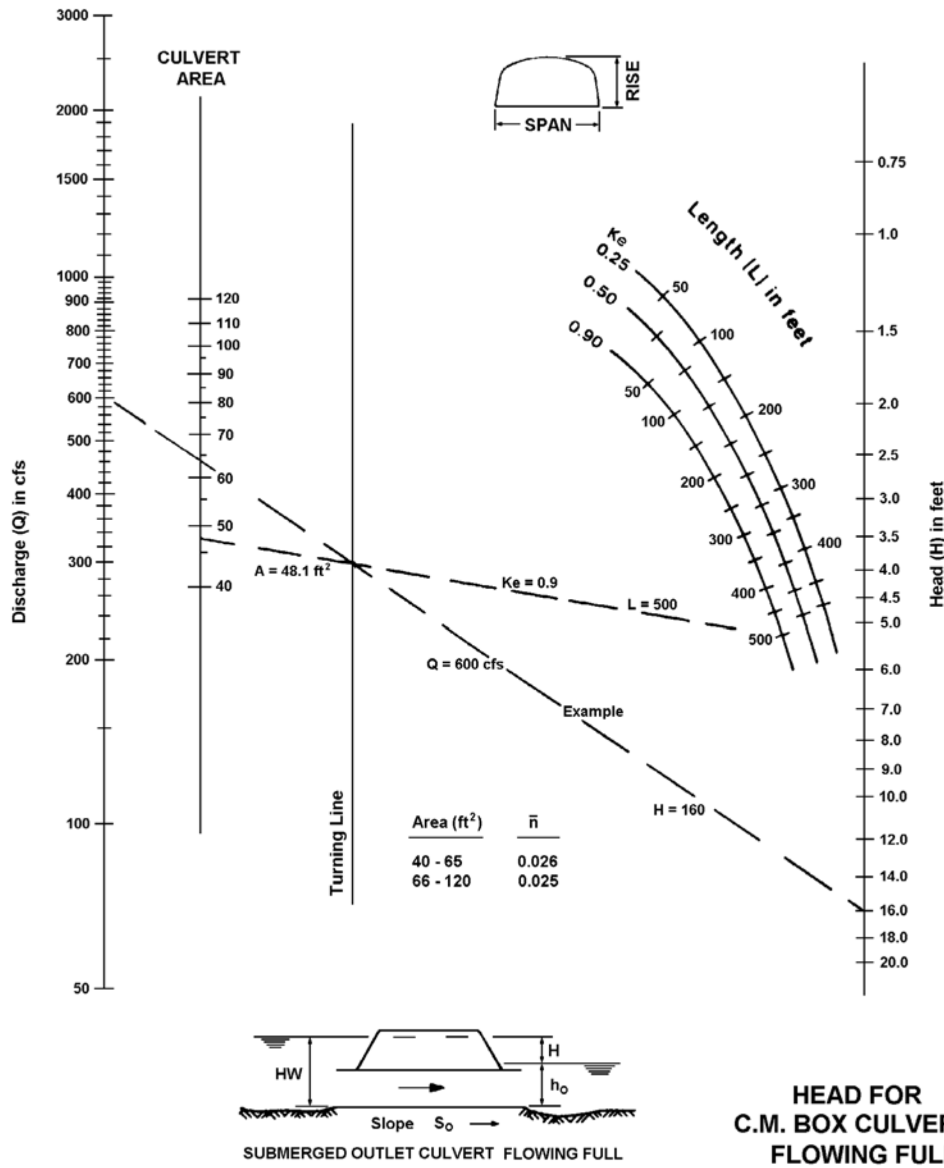
## CHART 23




**HEAD FOR C.M. BOX CULVERTS FLOWING FULL CONCRETE BOTTOM $0.4 \le$ RISE/SPAN < 0.5**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

# CHART 24

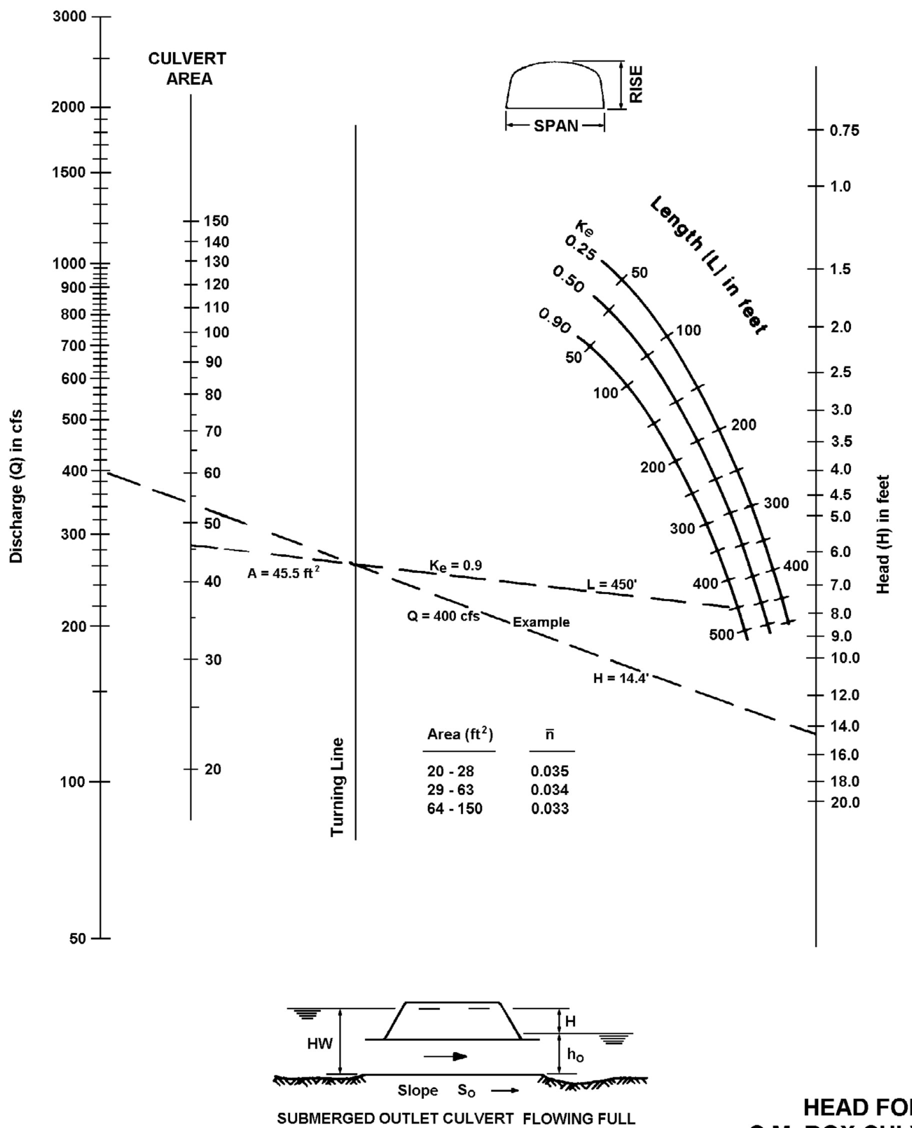



Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

**HEAD FOR C.M. BOX CULVERTS FLOWING FULL CONCRETE BOTTOM $0.5 \leq$ RISE/SPAN**

BACK TO TOC

## CHART 25

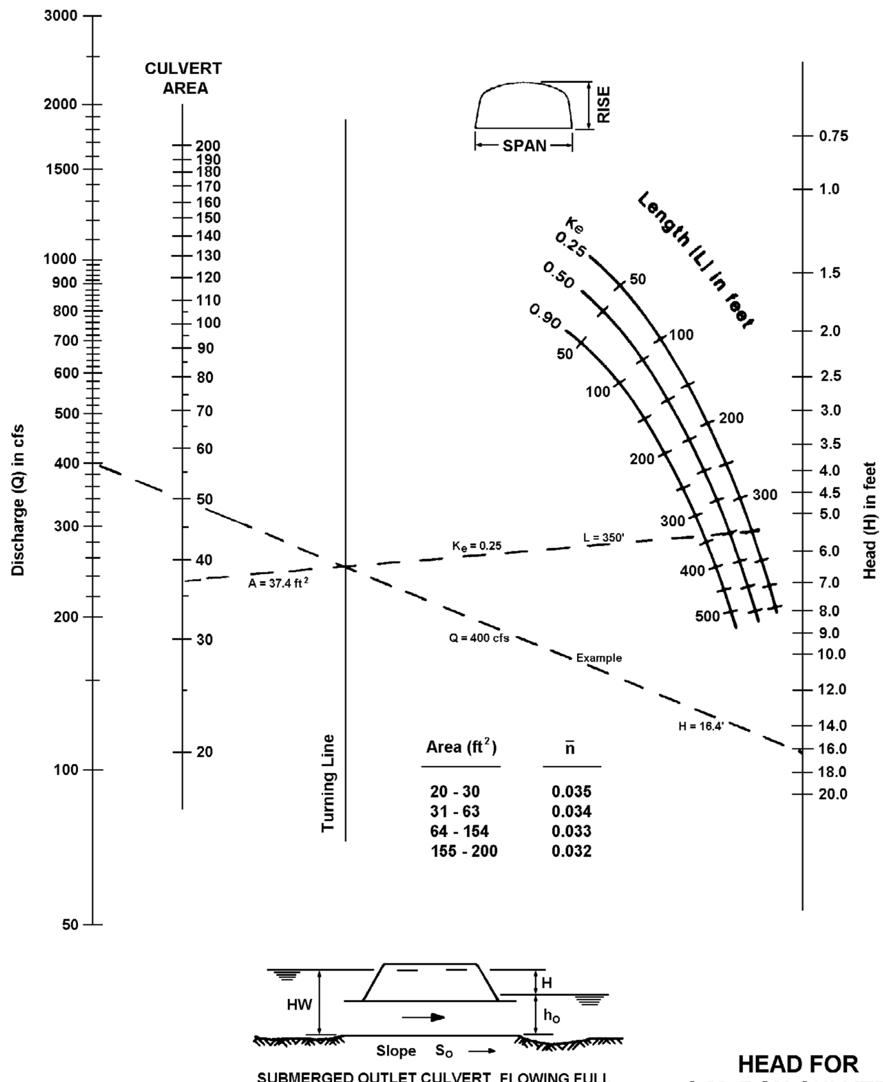



Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

**HEAD FOR C.M. BOX CULVERTS FLOWING FULL CORRUGATED METAL BOTTOM 0.3 < RISE/SPAN**

BACK TO TOC

## CHART 26

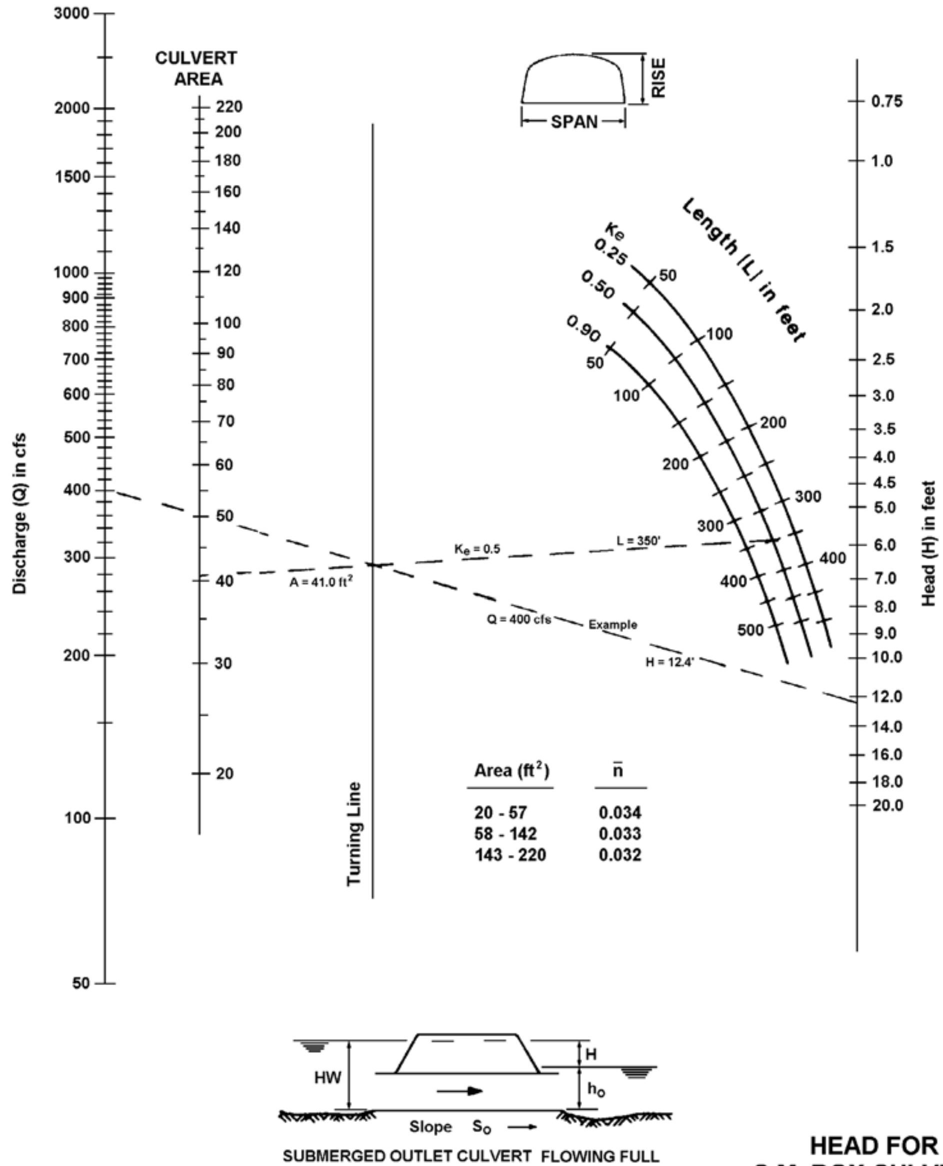



| Area ($ft^2$) | $\bar{n}$ |
|---|---|
| 20 - 30 | 0.035 |
| 31 - 63 | 0.034 |
| 64 - 154 | 0.033 |
| 155 - 200 | 0.032 |

**HEAD FOR**
**C.M. BOX CULVERTS**
**FLOWING FULL**
**CORRUGATED METAL BOTTOM**
**$0.4 \leq$ RISE/SPAN < 0.5**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

## CHART 27

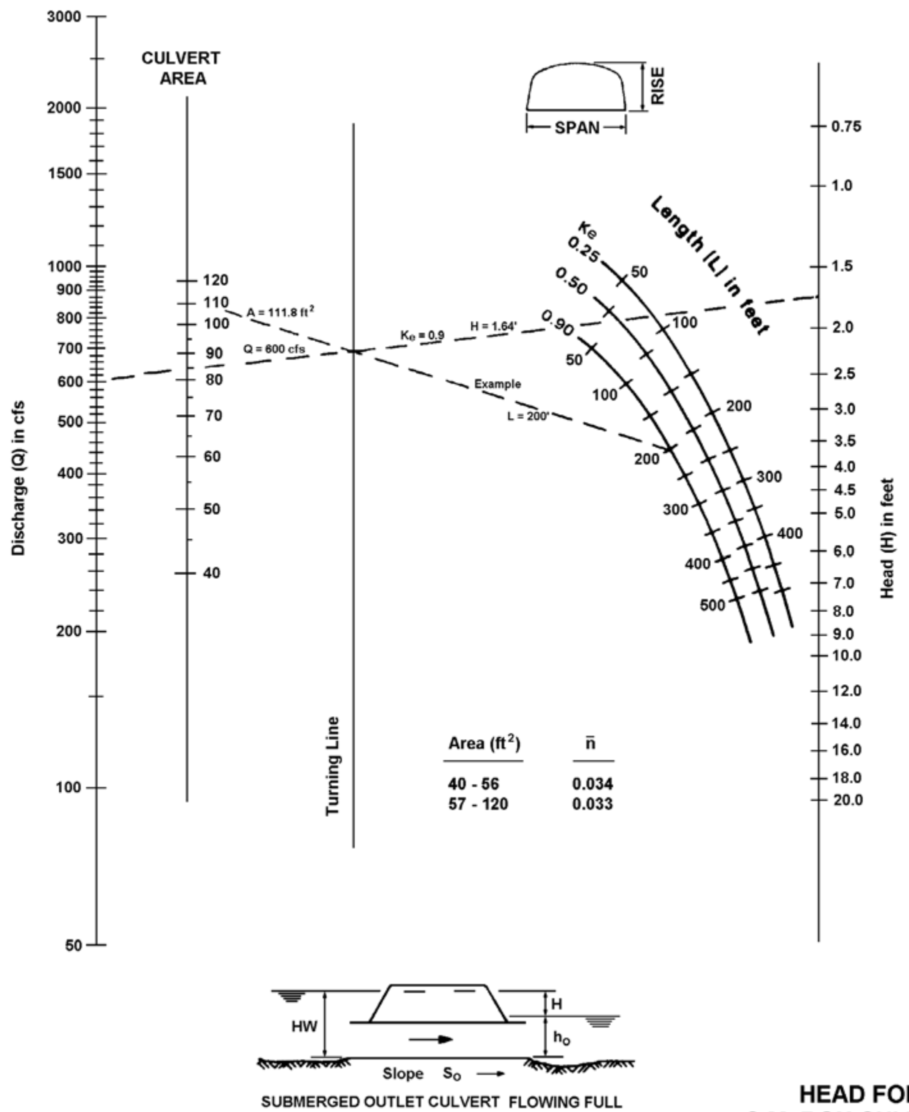



| Area (ft²) | n̄ |
|---|---|
| 20 - 57 | 0.034 |
| 58 - 142 | 0.033 |
| 143 - 220 | 0.032 |

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

**HEAD FOR**
**C.M. BOX CULVERTS**
**FLOWING FULL**
**CORRUGATED METAL BOTTOM**
**$0.4 \leq$ RISE/SPAN $< 0.5$**

## CHART 28



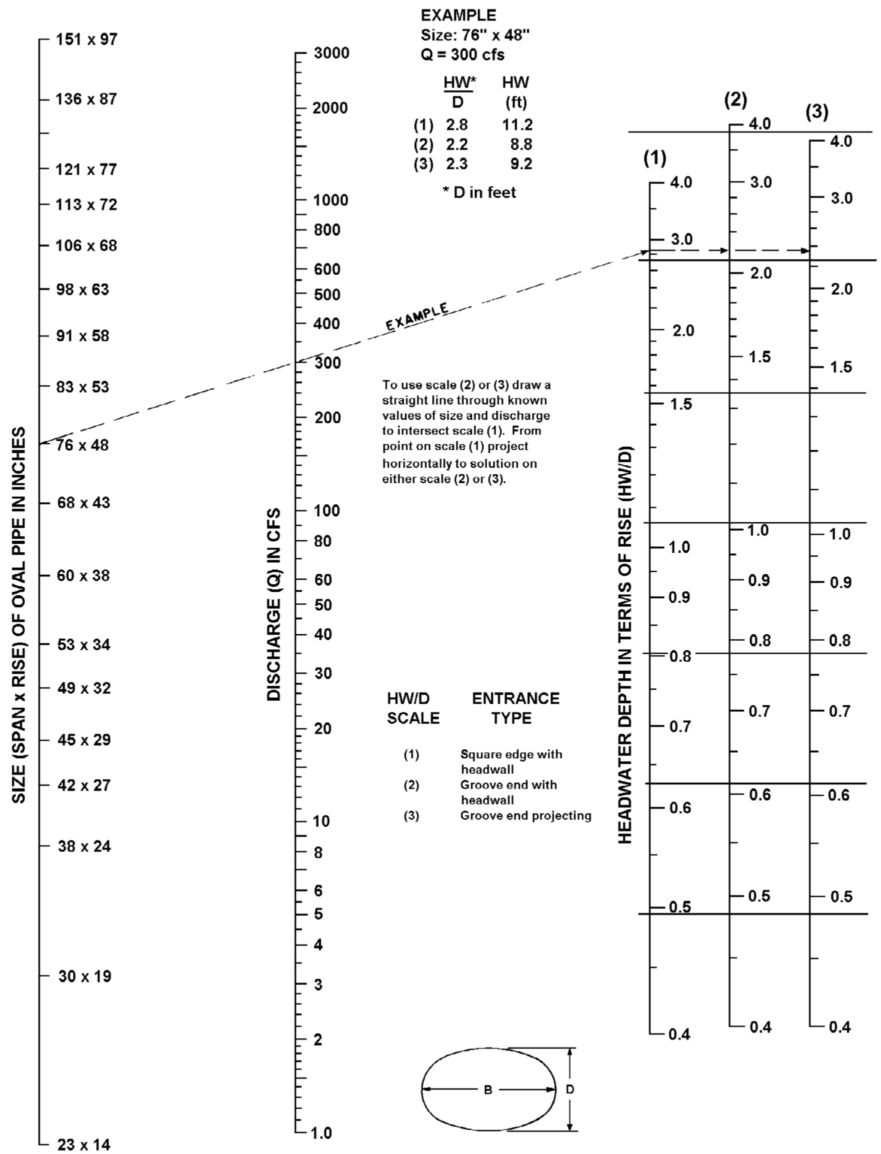

| Area ($ft^2$) | $\bar{n}$ |
|---|---|
| 40 - 56 | 0.034 |
| 57 - 120 | 0.033 |

**HEAD FOR
C.M. BOX CULVERTS
FLOWING FULL
CORRUGATED METAL BOTTOM
0.5 ≤ RISE/SPAN**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

BACK TO TOC

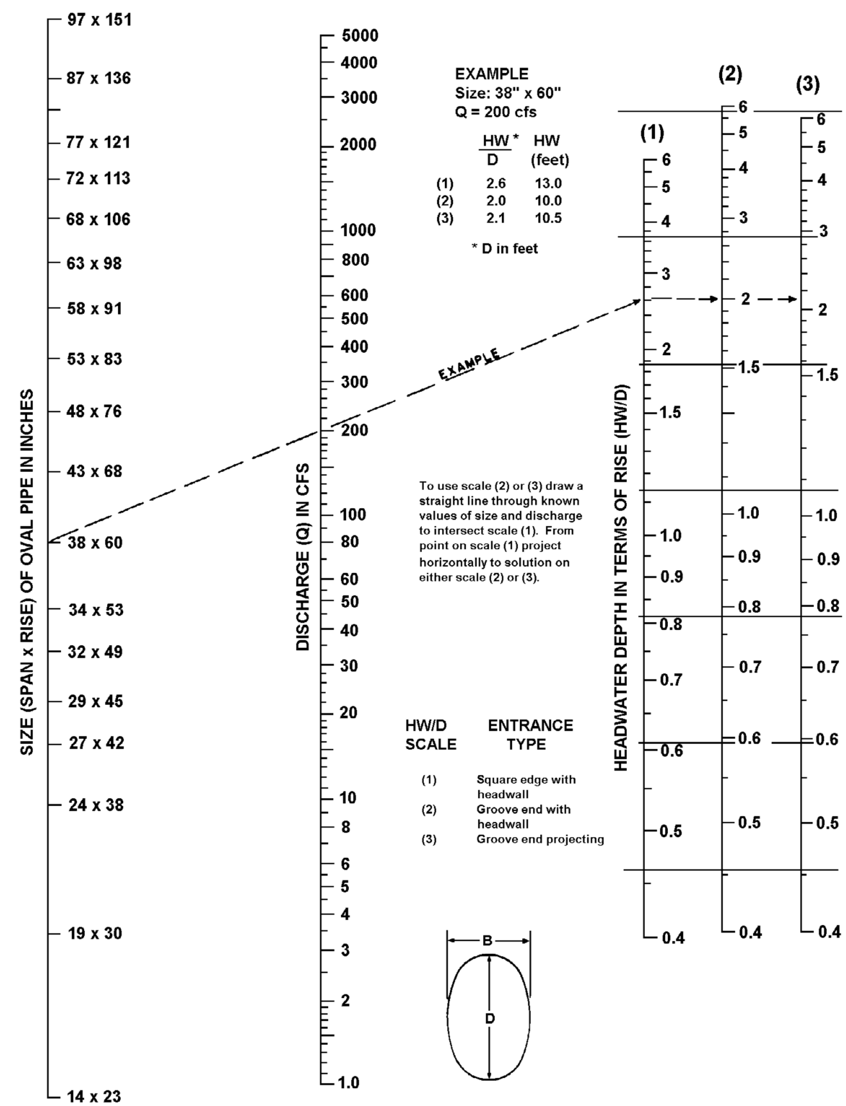
# CHART 29




## HEADWATER DEPTH FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

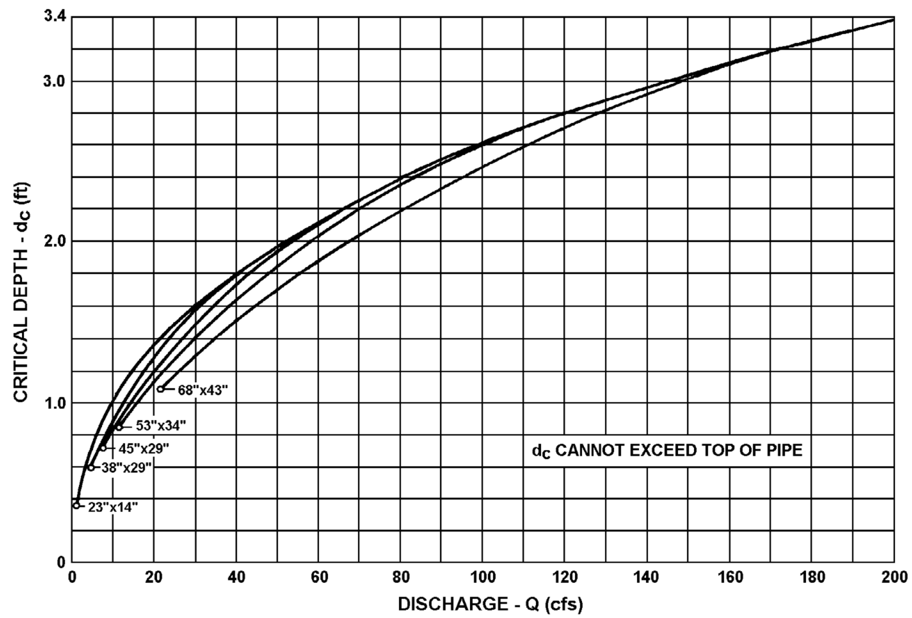
## CHART 30

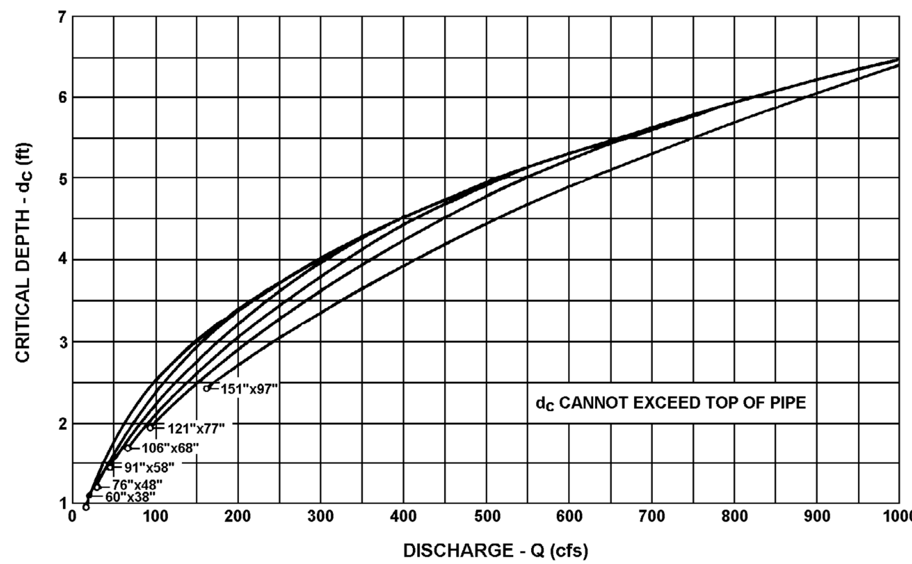



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS VERTICAL
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

# CHART 31



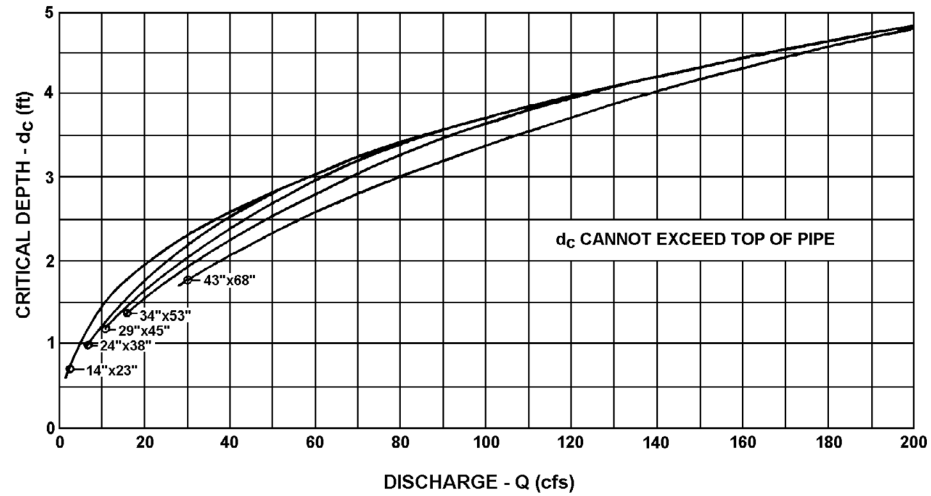



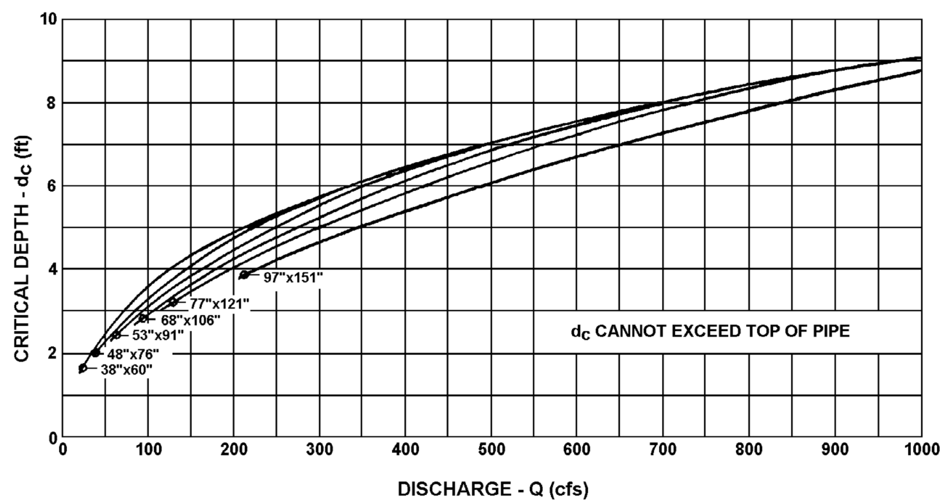

BUREAU OF PUBLIC ROADS JAN. 1964

**CRITICAL DEPTH**
**OVAL CONCRETE PIPE**
**LONG AXIS HORIZONTAL**

BACK TO TOC

## CHART 32



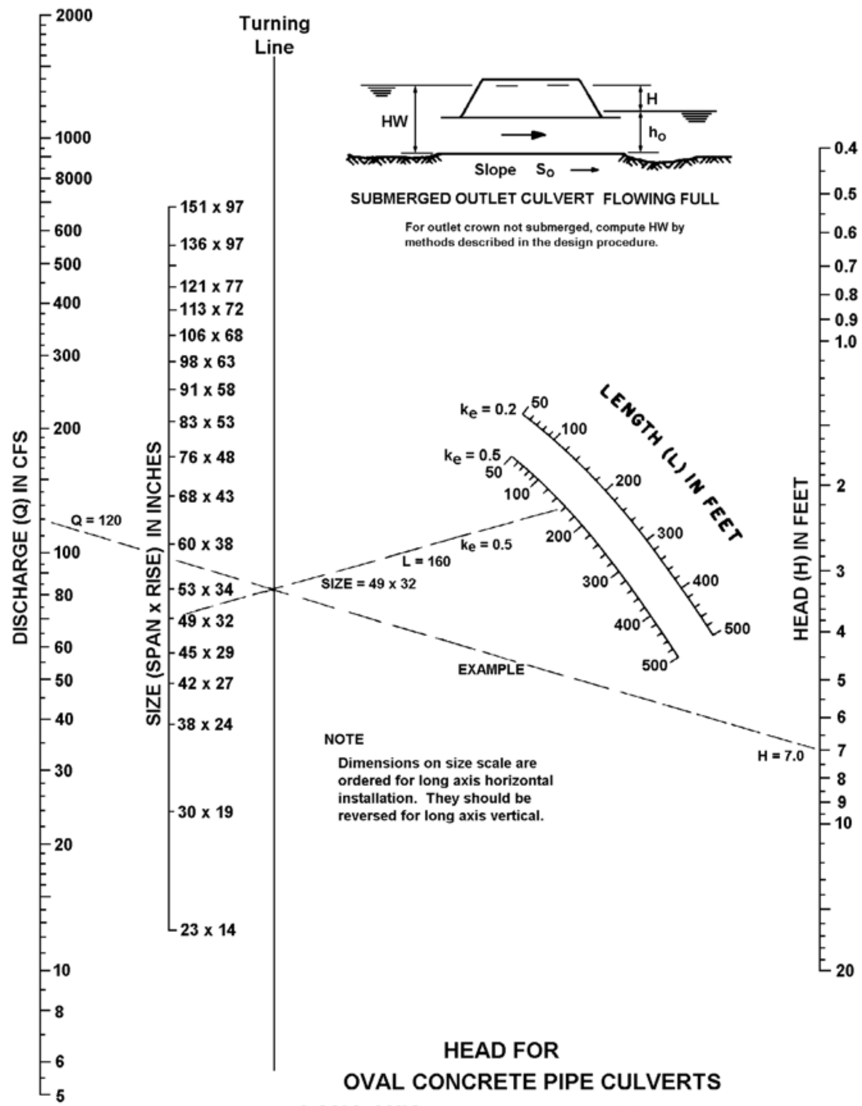

BUREAU OF PUBLIC ROADS JAN. 1964

**CRITICAL DEPTH**
**OVAL CONCRETE PIPE**
**LONG AXIS VERTICAL**



BACK TO TOC

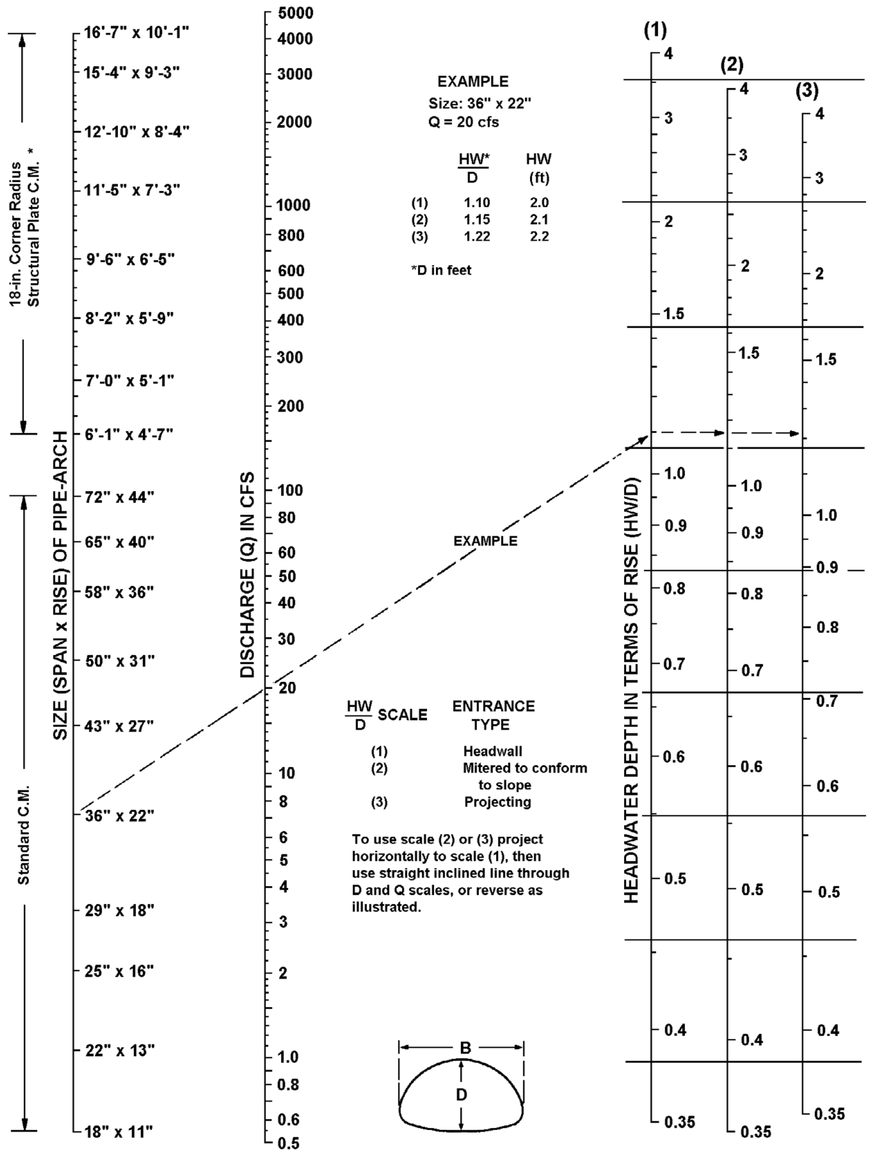
CHART 33




HEAD FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL OR VERTICAL
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

## CHART 34




* Additional sizes not dimensioned are listed in fabricator's catalog.

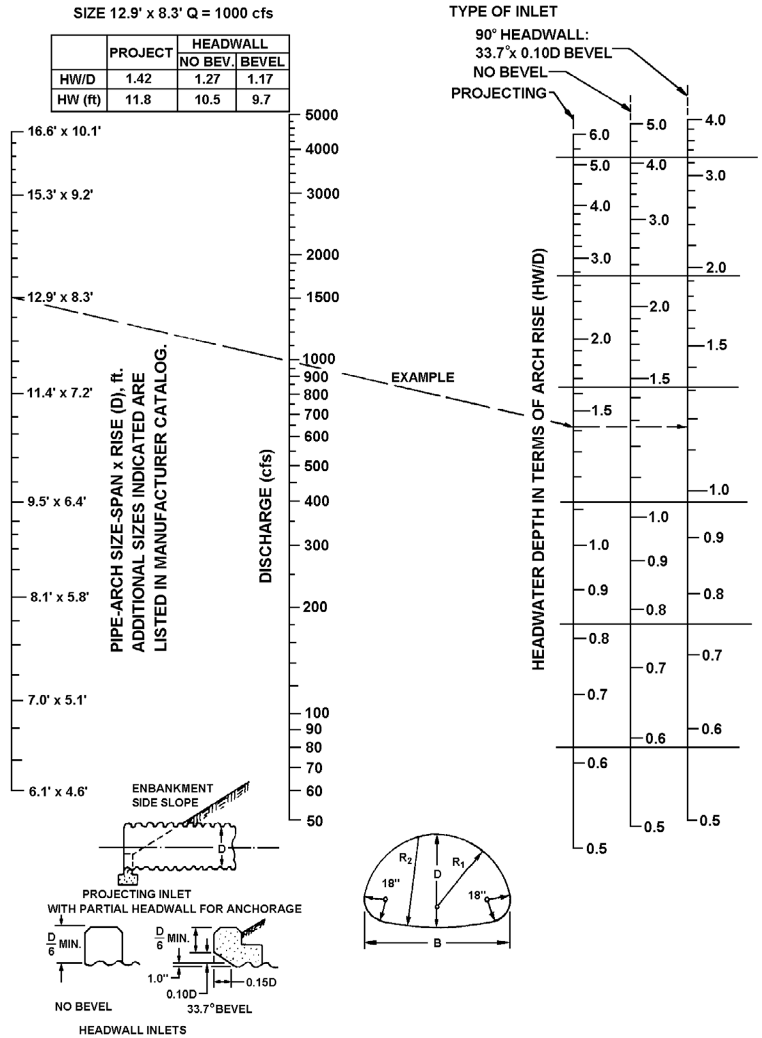
HEADWATER DEPTH FOR
C.M. PIPE-ARCH CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

## CHART 35

EXAMPLE
SIZE 12.9' x 8.3' Q = 1000 cfs

| | PROJECT | HEADWALL | |
|---|---|---|---|
| | | NO BEV. | BEVEL |
| HW/D | 1.42 | 1.27 | 1.17 |
| HW (ft) | 11.8 | 10.5 | 9.7 |




BUREAU OF PUBLIC ROADS
OFFICE OF R&D JULY 1968

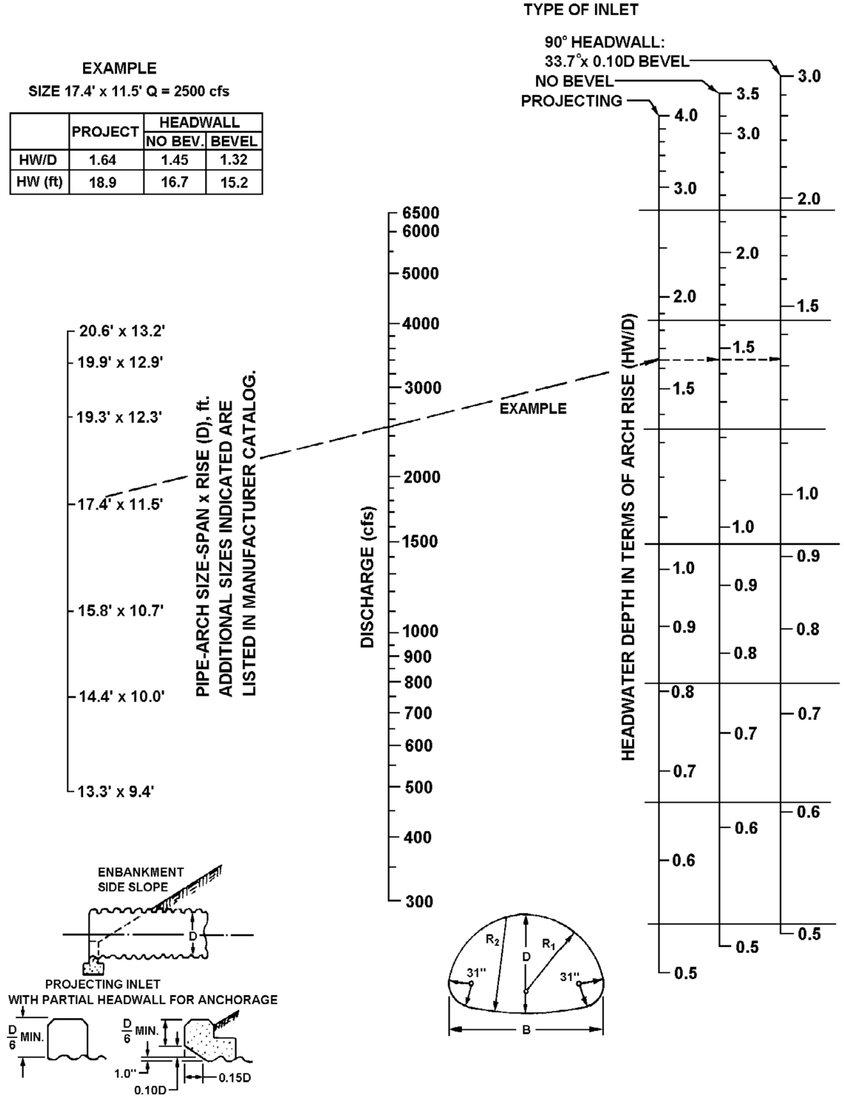
HEADWATER DEPTH FOR INLET CONTROLS
STRUCTURAL PLATE PIPE-ARCH CULVERTS
18 in. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

## CHART 36

EXAMPLE

SIZE 17.4' x 11.5' Q = 2500 cfs

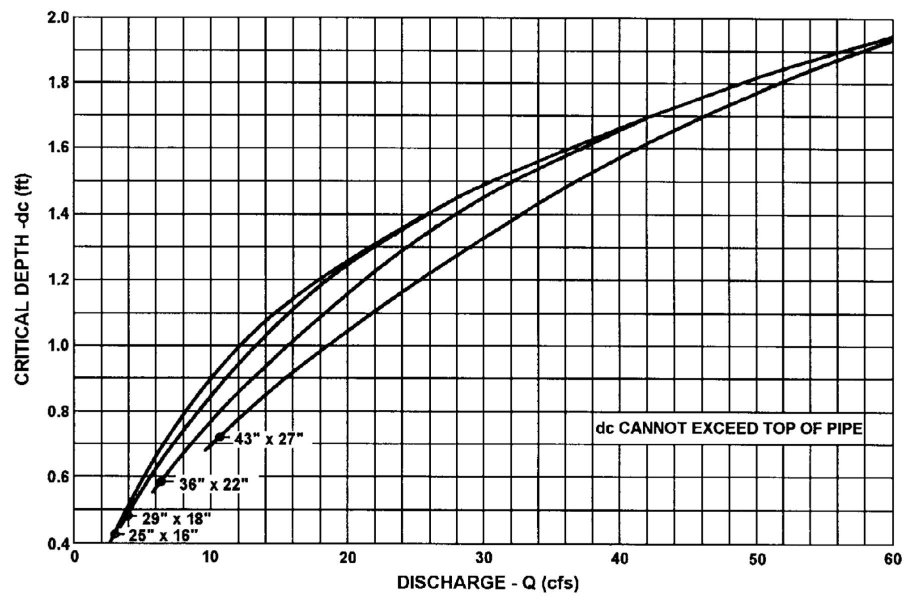
| | PROJECT | HEADWALL | |
|---|---|---|---|
| | | NO BEV. | BEVEL |
| HW/D | 1.64 | 1.45 | 1.32 |
| HW (ft) | 18.9 | 16.7 | 15.2 |

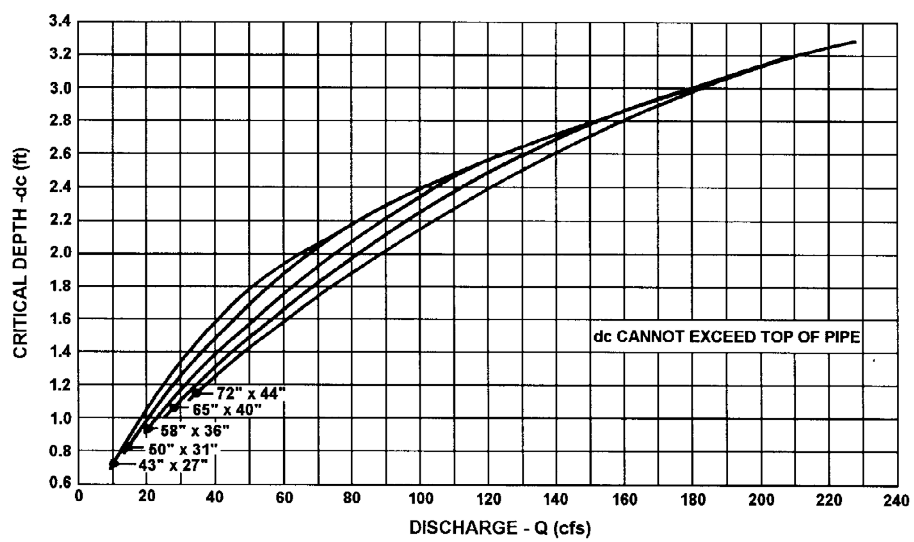



BUREAU OF PUBLIC ROADS
OFFICE OF R&D JULY 1966

HEADWATER DEPTH FOR INLET CONTROL
STRUCTURAL PLATE PIPE-ARCH CULVERTS
31 in. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

## CHART 37

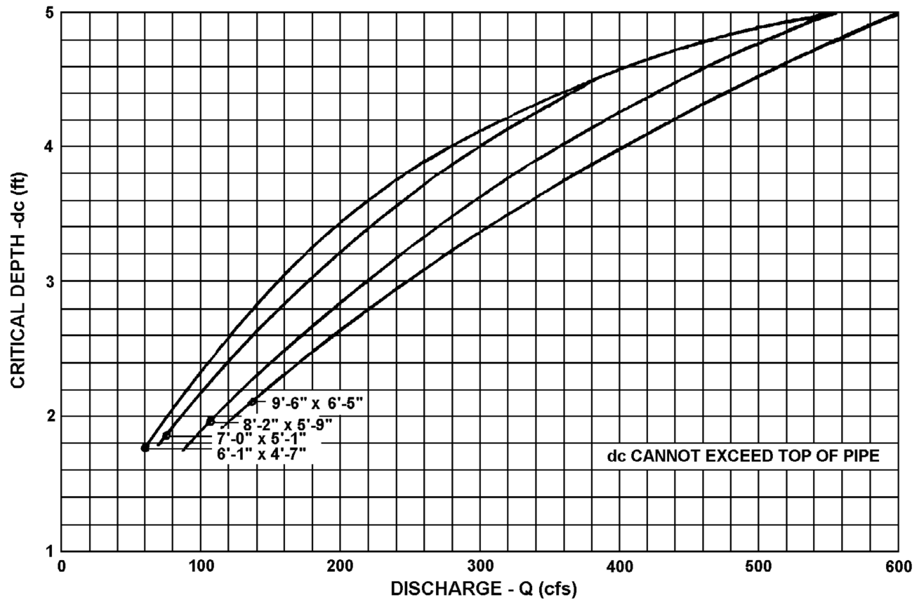


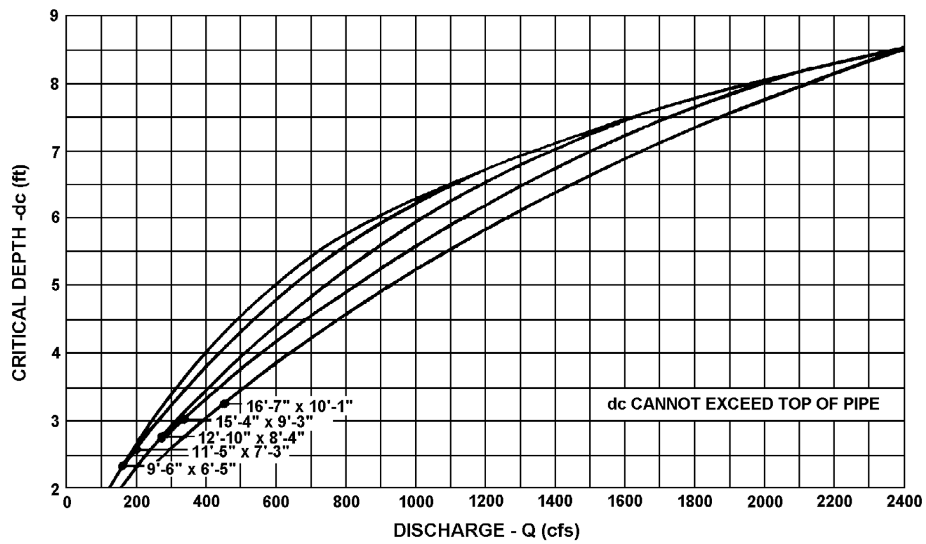




BUREAU OF PUBLIC ROADS
JAN. 1964

**CRITICAL DEPTH**
**STANDARD C.M. PIPE ARCH**

# CHART 38



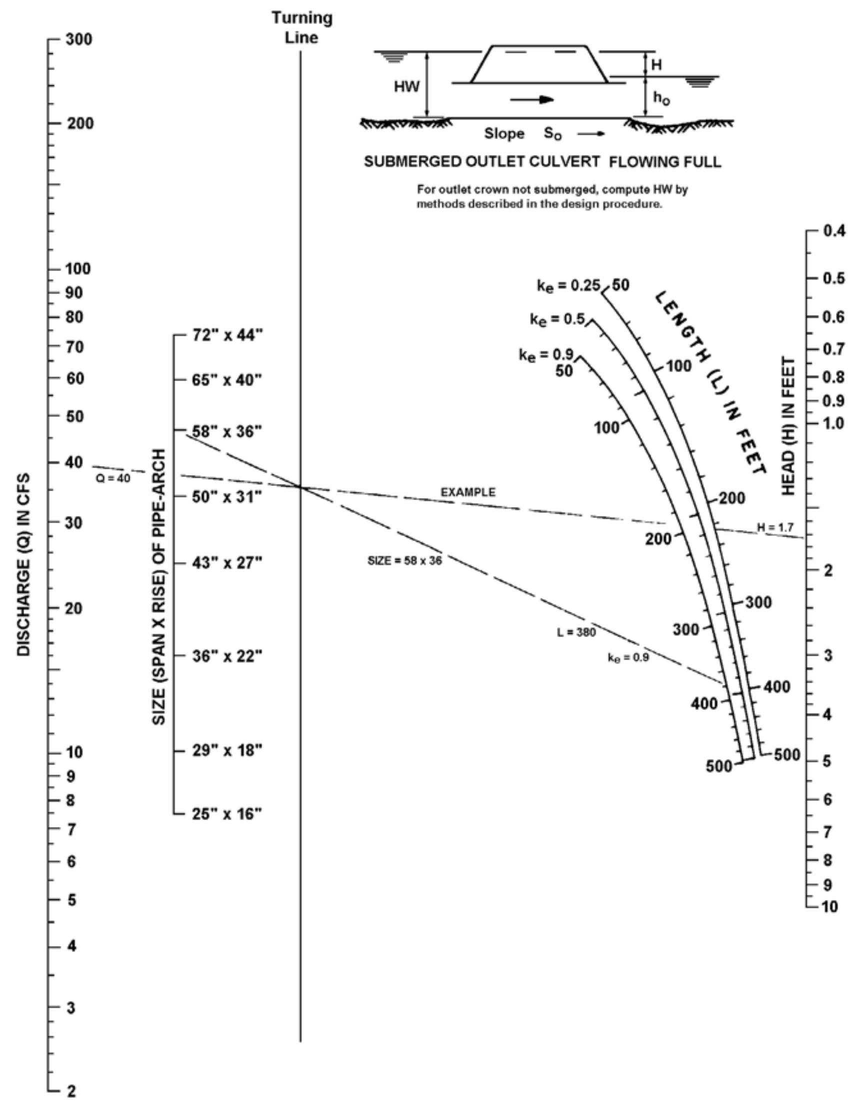

BUREAU OF PUBLIC ROADS
JAN. 1964

**CRITICAL DEPTH**
**STRUCTURAL PLATE**
**C.M. PIPE ARCH**
**18 in. CORNER RADIUS**



BACK TO TOC

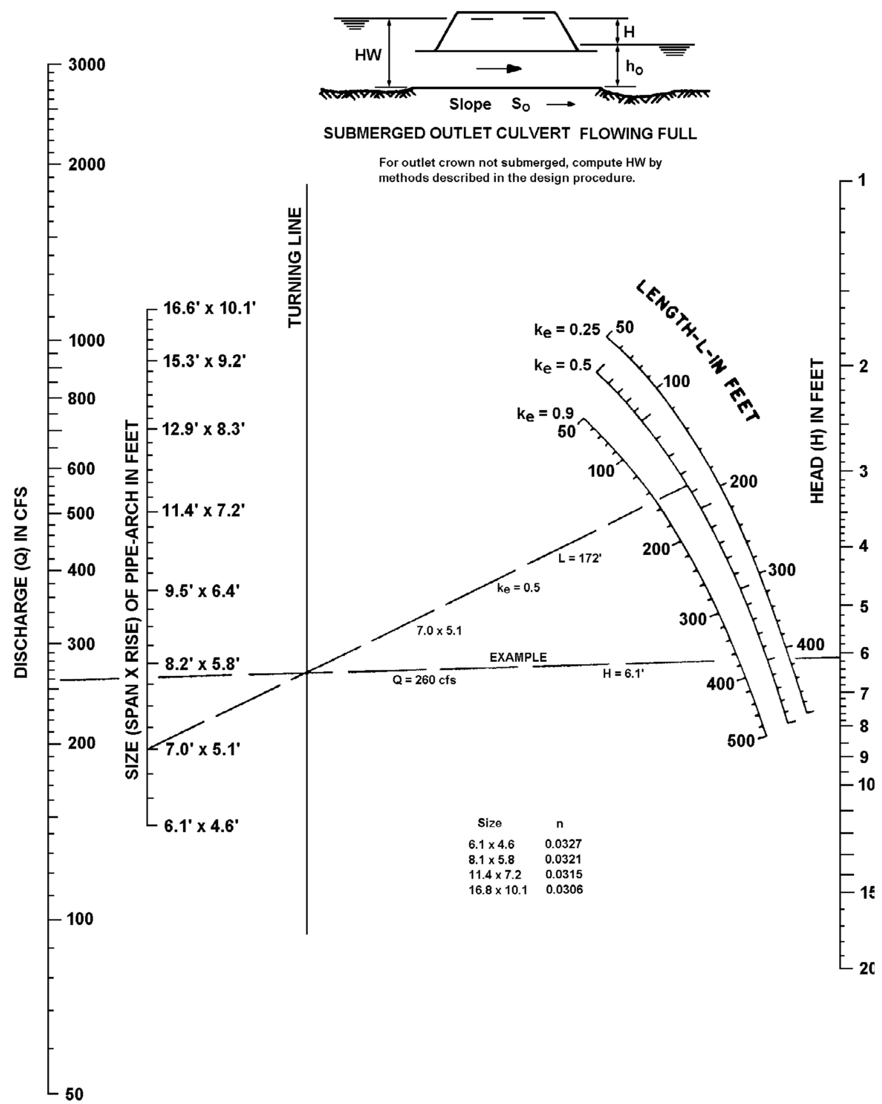
## CHART 39




BUREAU OF PUBLIC ROADS JAN. 1963

**HEAD FOR STANDARD C.M. PIPE-ARCH CULVERTS FLOWING FULL**
**n = 0.024**

## CHART 40

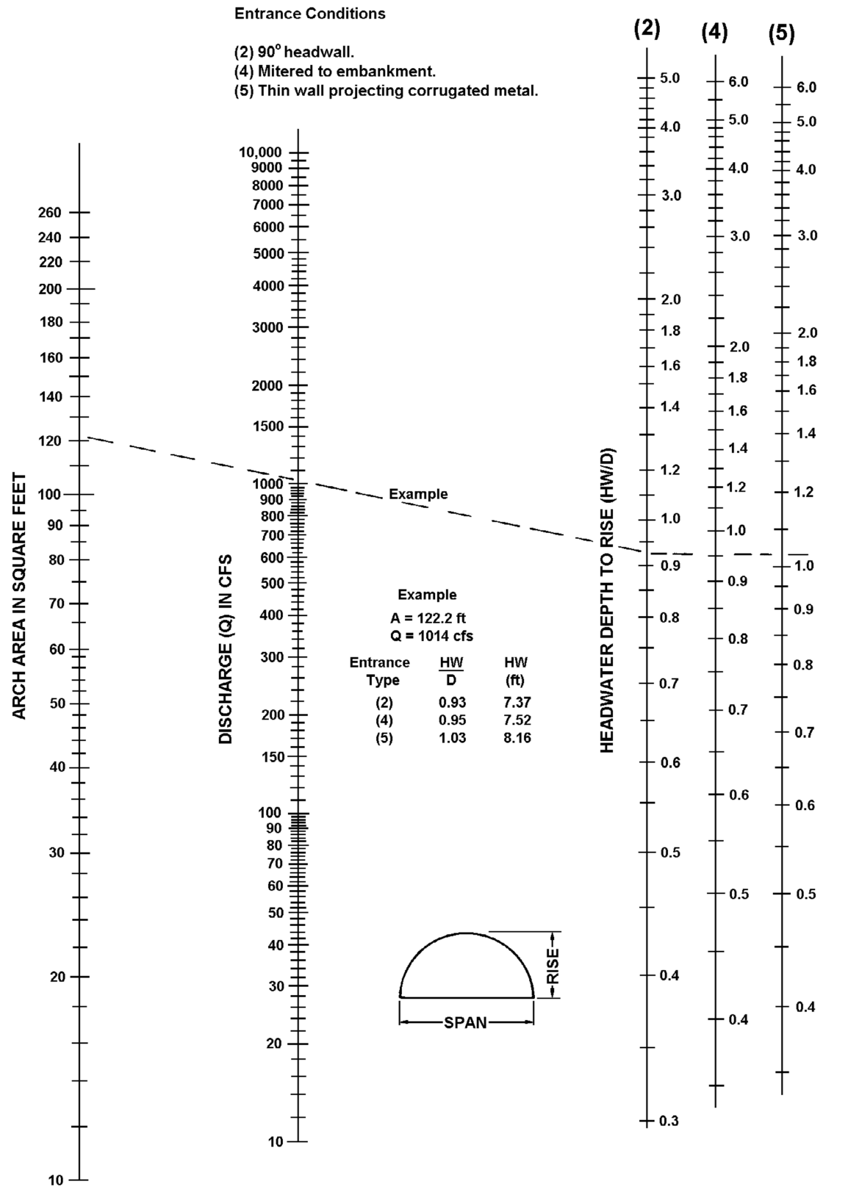



| Size | n |
|---|---|
| 6.1 x 4.6 | 0.0327 |
| 8.1 x 5.8 | 0.0321 |
| 11.4 x 7.2 | 0.0315 |
| 16.8 x 10.1 | 0.0306 |

BUREAU OF PUBLIC ROADS JAN. 1963

**HEAD FOR STRUCTURAL PLATE C.M. PIPE ARCH CULVERTS 18 in. CORNER RADIUS FLOWING FULL n = 0.0327 TO 0.0306**

## CHART 41

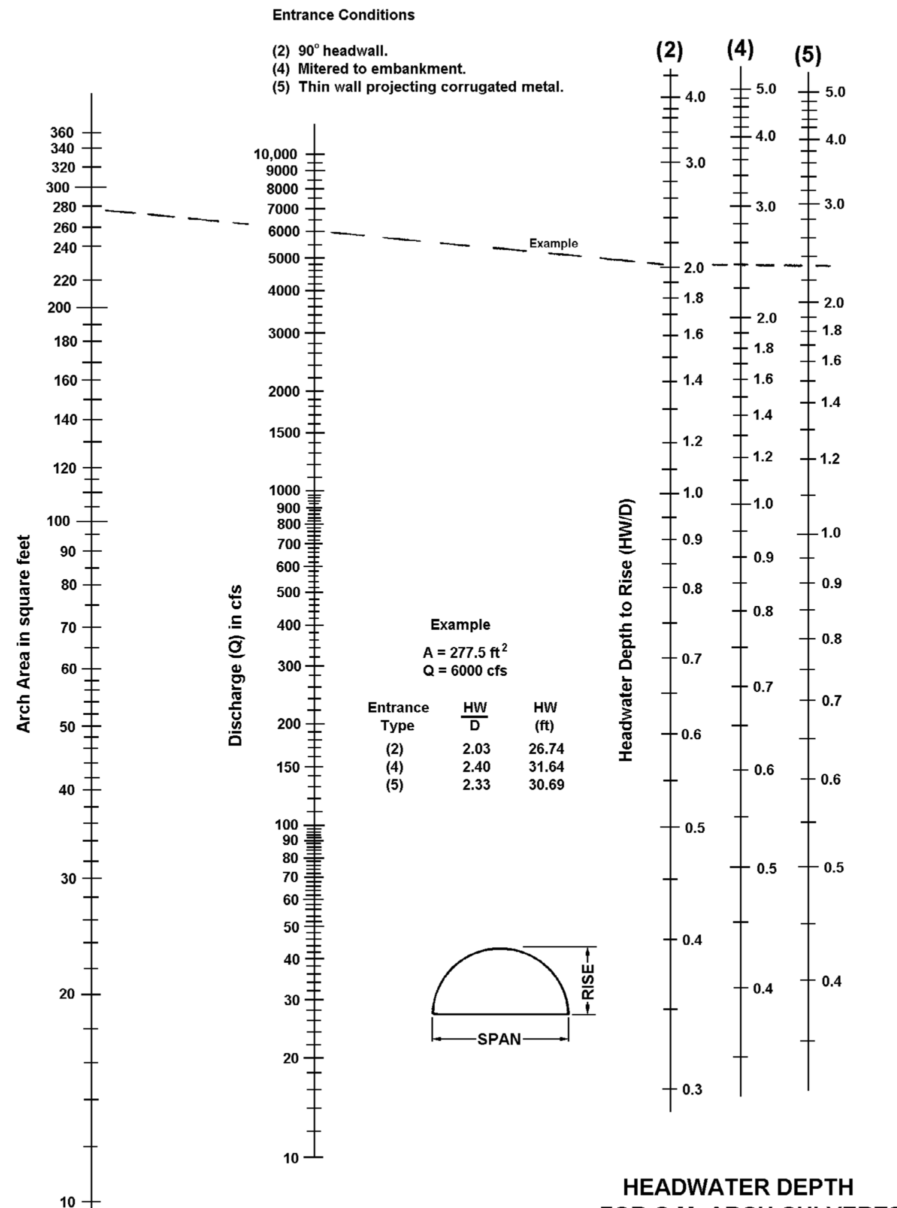



**HEADWATER DEPTH**
**FOR C.M. ARCH CULVERTS**
**0.3 ≤ RISE/SPAN < 0.4**
**WITH INLET CONTROL**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

BACK TO TOC

# CHART 42

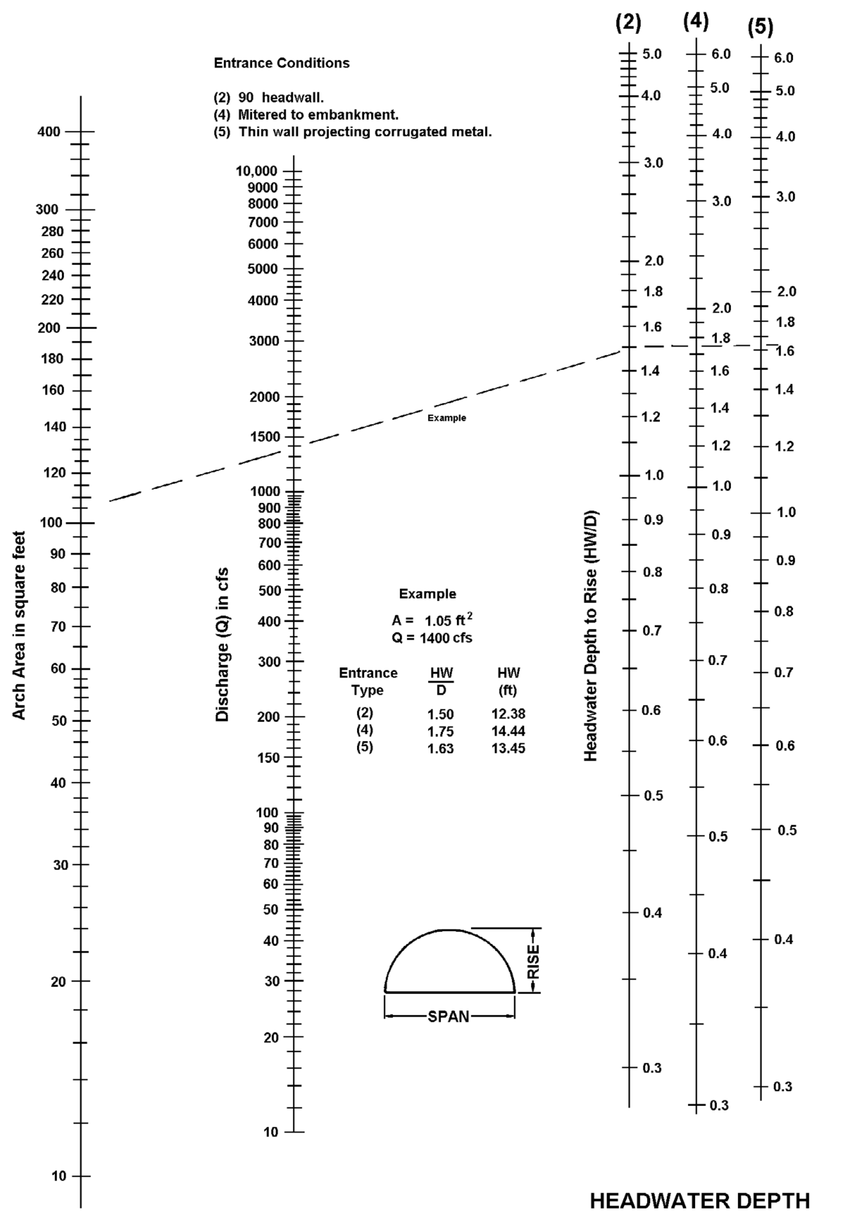



HEADWATER DEPTH
FOR C.M. ARCH CULVERTS
0.4 ≤ RISE/SPAN < 0.5
WITH INLET CONTROL

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

BACK TO TOC

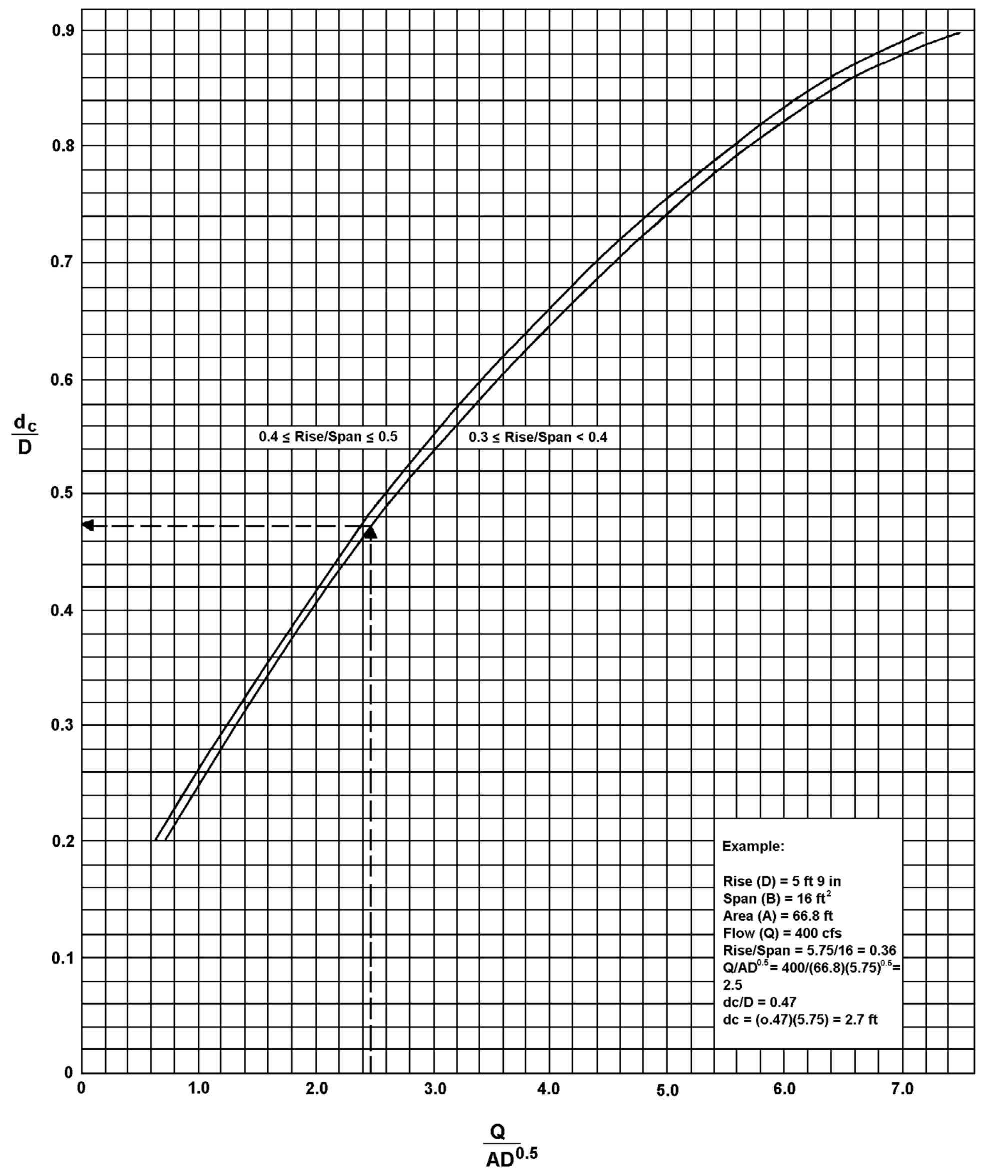
# CHART 43




HEADWATER DEPTH
FOR C.M. ARCH CULVERTS
0.5 ≤ RISE/SPAN
WITH INLET CONTROL

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

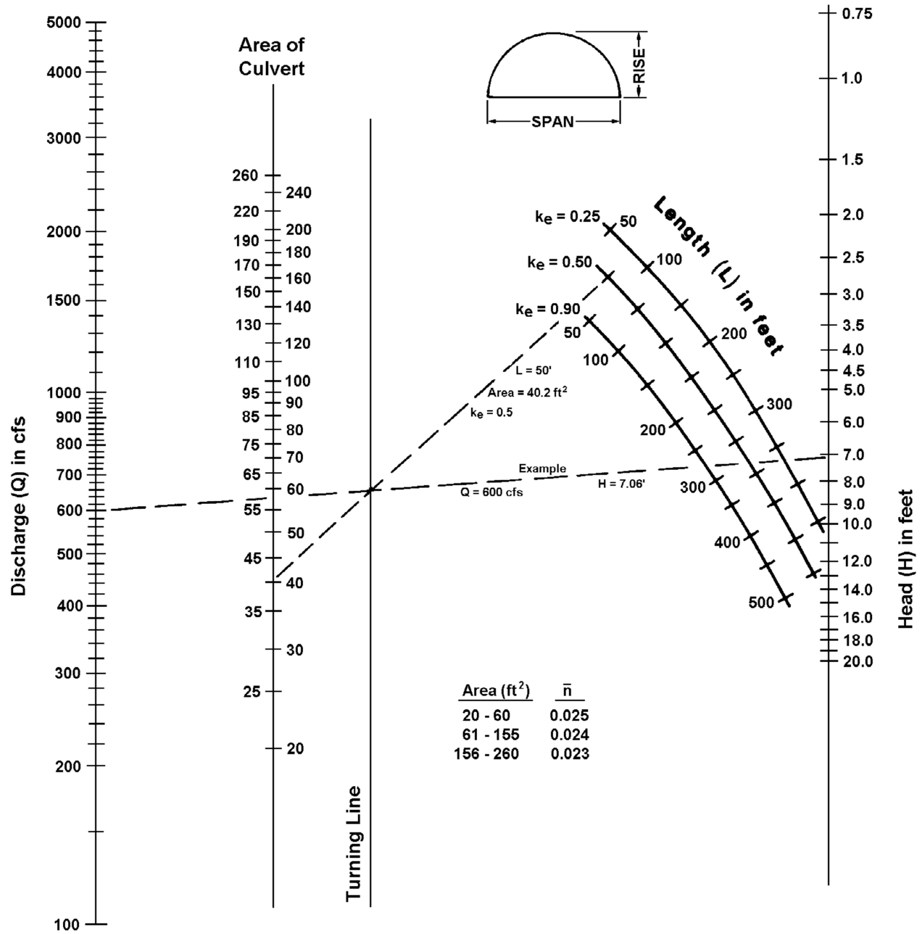
## CHART 44



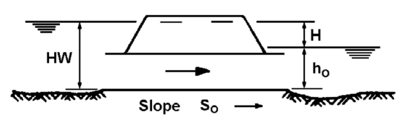

DIMENSIONLESS CRITICAL DEPTH CHART
FOR C.M. ARCH CULVERTS

BACK TO TOC

## CHART 45




| Area ($ft^2$) | $\bar{n}$ |
|---|---|
| 20 - 60 | 0.025 |
| 61 - 155 | 0.024 |
| 156 - 260 | 0.023 |



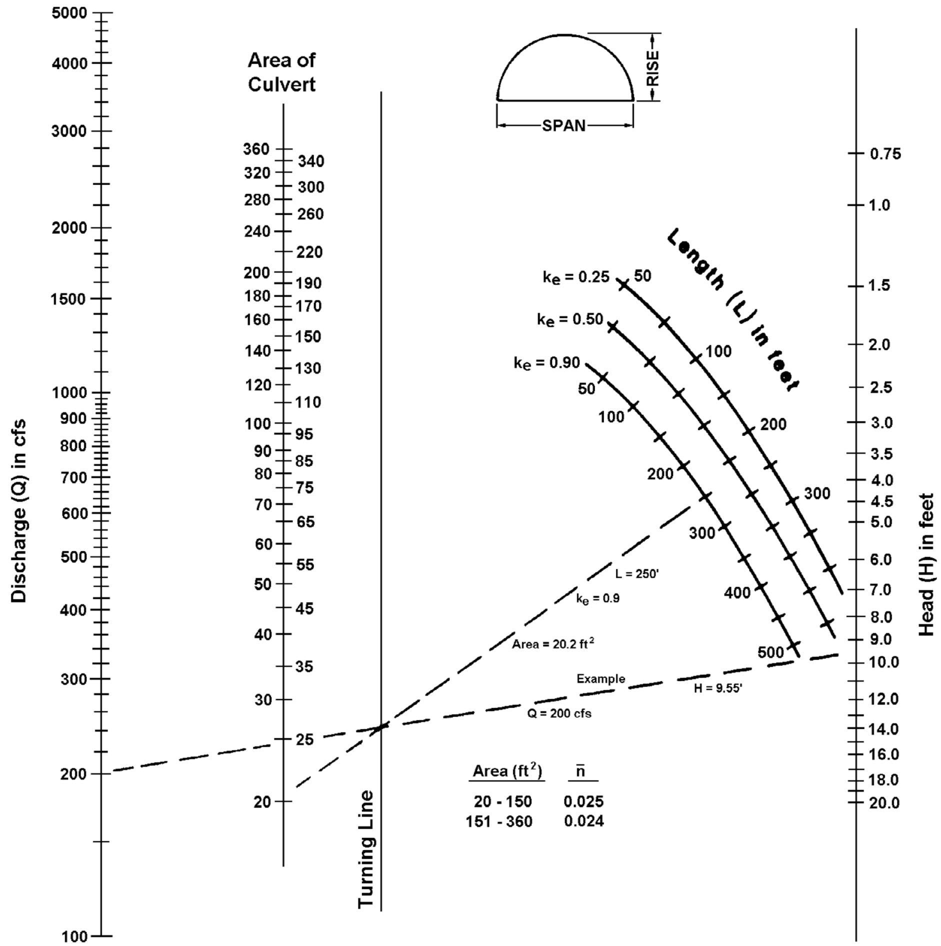
SUBMERGED OUTLET CULVERT FLOWING FULL

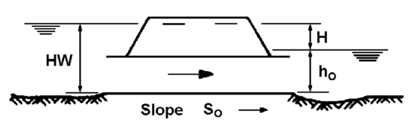
**HEAD FOR**
**C.M. ARCH CULVERTS**
**FLOWING FULL**
**CONCRETE BOTTOM**
**$0.3 \le$ RISE/SPAN $< 0.4$**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

BACK TO TOC

## CHART 46



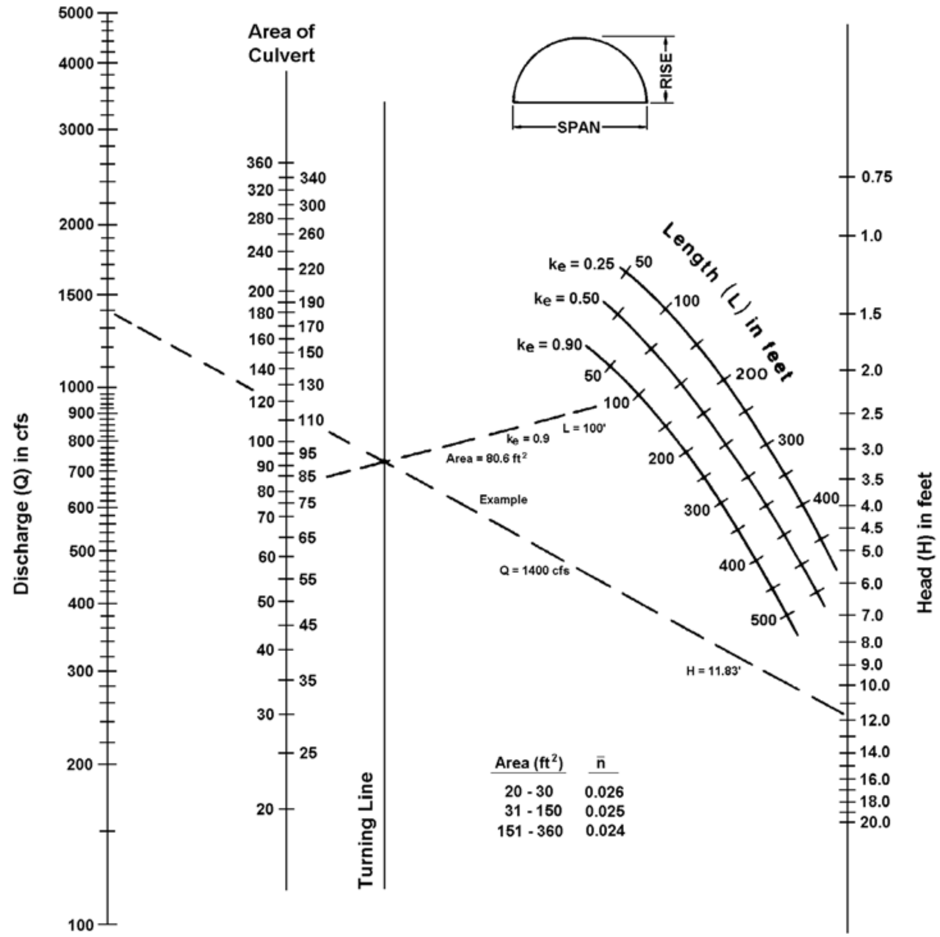



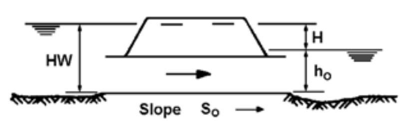

**HEAD FOR C.M. ARCH CULVERTS FLOWING FULL CONCRETE BOTTOM $0.4 \leq$ RISE/SPAN $< 0.5$**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.
Duplication of this nomograph may distort scale.

BACK TO TOC

# CHART 47



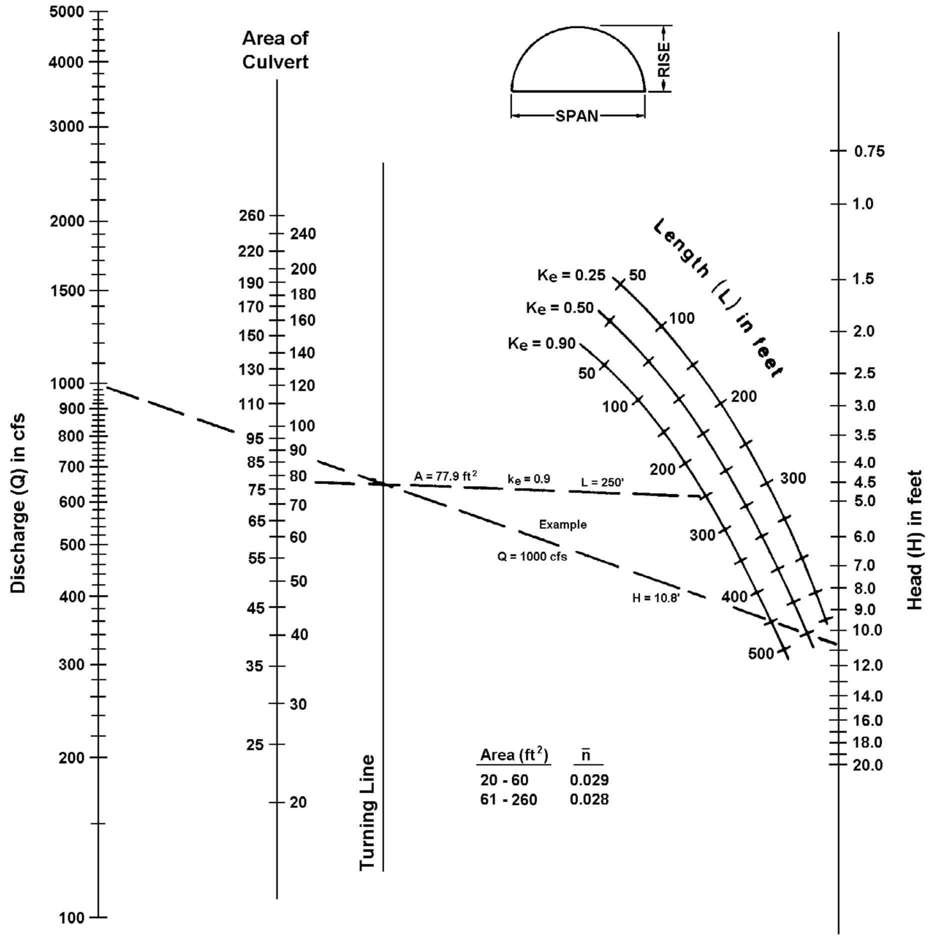




SUBMERGED OUTLET CULVERT FLOWING FULL

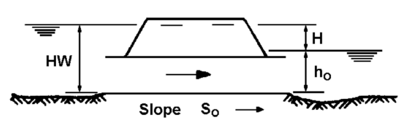
HEAD FOR
C.M. ARCH CULVERTS
FLOWING FULL
CONCRETE BOTTOM
0.5 ≤ RISE/SPAN

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

## CHART 48




| Area ($ft^2$) | $\bar{n}$ |
|---|---|
| 20 - 60 | 0.029 |
| 61 - 260 | 0.028 |



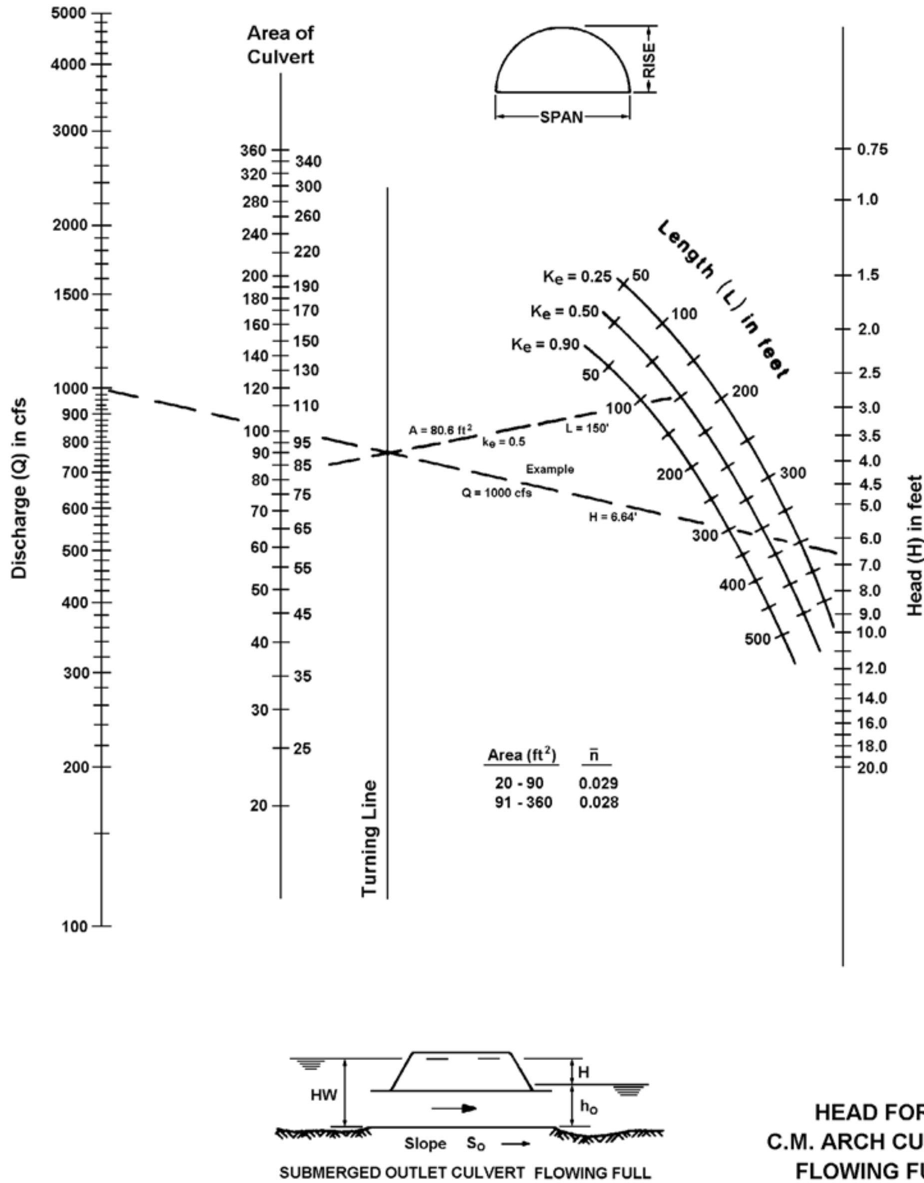

SUBMERGED OUTLET CULVERT FLOWING FULL

**HEAD FOR**
**C.M. ARCH CULVERTS**
**FLOWING FULL**
**EARTH BOTTOM ($n_b$ = 0.022)**
**0.3 ≤ RISE/SPAN < 0.4**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.
Duplication of this nomograph may distort scale.

BACK TO TOC

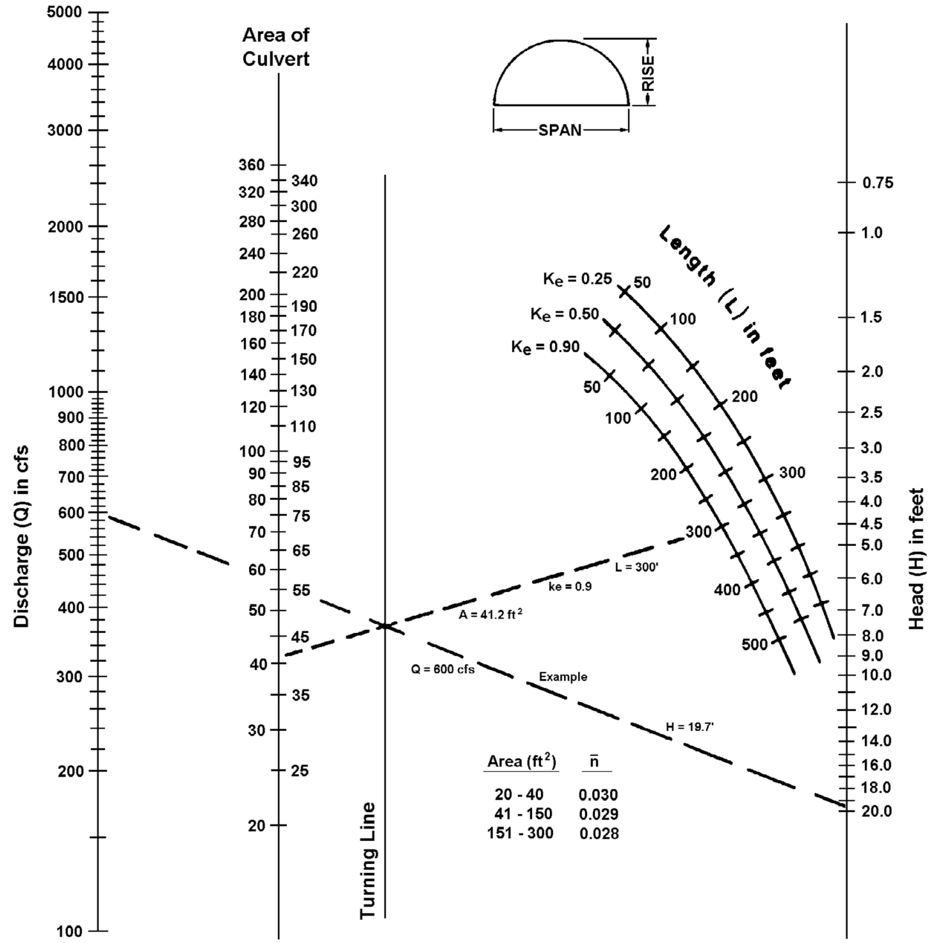
## CHART 49



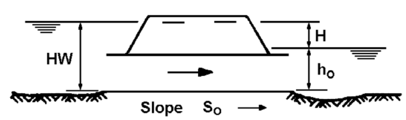

**HEAD FOR**
**C.M. ARCH CULVERTS**
**FLOWING FULL**
**EARTH BOTTOM ($n_b$ = 0.022)**
**0.4 ≤ RISE/SPAN < 0.5**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

## CHART 50




| Area (ft²) | n̄ |
|---|---|
| 20 - 40 | 0.030 |
| 41 - 150 | 0.029 |
| 151 - 300 | 0.028 |

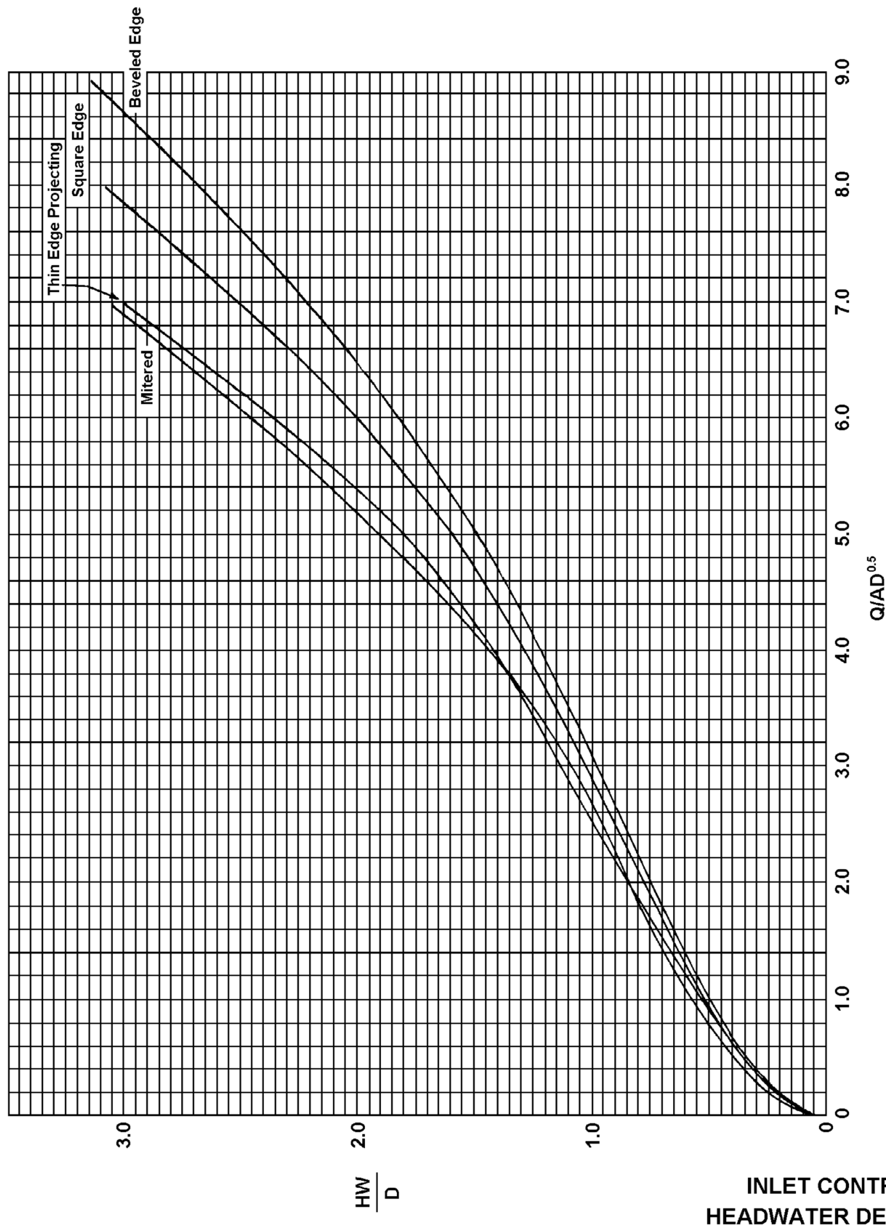



SUBMERGED OUTLET CULVERT FLOWING FULL

**HEAD FOR**
**C.M. ARCH CULVERTS**
**FLOWING FULL**
**EARTH BOTTOM ($n_b$ = 0.022)**
**0.5 ≤ RISE/SPAN**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.
Duplication of this nomograph may distort scale.

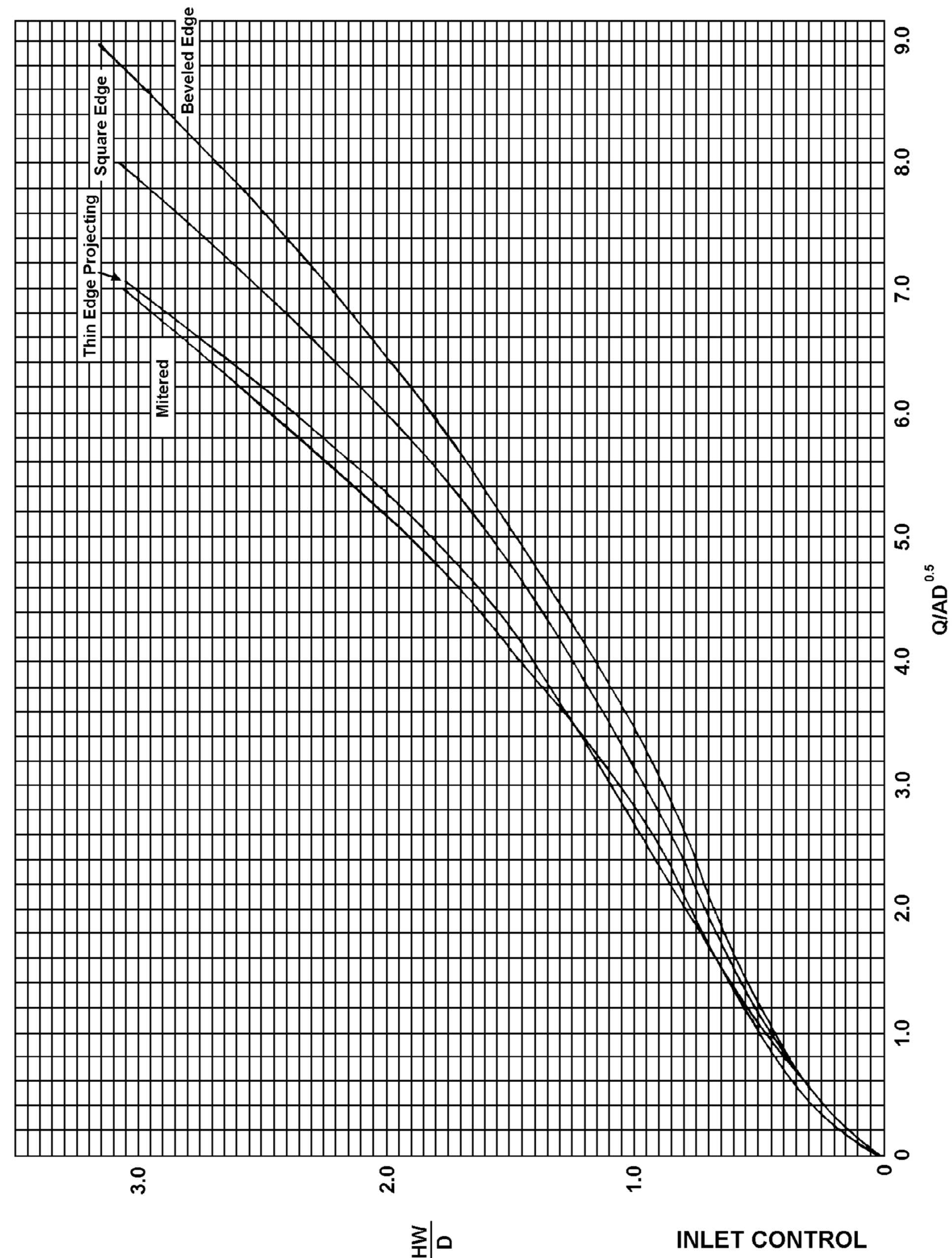
## CHART 51




**INLET CONTROL**
**HEADWATER DEPTH FOR**
**CIRCULAR OR ELLIPTICAL**
**STRUCTURAL PLATE**
**C.M. CONDUITS**

BACK TO TOC

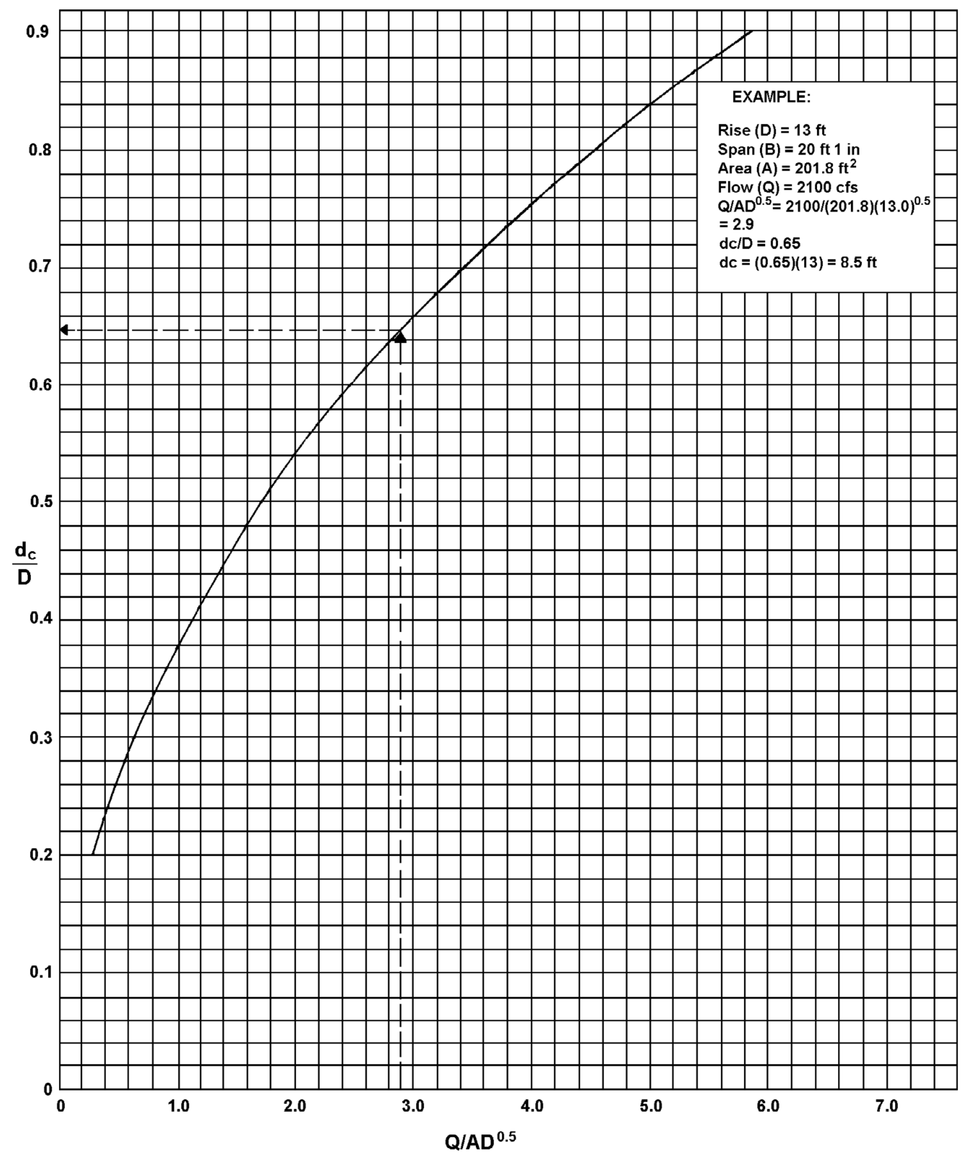
## CHART 52




**INLET CONTROL HEADWATER DEPTH FOR HIGH AND LOW PROFILE STRUCTURAL PLATE C.M. ARCH**

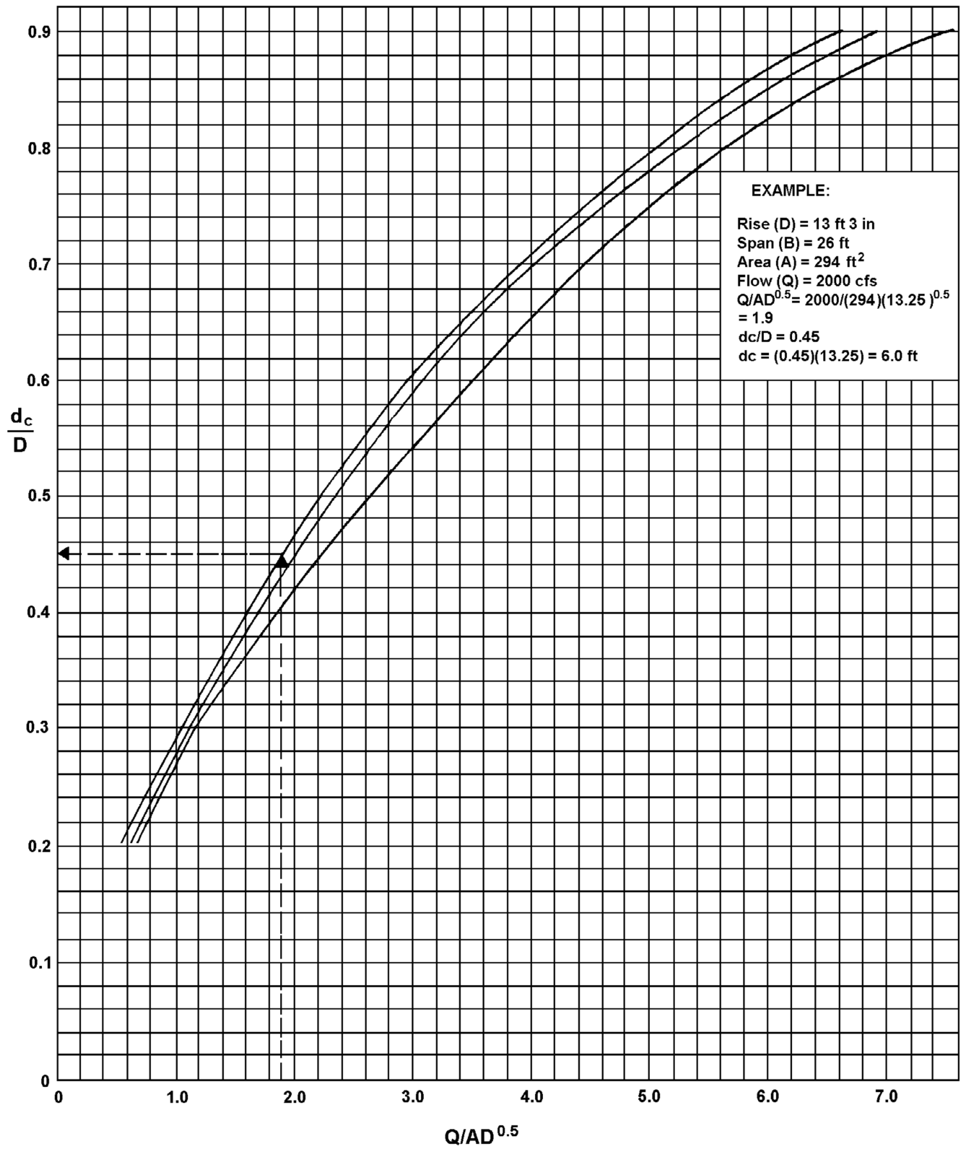
BACK TO TOC

## CHART 53




**DIMENSIONLESS CRITICAL DEPTH CHART
FOR STRUCTURAL PLATE
ELLIPSE LONG AXIS HORIZONTAL**

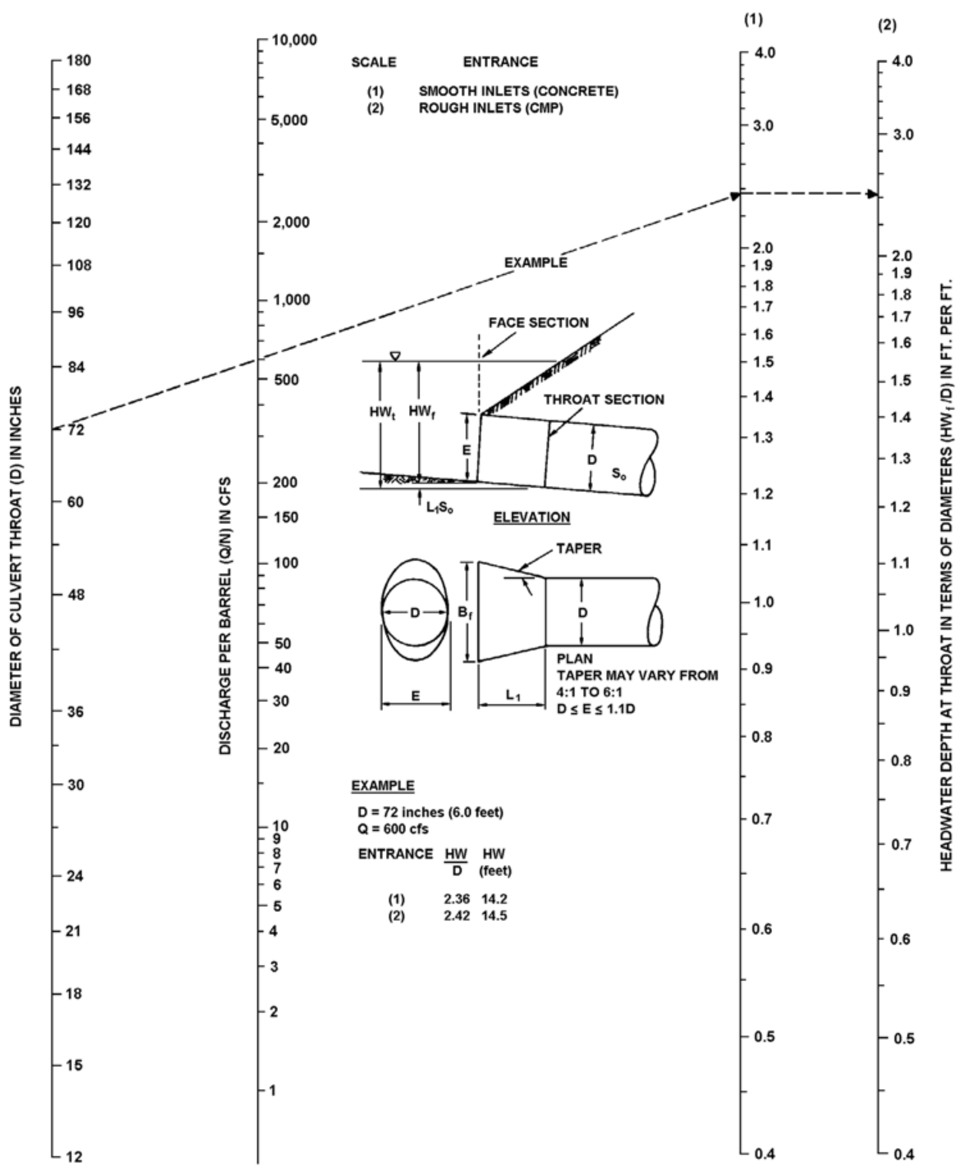
## CHART 54




DIMENSIONLESS CRITICAL DEPTH CHART
FOR STRUCTURAL PLATE
LOW- AND HIGH-PROFILE ARCHES

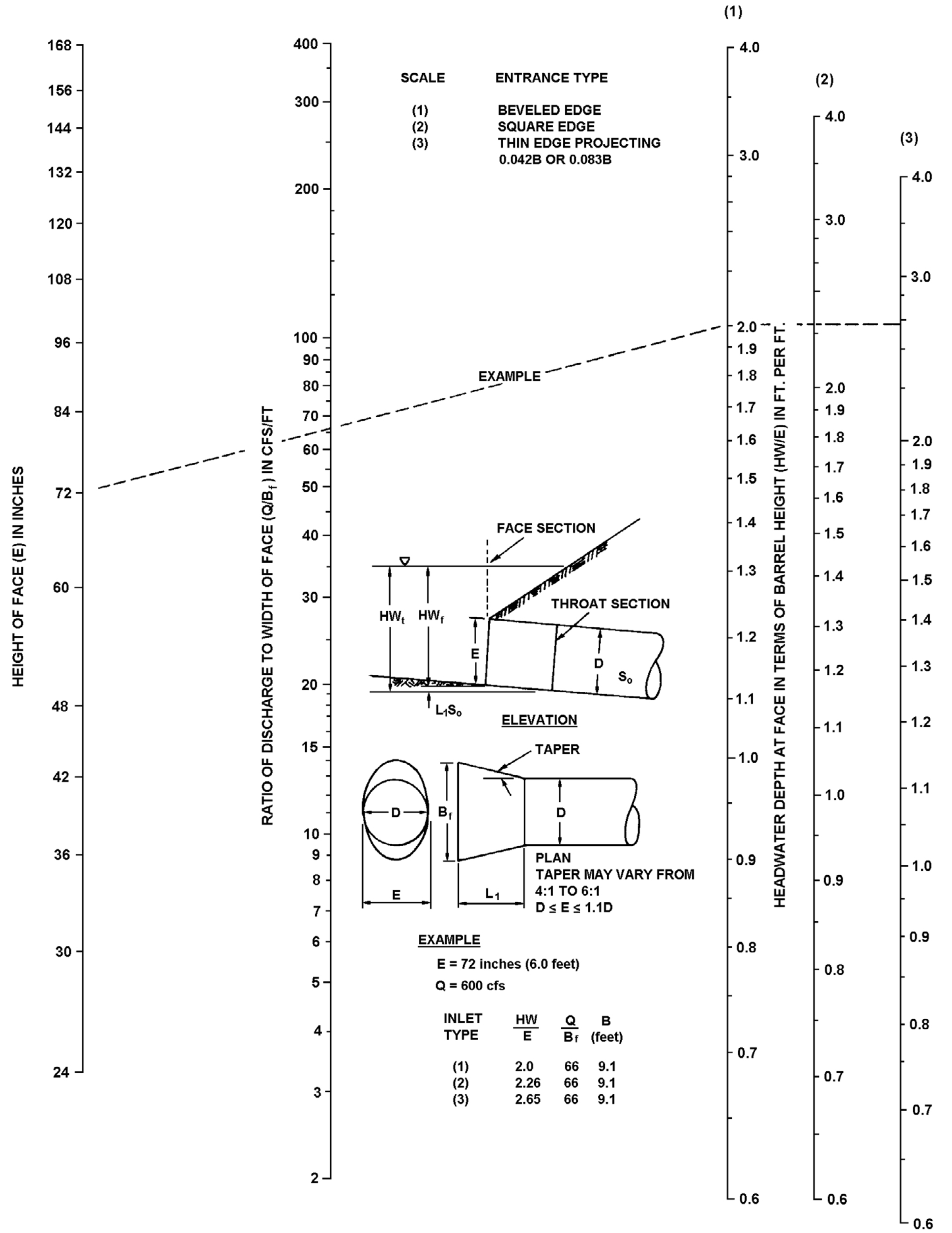
BACK TO TOC

## CHART 55




THROAT CONTROL
FOR SIDE-TAPERED INLETS
TO PIPE CULVERT
(CIRCULAR SECTION ONLY)

# CHART 56




EXAMPLE

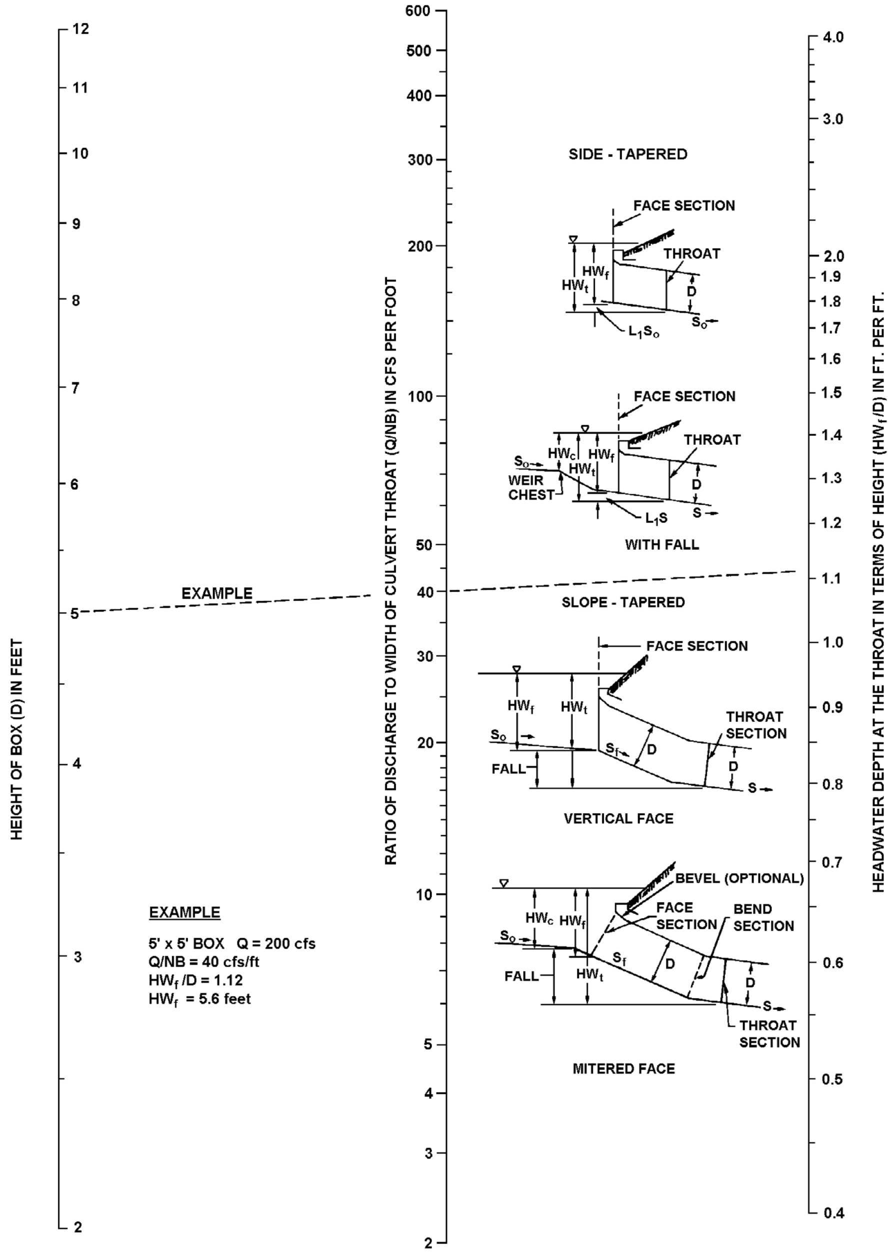
E = 72 inches (6.0 feet)

Q = 600 cfs

| INLET TYPE | $\frac{HW}{E}$ | $\frac{Q}{B_f}$ | B (feet) |
|---|---|---|---|
| (1) | 2.0 | 66 | 9.1 |
| (2) | 2.26 | 66 | 9.1 |
| (3) | 2.65 | 66 | 9.1 |

FACE CONTROL FOR SIDE-TAPERED
INLETS TO PIPE CULVERTS
(NON-RECTANGULAR SECTIONS ONLY)

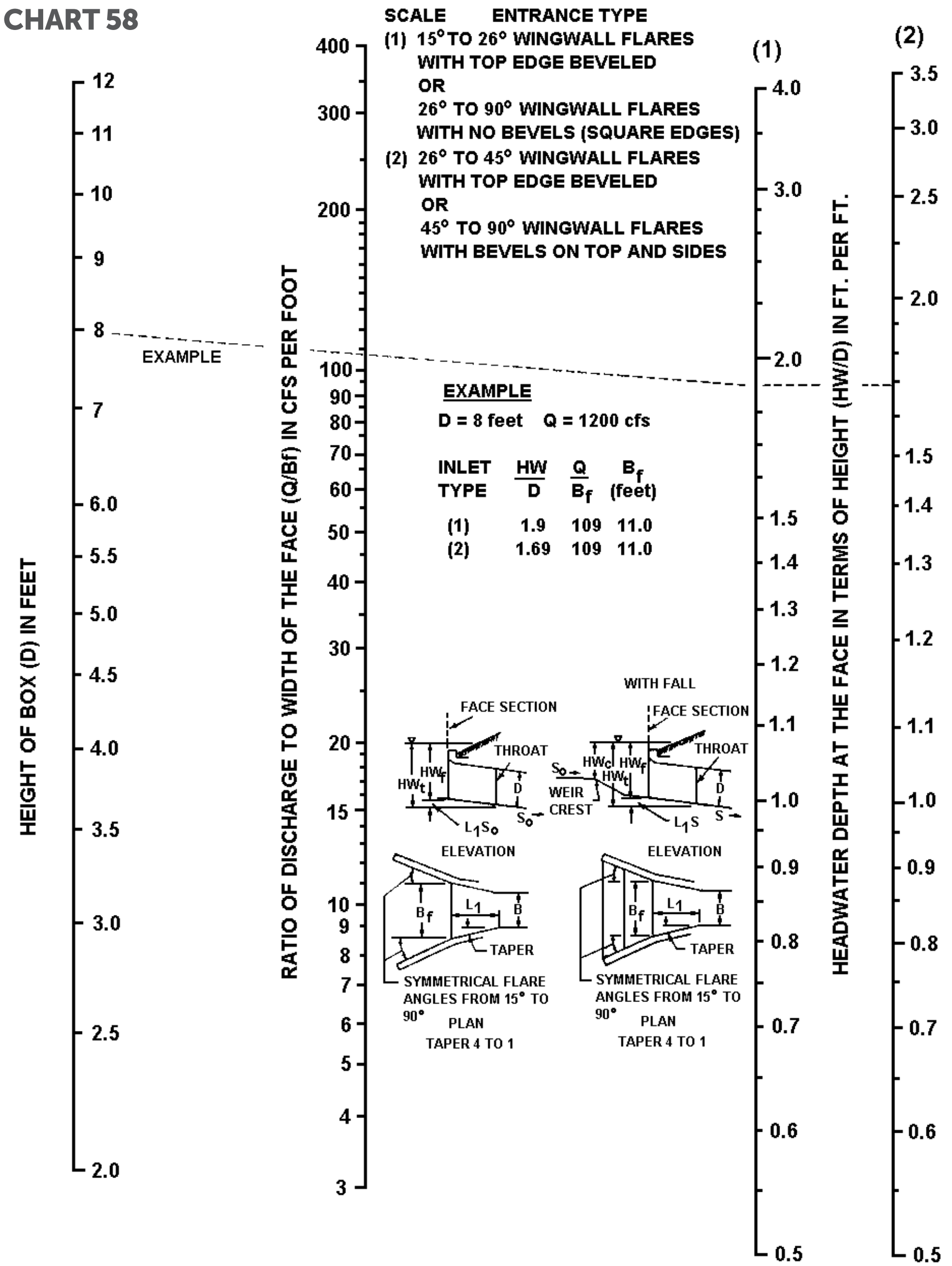
## CHART 57




**THROAT CONTROL FOR BOX CULVERTS WITH TAPERED INLETS**

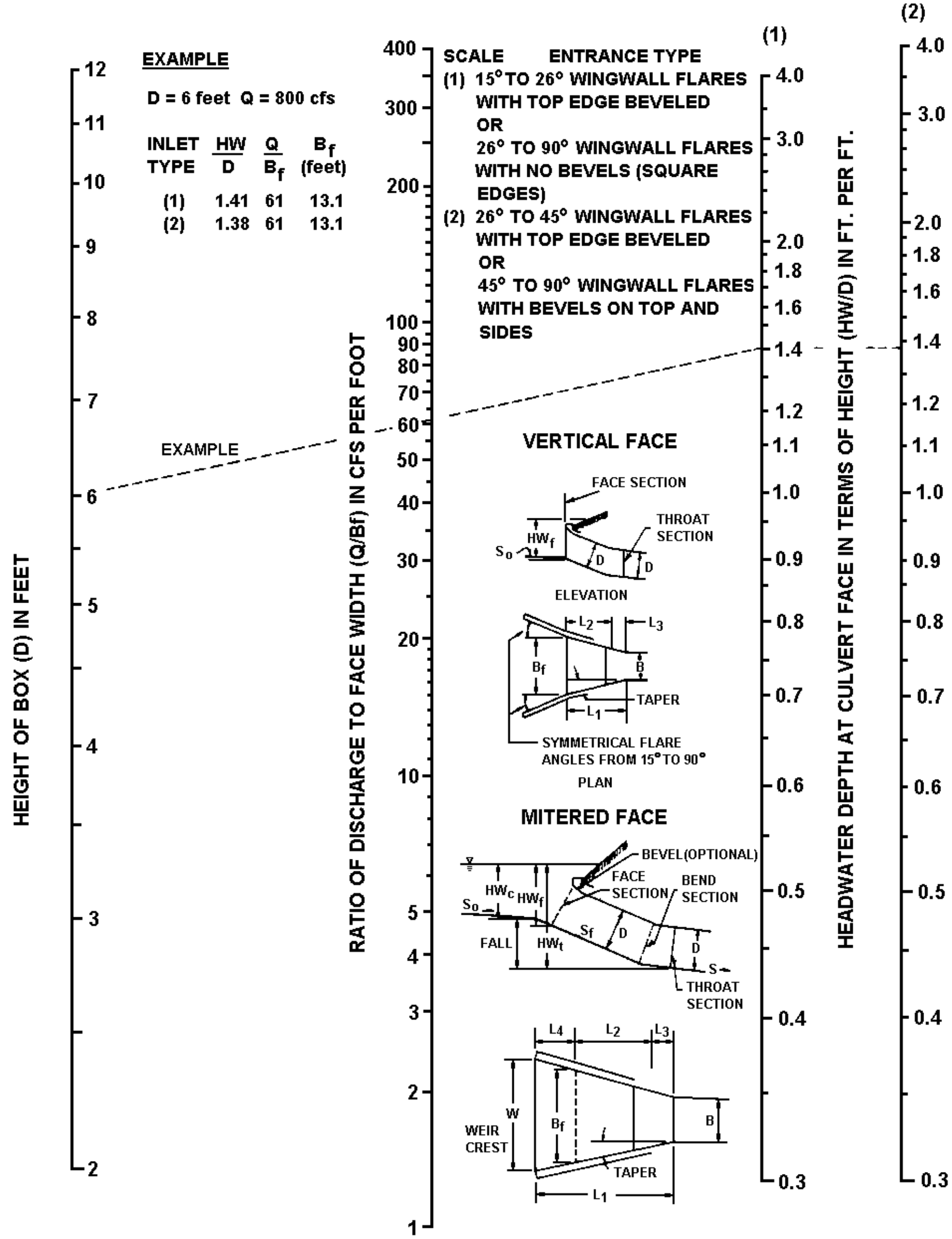
BACK TO TOC

CHART 58



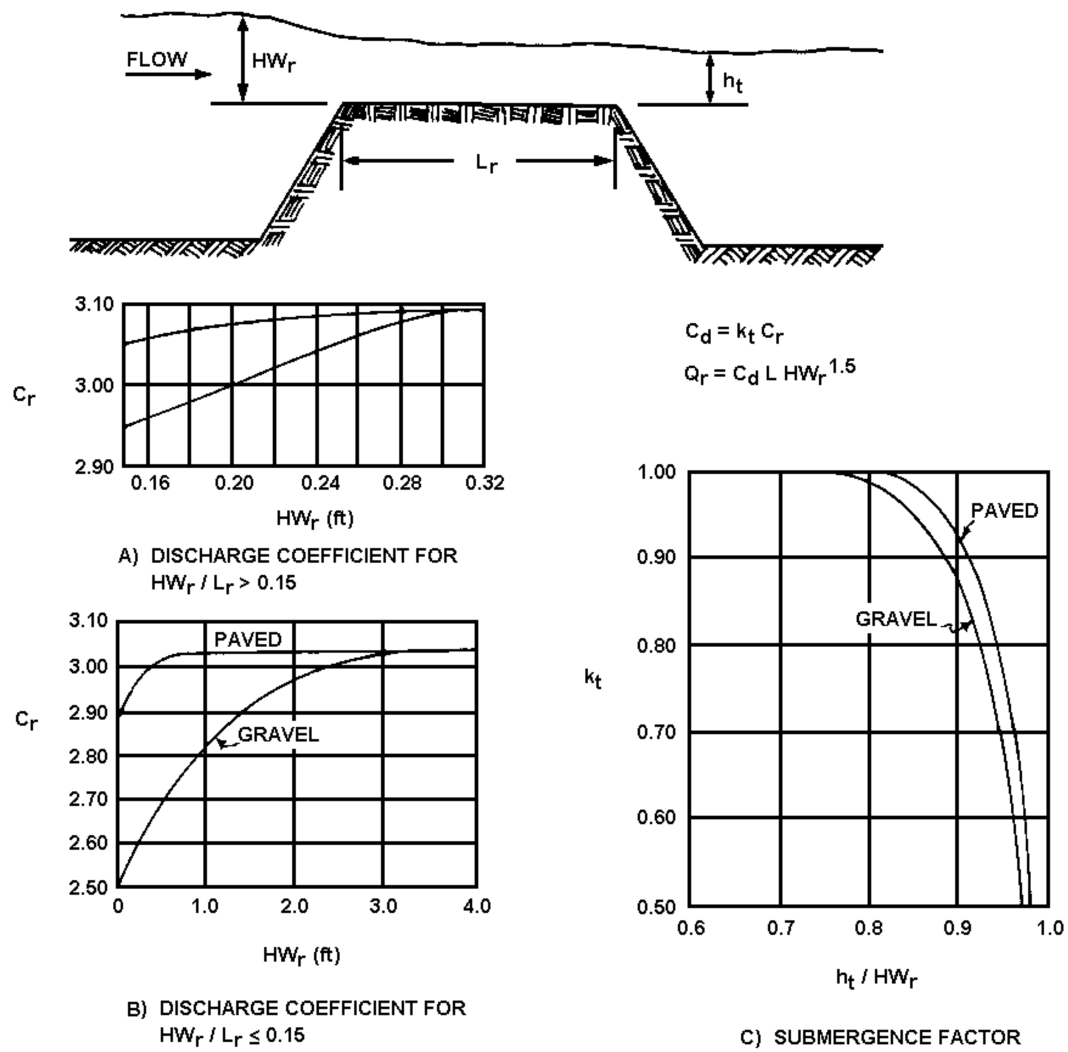

FACE CONTROL FOR BOX CULVERTS
WITH SIDE-TAPERED INLETS

CHART 59




FACE CONTROL FOR BOX CULVERTS
WITH SLOPE TAPERED INLETS

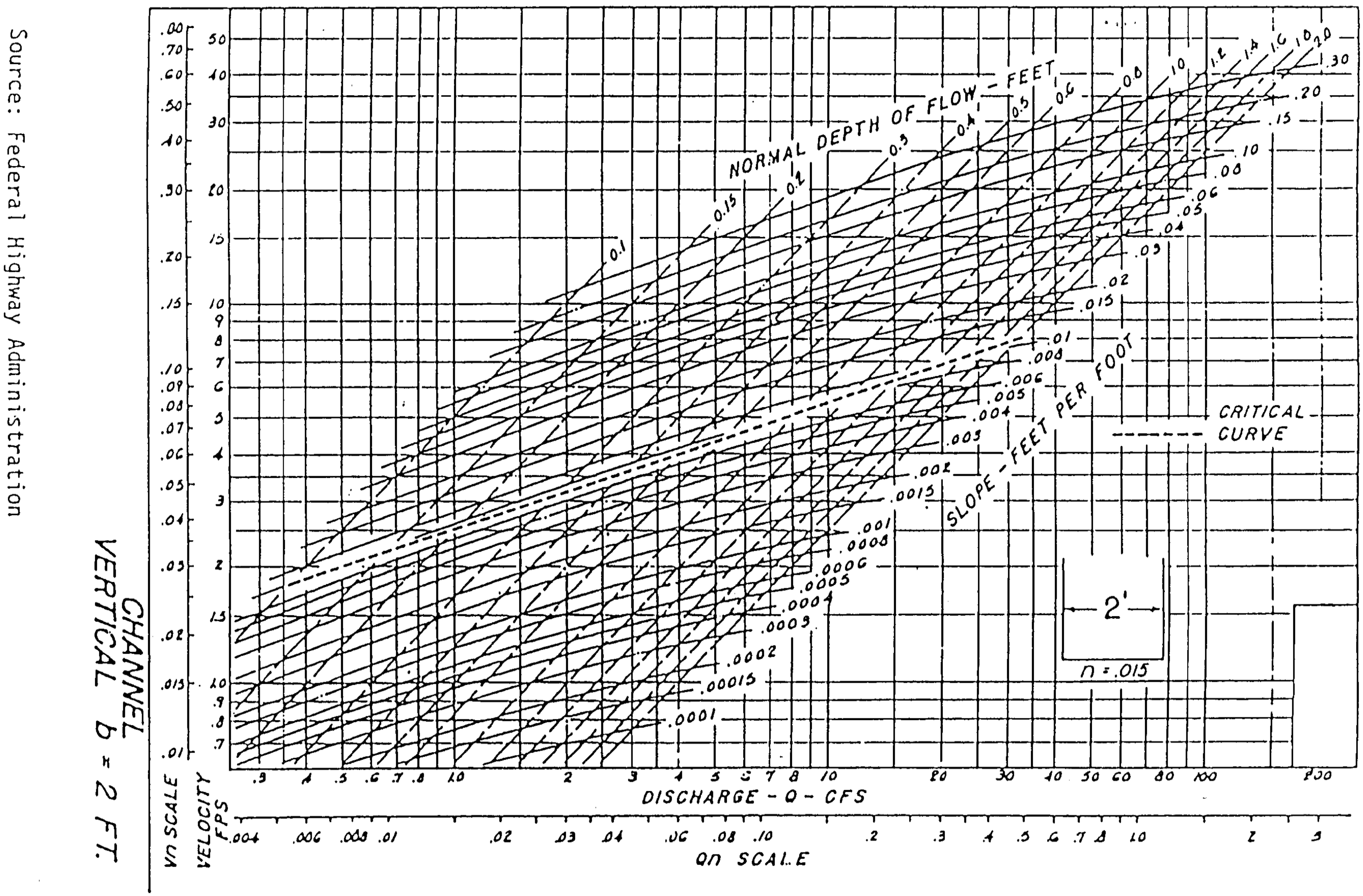
## CHART 60

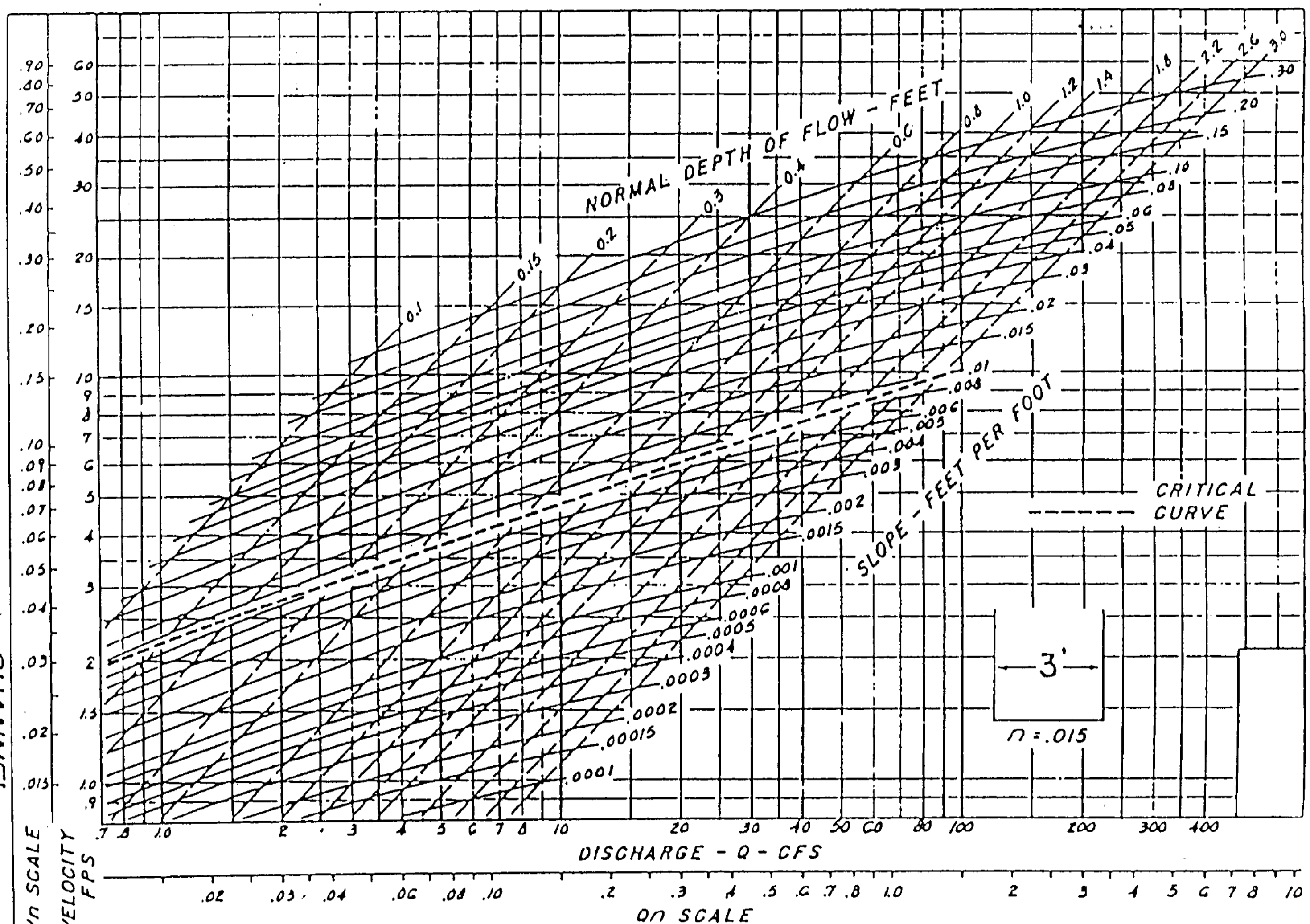



# DISCHARGE COEFFICIENTS FOR ROADWAY OVERTOPPING

BACK TO TOC

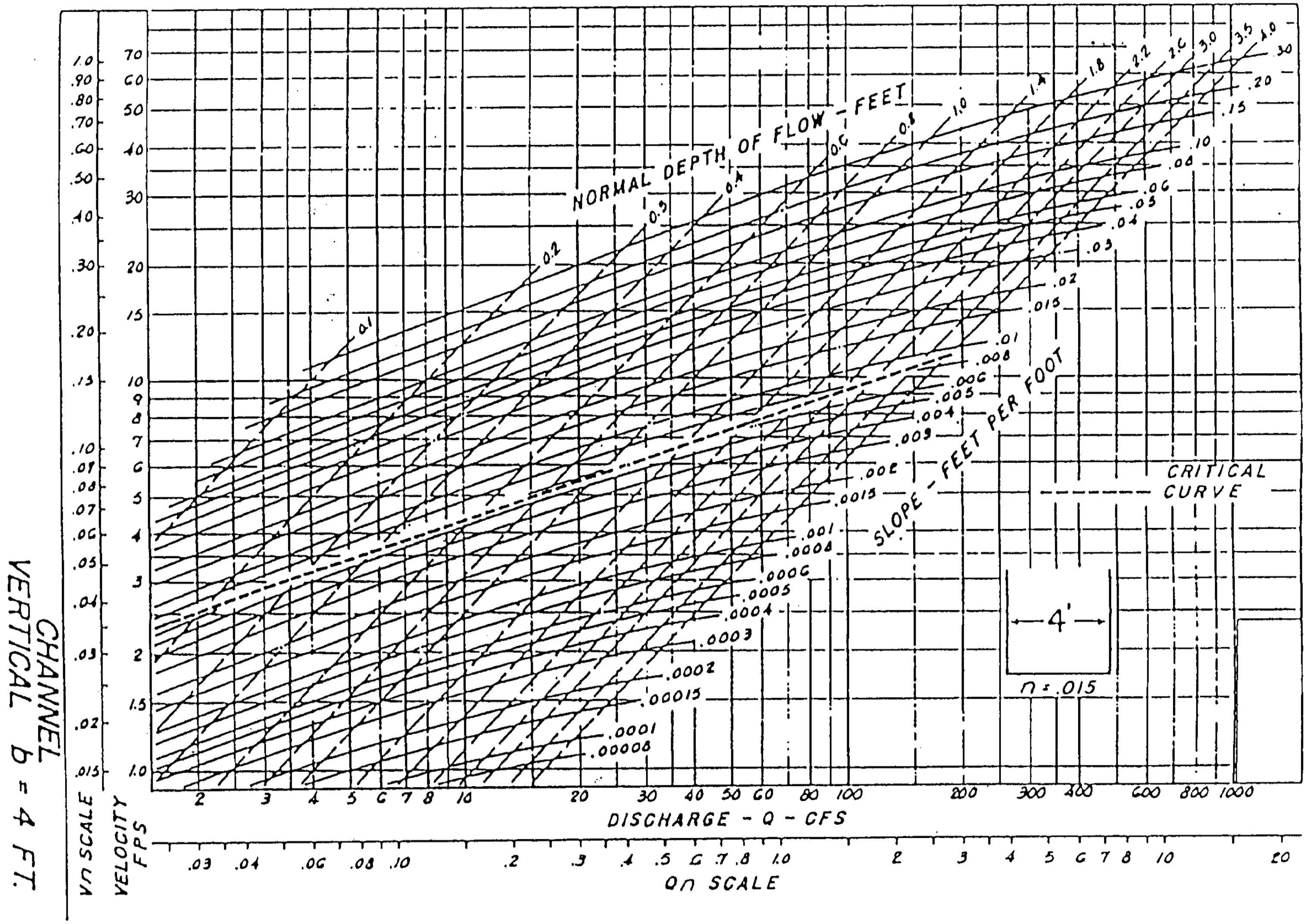
## C-2 Open Channel Design Figures





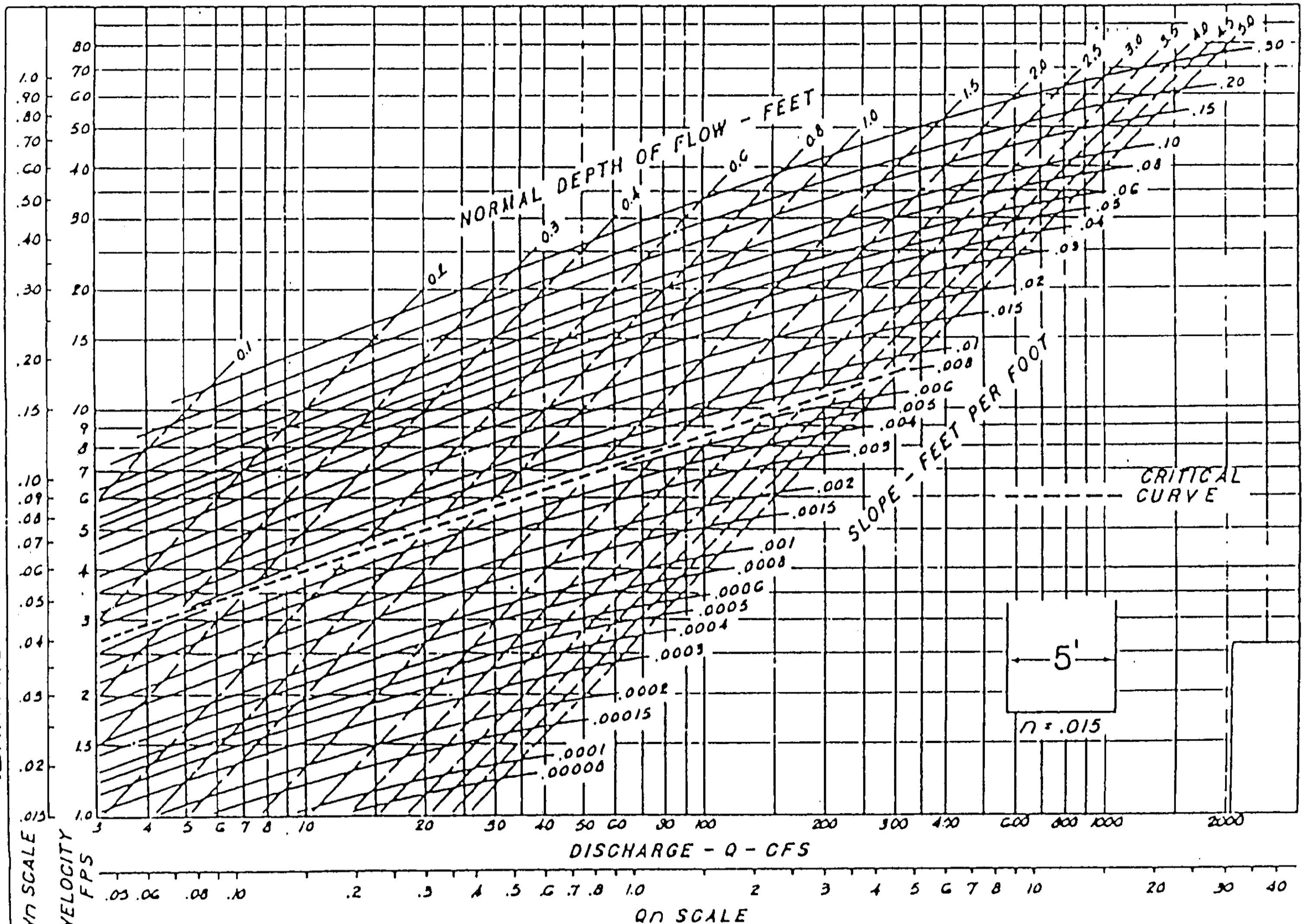

CHANNEL
VERTICAL b = 3 FT.

Source: Federal Highway Administration




Source: Federal Highway Administration

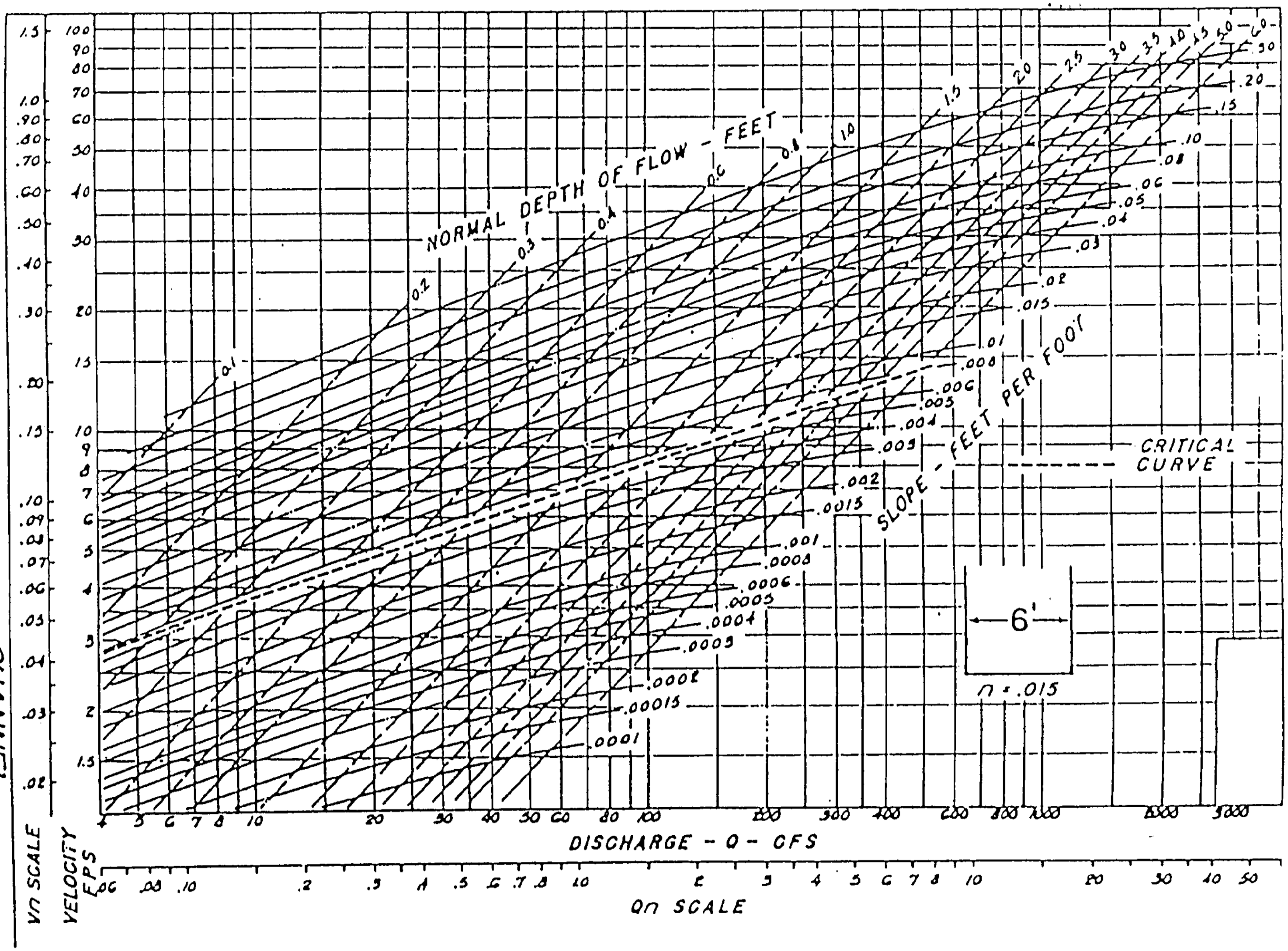
BACK TO TOC




VERTICAL CHANNEL b = 5 FT.

Source: Federal Highway Administration

BACK TO TOC



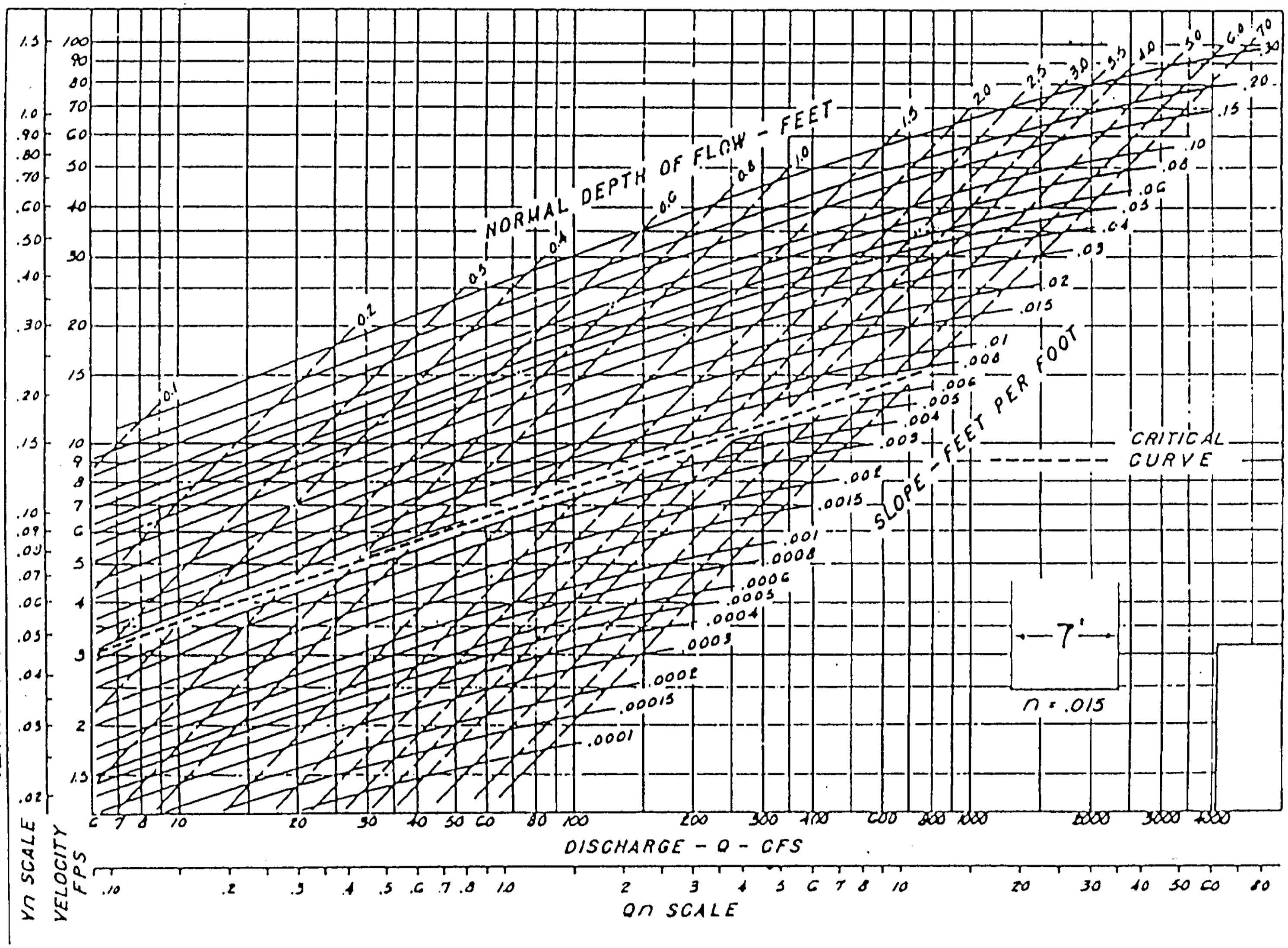

CHANNEL
VERTICAL D = 6 FT.

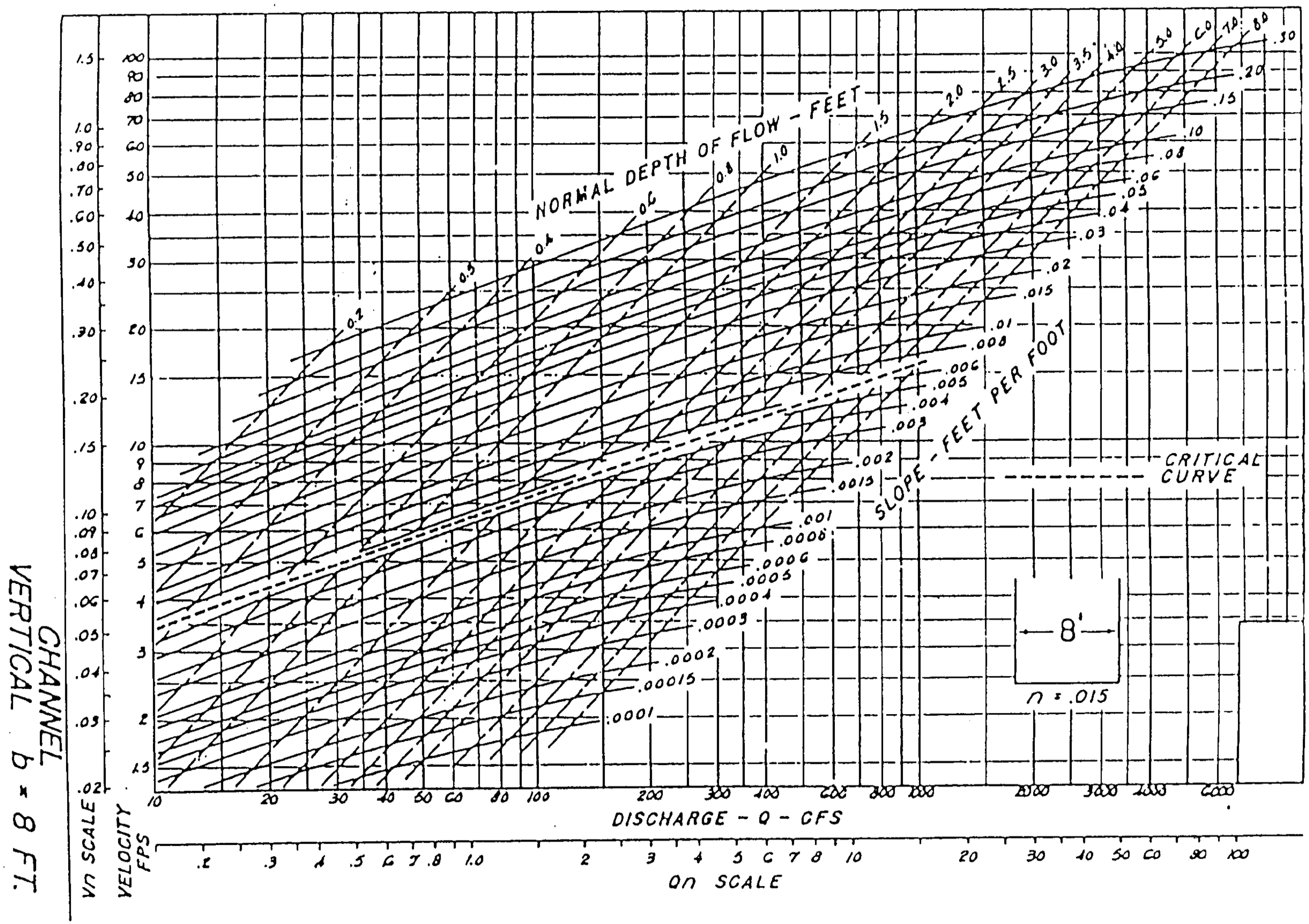
Source: Federal Highway Administration

BACK TO TOC

Source: Federal Highway Administration

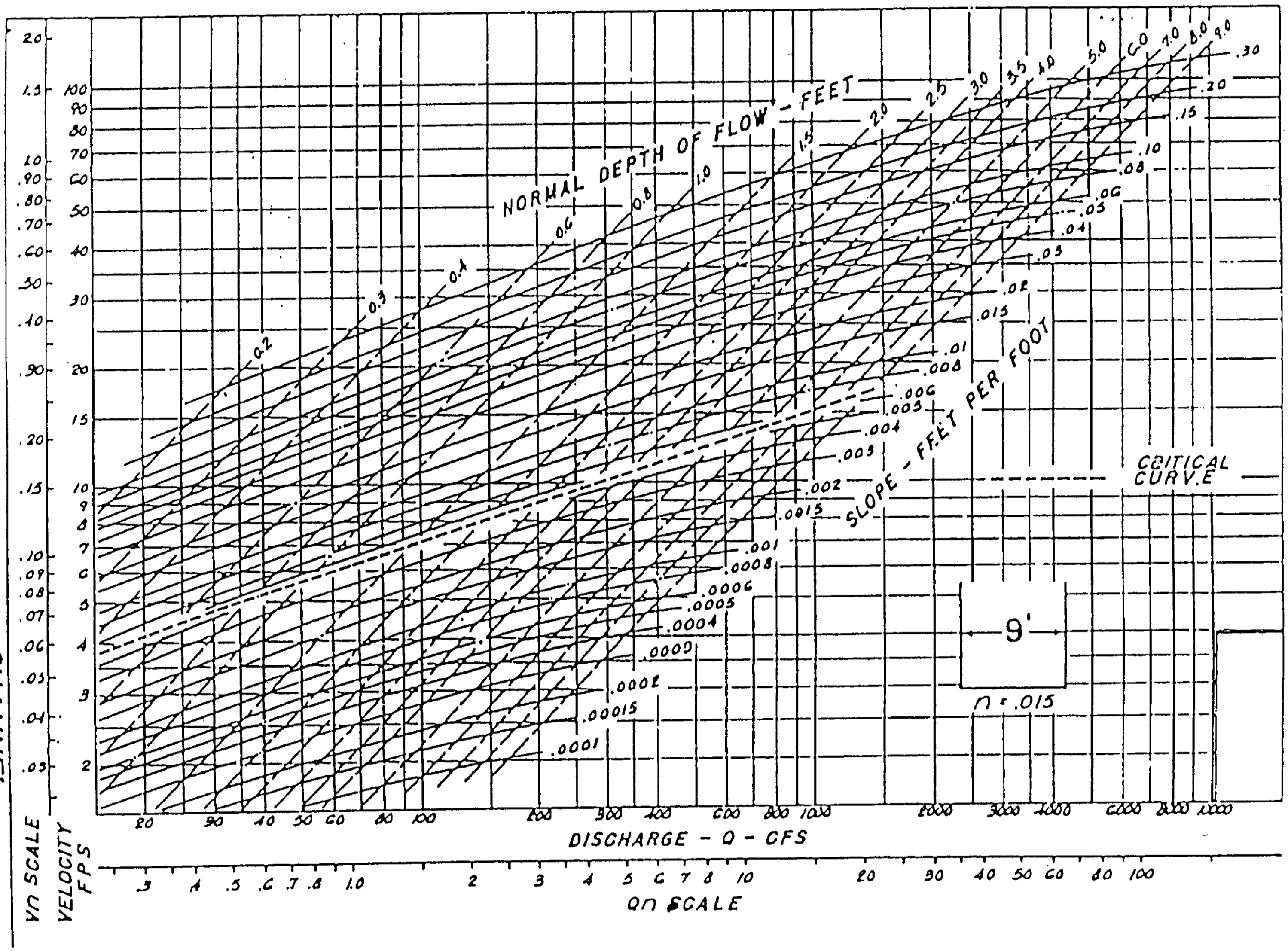
CHANNEL
VERTICAL b = 7 FT.






Source: Federal Highway Administration

# CHANNEL
# VERTICAL b = 9 FT.

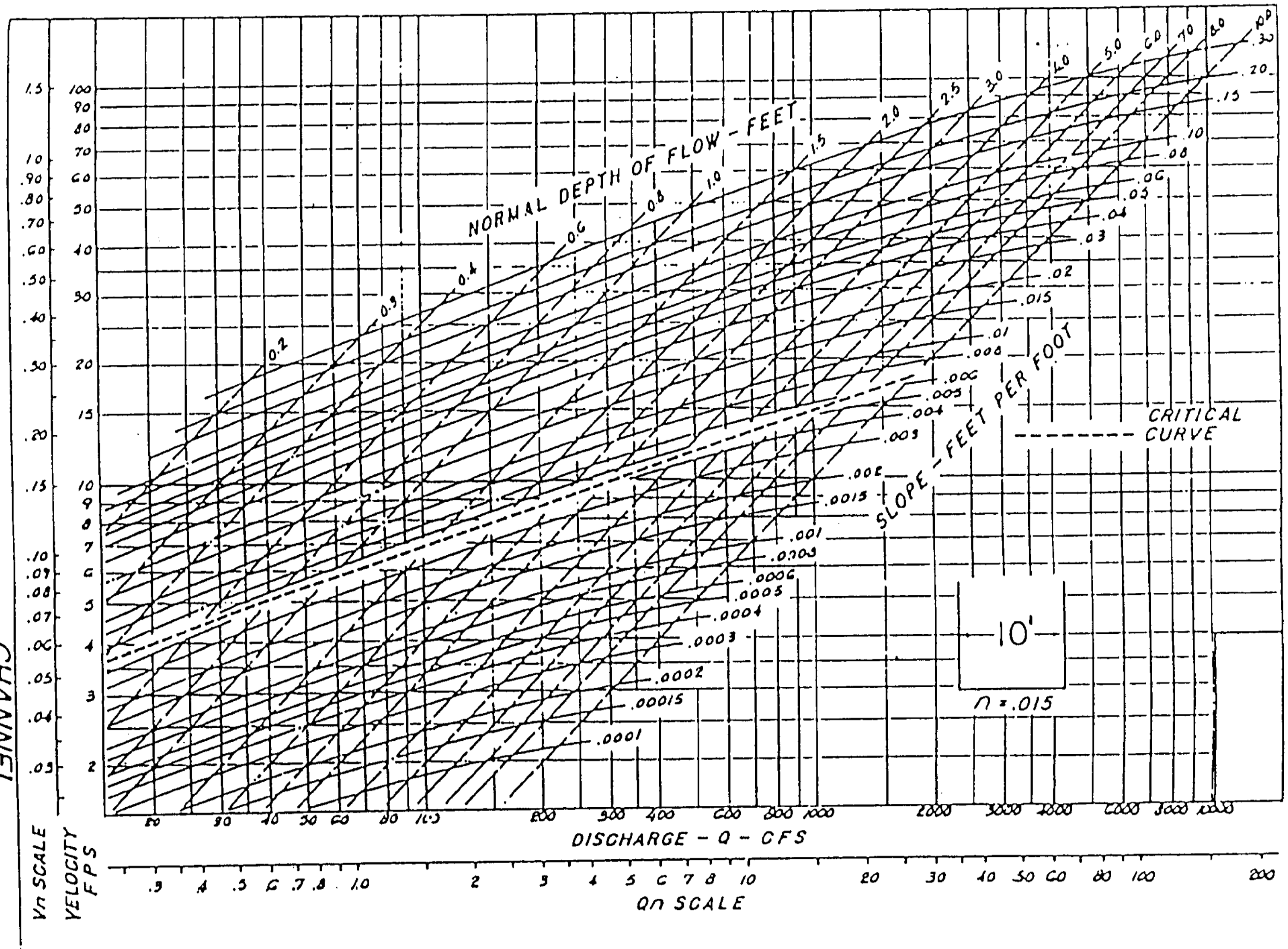


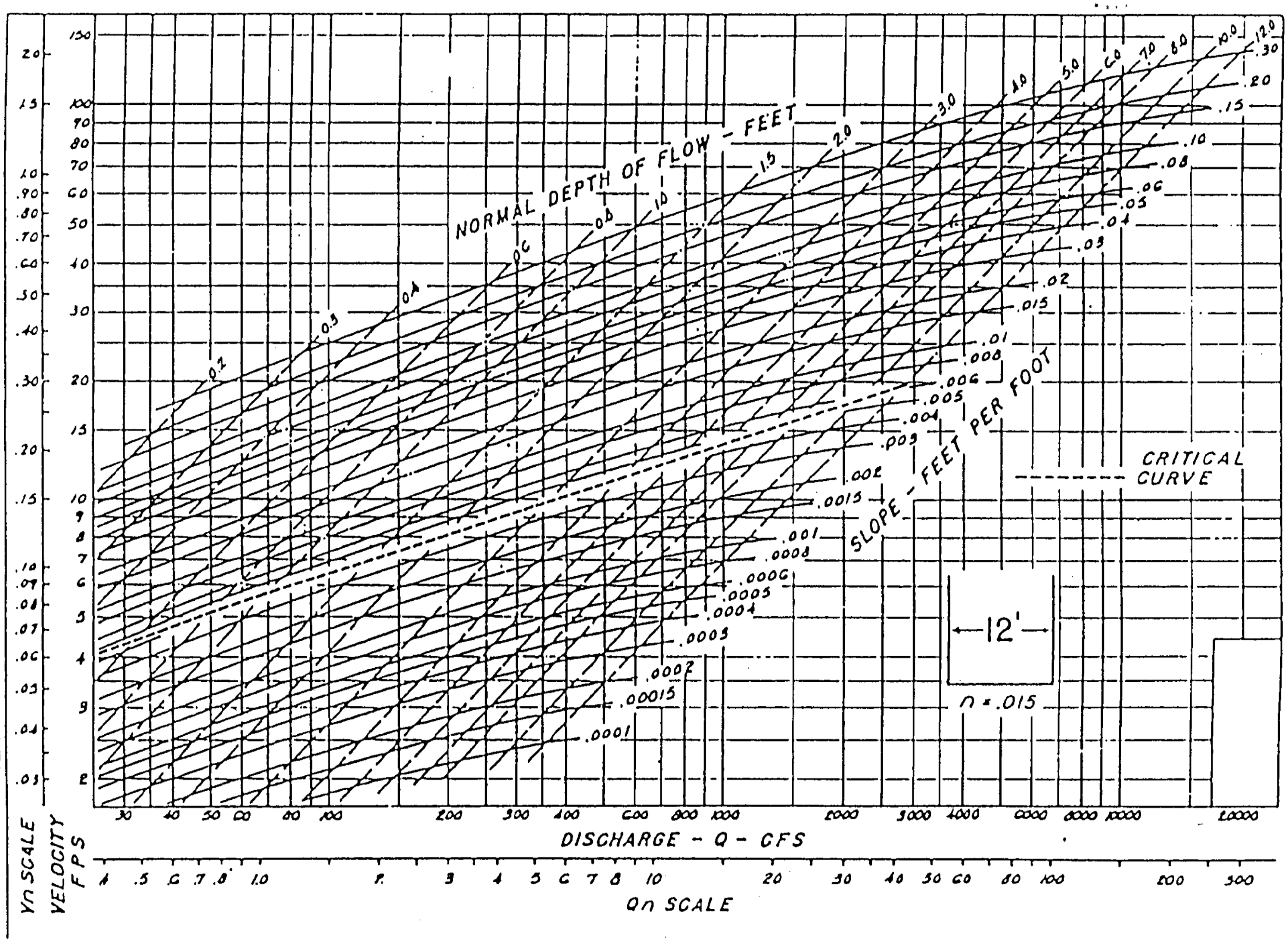

Source: Federal Highway Administration

BACK TO TOC

Source: Federal Highway Administration

CHANNEL
VERTICAL b = 10 FT.

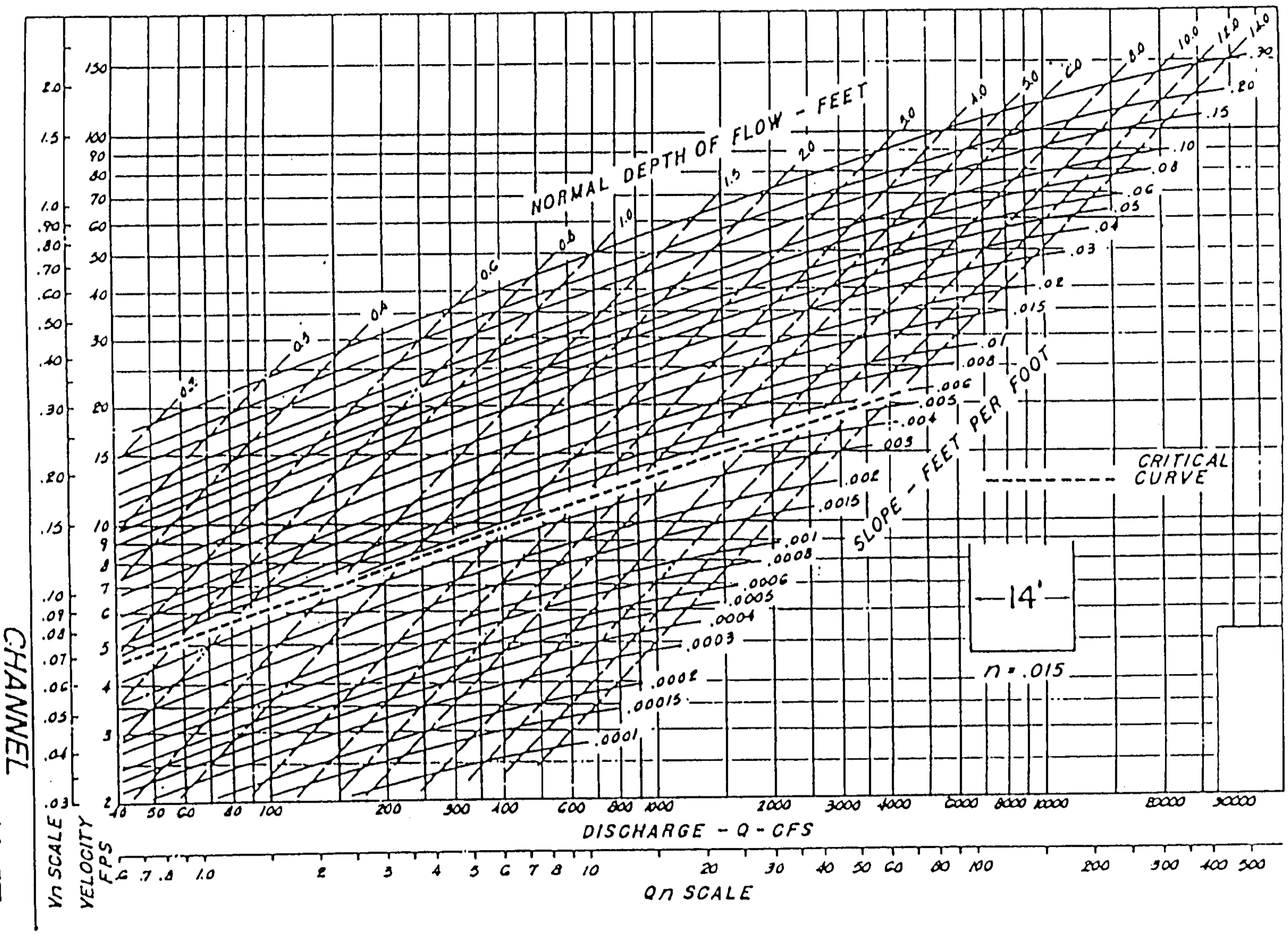





CHANNEL
VERTICAL b = 12 FT.

Source: Federal Highway Administration

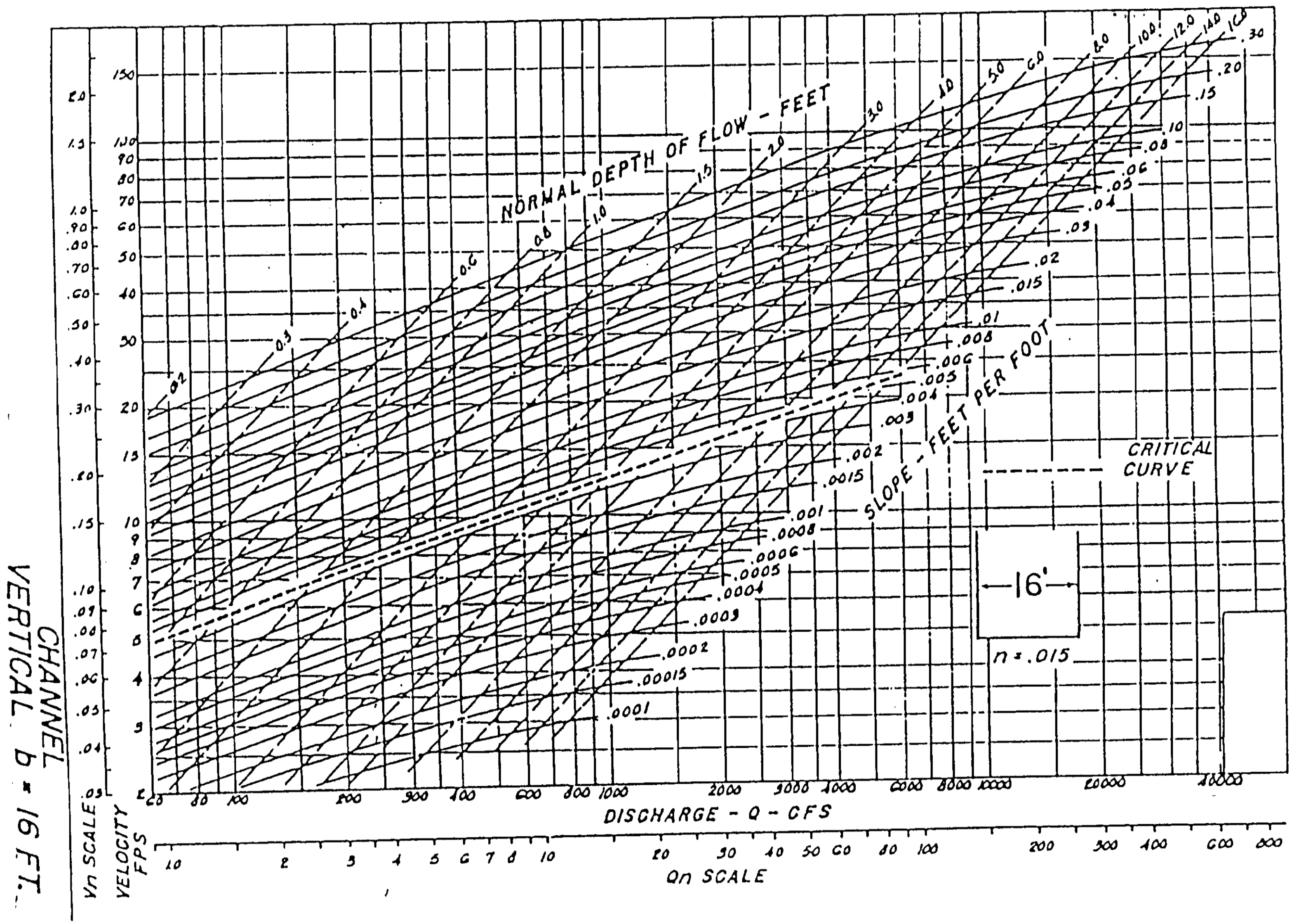
BACK TO TOC




CHANNEL
VERTICAL b = 14 FT.

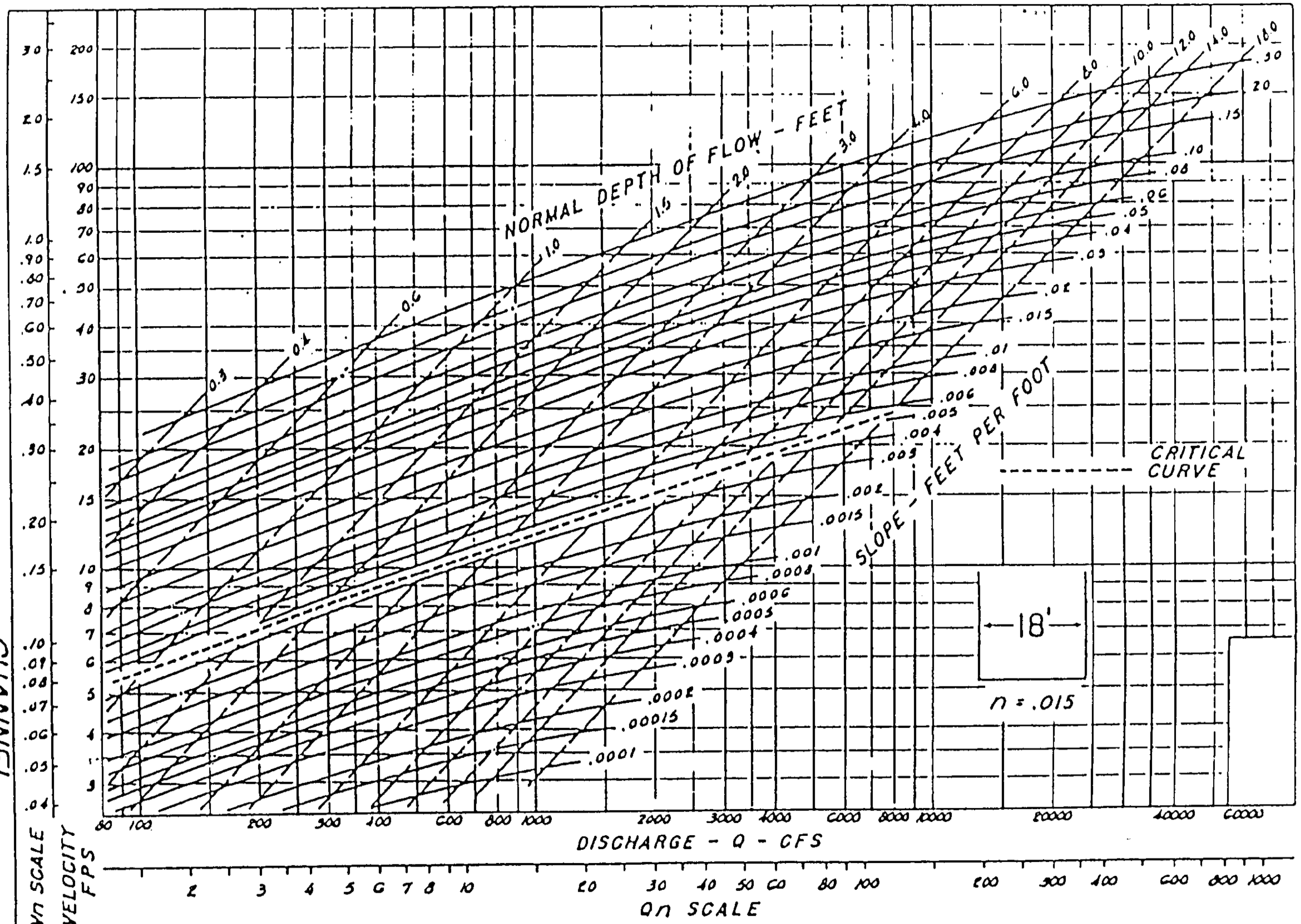
Source: Federal Highway Administration.

BACK TO TOC




Source: Federal Highway Administration

CHANNEL
VERTICAL b = 18 FT.

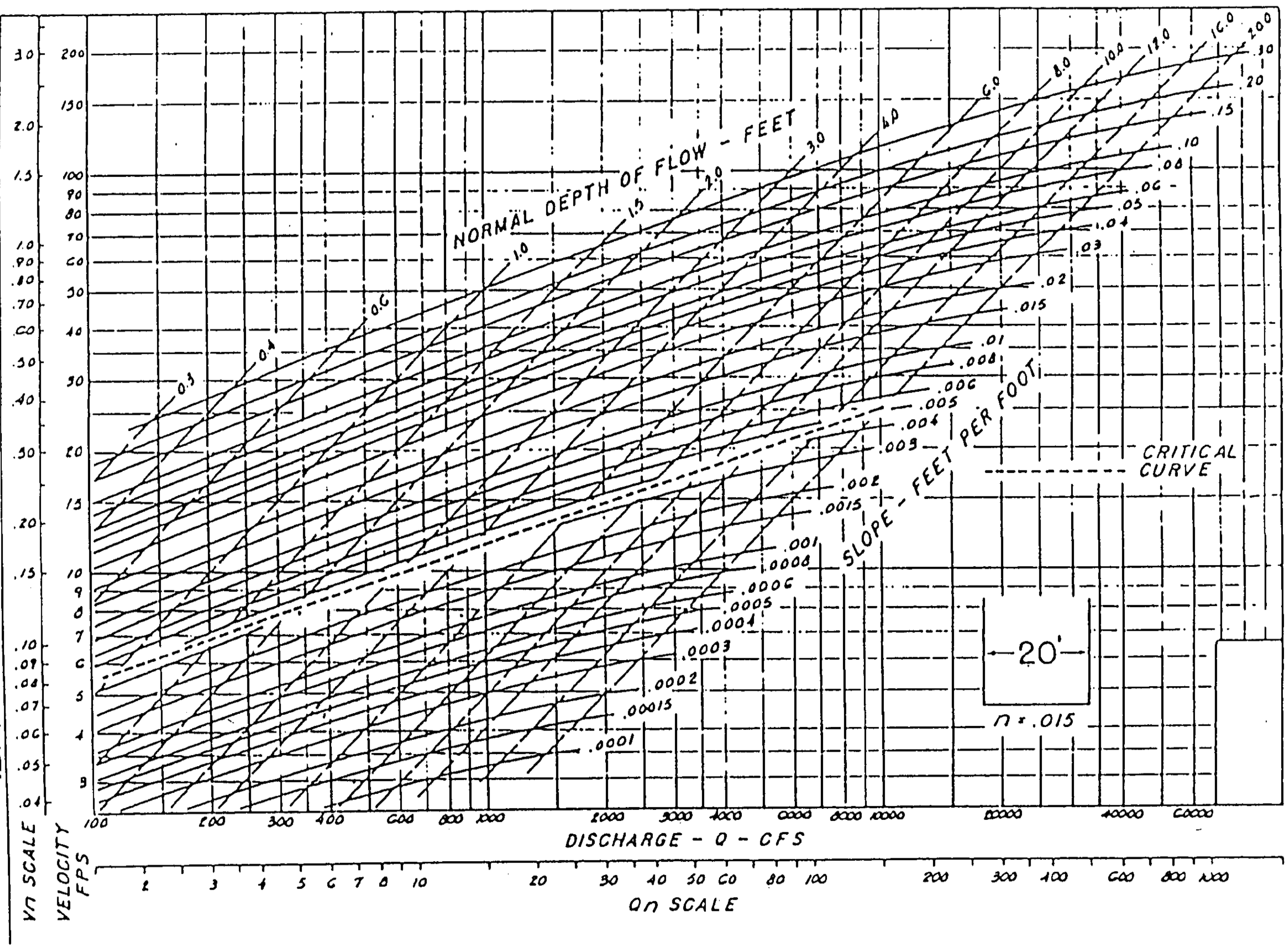



Source: Federal Highway Administration

BACK TO TOC

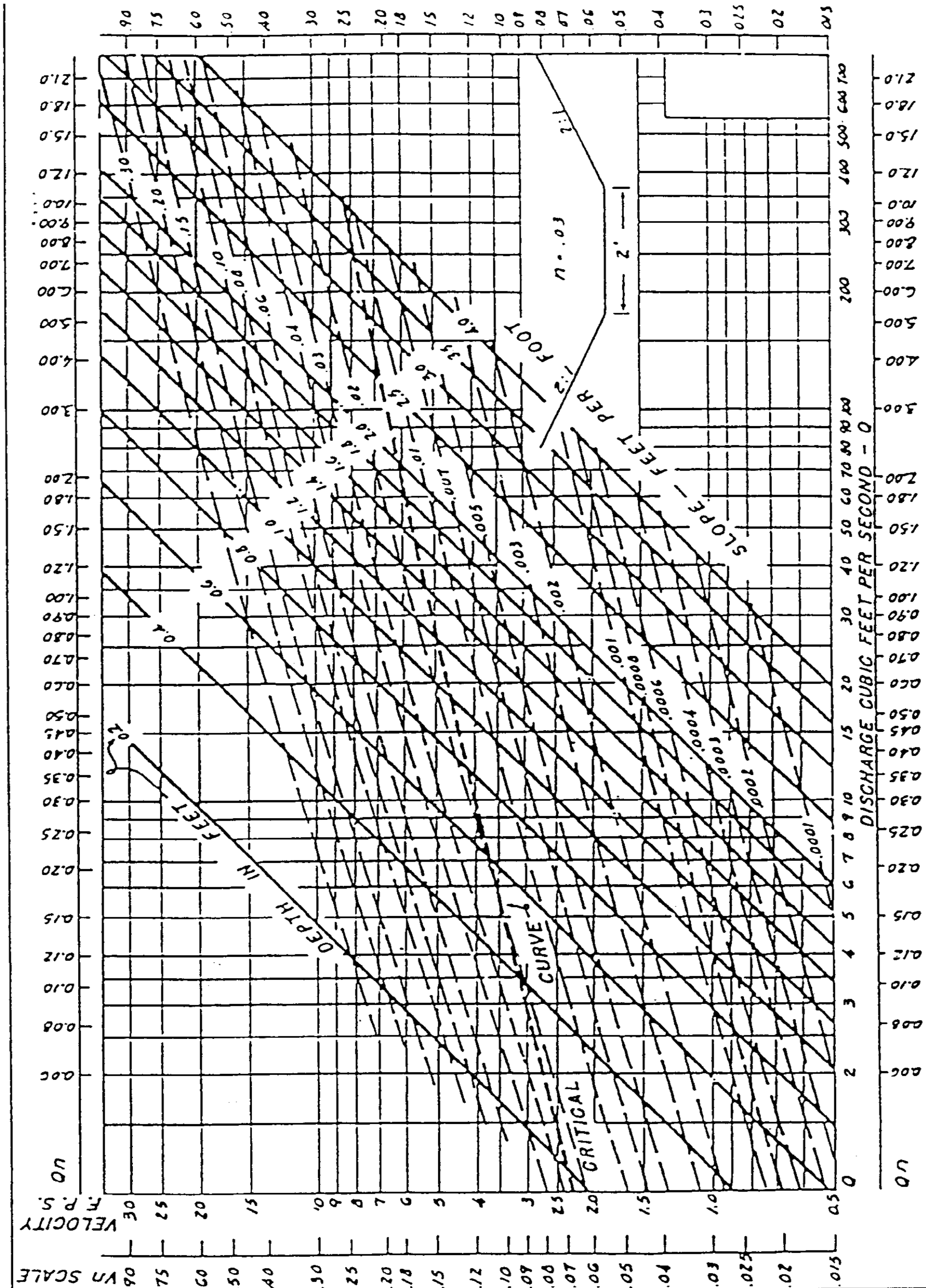
Source: Federal Highway Administration

CHANNEL
VERTICAL b = 20 FT.



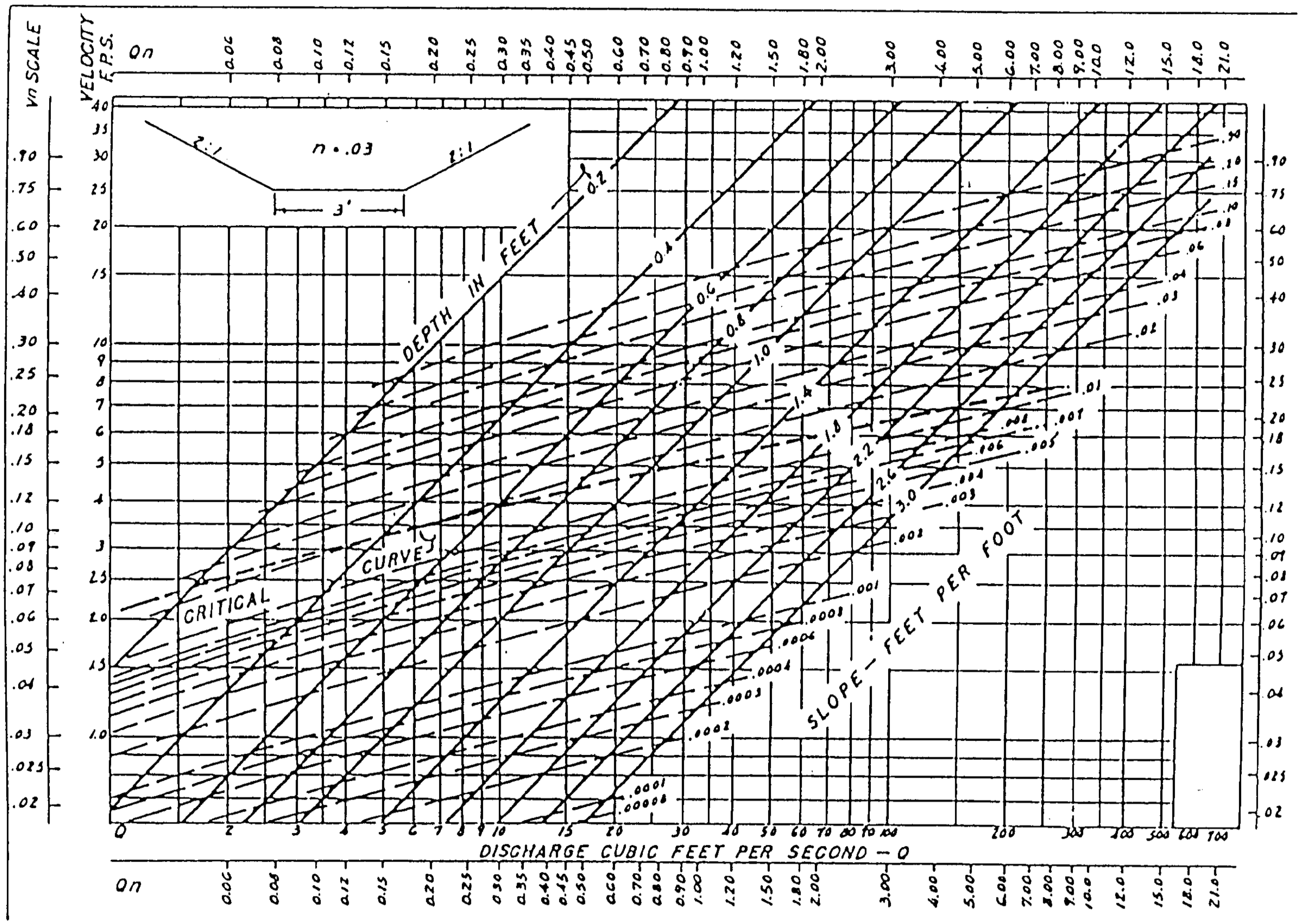



BACK TO TOC




CHANNEL
2:1 b = 2 FT.

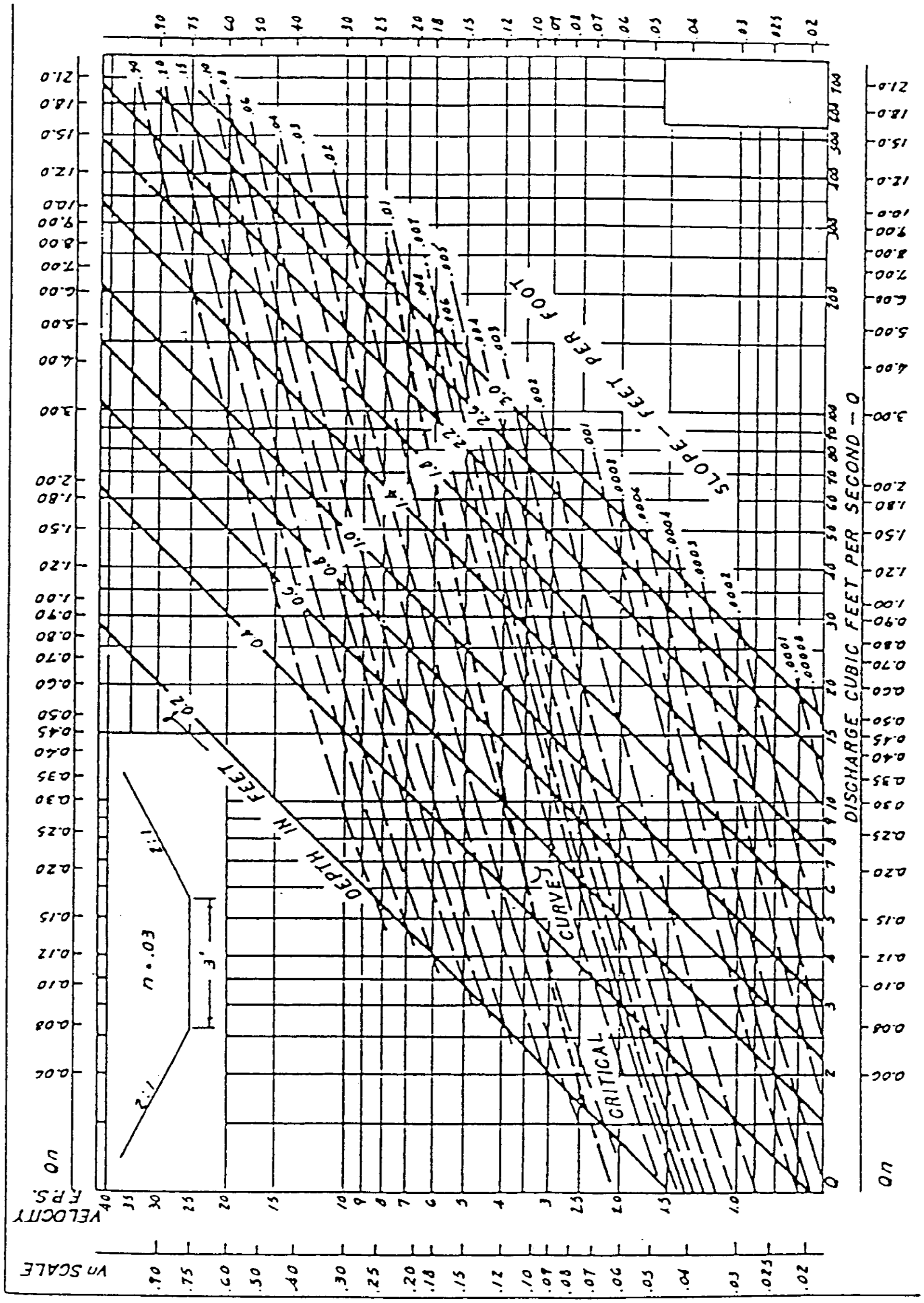
Source: Federal Highway Administration




CHANNEL
2:1 b = 3 FT.

Source: Federal Highway Administration

BACK TO TOC

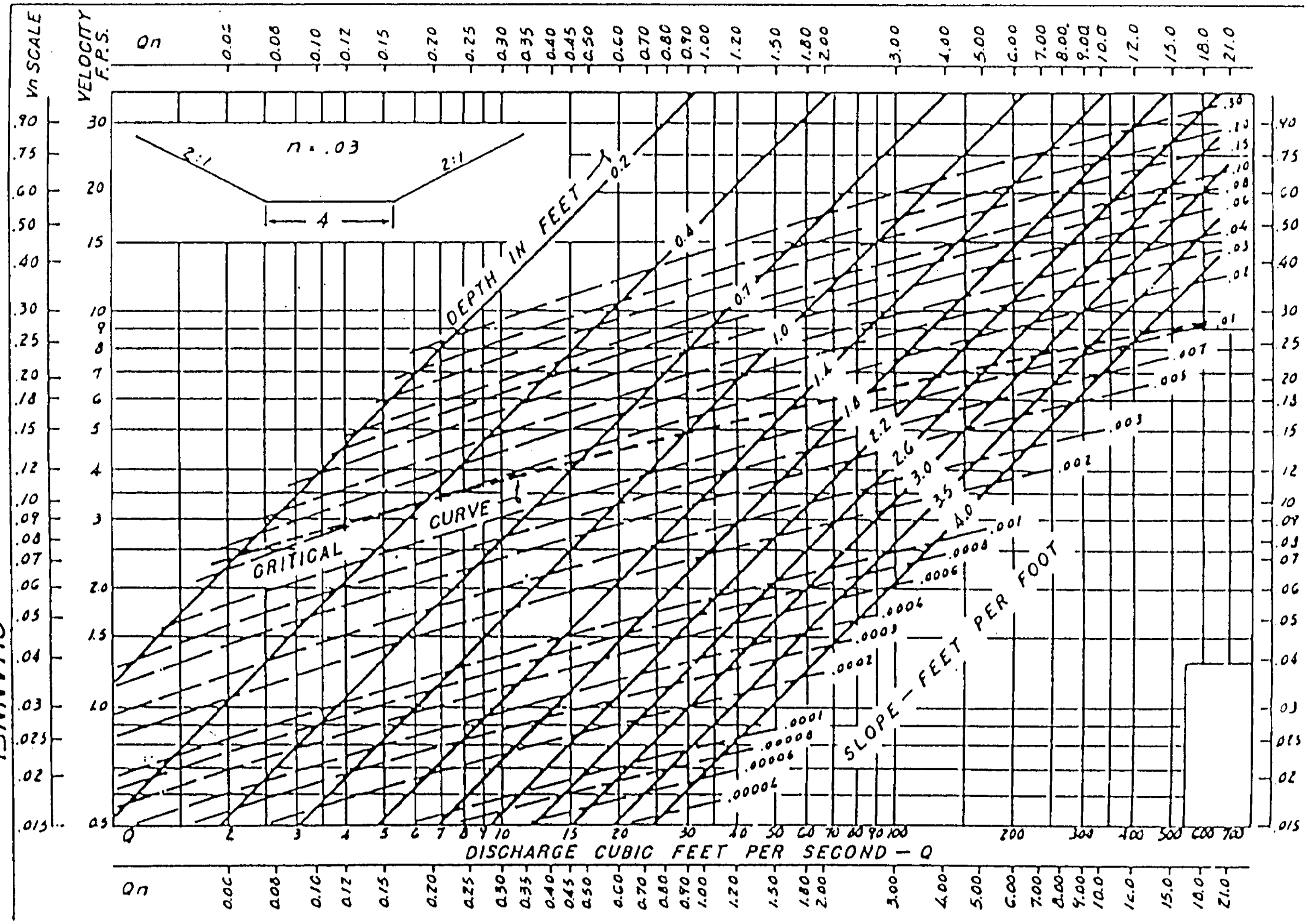



CHANNEL
2:1 b = 3 FT.

Source: Federal Highway Administration

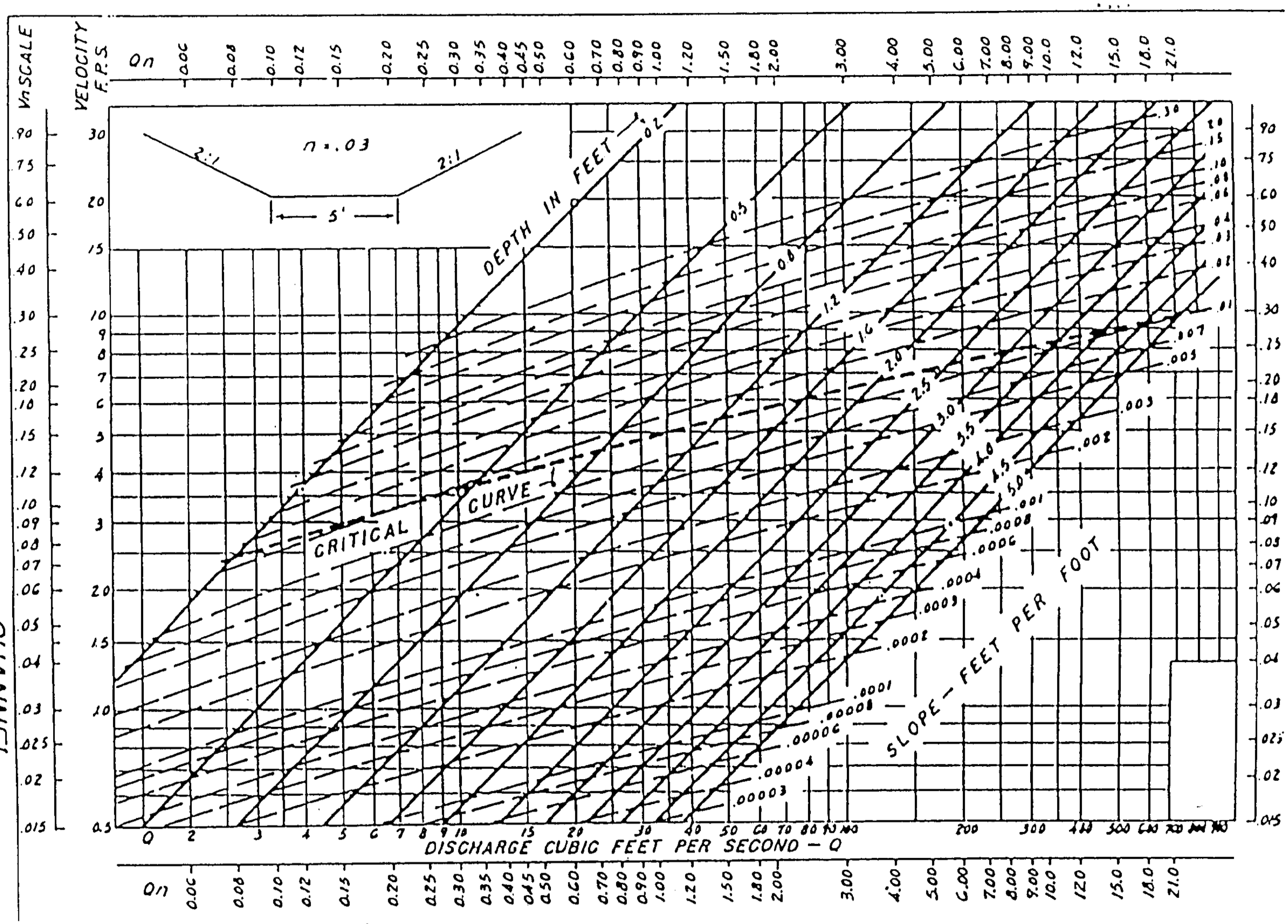
BACK TO TOC

CHANNEL
2:1 b = 4 FT.



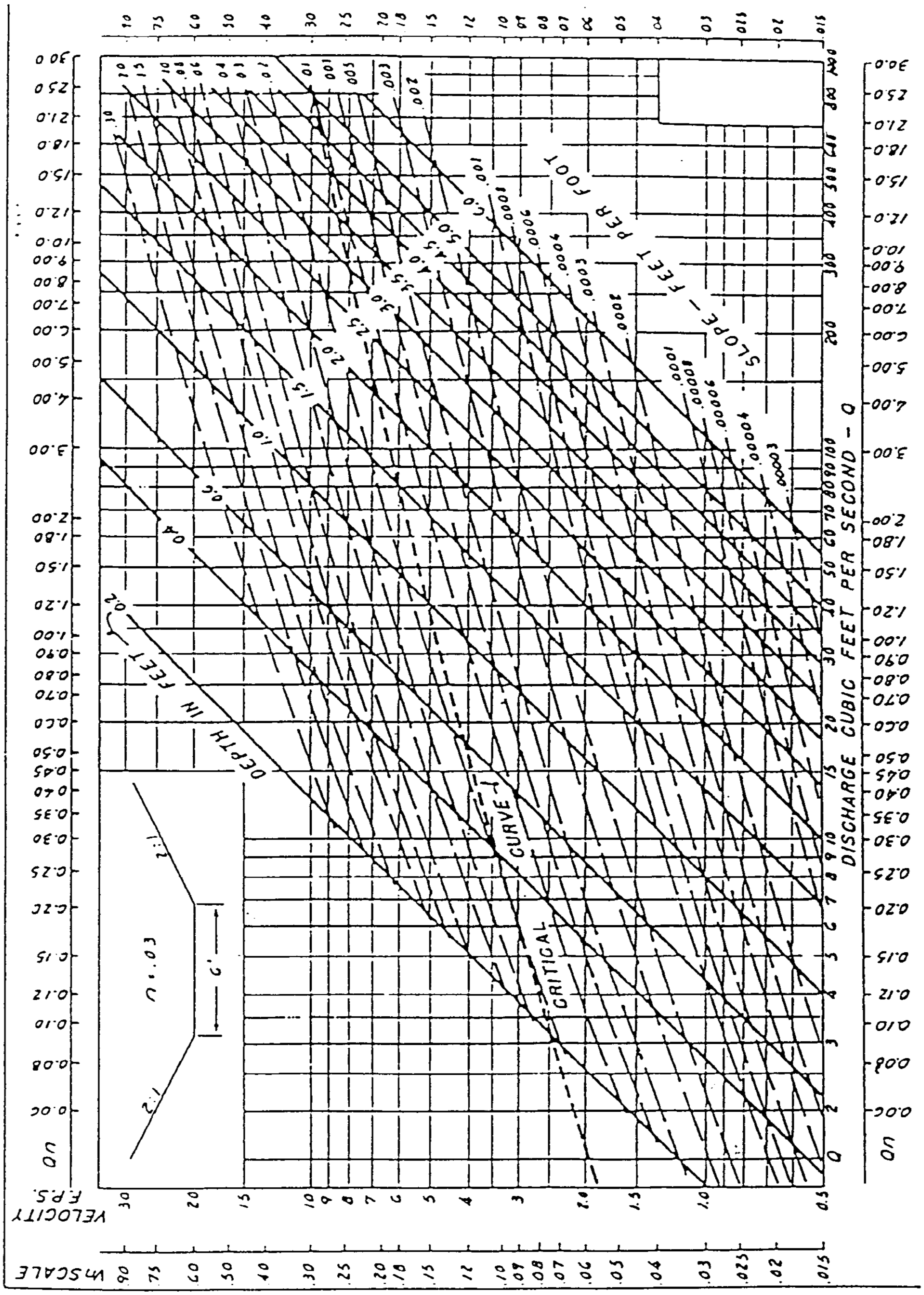

Source: Federal Highway Administration

CHANNEL
2:1 b = 5 FT.




Source: Federal Highway Administration

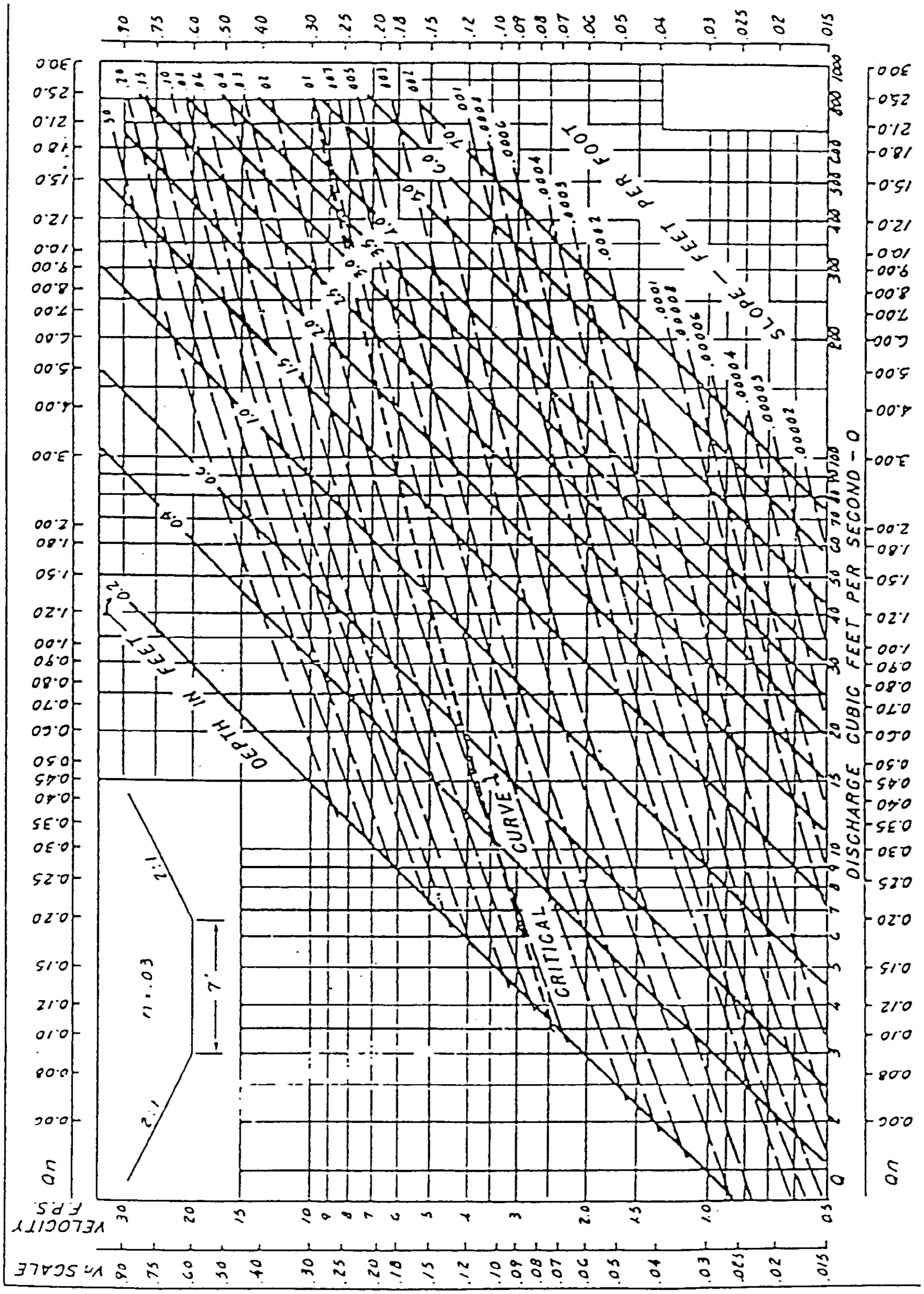
BACK TO TOC




CHANNEL
2:1 b = 6 FT.

Source: Federal Highway Administration

BACK TO TOC



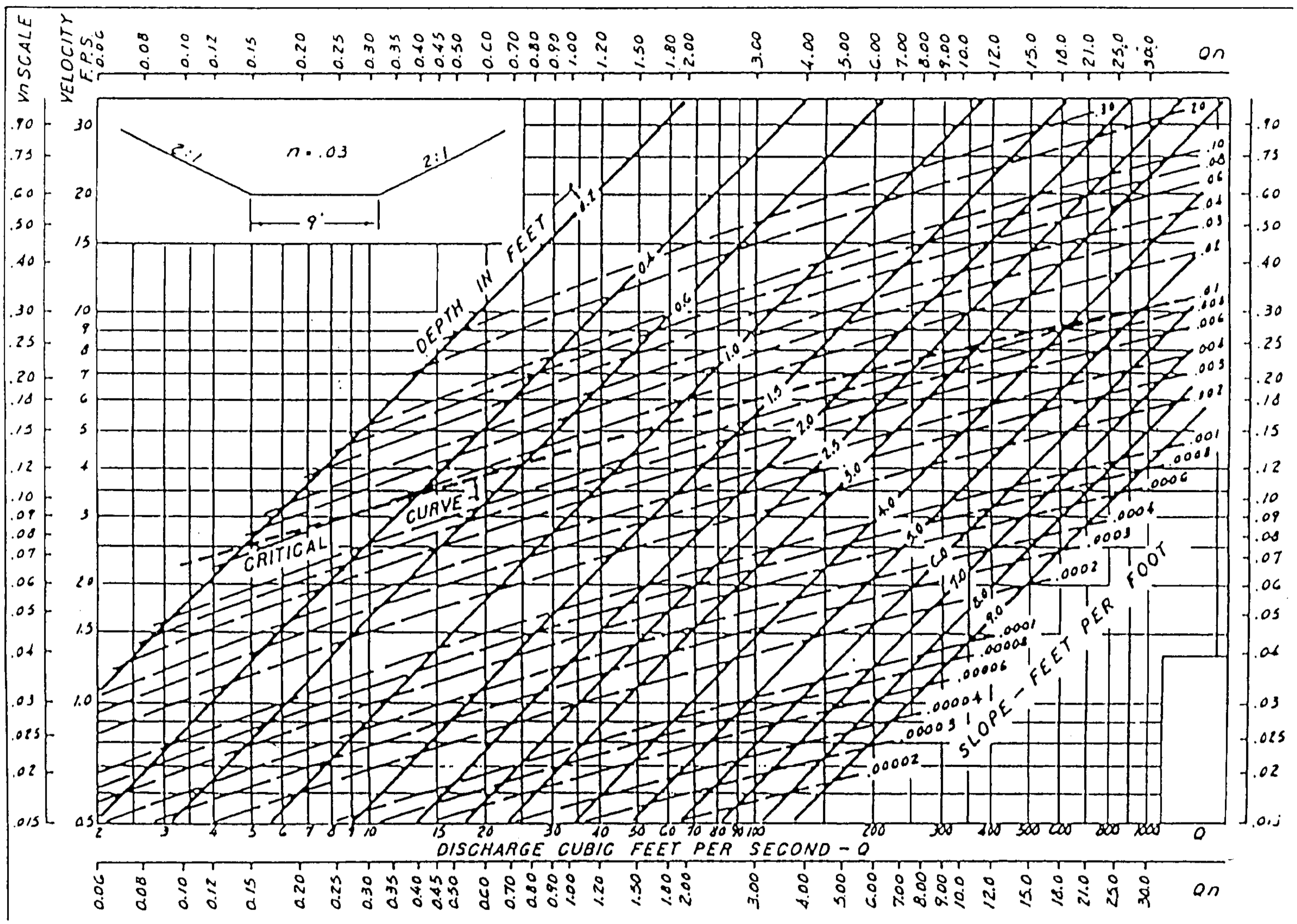

CHANNEL
2:1 b = 7 FT.

Source: Federal Highway Administration

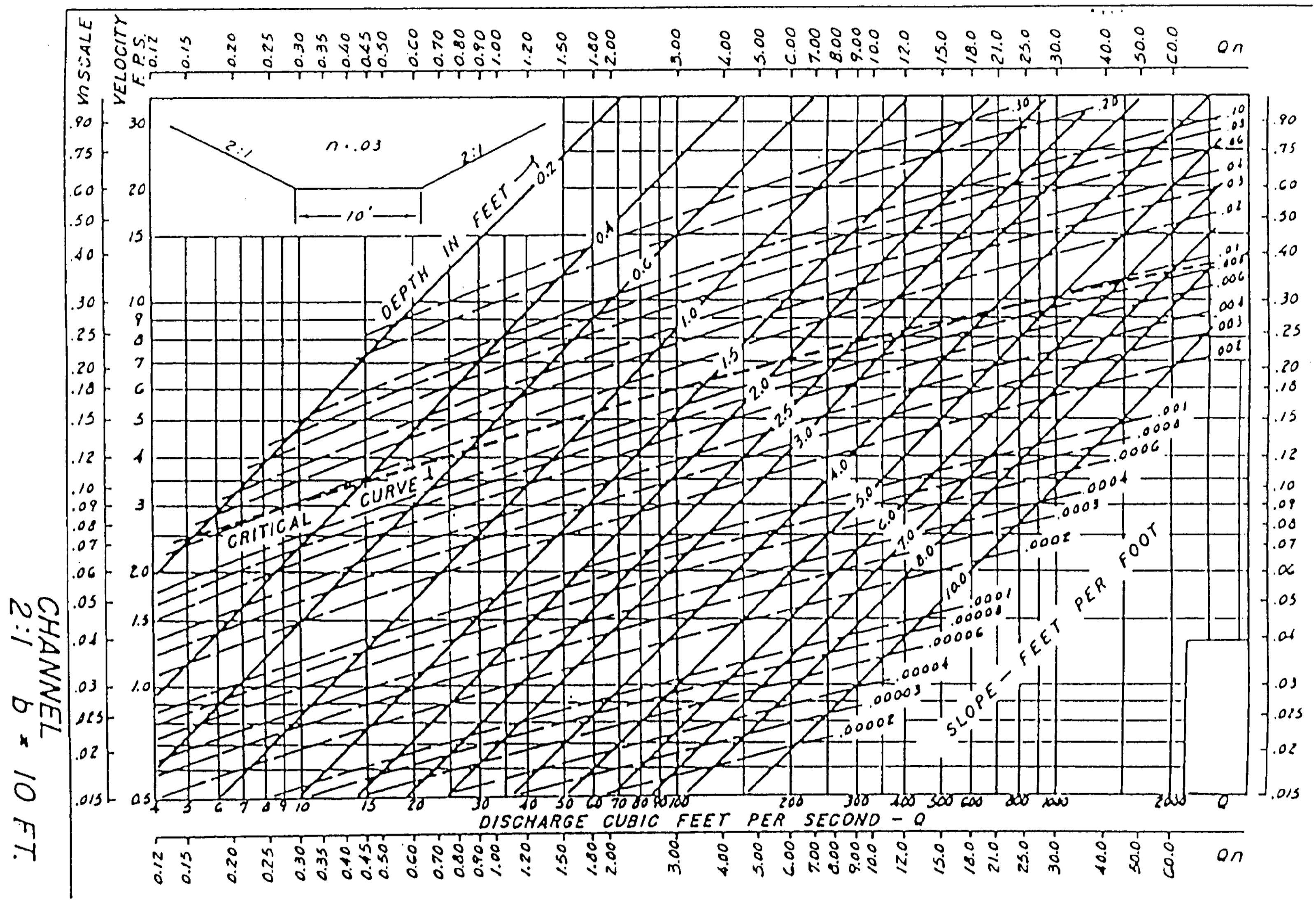
BACK TO TOC

Source: Federal Highway Administration

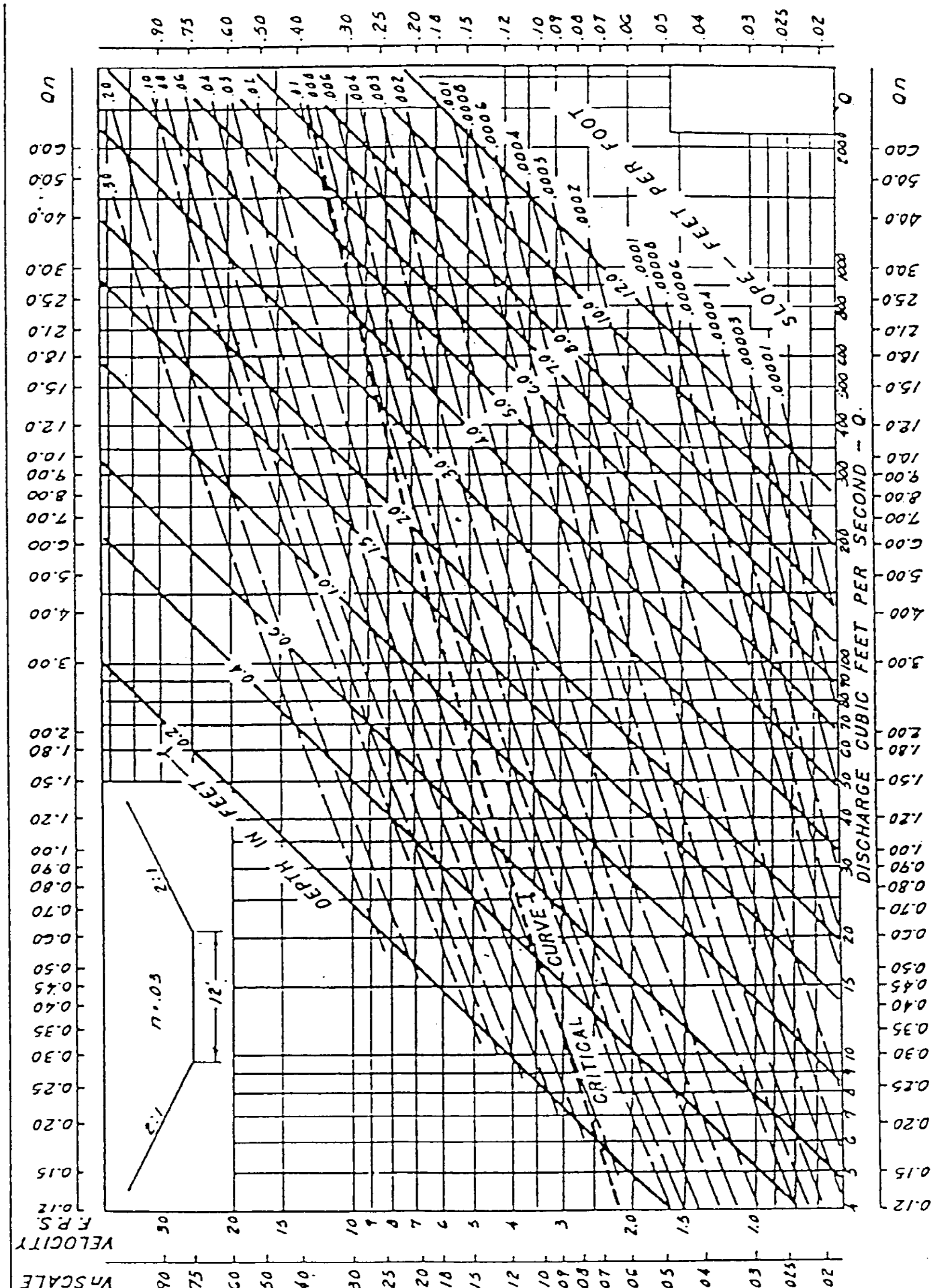
CHANNEL
2:1 b = 9 FT.



Source: Federal Highway Administration

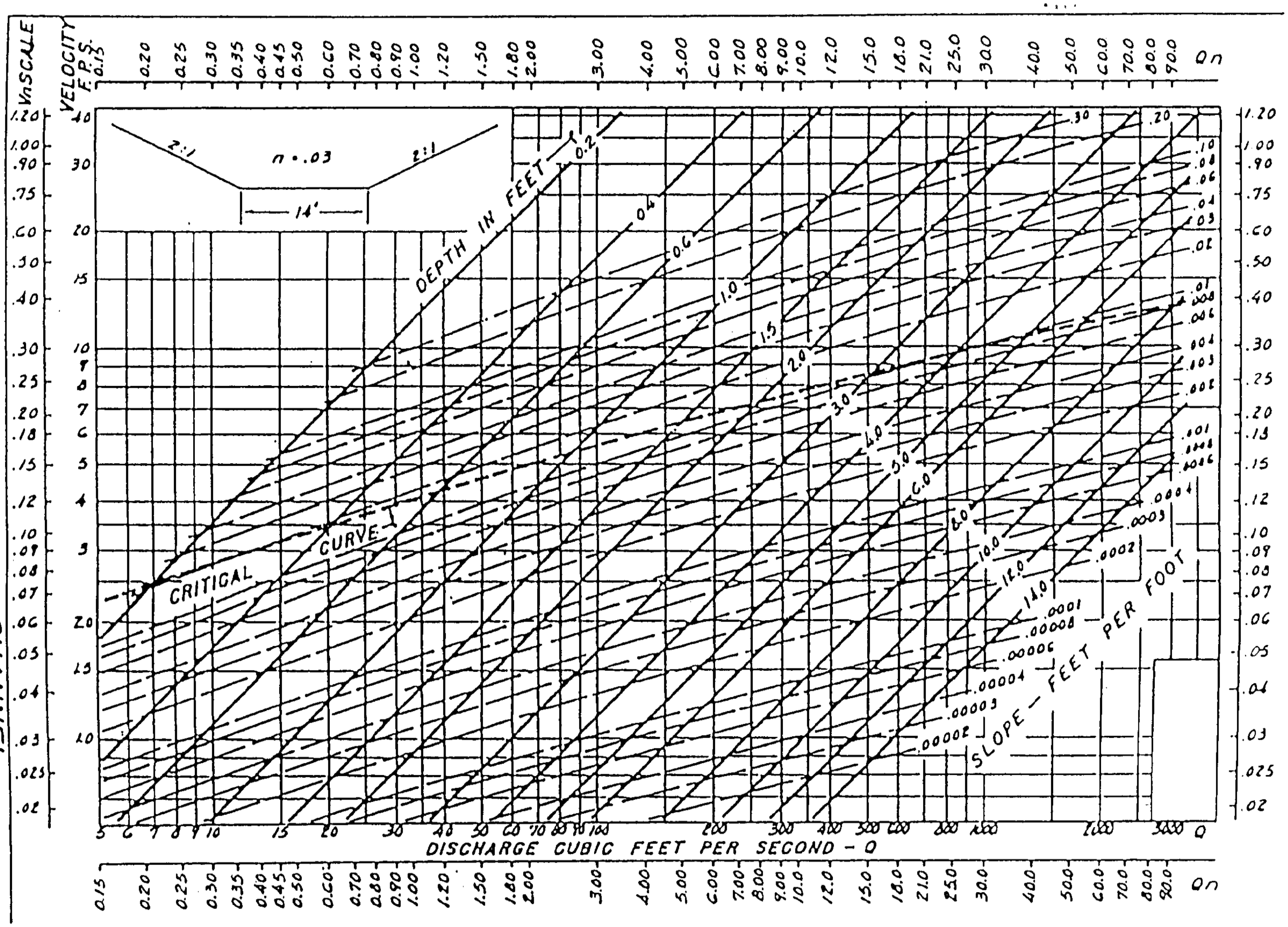



BACK TO TOC




CHANNEL
2:1 b = 12 FT.

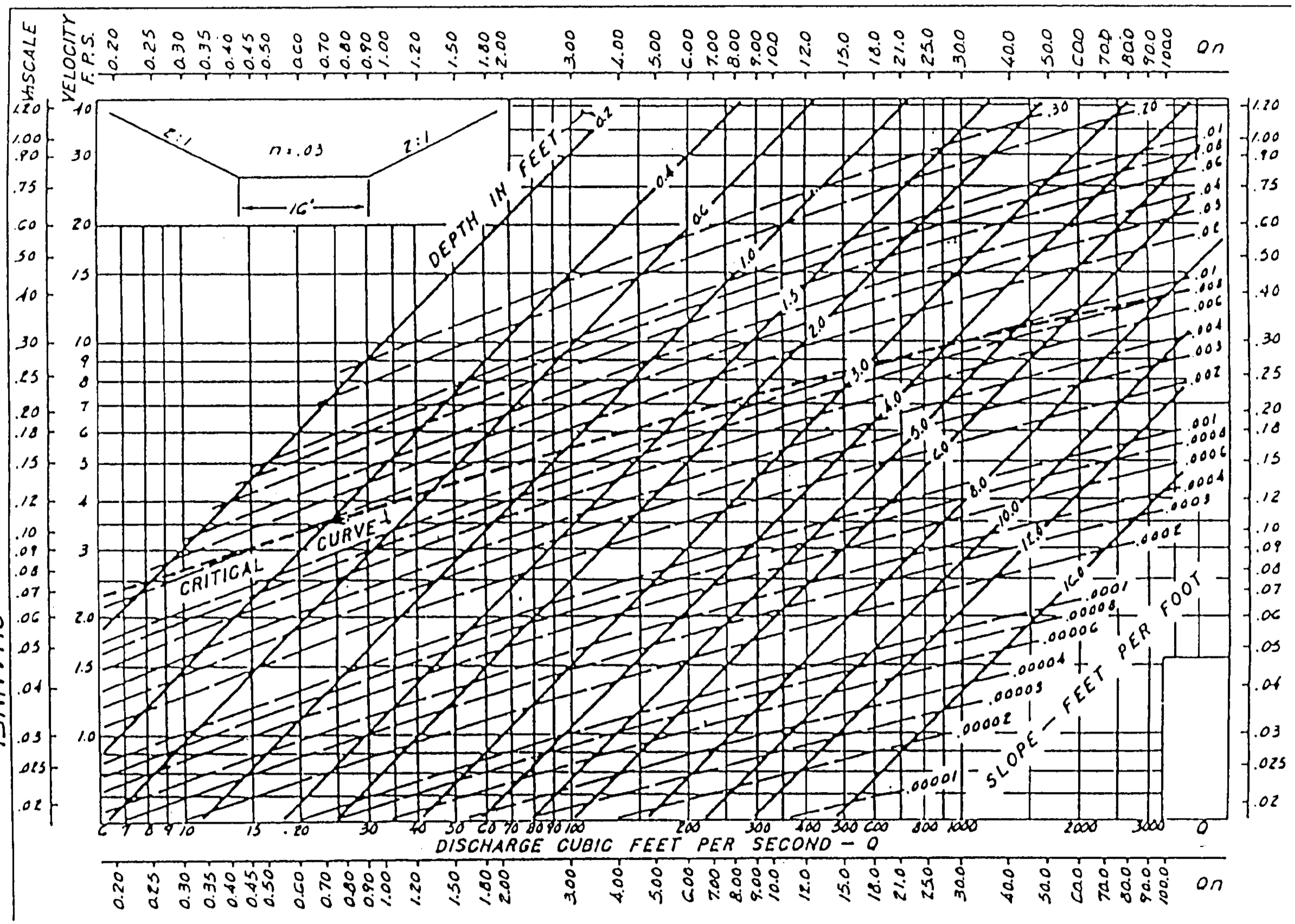
Source: Federal Highway Administration




CHANNEL
2:1 b = 14 FT.

Source: Federal Highway Administration

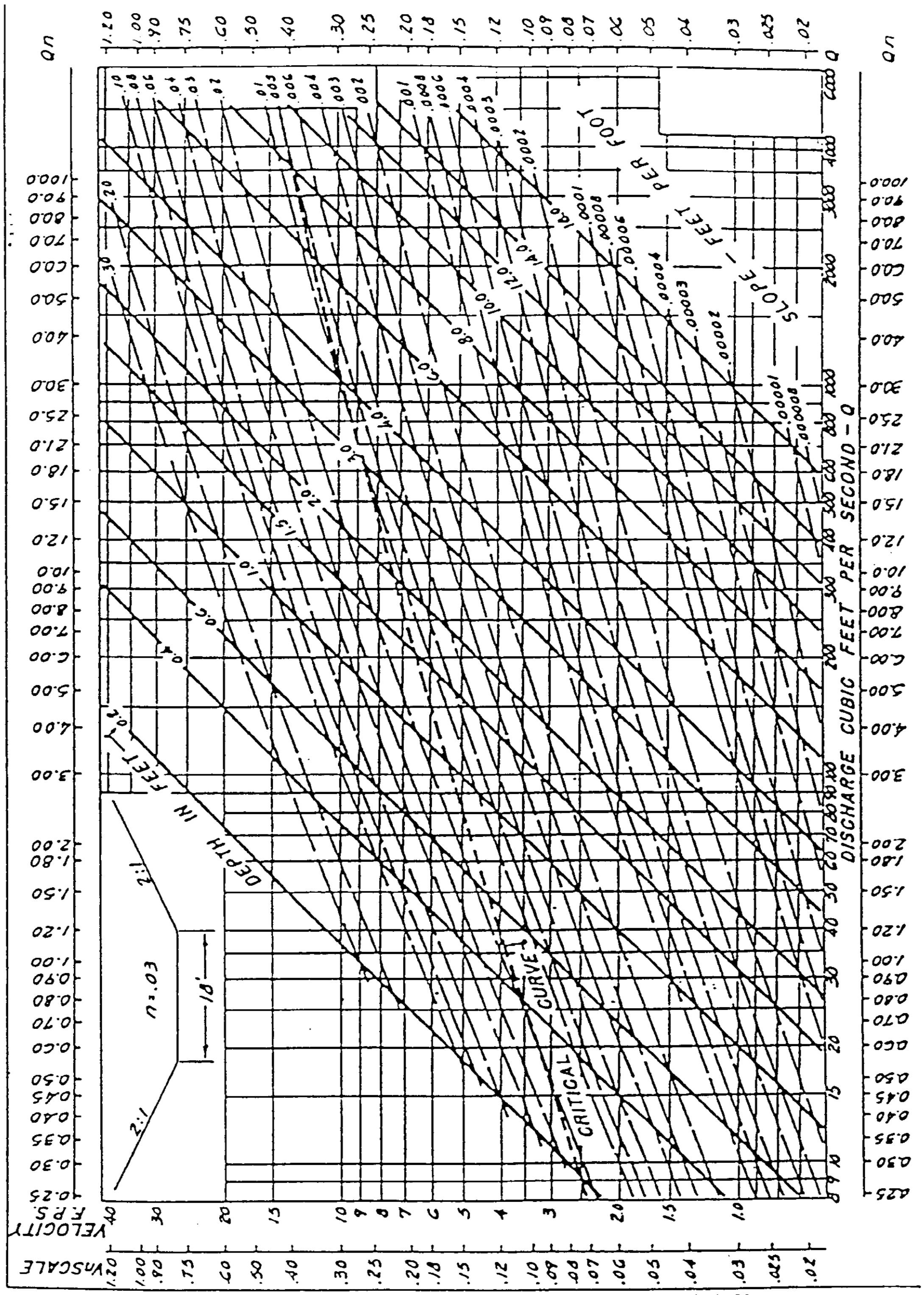
BACK TO TOC




CHANNEL
2:1 b = 16 FT.

Source: Federal Highway Administration

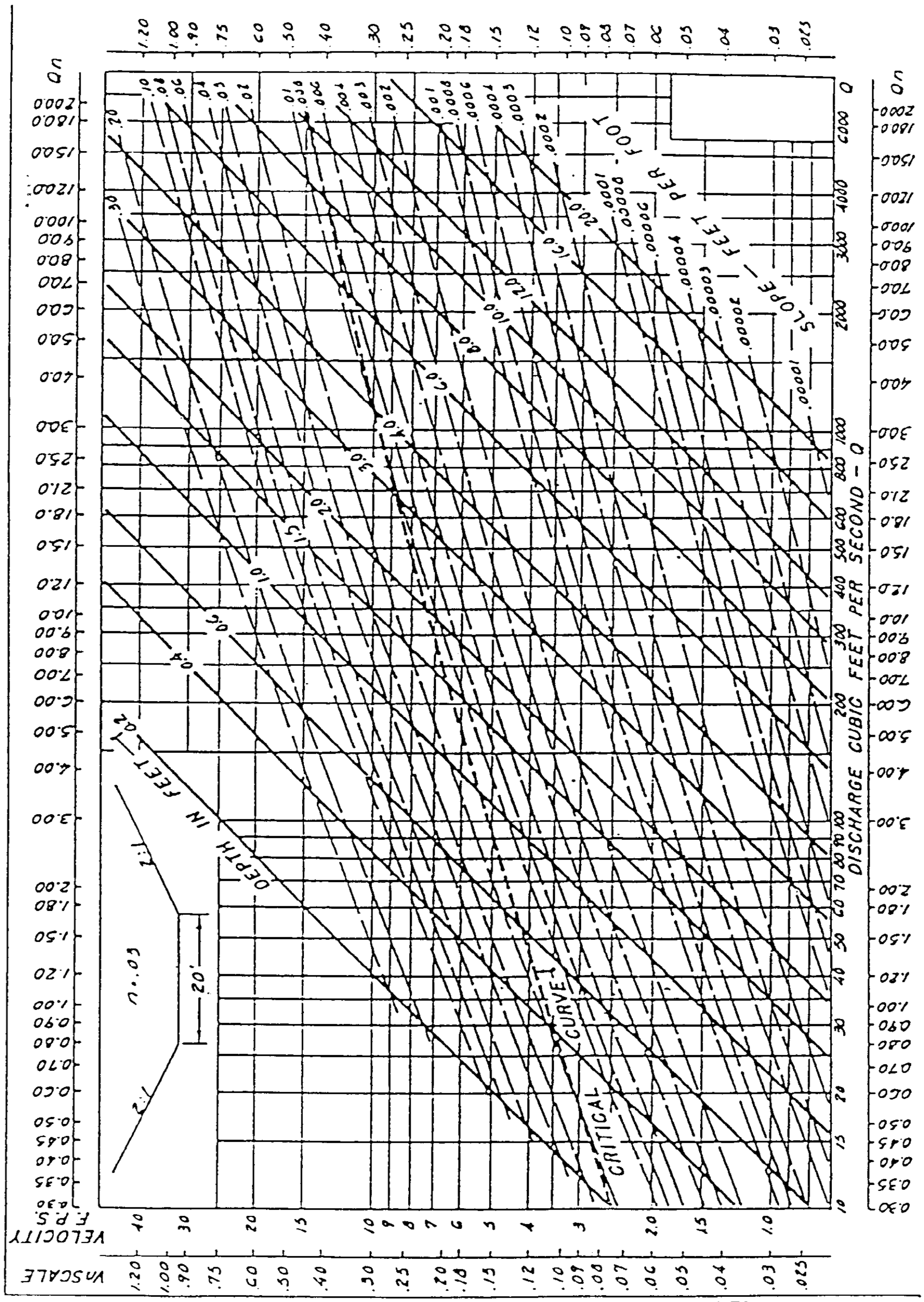
BACK TO TOC




CHANNEL
2:1 b = 18 FT.

Source: Federal Highway Administration

BACK TO TOC

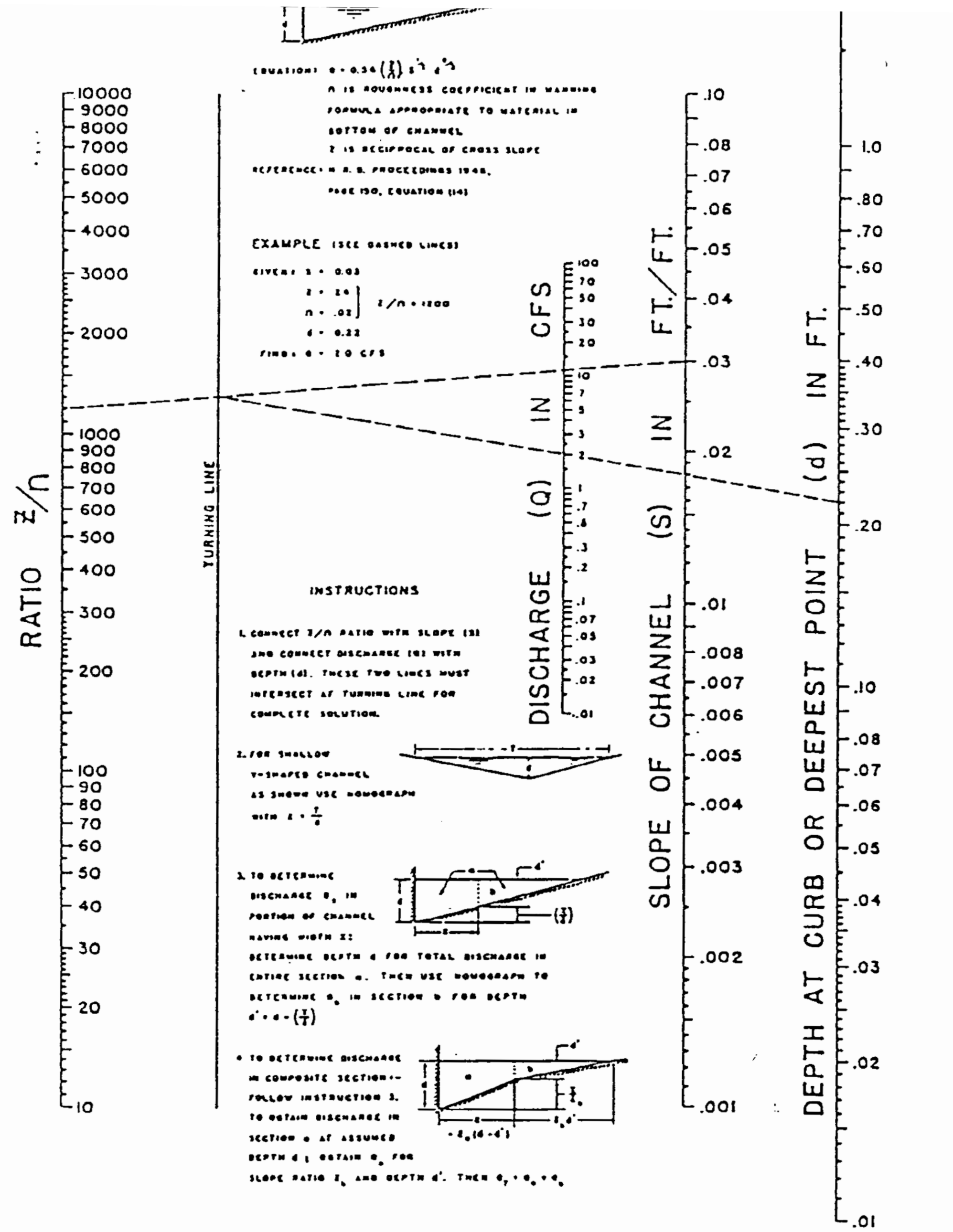



CHANNEL
2:1 b = 20 FT.

Source: Federal Highway Administration

BACK TO TOC

# C-3 Triangular Channel Nomograph

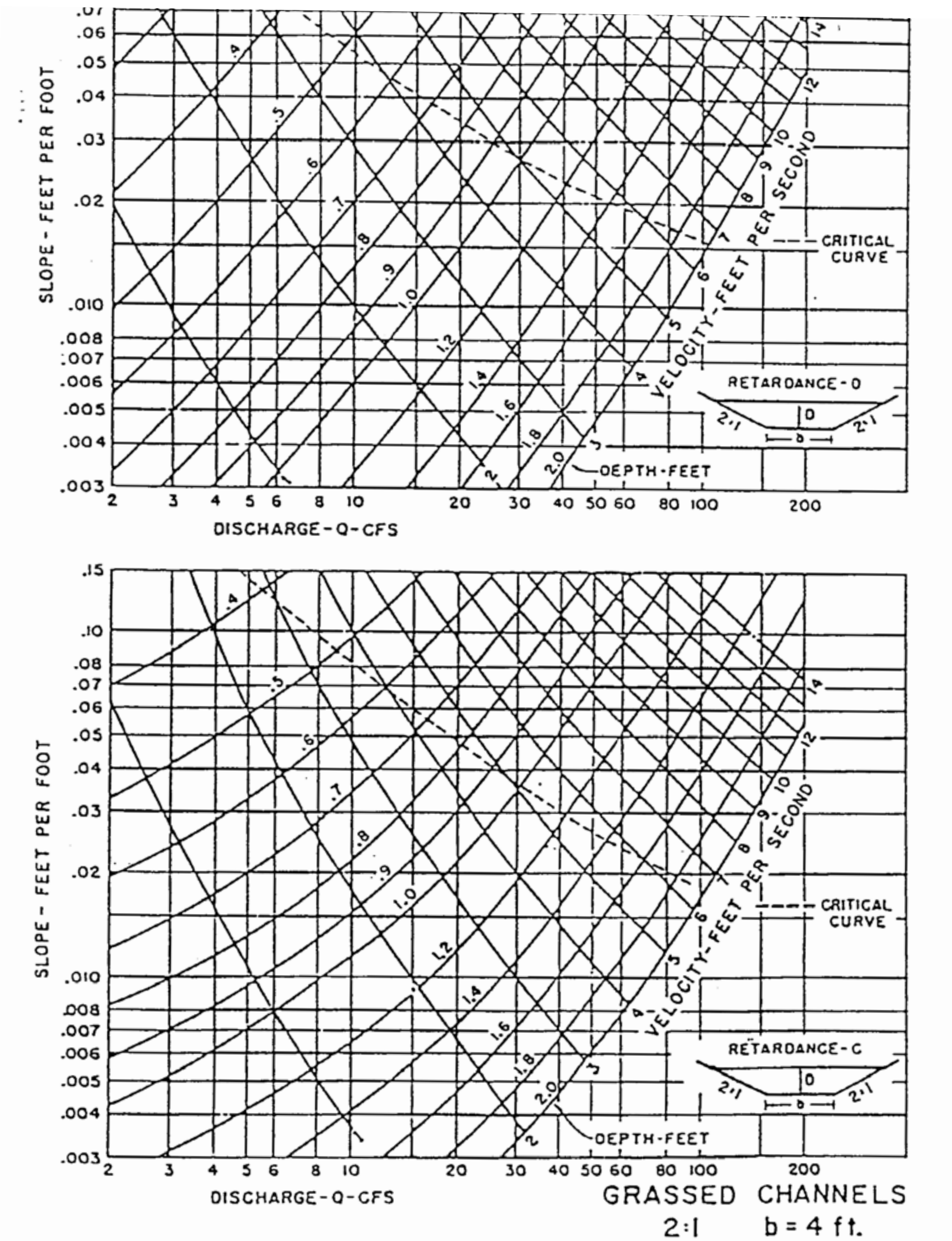



NOMOGRAPH FOR FLOW IN TRIANGULAR CHANNELS

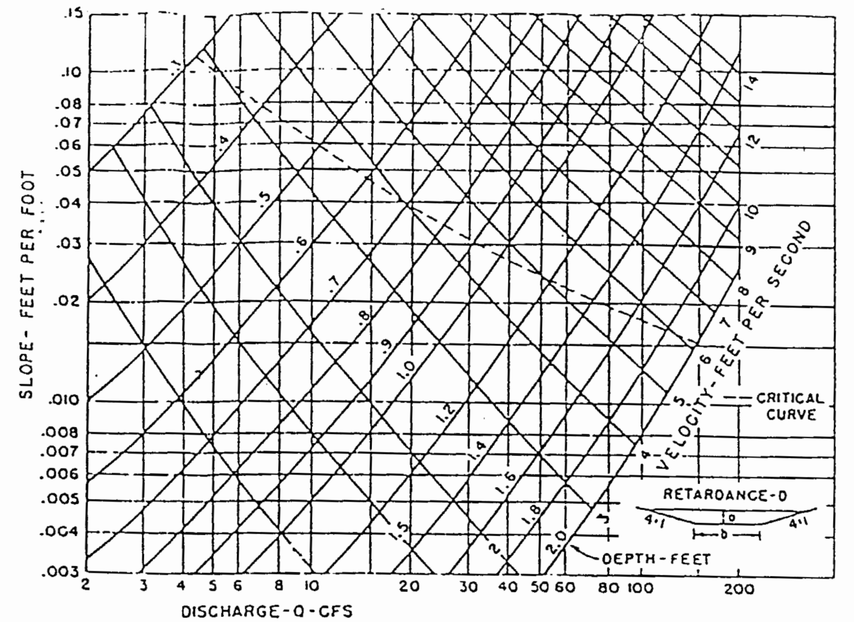
Source: Federal Highway Administration

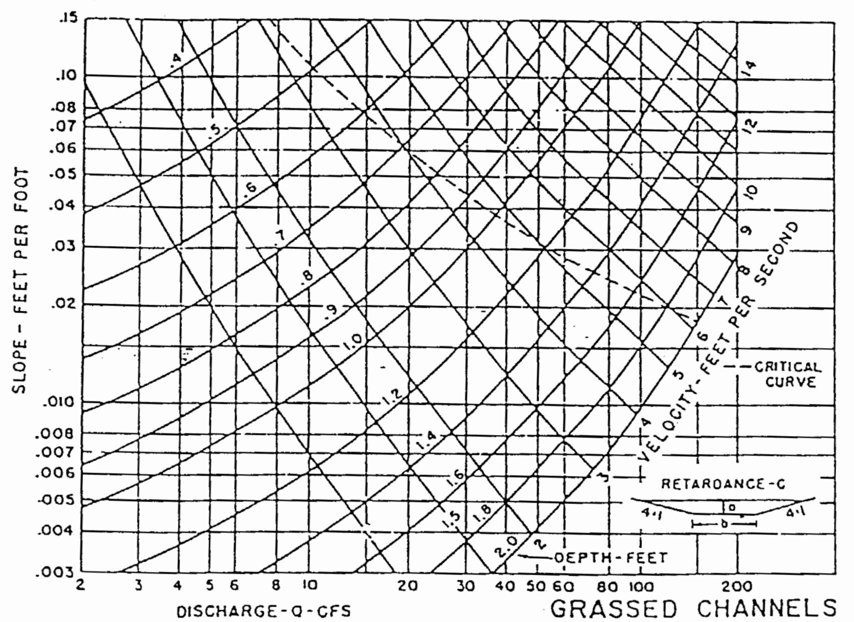
BACK TO TOC

# C-4 Grassed Channel Design Figures



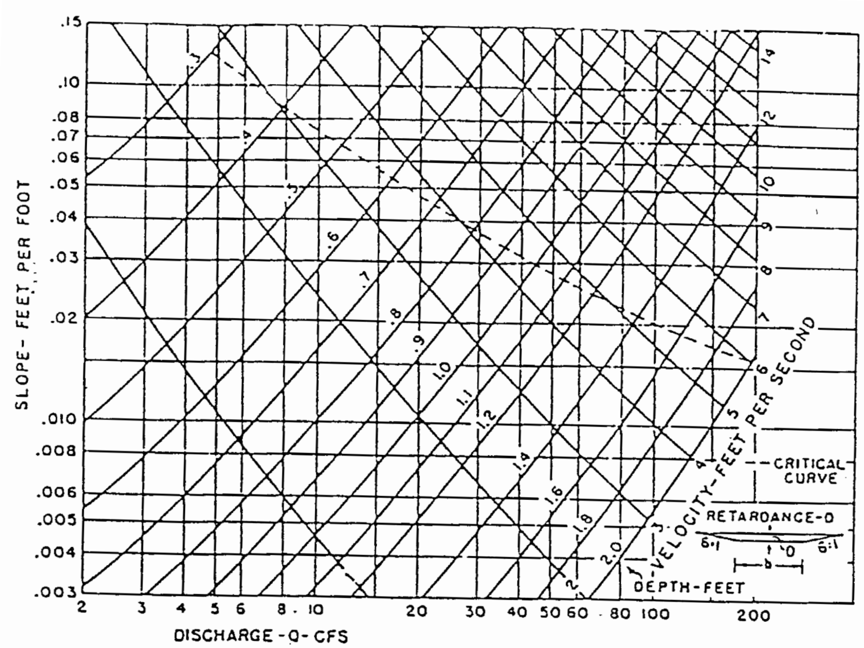

Source: Federal Highway Administration

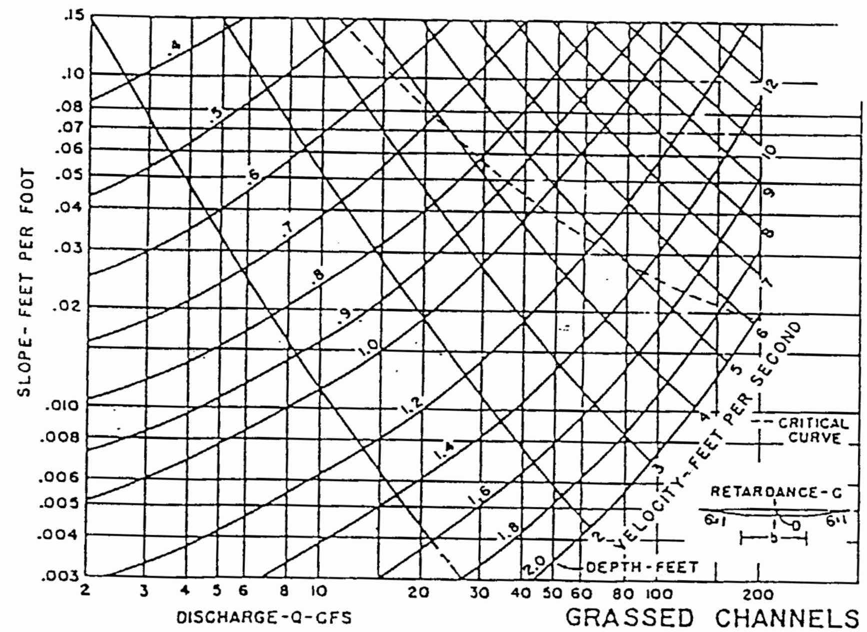



GRASSED CHANNELS
4:1 b = 4 ft.

Source: Federal Highway Administration

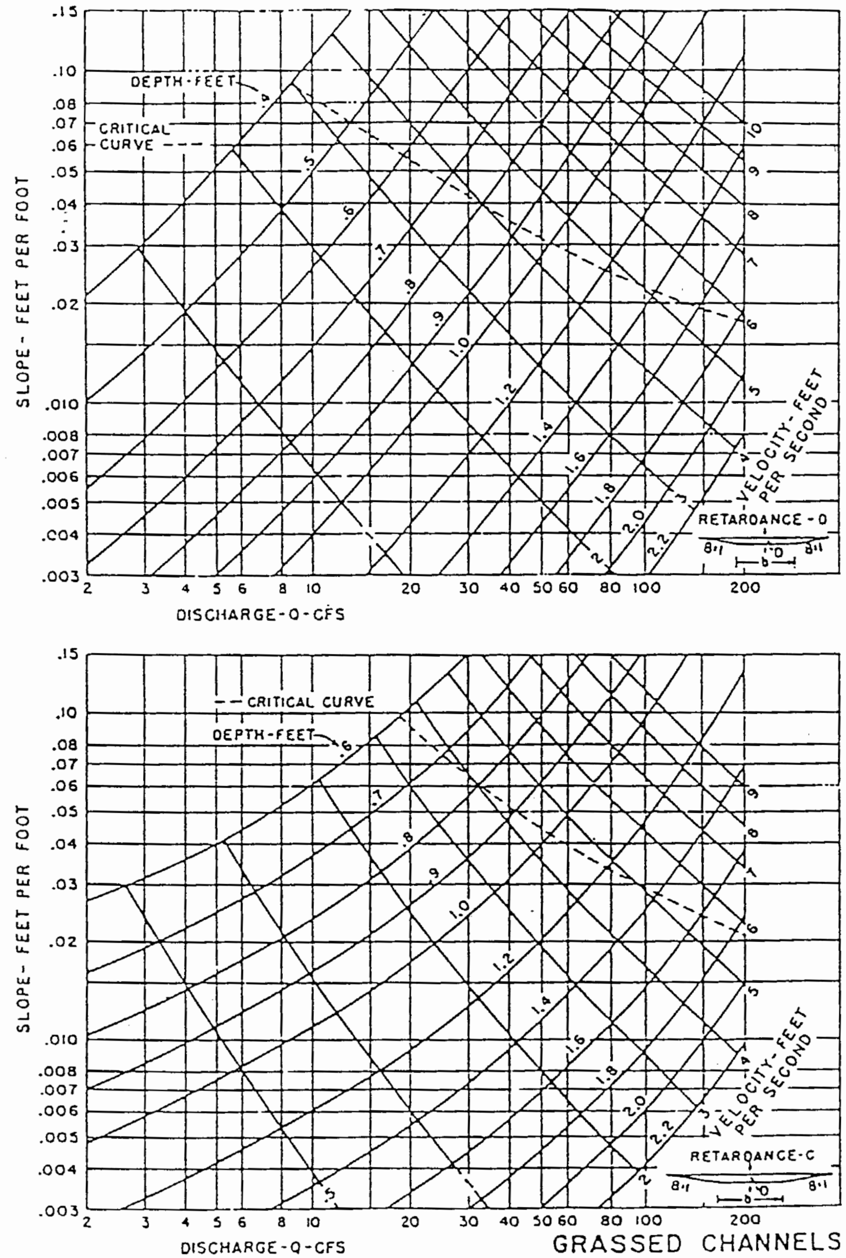



GRASSED CHANNELS
6:1 b = 4 ft.

Source: Federal Highway Administration

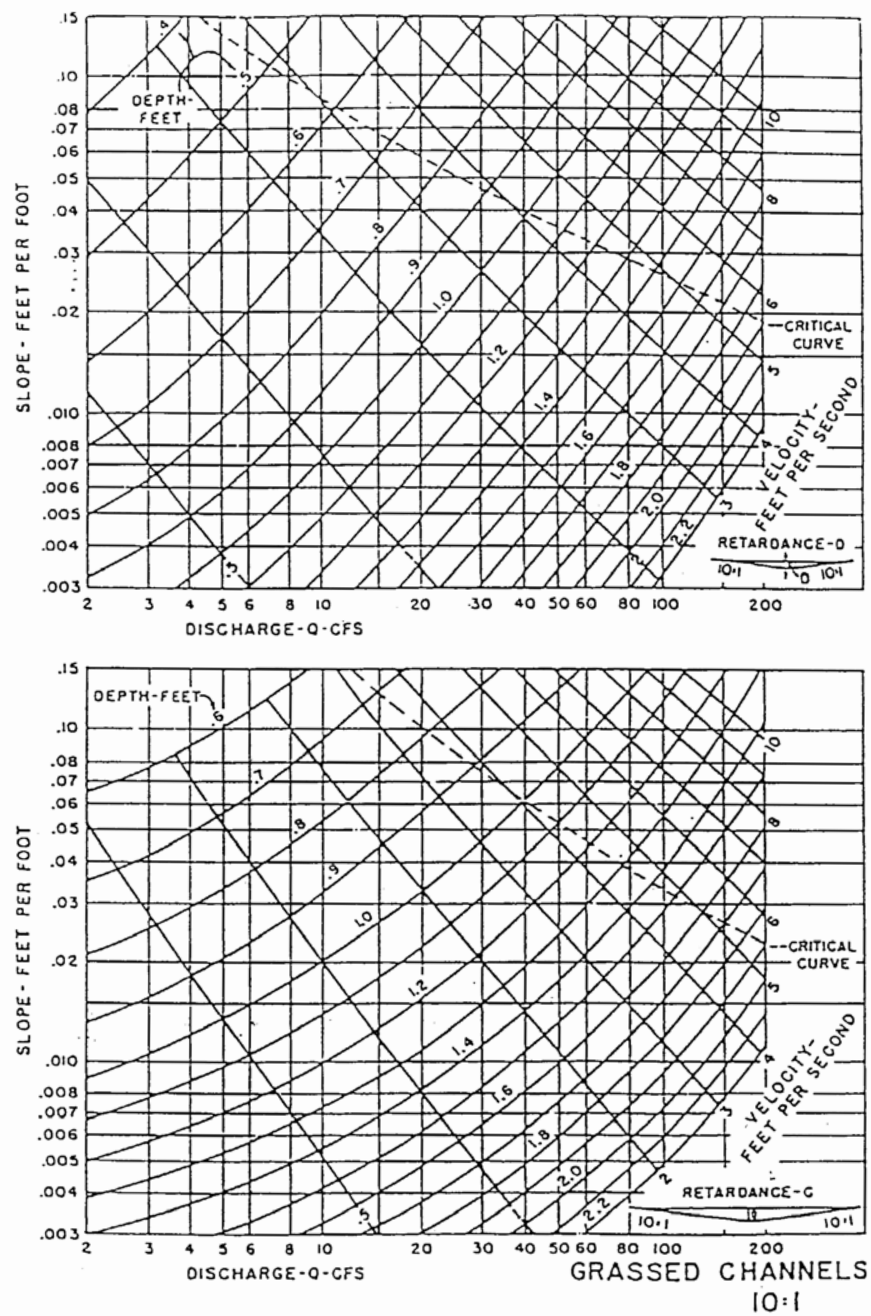
BACK TO TOC




GRASSED CHANNELS

8:1 b = 4 ft.

Source: Federal Highway Administration




Source: Federal Highway Administration

# Appendix D: Planting & Soil Guidance

## Introduction

The proper selection of plants and soil composition specifications is a critical aspect to the function and success of many stormwater BMPs.

**Plants**: Plants reduce erosion, increase pollutant removal, reduce runoff velocity, enhance infiltration, control access and contribute to wildlife habitat. Plants can also improve the appearance of stormwater facilities.

**Soil:** The soil composition of many stormwater BMPs is vital to their relative success or failure. Properly specified, mixed and placed soil and/or planting media aids in infiltration and natural detention as well as plant health. The purpose of this Appendix is to provide guidance on plant selection and planting media for stormwater facilities. This appendix is divided into the following sections:

Section 1: General Planting Guidance

Section 2: Site Characteristics and Soil - discusses the physical site factors and considerations involved in selecting plant material for stormwater facility landscaping.

Section 3: Plant Selection for Stormwater Facilities - reviews key factors to consider in selecting plant material for stormwater landscaping, including hardiness, physiographic regions, inundation tolerance, and other factors.

Section 4: Specific Landscaping Criteria for BMPs

Section 5: Plant List for Stormwater BMPs

Section 6: Minimum Requirements for Landscape Plans

Section 7:  Plant Establishment and Maintenance

## Section 1: General Planting Guidance

Below are general guidelines that should be followed in the planting of any stormwater control or conveyance facility. Native plant species should be specified over exotic or foreign species because they are well adapted to local soil and climactic conditions.

- A variety of species should be utilized for the best performance of the stormwater BMP and for survivability of the plants.
- Trees, shrubs and grasses should be placed and spaced according to their mature size.
- Turf areas should be minimized or eliminated because of the maintenance (mowing, pesticides and fertilizers) required after installation. Recent research indicates that disturbed soils and managed turf also impact stormwater quality (Law et al, 2008). Groundcovers can be utilized as an alternative.
- Trees and shrubs should not be planted within 15 feet of the toe of the slope of a dam or from a non-clogging, low flow orifice.
- Tree species with tap roots that seek water (e.g., willow and poplar), should be avoided within 50 feet of sewer or water pipes or manmade drainage structures that continually hold water.
- Woody vegetation should not be planted on an embankment and herbaceous embankment plantings should be limited to 10" in height to allow inspectors to look for integrity compromising burrowing rodents.
- Evergreen trees and trees which produce relatively little leaf-fall (once per year) are preferred in areas draining to a detention practice.
- Site characteristics (e.g. views to be preserved, buffer, screens, slopes, existing soil) and plant characteristics, including plant maintenance should be considered when selecting plant material. Larger plant material should not block views at entrances, exits, or difficult road curves.
- Access for maintenance and blocking of access to pedestrians should both be considered.
- Drought tolerant species are recommended.
- Plants should be purchased from a source near the installation site for the best chance of survival.
- Existing vegetation should be considered before siting of stormwater BMP and preserved when possible.
- Water tolerances of existing vegetation should be analyzed.
- Water availability should be considered (see Section 7: Plant Establishment and Maintenance)
- Educational signage should be provided near stormwater BMPs to help educate the public and preserve/identify existing natural vegetation.

BACK TO TOC

## Section 2: Site Characteristics and Soil

A development site's characteristics often will help to determine which plant materials and planting methods the site designer should select and will help improve plant establishment. Primary site considerations include:

1. Soil Characteristics
2. Drainage
3. Slope
4. Orientation

Soils provide nutrients for plant growth, filtration of pollutants as well as the storage/release of storm flows which makes soils a critical piece of a successful stormwater BMP.

**Soil Tests**: To determine the characteristics of soil on your site, samples of on-site soils should be analyzed by experienced and qualified individuals who can explain the results and provide information on any soil amendments that are required for the intended stormwater BMP. For information about testing your own soil and to order a soil test kit you can contact your local extension service (UGA-Cooperative Extension Office here: http://extension.uga.edu/about/county/index.cfm or click here: http://aesl.ces.uga.edu/soiltest123/Georgia.htm to order a soil test kit). The test should indicate soil PH, content and texture. Where poor soils can't be amended, seed mixes and plant material must be selected to establish ground cover as quickly as possible.

**Utilizing On-Site Soils:** A soil test will help to determine if on-site soils can be utilized for your BMP. There are drawbacks to utilizing on-site soils which include the staging area required to mix soils, weather conditions making the blending process difficult, availability of clean organic content and control over the performance of the soil created which is particularly important for bioretention area soils. If it is decided that on-site soils can be utilized, recognize that both weight and volume of soil media components are routinely specified and stay consistent between the two.

**Utilizing a Manufactured Soil Media:** Several companies in Georgia are now manufacturing soil media for BMP's and can create mixes that meet the nutrient levels and textures that you require. The manufacturer should be able to provide a specification for their mix and to show the soil components, installation instructions, and infiltration rates (if applicable).

**Soil Performance:** Different planting media with different ratios of organic content, sand/clay content, permeability and pH will perform in different ways. Higher organic content is better for plants but can cause nutrient export. Higher sand content will increase permeability but can decrease the ability of the soil to hold proper nutrients to sustain plant life. Increasing or decreasing the clay in the planting media can slow or speed infiltration. An analysis of desired plant material, specific site conditions and desired function of the proposed BMP will need to be performed to determine the mix that is best suited. At this time there is no universal ASTM standard for planting media. Links are provided below for different planting media.

- See guidance for bioretention media composition: http://stormwater.pca.state.mn.us/index.php/Design_criteria_for_bioretention
- http://www.lowimpactdevelopment.org/epa03/biospec_left.htm
- North Carolina State University Stormwater Engineering Group Soil Media Mix: https://www.bae.ncsu.edu/topic/bioretention/design-soil.html
- Page 367 / G1 http://www.sandiegocounty.gov/content/dam/sdc/dpw/WATERSHED_PROTECTION_PROGRAM/susmppdf/lid_handbook_2014sm.pdf

BACK TO TOC

## D.2.1 Infiltration Testing

The purpose of green infrastructure (GI) or green management practices (GMPs) is to store, treat and infiltrate stormwater into the soil, mimicking natural systems. Subsurface conditions are key in assessing the feasibility of infiltration in the design of GI. Infiltration capacity testing and design of best management practices (BMPs) that rely on infiltration to treat the stormwater runoff shall follow the specifications summarized in this Section. While the Natural Resources Conservation Service (NRCS) soil classification of the site is encouraged as part of a desktop analysis to gain familiarity with potential native soil conditions, it is not adequate justification for infiltration testing results and cannot be substituted for infiltration testing using infiltrometers, test pits or other infiltration testing methods.

### Background and Desktop Analysis

A desktop analysis of soils data, topography, the location of streams, waterbodies, existing/previous land uses, and structures is encouraged to identify potential BMP locations and types. Existing or previous soil investigation or lab data may also be used to support preliminary siting of BMPs and infiltration testing. While NRCS soil classification of the site is encouraged as part of a desktop analysis to gain familiarity with potential native soil conditions, it is not adequate justification for infiltration testing results and cannot be substituted for infiltration testing using infiltrometers or test pits.

### Infiltration Testing Requirements

Infiltration tests shall not be conducted in the rain or within 48 hours of significant rainfall events (greater than 0.5 inches), or when the temperature is below freezing.

Infiltration testing performed; including testing procedures followed, shall be documented and submitted (as required) to the governing agency.

Infiltration testing shall be conducted by a qualified professional and plans including infiltration testing results must be certified by a professional engineer or professional geologist. However, homeowners may perform their own simple infiltration tests to site rain gardens in the proper location.

Portions of this Section present testing methods at the bottom of an excavation. It is the testing personnel's responsibility to be aware of and take proper health and safety precautions for activities in an excavation. See the U.S. Occupational Health and Safety Administration (OSHA) for guidelines and requirements (www.osha.gov).

### Tiered Testing Approach

A tiered approach to infiltration testing recognizes the importance of accurate in situ conditions while screening out sites unsuitable for infiltration practices and thereby reducing soil investigation and testing costs. The following tiers include:

1. Feasibility Analysis
2. Conceptual Design Testing

Minimum testing recommendations for each tier are summarized in **Table D-1**, on the following page.

**Tier 1: Feasibility Analysis**

A minimum of one single-ring infiltrometer test must initially be performed on site.

*Single-Ring Infiltrometer Infiltration Test*

This test method utilizes perforated 200 mm to 250 mm (8-inch to 10-inch) plastic or metal canisters with bottom, set in coarse drainage sand, to minimize disturbance to in-place soils and to prevent siltation of the test hole during testing.

1. Holes in the test canister should be 3 mm (1/8 inch) diameter and spaced on 25 mm (1 inch) centers.
2. Excavate a test hole to the depth of the infiltration plane, or the bottom of the BMP and approximately 25 mm (1 inch) larger diameter and approximately 25 mm (1 inch deeper than the dimensions of a test canister. If the depth of testing is greater than 18", it may be necessary to excavate a shallow test pit to conduct testing.
3. Check that the sides of the test hole are not smooth, but scarified.
4. Place coarse drainage sand in the bottom of the hole and place the canister firmly into the hole. The bottom of the hole should be uncompacted.
5. Backfill the space around the canister with soil and tamp the soil into place.
6. Fill canister with water and allow to drain completely or to soak the surrounding soils for a minimum of one hour, whichever occurs first. Re-fill the canister and measure the rate at which the water level drops.
7. Record the infiltration rate as the decrease in depth of water per hour (inches/hour).

Where the feasibility analysis does not meet minimum infiltration criteria, the designer may prefer the use of an underdrain rather than continue with further testing.

Where the feasibility analysis meets the minimum infiltration criteria, the test pits are necessary for conducting infiltration testing per **Table D-1** to further verify site information characteristics.

**Table D-1** Testing/Design Considerations

| BMP Type | **Tier 1:** Feasibility Analysis | **Tier 2:** Conceptual Design Testing | |
|---|---|---|---|
| | | For initial yields equal to or greater than 0.5"/hr | For initial yields less than 0.5"/hr |
| **Linear practices** (i.e., bioswales, interconnected tree boxes, infiltration trenches, etc.) | 1 single-ring infiltrometer test per site | 1 single-ring infiltrometer test and 1 test pit per 400 linear feet (minimum 1 infiltration test per test pit) of BMP practice | Underdrain required |
| **Non-linear practices** (rain gardens, basins, etc.) | 1 single-ring infiltrometer test per site | 1 single-ring infiltrometer test and 1 test pit per 400 square feet of practice area (minimum 1 infiltration test per test pit) | Underdrain required |

**Tier 2: Conceptual Design Testing**

*Test Pit Infiltration Test*

This test method consists of a trench or pit that allows visual observation of the soil horizons and overall soil conditions at a particular location on the site. Multiple test pit observations can be made for a relatively low cost and in a short time period. The use of soil borings shall not be substituted for test pits. Test pits (see **Figure D-1**) allow in-situ visual observation of soil conditions, where soil borings do not. Soil borings are encouraged to supplement data collection, but cannot be substituted for infiltrometer or test pits.

1. Dig a backhoe-excavated trench/pit, 2-1/2 to 3 feet wide, to the proposed depth of the infiltration plane of the practice, or until bedrock or fully saturated conditions are encountered. Safe test pit entry should always be observed. A test pit should never be accessed if it is not safe to do so. OSHA regulations should always be observed.
2. Document soil profile (soil horizons, soil texture and color and depth below ground surface, depth to water table, depth to bedrock, etc).
3. Based on observed field conditions, the qualified professional should consider modifying the proposed infiltration plane of the practice and adjust infiltration testing locations as necessary.
4. Perform Single-Ring Infiltrometer test (above) at depth of infiltration plane of the proposed practice.
5. Soil samples may be collected at various horizons for additional analysis at the designer's discretion.
6. After testing is complete, re-fill test pit with original native soils and stake the location of the test pit.

**Other Infiltration Testing and Verification Methods**

Other infiltration testing standards that are acceptable include ASTM D3385—09 Standard Test Method for Infiltration Rate of Soils in Field Using Double Ring Infiltrometer.

Several simple infiltration tests do exist for homeowners and small sites to determine if a rain garden or small bioretention area can be sited in a specific place. One example of a simple test can be found here: http://www.phillywatersheds.org/whats_in_it_for_you/residents/infiltration-test.

Verification methods such as soil borings may be used to verify site conditions where final locations of BMPs are adjusted and do not fall within the original testing location. Test results must verify that the soil conditions are the same as those from the original test results.

Designers should also consider construction access and staging during the design process. Activities that could compact soils where BMPs are sited should be avoided. Where site constraints make this unavoidable, the designer shall compensate accordingly in the design of the BMP.

**Construction Equipment and Minimizing Compaction**

Soils should not be compromised by compaction from construction equipment. Care should be taken to minimize soil compaction throughout the BMP and especially at the plane of infiltration so that infiltration rates of native soils are not impacted. Acceptable excavation methods at infiltration practices include hand labor with shovels or the use of an excavator such as a backhoe or trackhoe (located outside the perimeter or footprint of the practice). Heavy equipment should never be used over the footprint of existing or planned infiltration practices. Prior to site disturbance, the perimeter of the practice should be partitioned off with temporary fencing/tape to keep heavy equipment from crossing the perimeter throughout time of active construction. In cases where the BMP is sufficiently large that equipment must enter it, methods proposed to limit and restore compacted soil must be approved in advance.

**Long-term Infiltration Rates**

Infiltration rates may decrease over time due to settlement of filter media, compaction, or accumulation of sediment in the practice. To sustain infiltration rates long-term, it is important that a maintenance plan is in place. Regular maintenance should be conducted to optimize operating infiltration rates.

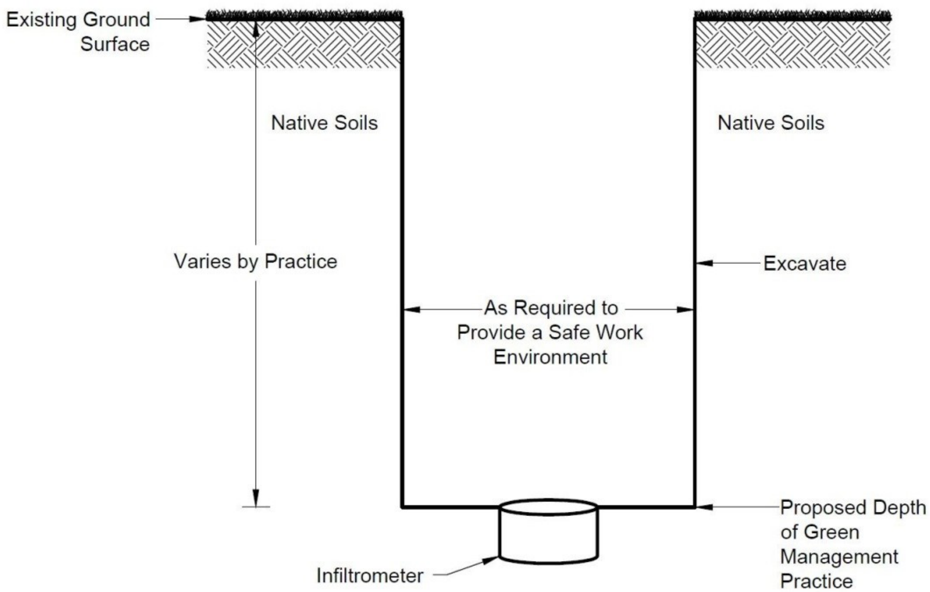



Figure D-1: Test Pit

### D.2.2 Compaction, Construction and Soil

Areas that have recently been involved in construction as well as some native soils can become compacted so that plant roots cannot penetrate the soil. Seeds lying on the surface of compacted soils can be washed away or be eaten by birds. Soils should be loosened to a minimum depth of four inches, preferably to a six-inch depth. Hard soils may require a deeper depth. Loosening soils will improve seed contact with the soil, provide greater germination rates, and allow the roots to penetrate into the soil. Sod and other plantings will also benefit from loosened soil.

### D.2.3 Soil Characteristics

The ability of the soil to store and release water and to provide plant establishment and plant growth can be limited by a number of different soil characteristics such as:

- Soil texture
- Soil Permeability
- pH -- whether acid, neutral, or alkali
- Nutrient levels -- nitrogen, phosphorus, potassium
- Minerals -- such as chelated iron, lime
- Salinity
- Toxicity

**Soil texture**: is determined by the percentage of sand, silt, and clay in the soil. The structure of a soil is influenced by soil texture and also by the combination of small soil particles into larger particles. The amount of aggregation in a soil is strongly influenced by the amount of organic matter present. Soils are made up of four basic ingredients: mineral elements, pore space, organic matter and other items consisting mainly of living organisms including fungi, bacteria, and nematodes. One classification of soils is based upon the mineral part of soil and consists of four sizes of particles. Clay particles are the smallest, followed by silt, sand, and gravel. The USDA has devised another system of classifying soil particles. In this system soil is divided into seven categories: clay, silt, and five sizes of sand.

**Soil Permeability**: Soil permeability is an important design factor in stormwater BMPs. It is advantageous and sometimes necessary to have high permeability in-situ soils for systems where infiltration may be desired (e.g. bioretention, infiltration practices, etc.). It is also advantageous and sometimes necessary to have low permeability in-situ soil for systems where permanent ponded water is required (e.g. stormwater wetlands, wet detention basins, etc.). In some BMP systems (e.g. sand filters, bioretention, etc.), high permeability media is required within the BMP, but since relatively small quantities are typically required, suitable soils can be imported to a site if necessary.

**pH**: Soil pH affects the acidity of a soil. A pH of 7 is considered to be neutral. A pH less than 7 is acidic and solutions with a pH greater than 7 are basic. Different trees, shrubs and grasses thrive in different pH levels and the availability of nutrients is optimal at a pH between 6.5 and 7.5.

**Organic Content/Compost**: Vegetation cannot survive without the proper amount of organic matter. Organic content requirements vary based on the site, the BMP and plants that are being selected. Organic content affects pollutant removal rates: higher organic content filters more pollutants than lower organic content but depending on the sources of the organic amendment, it can raise phosphorus levels which can become a contaminat. Compost utilized to meet the organic matter content within planting media should be a well-decomposed, stable, weed-free organic matter source derived from waste materials including yard debris, wood wastes or other organic materials, not including manure or biosolids. Compost shall have a dark brown color and a soil-like odor. Compost that is exhibiting a sour or putrid smell, contains recognizable grass or leaves, or is hot (120 degrees Fahrenheit) upon delivery is not acceptable. Examples of acceptable material include pine bark fines and leaf compost.

**Topsoil**: Topsoil provides organic matter, nutrients, and microbes beneficial to the plant material. This also allows the stabilizing plant materials to become established faster, while the roots are able to penetrate deeper and stabilize the soil, making it less likely that the plants will wash out during a heavy storm. It is important to understand the composition of the topsoil that is being utilized. For example, the clay content of topsoil affects the infiltration rate of the planting media. Also, the organic content in topsoil must be taken into consideration when calculating the total organic content in a bioretention planting media.

Whenever possible, the topsoil on a site should be stripped and stockpiled prior to any grading activities, After final grading, stockpiled topsoil should be spread to a depth of six inches (or more depending on the size of the plant material to be sustained) over the entire area to be planted. If stockpiled topsoil is insufficient, off-site soils and amendments may need to be brought in. If topsoil has been stockpiled in deep mounds for a long period of time, it is desirable to test the soil for pH as well as microbial activity. If the microbial activity has been destroyed, it may be necessary to inoculate the soil after application.

Topsoil should conform to ASTM D5268 – 13:

| Total Sample Compositional Category: | Percentage by Mass |
|---|---|
| Deleterious materials (rock, gravel, slag, cinder, roots, sod) | 5 max material passing the No. 10 (2 mm) sieve |
| Organic material | 2 to 20 |
| Sand content | 20 to 60 |
| Silt and clay content | 35 to 70 |
| pH | 5 to 7 |

Source: (ASTM D5268-13, Standard Specification for Topsoil Used for Landscaping Purposes, ASTM International, West Conshohocken, PA, 2013, www.astm.org)

On or off-site topsoil should be obtained from sources that are naturally well-drained sites where topsoil occurs at least 4 inches (100 mm) deep, not from bogs, or marshes; and that do not contain undesirable organisms; disease-causing plant pathogens; or obnoxious weeds and invasive plants including, but not limited to, quackgrass, Johnsongrass, poison ivy, nutsedge, nimblewill, Canada thistle, bindweed, bentgrass, wild garlic, ground ivy, perennial sorrel, and bromegrass.

BACK TO TOC

## D.2.4 Drainage

Soil moisture and drainage have a direct bearing on the plant species and communities that can be supported on a site. Factors such as soil texture, topography, groundwater levels and climatic patterns all influence soil drainage and the amount of water in the soil. Identifying the topography and drainage of the site will help determine potential moisture gradients. The following categories can be used to describe the drainage properties of soils on a site:

- ❑ Flooded
  Areas where standing water is present most of the growing season.
- ❑ Wet
  Areas where standing water is present most of the growing season, except during times of drought. Wet areas are found at the edges of ponds, rivers, streams, ditches, and low spots. Wet conditions exist on poorly drained soils, often with a high clay content.
- ❑ Moist
  Areas where the soil is damp. Occasionally, the soil is saturated and drains slowly. These areas usually are at slightly higher elevations than wet sites. Moist conditions may exist in sheltered areas protected from sun and wind.
- ❑ Well-drained
  Areas where rain water drains readily, and puddles do not last long. Moisture is available to plants most of the growing season. Soils usually are medium textures with enough sand and silt particles to allow water to drain through the soil.
- ❑ Dry
  Areas where water drains rapidly through the soil. Soils are usually coarse, sandy, rocky or shallow. Slopes are often steep and exposed to sun and wind. Water runs off quickly and does not remain in the soil.

## D.2.5 Slope

The degree of slope can also limit its suitability for certain types of plants. Plant establishment and growth requires stable substrates for anchoring root systems and preserving propagules such as seeds and plant fragments, and slope is a primary factor in determining substrate stability. Establishing plants directly on or below eroding slopes is not possible for most species. In such instances, plant species capable of rapid spread and anchoring soils should be selected or bioengineering techniques should be used to aid the establishment of a plant cover.

In addition, soils on steep slopes generally drain more rapidly than those on gradual slopes. This means that the soils may remain saturated longer on gradual slopes. If soils on gradual slopes are classified as poorly drained, care should be taken that plant species are selected that are tolerant of saturation.

Site topography also affects maintenance of plant species diversity. Small irregularities in the ground surface (e.g., depressions, etc.) are common in natural systems. More species are found in areas with many micro-topographic features than in areas without such features. Raised sites are particularly important in wetlands because they allow plants that would otherwise die while flooded to escape inundation.

In wetland plant establishment, ground surface slope interacts with the site hydrology to determine water depths for specific areas within the site. Depth and duration of inundation are principal factors in the zonation of wetland plant species. A given change in water levels will expose a relatively small area on a steep slope in comparison with a much larger area exposed on a gradual or flat slope. Narrow planting zones will be delineated on steep slopes for species tolerant of specific hydrologic conditions, whereas gradual slopes enable the use of wider planting zones.

## D.2.6 Orientation

Slope exposure should be considered for its effect on plants. A southern-facing slope receives more sun and is warmer and drier, while the opposite is true of a northern slope. Eastern- and western-facing slopes are intermediate, receiving morning and afternoon sun, respectively. Western-facing slopes tending to be drier and receive more wind.

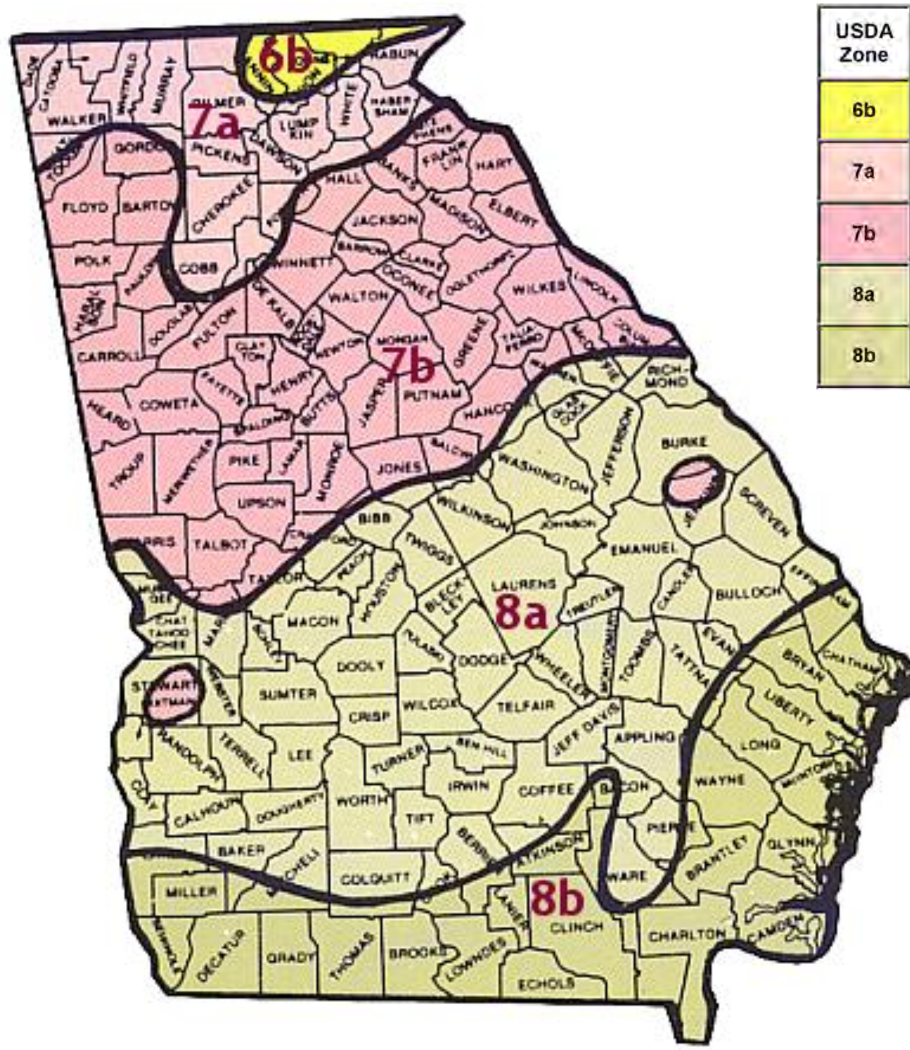
BACK TO TOC

## Section 3: Plant Selection for Stormwater Facilities

### D.3.1 Hardiness Zones

Hardiness zones are based on historical average annual minimum temperatures recorded in an area. A site's location in relation to plant hardiness zones is important to consider first because plants differ in their ability to withstand very cold winters. This does not imply that plants are not affected by summer temperatures. Given that Georgia summers can be very hot, heat tolerance is also a characteristic that should be considered in plant selection.

It is best to recommend plants known to thrive in specific hardiness zones. The plant list included at the end of this appendix identifies the hardiness zones for each species listed as a general planting guide. It should be noted, however, that certain site factors can create microclimates or environmental conditions which permit the growth of plants not listed as hardy for that zone. By investigating numerous references and based on personal experience, a designer should be able to confidently recommend plants that will survive in microclimates.




| USDA Zone | Average Annual Minimum Temperature Range |
|---|---|
| 6b | -5 to 0 F<br>(-17.8 to -20.5 C) |
| 7a | 0 to 5 F<br>(-15.0 to -17.7 C) |
| 7b | 5 to 10 F<br>(-12.3 to -14.9 C) |
| 8a | 10 to 15 F<br>(-9.5 to -12.2 C) |
| 8b | 15 to 20 F<br>(-6.7 to -9.4 C) |

Figure D-2: USDA Plant Hardiness Zones in Georgia

BACK TO TOC

### D.3.2 Physiographic Provinces

There are five physiographic provinces in Georgia that describe distinct geographic regions in the state with similar physical and environmental conditions (**Figure F-2**). These physiographic provinces include, from northwest to southeast, Appalachian Plateau, Ridge and Valley, Blue Ridge, Piedmont and Coastal Plain (subdivided into upper and lower regions). Each physiographic region is defined by unique geological strata, soil type, drainage patterns, moisture content, temperature and degree of slope which often dictate the predominant vegetation. Because the predominant vegetation has evolved to live in these specific conditions it is important to understand the region in which your stormwater facility will reside. The five physiographic regions are described here: http://www.georgiawildlife.com/node/1704 with an associated map: http://www1.gadnr.org/cwcs/PDF/ga_eco_l3_pg.pdf.

### D.3.3 Other Considerations in Plant Selection

The landscape planting design must include elements that ensure plant survival and overall stormwater BMP functional success. Plant selection is a complex task, involving matching the plant's physiological characteristics with a site's particular environmental conditions. The following factors should be considered:

- Full grown plant size and shape (e.g., limbs growing into power lines, maintenance access impediment).
- Growth rate.
- Site conditions (e.g., wind direction and intensity, street lighting, type and quantity of pollutants contained within stormwater runoff, etc.).
- Soil moisture and drought tolerance.
- Tolerance of periodic inundation.
- Sediment and organic matter build-up.
- Potential for outlet structure clogging (e.g., root structure).
- Maintenance requirements.
- Wildlife use

Plants need to provide aesthetics/ability to meet both landscape and stormwater BMP requirements. In urban or suburban settings, a plant's aesthetic interest may be of greater importance. Residents living next to a stormwater system may desire that the facility be appealing or interesting to look at throughout the year. Aesthetics is an important factor to consider in the design of these systems. Failure to consider the aesthetic appeal of a facility to the surrounding residents may result in reduced value to nearby lots. Careful attention to the design and planting of a BMP can result in maintained or increased values of a property.

Individual plants often have physiological characteristics difficult to convey in a general list. It is necessary to investigate specific information to ensure successful plant selection. There are many resources available to guide designers in the selection of plant material for stormwater BMPs. Knowledgeable landscape architects, wetland scientists, urban foresters, and nursery suppliers provide valuable information for considering specific conditions for successful plant establishment and accounting for the variable nature of stormwater hydrology.

#### D.3.3.1 AVAILABILITY AND COST

Often overlooked in plant selection is the availability from wholesalers and the cost of the plant material. There are many plants listed in landscape books that are not readily available from the nurseries. Without knowledge of what is available, time spent researching and finding the one plant that meets all the needs will be wasted, if it is not available from the growers. It may require shipping, therefore, making it more costly than the budget may allow. Some planting requirements, however, may require a special effort to find the specific plant that fulfills the needs of the site and the function of the plant in the landscape. Plants should be sourced from similar geographic regions to optimize survivability.

BACK TO TOC

#### D.3.3.2 NATIVE VERSUS NONNATIVE SPECIES

This Manual encourages the use of native plants in stormwater management facilities since they are best suited to thrive and can provide benefits to pollinators and other native fauna. Unfortunately native plants may not always be available in quantity from local nurseries. Therefore, non-invasive naturalized plants that can survive under difficult conditions may be a useful alternative. Great care should be taken, however, when introducing plant species so as not to create a situation where they may become invasive and take over adjacent natural plant communities. The Georgia Exotic Pest Plant Council keeps a list of species that are invasive or could become invasive: http://www.gaeppc.org/list/.

#### D.3.3.3 MOISTURE STATUS

In landscaping stormwater management facilities, hydrology plays a large role in determining which species will survive in a given location.

For areas that are to be planted within a stormwater management facility it is necessary to determine what type of hydrologic zones will be created.

The six zones in the next section describe the different conditions encountered in stormwater management facilities. Every facility does not necessarily reflect all of these zones. The hydrologic zones designate the degree of tolerance the plant exhibits to differing degrees of inundation by water. Each zone has its own set of plant selection criteria based on the hydrology of the zone, the stormwater functions required of the plant and the desired landscape effect.

BACK TO TOC

# Section 4: Specific Landscaping Criteria for Structural Stormwater Controls

## D.4.1 Stormwater Ponds and Wetlands

Stormwater ponds and wetlands are engineered basins and wetland areas designed to control and treat stormwater runoff. Aquatic vegetation plays an important role in pollutant removal in both stormwater ponds and wetlands. In addition, vegetation can enhance the appearance of a pond or wetland, stabilize side slopes, serve as wildlife habitat, and can temporarily conceal unsightly trash and debris.

Within a stormwater pond or wetland, there are various hydrologic zones as shown in **Table D-2** that must be considered in plant selection. These hydrologic zones designate the degree of tolerance a plant must have to differing degrees of inundation by water. Hydrologic conditions in an area may fluctuate in unpredictable ways; thus the use of plants capable of tolerating wide varieties of hydrologic conditions greatly increases the successful establishment of a planting. Plants suited for specific hydrologic conditions may perish when those conditions change, exposing the soil, and therefore, increasing the chance for erosion. Each of the hydrologic zones is described in more detail below along with examples of appropriate plant species.

**Table D-2** Hydrolic Zones

| Zone # | Zone Description | Hydrologic Conditions |
|---|---|---|
| Zone 1 | Deep Water Pool | 1-6 feet depth (permanent pool) |
| Zone 2 | Shallow Water Bench | Normal pool elevation to 1 foot depth |
| Zone 3 | Shoreline Fringe | Regularly inundated |
| Zone 4 | Riparian Fringe | Periodically inundated |
| Zone 5 | Floodplain Terrace | Infrequently inundated |
| Zone 6 | Upland Slopes | Seldom or never inundated |

**Zone 1: Deep Water Area (1- 6 Feet)**

Ponds and wetlands both have deep pool areas that comprise Zone 1. These pools range from one to six feet in depth, and are best colonized by submergent plants, if at all.

This pondscaping zone is not routinely planted for several reasons. First, the availability of plant materials that can survive and grow in this zone is limited, and it is also feared that plants could clog the stormwater facility outlet structure. In many cases, these plants will gradually become established through natural recolonization (e.g., transport of plant fragments from other ponds via the feet and legs of waterfowl). If submerged plant material is commercially available and clogging concerns are addressed, this area can be planted. The function of the planting is to reduce resedimentation and improve oxidation while creating a greater aquatic habitat.

- ❏ Plant material must be able to withstand constant inundation of water of one foot or greater in depth.
- ❏ Plants may be submerged partially or entirely.
- ❏ Plants should be able to enhance pollutant uptake.
- ❏ Plants may provide food and cover for waterfowl, desirable insects, and other aquatic life.

Some suggested emergent or submergent species include, but are not limited to: Water Lily, Deepwater Duck Potato, Spatterdock, Wild Celery and Redhead Grass.

**Zone 2: Shallow Water Bench (Normal Pool To 1 Foot)**
Zone 2 includes all areas that are inundated below the normal pool to a depth of one foot, and is the primary area where emergent plants will grow in stormwater wetlands. Zone 2 also coincides with the aquatic bench found in stormwater ponds. This zone offers ideal conditions for the growth of many emergent wetland species. These areas may be located at the edge of the pond or on low mounds of earth located below the surface of the water within the pond. When planted, Zone 2 can be an important habitat for many aquatic and nonaquatic animals, creating a diverse food chain. This food chain includes predators, allowing a natural regulation of mosquito populations, thereby reducing the need for insecticidal applications.

- ❏ Plant material must be able to withstand constant inundation of water to depths between six inches and one foot deep.
- ❏ Plants will be partially submerged.
- ❏ Plants should be able to enhance pollutant uptake.
- ❏ Plants may provide food and cover for waterfowl, desirable insects and other aquatic life.

Common emergent wetland plant species used for stormwater wetlands and on the aquatic benches of stormwater ponds include, but are not limited to: Arrowhead/Duck Potato, Soft Rush, various Sedges, Softstem Bulrush, Cattail, Switchgrass, Southern Blue-Flag Iris, Swamp Hibiscus, Swamp Lily, Pickerelweed, Pond Cypress and various Asters.

**Zone 3: Shoreline Fringe (Regularly Inundated)**
Zone 3 encompasses the shoreline of a pond or wetland, and extends vertically about one foot in elevation from the normal pool. This zone includes the safety bench of a pond, and may also be periodically inundated if storm events are subject to extended detention. This zone occurs in a wet pond or shallow marsh and can be the most difficult to establish since plants must be able to withstand inundation of water during storms, when wind might blow water into the area, or the occasional drought during the summer. In order to stabilize the soil in this zone, Zone 3 must have a vigorous cover.

- ❏ Plants should stabilize the shoreline to minimize erosion caused by wave and wind action or water fluctuation.
- ❏ Plant material must be able to withstand occasional inundation of water. Plants will be partially submerged partially at this time.
- ❏ Plant material should, whenever possible, shade the shoreline, especially the southern exposure. This will help to reduce the water temperature.
- ❏ Plants should be able to enhance pollutant uptake.
- ❏ Plants may provide food and cover for waterfowl, songbirds, and wildlife. Plants could also be selected and located to control overpopulation of waterfowl.
- ❏ Plants should be located to reduce human access, where there are potential hazards, but should not block the maintenance access.
- ❏ Plants should have very low maintenance requirements, since they may be difficult or impossible to reach.
- ❏ Plants should be resistant to disease and other problems which require chemical applications (since chemical application is not advised in stormwater ponds).

Many of the emergent wetland plants that perform well in Zone 2 also thrive in Zone 3. Some other species that do well include Broom Grass, Upland Sea-Oats, Dwarf Tickseed, various Ferns, Hawthorns. If shading is needed along the shoreline, the following tree species are suggested: Boxelder, Ash, Willow, Red Maples and Willow Oak.

**Zone 4: Riparian Fringe (Periodically Inundated)**
Zone 4 extends from one to four feet in elevation above the normal pool. Plants in this zone are subject to periodic inundation after storms, and may experience saturated or partly saturated soil inundation. Nearly all of the temporary extended detention (ED) storage area is included within this zone.

- ❏ Plants must be able to withstand periodic inundation of water after storms, as well as occasional drought during the warm summer months.
- ❏ Plants should stabilize the ground from erosion caused by run-off.
- ❏ Plants should shade the low flow channel to reduce the pool warming whenever possible.
- ❏ Plants should be able to enhance pollutant uptake.
- ❏ Plant material should have very low maintenance, since they may be difficult or impossible to access.
- ❏ Plants may provide food and cover for waterfowl, songbirds and wildlife. Plants may also be selected and located to control overpopulation of waterfowl.
- ❏ Plants should be located to reduce pedestrian access to the deeper pools.

Some frequently used plant species in Zone 4 include Broom Grass, Yellow Indian Grass, Ironweed, Joe Pye Weed, Lilies, Flatsedge, Native Hollies, Forsythia, Lovegrass, Hawthorn and Sugar Maples.

**Zone 5: Floodplain Terrace (Infrequently Inundated)**
Zone 5 is periodically inundated by flood waters that quickly recede in a day or less. Operationally, Zone 5 extends from the maximum two year or $CP_v$ water surface elevation up to the 25 or 100 year maximum water surface elevation. Key landscaping objectives for Zone 5 are to stabilize the steep slopes characteristic of this zone, and establish a low maintenance, natural vegetation.

- ❏ Plant material should be able to withstand occasional but brief inundation during storms, although typical moisture conditions may be moist, slightly wet, or even swing entirely to drought conditions during the dry weather periods.
- ❏ Plants should stabilize the basin slopes from erosion.
- ❏ Ground cover should be very low maintenance, since they may be difficult to access on steep slopes or if the frequency of mowing is limited. A dense tree cover may help reduce maintenance and discourage resident geese.
- ❏ Plants may provide food and cover for waterfowl, songbirds, and wildlife.
- ❏ Placement of plant material in Zone 5 is often critical, as it often creates a visual focal point and provides structure and shade for a greater variety of plants.

Some commonly planted species in Zone 5 include many wildflowers or native grasses, many Fescues, many Viburnums, Witch Hazel, Blueberry, American Holly, American Elderberry and Red Oak.

**Zone 6: Upland Slopes (Seldom or Never Inundated)**
The last zone extends above the maximum 100 year water surface elevation, and often includes the outer buffer of a pond or wetland. Unlike other zones, this upland area may have sidewalks, bike paths, retaining walls, and maintenance access roads. Care should be taken to locate plants so they will not overgrow these routes or create hiding places that might make the area unsafe.

- ❏ Plant material is capable of surviving the particular conditions of the site. Thus, it is not necessary to select plant material that will tolerate any inundation. Rather, plant selections should be made based on soil condition, light, and function within the landscape.
- ❏ Ground covers should emphasize infrequent mowing to reduce the cost of maintaining this landscape.
- ❏ Placement of plants in Zone 6 is important since they are often used to create a visual focal point, frame a desirable view, screen undesirable views, serve as a buffer, or provide shade to allow a greater variety of plant materials. Particular attention should be paid to seasonal color and texture of these plantings.

Some frequently used plant species in Zone 6 include most ornamentals (as long as soils drain well, many wildflowers or native grasses, Linden, False Cypress, Magnolia, most Spruce, Mountain Ash and most Pine.



BACK TO TOC

Specific plants appropriate for wetlands are listed in the Plant List in Section 5.

The figures below (**Figures D-3** to **D-6**) illustrate a sample design of plantings within a constructed wetland:

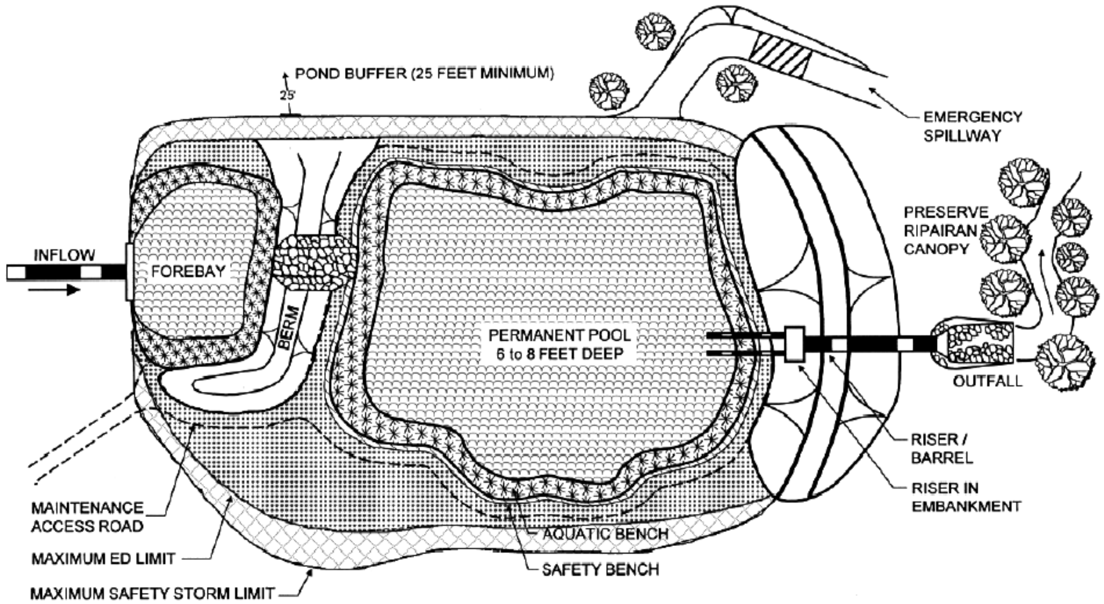



Figure D-3: Plan View of Hydrologic Zones around Stormwater Wet ED Pond

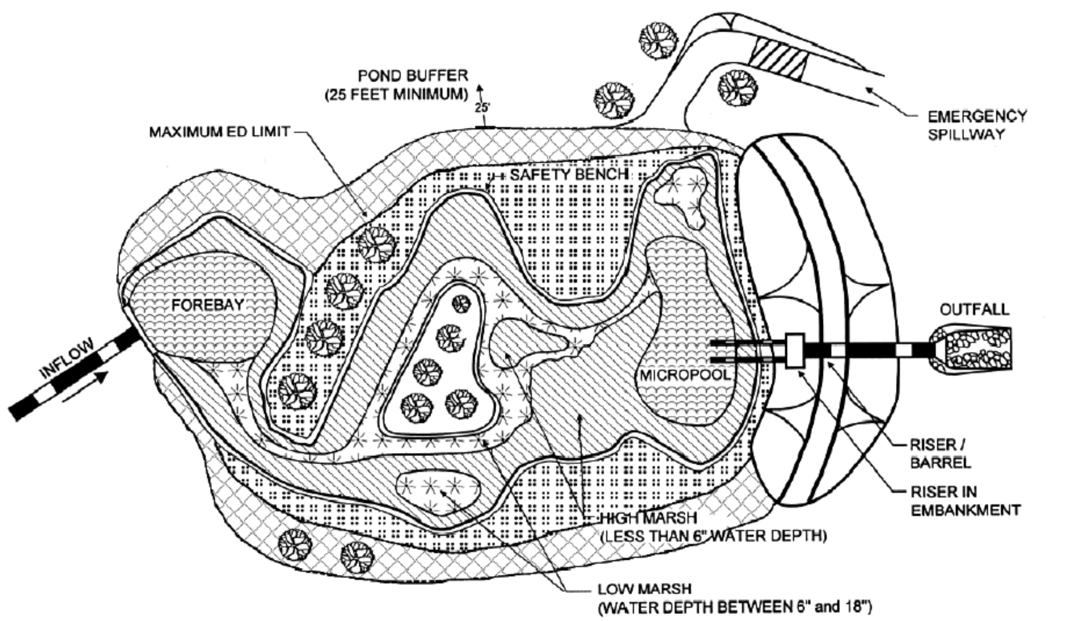



Figure D-4: Plan View of Hydrologic Zones around Stormwater ED Shallow Wetland

BACK TO TOC

*12 to 36 inch depth below normal pool elevation*
Water Lily, Deep Water Duck Potato, Spatterdock, Wild Celery, Redhead Grass

*0 to 12 inch depth below normal pool elevation*
Arrowhead/Duck Potato, Soft Rush, various Sedges, Softstem Bulrush, Cattail, Switchgrass, Southern Blue Flag Iris, Swamp Hibiscus, Swamp Lily, Pickerelweed, Pond Cypress, various Asters

*0 to 12 inch elevation above normal pool elevation*
Various species from above, Broom Grass, Upland Sea-Oats, Dwarf Tickseed, various Ferns, Hawthorns, Boxelder, Ash, Willow, Red Maple, Willow Oak

*1 to 4 foot elevation above normal pool elevation*
Broom Grass, Yellow Indian Grass, Ironweed, Joe Pye Weed, various Lilies, Flatsedge, Hollies, Lovegrass, Hawthorn, Sugar Maple

*$CP_v$ to $Qp_{25}$ or $Q_f$ water surface elevation*
Many wildflowers or native grasses, many Fescues, many Viburnums, Witch Hazel, Blueberry, American Holly, American Elderberry, Red Oak

*$Q_f$ water surface elevation and above*
Many ornamentals as long as soils drain well, many wildflowers or native grasses, Linden, False Cypress, Magnolia, most Spruce, Mountain Ash, most Pine

Figure D-5 Legend of Hydrologic Zones Around Stormwater Facilities

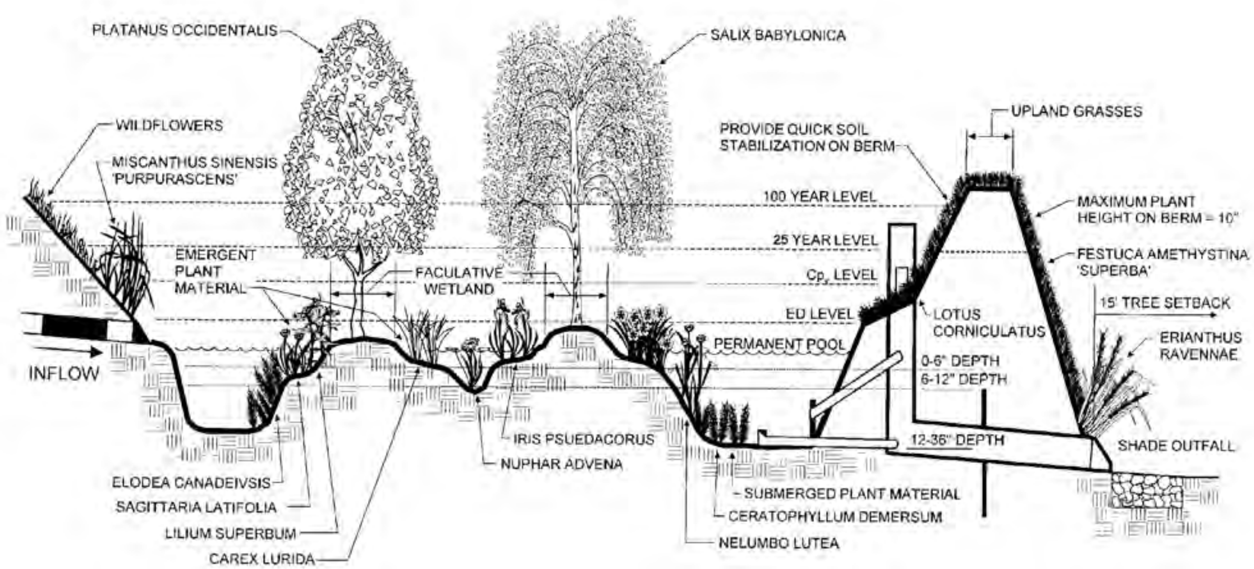



Figure D-6: Section of Typical Shallow ED Wetland

## D.4.2 Bioretention Areas

Bioretention areas are structural stormwater controls that capture and treat runoff using soils and vegetation in shallow basins or landscaped areas. Landscaping is therefore critical to the performance and function of these facilities. Below are guidelines for soil characteristics, mulching, and plant selection for bioretention areas.

### D.4.2.1 PLANTING MEDIA CHARACTERISTICS

Specifying the correct soils is critical in order to achieve stormwater objectives and plant health. Soils must balance three primary design objectives: 1) High enough infiltration rates to meet surface water draw down requirements, 2) infiltration rates that are not so high that they preclude pollutant removal function of soils and 3) soil composition that supports plant establishment and long-term health.

If the native soils cannot suffice for the planting media used within the bioretention area planting beds, then an engineered planting media should be provided that meets the specifications in the BMP write up. Keep in mind that increasing sand content increases permeability in the media but lowers phosphorus levels in the soil which hurts plant survivability.

### D.4.2.2 MULCH LAYER

The mulch layer plays an important role in the performance of the bioretention system. The mulch layer helps maintain soil moisture and avoids surface sealing which reduces permeability. Mulch helps prevent erosion, and provides a micro-environment suitable for soil biota at the mulch/soil interface. It also serves as a pretreatment layer, trapping the finer sediments which remain suspended after the primary pretreatment. The mulch layer should be standard landscape style, single or double, shredded hardwood mulch or chips. The mulch layer should be well aged (stockpiled or stored for at least 12 months), uniform in color, and free of other materials, such as weed seeds, soil, roots, etc. The mulch should be applied to a maximum depth of three inches. Grass clippings should not be used as a mulch material.

Plant material selection should be based on the goal of simulating a terrestrial forested community of native species. Bioretention simulates an ecosystem consisting of an upland-oriented community dominated by trees, but having a distinct community, or sub-canopy, of understory trees, shrubs and herbaceous materials. The intent is to establish a diverse, dense plant cover to treat stormwater runoff and withstand urban stresses from insect and disease infestations, drought, temperature, wind, and exposure.

The proper selection and installation of plant materials is key to a successful system. There are essentially three zones within a bioretention facility (**Figure D-7**). The lowest elevation supports plant species adapted to standing and fluctuating water levels. The middle elevation supports a slightly drier group of plants, but still tolerates fluctuating water levels. The outer edge is the highest elevation and generally supports plants adapted to dryer conditions. A sample of appropriate plant materials for bioretention facilities is included in Section 5.

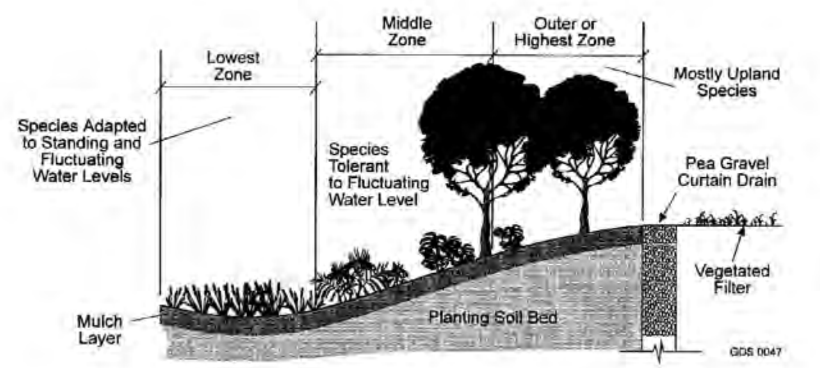



Figure D-7: Planting Zones for Bioretention Facilities

BACK TO TOC

The layout of plant material should be flexible, but should follow the general principals described below. The objective is to have a system that resembles a random and natural plant layout, while maintaining optimal conditions for plant establishment and growth. A sample bioretention planting plan is presented in Figure D-8.

- ❑ Trees provide aesthetic and performance benefits in bioretention areas. When including trees, refer to the following guidelines:
  - Provide sufficient landscape width (a rule of thumb is 8′ min.)
  - Locate trees on the side slopes, not in areas that pond
  - Select trees that will tolerate wet soils.
  - Locate trees with invasive roots away from perforated pipes.
  - Consider behavior and maintenance of plant material. Deciduous plant material that will drop leaves can impair the function of a bioretention area if not located properly or maintained.
- ❑ Native plant species should be specified.
- ❑ Appropriate vegetation should be selected based on the zone of hydric tolerance
- ❑ Species layout should generally be random and natural.
- ❑ The tree-to-shrub ratio should be 2:1 to 3:1.
- ❑ Plants should be placed at regular intervals based on their mature size to replicate a natural forest.
- ❑ Woody vegetation should not be specified at inflow locations.
- ❑ A canopy should be established with an understory of shrubs and herbaceous materials.
- ❑ Woody vegetation should not be specified in the vicinity of inflow locations.
- ❑ Urban stressors (e.g., wind, sun, exposure, insect and disease infestation, drought) should be considered when laying out the planting plan.
- ❑ Noxious weeds should not be specified.
- ❑ Aesthetics and visual characteristics should be a prime consideration.
- ❑ Traffic and safety issues must be considered.
- ❑ Existing and proposed utilities must be identified and considered.

See plant list in Section 5 for species appropriate to bioretention areas.

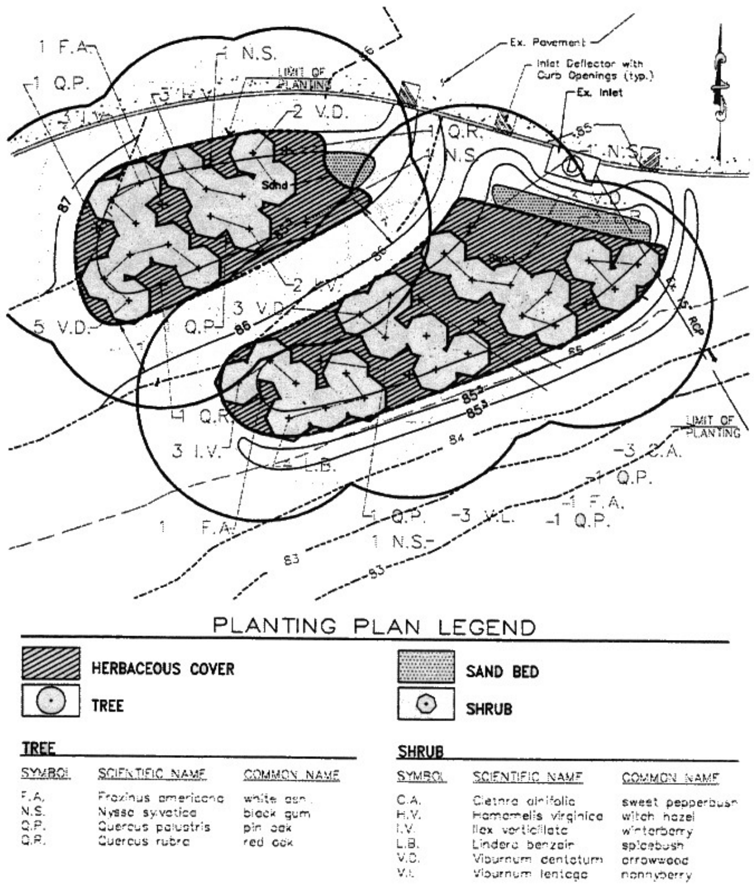



Figure D-8: Sample Bioretention Area Planting Plan
(Source: VDCR, 1999)

BACK TO TOC

# Section 5: Plant Selection for Stormwater Facilities

The following plant list is not representative of every plant that can be utilized in a stormwater bmp, but is meant as a starting point for native plant material. Many factors must be analyzed before selecting plant material types including sun/shade tolerance, flowering and/or fruiting habits, shape and full grown size of species, and local availability.

Many additional online and printed resources exist, including the following:

- Georgia Native Plant Society: http://gnps.org/indexes/Plant_Gallery_Index.php
- UGA Extension Native Plants for Georgia:
  - » Trees, Shrubs and Woody Vines: http://extension.uga.edu/publications/files/pdf/B%20987_8.PDF
  - » Grasses and Sedges: http://extension.uga.edu/publications/files/pdf/B%20987-4_1.PDF
  - » Ferns: http://extension.uga.edu/publications/detail.cfm?number=B987-2
  - » Wildflowers: http://extension.uga.edu/publications/files/pdf/B%20987-3_4.PDF

# Georgia Native Plant List

TREES

| Scientific Name | Common Name | Habit | | | H ZONE* | Hardiness |
|---|---|---|---|---|---|---|
| *Acer negundo* | Boxelder | Tree | Deciduous | Native | 3,4,5 | USDA Zone 2-10 |
| *Acer rubrum* | Red Maple | Tree | Deciduous | Native | 3,4,5 | USDA Zone 3-9 |
| *Asimina triloba* | Common Pawpaw | Tree | Deciduous | Native | 3,4,5 | USDA Zone 5-9 |
| *Betula nigra* | River Birch | Tree | Deciduous | Native | 2,3,4,5 | USDA Zone 4-9 |
| *C Carya aquatica* | Water Hickory | Tree | Deciduous | Native | 3,4 | USDA Zone 4-8 |
| *Carpinus caroliniana* | American Hornbeam | Tree | Deciduous | Native | 4,5 | USDA Zone 3-9 |
| *Carya cordiformus* | Bitternut Hickory | Tree | Deciduous | Native | 3,4,5 | USDA Zone 4-9 |
| *Carya illinoensis* | Pecan | Tree | Deciduous | Native | 3,4,5 | USDA Zone 5b-9a |
| *Carya laciniosa* | Shellbark Hickory | Tree | Deciduous | Native | 2,3,4,5 | USDA Zone 5-8 |
| *Celtis laevigata* | Sugarberry | Tree | Deciduous | Native | 3,4,5 | USDA Zone 6-9 |
| *Chemaecyparis thyoides* | Atlantic White Cedar | Tree | Evergreen | Native | 2,3,4,5 | USDA Zone 4-8 |
| *Cornus drummondii* | Rough-Leaf Dogwood | Tree | Deciduous | Native | 3,4,5 | USDA Zone 5-8 |
| *Crataegus spp.* | Hawthornes | Tree | Deciduous | Native | 4,5 | USDA Zone 5-8 |
| *Diospyros virginiana* | Common Persimmon | Tree | Deciduous | Native | 4,5 | USDA Zone 4-9 |
| *Fraxinus caroliniana* | Carolina Ash | Tree | Deciduous | Native | 3,4,5 | USDA Zone 4-8 |
| *Fraxinus pennsylvanica* | Green Ash | Tree | Deciduous | Native | 3,4,5 | USDA Zone 3-9a |
| *Fraxinus profonda* | Pumpkin Ash | Tree | Deciduous | Native | 3,4,5 | USDA Zone 5-9 |
| *Gordonia laisianthus* | Loblolly Bay | Tree | Evergreen | Native | 3,4 | USDA Zone 6-9 |
| *Juniperus silicicola* | Southern Red Cedar | Tree | Evergreen | Native | 3,4,5 | USDA Zone 8a-10b |
| *Juniperus virginiana* | Eastern Red Cedar | Tree | Evergreen | Native | 3,4,5 | USDA Zone 2-9 |
| *Liquidamber styraciflua* | Sweetgum | Tree | Deciduous | Native | 3,4,5 | USDA Zone 5b-10a |
| *Liriodendron tulipifera* | Yellow Poplar | Tree | Deciduous | Native | 3,4,5 | USDA Zone 4-9 |
| *Magnolia virginiana* | Sweetbay | Tree | Semi-Evergreen | Native | 3,4,5 | USDA Zone 5-10 |
| *Morus rubra* | Red Mulberry | Tree | Deciduous | Native | 4,5 | USDA Zone 4-8 |
| *Myrica cerifera* | Southern Bayberry | Tree | Evergreen | Native | 3,4,5 | USDA Zone 7-10 |
| *Nyssa aquatica* | Water Tupelo | Tree | Deciduous | Native | 3,4,5 | USDA Zone 6-9 |
| *Nyssa ogeche* | Ogeechee Tupelo | Tree | Deciduous | Native | 3,4,5 | USDA Zone 7a-9b |
| *Nyssa sylvatica* | Black Gum/ Swamp Tupelo | Tree | Deciduous | Native | 3,4,5 | USDA Zone 4b-9 |
| *Pinus elliottii* | Slash Pine | Tree | Evergreen | Native | 4,5 | USDA Zone 7-11 |
| *Pinus glabra* | Spruce Pine | Tree | Evergreen | Native | 4,5 | USDA Zone 8,9 |
| *Pinus serotina* | Pond Pine | Tree | Evergreen | Native | 4,5 | USDA Zone 7-9 |
| *Pinus taeda* | Loblolly Pine | Tree | Evergreen | Native | 5 | USDA Zone 6b-9 |
| *Platanus occidentalis* | American Sycamore | Tree | Deciduous | Native | 3,4,5 | USDA Zone 4-9 |

**Hydrologic Zone for Stormwater Pond or Wetland*

BACK TO TOC

# Georgia Native Plant List (continued)

| | Scientific Name | Common Name | Habit | | | H ZONE* | Hardiness |
|---|---|---|---|---|---|---|---|
| TREES | *Populus deltoides* | Eastern Cottonwood | Tree | Deciduous | Native | 4,5 | USDA Zone 3-9 |
| | *Populus heterophylla* | Swamp Cottonwood | Tree | Deciduous | Native | 3,4,5 | USDA Zone 7-8 |
| | *Ptelea trifoliata* | Wafer Ash | Tree | Deciduous | Native | 5 | USDA Zone 4-9 |
| | *Quercus bicolor* | Swamp White Oak | Tree | Deciduous | Native | 3,4,5 | USDA Zone 3-8 |
| | *Quercus laurifolia* | Laurel Oak | Tree | Deciduous | Native | 4,5 | USDA Zone 7-9 |
| | *Quercus lyrata* | Overcup Oak | Tree | Deciduous | Native | 3,4,5 | USDA Zone 5-9 |
| | *Quercus michiauxii* | Swamp Chestnut Oak | Tree | Deciduous | Native | 4,5 | USDA Zone 6-8 |
| | *Quercus nigra* | Water Oak | Tree | Deciduous | Native | 4,5 | USDA Zone 5-9 |
| | *Quercus pagoda* | Cherrybark Oak | Tree | Deciduous | Native | 4,5 | USDA Zone 4-8 |
| | *Quercus palustris* | Pin Oak | Tree | Deciduous | Native | 4,5 | USDA Zone 4-8 |
| | *Quercus phellos* | Willow Oak | Tree | Deciduous | Native | 3,4,5 | USDA Zone 5-9 |
| | *Quercus shumardii* | Shumard Oak | Tree | Deciduous | Native | 4,5 | USDA Zone 5-9 |
| | *Salix caroliniana* | Coastal Plain Willow | Tree | Deciduous | Native | 3,4,5 | USDA Zone 7-8 |
| | *Salix nigra* | Black Willow | Tree | Deciduous | Native | 3,4,5 | USDA Zone 2-8 |
| | *Taxodium distichum var. distichum* | Baldcypress | Tree | Deciduous | Native | 2,3,4 | USDA Zone 4-9 |
| | *Taxodium distichum var. nutans* | Pondcypress | Tree | Deciduous | Native | 2,3,4 | USDA Zone 4-9 |
| | *Ulmus americana* | American Elm | Tree | Deciduous | Native | 3,4,5 | USDA Zone 2-9 |
| | *Ulmus rubra* | Slippery Elm | Tree | Deciduous | Native | 3,4,5 | USDA Zone 3-9 |
| SHRUBS | *Aesculus pariviflora* | Bottlebrush Buckeye | Shrub | Deciduous | Native | 3,4,5 | USDA Zone 4-8 |
| | *Aesculus pavia* | Red Buckeye | Shrub | Deciduous | Native | 3,4,5 | USDA Zone 4-8 |
| | *Alnus serrulata* | Hazel Alder | Shrub | Deciduous | Native | 3,4,5 | USDA Zone 5-8 |
| | *Aronia arbutifolia* | Red Chokeberry | Shrub | Deciduous | Native | 3,4,5 | USDA Zone 4-9 |
| | *Cephalanthus occidentalis* | Common Buttonbush | Shrub | Deciduous | Native | 2,3,4 | USDA Zone 5-9 |
| | *Euonymus atropurpuresu* | Eastern Burning Bush | Shrub | Deciduous | Native | 4,5 | USDA Zone 3-7 |
| | *Fothergilla gardenii* | Fothergilla | Shrub | Deciduous | Native | 4,5 | USDA Zone 5-8 |
| | *Hamemelis virginiana* | Witch Hazel | Shrub | Deciduous | Native | 3,4,5 | USDA Zone 3-8 |
| | *Hypericum densiflorum* | Common St. Johns Wort | Shrub | Deciduous | Native | 4,5 | USDA Zone 5-9 |
| | *Ilex glabra* | Inkberry | Shrub | Evergreen | Native | 3,4,5 | USDA Zone 4-9 |
| | *Ilex verticillata* | Winterberry | Shrub | Deciduous | Native | 2,3,4 | USDA Zone 3-9 |
| | *Illex decidua* | Decidious Holly | Shrub | Deciduous | Native | 3,4,5 | USDA Zone 5-9 |
| | *Juniperus horizontalis* | Creeping Juniper | Shrub | Evergreen | Native | 5 | USDA Zone 3-9 |
| | *Lindera benzoin* | Spicebush | Shrub | Deciduous | Native | 3,4,5 | USDA Zone 4-9 |

**Hydrologic Zone for Stormwater Pond or Wetland*



BACK TO TOC

## Georgia Native Plant List (continued)

| | Scientific Name | Common Name | Habit | | | H ZONE* | Hardiness |
|---|---|---|---|---|---|---|---|
| GRASSES/HERBACEOUS | *Andropogon glomeratus* | Bushy Broom Grass | Grass | Perennial | Native | 3 | USDA Zone 5-9 |
| | *Andropogon virginicus* | Broom Grass | Grass | Perennial | Native | 4 | USDA Zone 5-8 |
| | *Chasmanthium latifolium* | Upland Sea-Oats | Grass | Perennial | Native | 3 | USDA Zone 3-8 |
| | *Leersia oryzoides* | Rice Cut Grass | Grass | Perennial | Native | 2 | USDA Zone 3a-9b |
| | *Panicum virgatum* | Switchgrass | Grass | Perennial | Native | 2 | USDA Zone 5-9 |
| | *Sorghastrum nutans* | Yellow Indian Grass | Grass | Perennial | Native | 4 | USDA Zone 5-9 |
| | *Osmunda cinnamomea* | Cinnamon Fern | Fern | Perennial | Native | 3 | USDA Zone 2-10 |
| | *Osmunda regalis* | Royal Fern | Fern | Perennial | Native | 3 | USDA Zone 3-9 |
| | *Woodwardia virginica* | Virginia Chain Fern | Fern | Perennial | Native | 2 | USDA Zone 3-10 |
| | *Carex spp.* | Carex Sedges | Sedge | | Use only Native | 2 | Varies |
| | *Cyperus odoratus* | Flat Sedge | Sedge | | Native | 2 | USDA Zone 7-11 |
| | *Juncus effusus* | Soft Rush | Sedge | | Native | 2 | USDA Zone 4-9 |
| | *Scirpus californicus* | Giant Bulrush | Sedge | | Native | 2 | USDA Zone 6-9 |
| | *Scirpus cyperinus* | Woolgrass | Sedge | | Native | | USDA Zone 4-8 |
| | *Scirpus validus* | Softstem Bulrush | Sedge | | Native | 2 | USDA Zone 3-9 |
| | *Canna flaccida* | Golden Canna | Perennial | Perennial | Native | 2 | USDA Zone 8-11 |
| | *Coreopsis leavenworthii* | Tickseed | Perennial | Perennial | Native | 2 | USDA Zone 8-11 |
| | *Coreopsis tinctoria* | Dwarf Tickseed | Perennial | Perennial | Native | 3 | USDA Zone 3-11 |
| | *Crinum americanum* | Swamp Lily | Perennial | Perennial | Native | 2 | USDA Zone 7-11 |
| | *Eleocharis cellulosa* | Coastal Spikerush | Perennial | Perennial | Native | 2 | USDA Zone 8-11 |
| | *Eleocharis interstincta* | Jonited Spikerush | Perennial | Perennial | Native | 2 | USDA Zone 8-10 |
| | *Eupatorium fistolosum* | Joe Pye Weed | Perennial | Perennial | Native | 4 | USDA Zone 4-8 |
| | *Eupatorium perpurea* | Joe Pye Weed | Perennial | Perennial | Native | | USDA Zone 4-9 |
| | *Helianthus angustifolius* | Swamp Sunflower | Perennial | Perennial | Native | 2 | USDA Zone 6-9 |
| | *Hibiscus coccinieus* | Swamp Hibiscus | Perennial | Perennial | Native | 2 | USDA Zone 6-9 |
| | *Iris louisiana* | Louisiana Iris | Perennial | Perennial | Native | 2 | USDA Zone 5-9 |
| | *Iris virginica* | Southern Blue-Flag | Perennial | Perennial | Native | 2 | USDA Zone 5-9 |
| | *Liatris spicata* | Spiked Gayfeather | Perennial | Perennial | Native | 3 | USDA Zone 3-8 |
| | *Lobelia cardinalis* | Cardinal Flower | Perennial | Perennial | Native | 3 | USDA Zone 3-9 |
| | *Peltandra virginica* | Green Arum | Perennial | Perennial | Native | 2 | USDA Zone 5-9 |
| | *Polygonum hydropiperoides* | Smartweed | Perennial | Perennial | Native | 2 | USDA Zone 3-10 |
| | *Pontederia cordata* | Pickerelweed | Perennial | Perennial | Native | 2 | USDA Zone 3-10 |

**Hydrologic Zone for Stormwater Pond or Wetland*

## Georgia Native Plant List (continued)

GRASSES/HERBACEOUS

| Scientific Name | Common Name | Habit | | | H ZONE* | Hardiness |
|---|---|---|---|---|---|---|
| *Pontederia lanceolata* | Pickerelweed | Perennial | Perennial | Native | 2 | USDA Zone 3-10 |
| *Rudbeckia hirta* | Black-eyed Susan | Perennial | Perennial | Native | 4 | USDA Zone 3-9 |
| *Rudbeckia laciniata* | Greenhead Coneflower | Perennial | Perennial | Native | 4 | USDA Zone 3-9 |
| *Sagittaria lancifolia* | Lance-leaf Arrowhead | Perennial | Perennial | Native | 2 | USDA Zone 5-10 |
| *Sagittaria latifolia* | Duck Potato | Perennial | Perennial | Native | 2 | USDA Zone 5-10 |
| *Saururus cernuus* | Lizard's Tail | Perennial | Perennial | Native | 2 | USDA Zone 3-9 |
| *Scirpus americanus* | Three-square | Perennial | Perennial | Native | 2 | USDA Zone 3-9 |
| *Thalia geniculata* | Alligator Flag | Perennial | Perennial | Native | 2 | USDA Zone 7-9 |
| *Typha latifolia* | Broadleaf Cattail | Perennial | Perennial | Native | 2 | USDA Zone 3-10 |
| *Vernonia gigantea* | Ironweed | Perennial | Perennial | Native | 4 | USDA Zone 5-8 |
| *Nuphar luteum* | Water Lily | Water Lily | Perennial | Native | 1 | USDA Zone 4-10 |
| *Nymphaea mexicana* | Yellow Water Lily | Water Lily | Perennial | Native | 1 | USDA Zone 3-11 |
| *Nymphaea odorata* | Fragrant Water Lily | Water Lily | Perennial | Native | 1 | USDA Zone 3-11 |

**Hydrologic Zone for Stormwater Pond or Wetland*

## Section 6: Minimum Requirements for Landscape Plans

Landscape plans must be prepared by a qualified design professional. They must include the following items, at a minimum:

1. Landscape plan sheet – A scaled construction drawing (typically at 1" = 20') to accurately locate and represent the plant material used within the BMP facility. Representation of plant material should be to scale and depicted at the mature width or spread.
2. A key that identifies all plant material used in the planting plan. The symbols used to identify the plants will correlate with the plant schedule. Plant groupings on the drawing are usually shown by an identifying symbol and the number of plants in that particular group.
3. List any other necessary information to communicate special construction requirements, materials, or methods such as specific plants that must be field located or approved by the designer and size or form matching of an important plant grouping.
4. Plant list/table – This must include scientific name, common name, quantity, nursery container size, container type (e.g., bare root, b&b, plug, container, etc.), appropriate planting season, and other information in accordance with the BMP facility-specific planting section and landscape industry standards.
5. Soil media specifications. If topsoil is specified, indicate the topsoil stockpile location, including source of the topsoil if imported to the site.
6. Construction notes with sequencing, soil and plant installation instructions, and initial maintenance requirements.
7. A description of the landscape contractor's responsibilities.
8. A minimum two-year warranty period stipulating requirements for plant survival/replacement.

BACK TO TOC

# Section 7: Plant Establishment and Maintenance

### Establishment

Slope stabilization methods (such as planted erosion control mats or fiber rolls) should be utilized for slopes susceptible to washout. Erosion control mats and fabrics should also be utilized to protect channels that are susceptible to washing out. Flows should be diverted temporarily from seeded areas until they are stabilized. Aquatic and safety benches should be stabilized with emergent wetland plants and wet seed mixes

### Irrigation

Planting design should minimize the need for a permanent irrigation system, however, irrigation is an important aspect of any landscape establishment. New plantings need two to three years of irrigation to become established but this varies by location and seasonal conditions. Temporary irrigation systems, hand watering or alternative methods of irrigation for landscape establishment should be specified. After that period, native plants will need little to no supplemental irrigation. Where permanent irrigation systems are utilized, they should include a weather-based controller to avoid watering during wet weather. Because bioretention soils are formulated to infiltrate, irrigation application rates must be properly designed to avoid overwatering and prevent potential discharges via underdrains.

### Staking

Provide extra support to trees, especially in high wind areas. They should be securely staked during establishment and inspected once or twice a year and following storm events. Stakes should be removed as soon as they are no longer needed to stabilize the tree (between one and two years).

### Weeding

Weeds compete with plants for nutrients, water and sunlight. They should be regularly removed, with their roots, by hand pulling or with manual pincer-type weeding tools. Care should be given to avoid unnecessary compaction of soils while weeding.

Leaf litter and trimmings present during maintenance should be removed from BMP rather than left to decompose because nitrogen levels can be affected and can change the function of the BMP.

### Mulching

Compost Mulch (1" - 2") should be applied to specified areas to retain moisture, prevent erosion and suppress weed growth. Reapply annually as the mulch breaks down. Use a compost mulch and avoid bark mulches that can float during storm events.

### Fertilization

The design for plantings shall minimize the need for herbicides, fertilizers, pesticides, or soil amendments at any time before, during, and after construction and on a long-term basis. Instead, a compost top dressing or application of compost tea can be used to introduce nutrients and beneficial microorganisms to the soil.
Apply compost mulch once per year in spring or fall or spray apply compost tea once per year between March and June.

### Plant Replacement

At the end of the first year and again at the end of the two-year warranty period, all plants that do not survive must be replaced to avoid spreading disease, establishment of weeds in bare areas and reduced LID function. Before replacing with the same species, determine if another species may be better suited to the conditions.



BACK TO TOC

# References

Art, Henry W., 1986. *A Garden of Wildflowers, 101 Native Species and How to Grow Them*, Storey Communications, Inc., Pownal, VT.

Clausen, Ruth Rogers and Ekstrom, Nicolas, H., 1989. *Perennials for American Gardens*, Random House, New York, NY.

Dirr, Michael A., 1990. *Manual of Woody Landscape Plants, Their Identification, Ornamental Characteristics, Culture, Propagation, and Uses, 4th Edition*, Stipes Publishing Company, Champaign, IL.

Engineering Technology Associates Inc. and Biohabitats, Inc. (ETA&B), 1993. *Design Manual for Use of Bioretention in Stormwater Management*, Prince Georges County Dept. of Environmental Resources, Upper Marlboro, MD.

Garber, M.P. and Moorhead, D.J., 1999. *Selection, Production and Establishment of Wetland Trees and Shrubs*. University of Georgia, College of Agricultural & Environmental Sciences & Daniel B. Warnell School of Forest Resources Cooperative Extension Service.

Georgia Wildlife Web: http://fishesofgeorgia.uga.edu/gawildlife/index.php

Greenlee, John, (photographed by Derek Fell) 1992. *The Encyclopedia of Ornamental Grasses, How to Grow and Use Over 250 Beautiful and Versatile Plants*, Rodale Press, Emmas, PA.

Hodler, T.W. and H.A. Schretter. 1986. *The Atlas of Georgia*. University of Georgia Press, Athens.

Law, N.L., K. Cappiella and M.E. Novotney. 2008. “The Need to Address Both Impervious and Pervious Surfaces in Urban Watershed and Stormwater Management.” Journal of Hydrologic Engineering. 14(4): 305-308.

Miles, Bebe, 1996. *Wildflower Perennials for Your Garden, A Detailed Guide to Years of Bloom from America’s Native Heritage*, Stackpole Books, Mechanicsburg, PA.

Newcomb, Lawrence, 1977. *Newcomb’s Wildflower Guide*, Little Brown and Company, Boston, MA.

Schueler, Thomas R., July 1987. *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMP’s*, Department of Environmental Programs Metropolitan Washington Council of Governments, Metropolitan Information Center, Washington, DC.

Schueler, Thomas R., October 1996. *Design of Stormwater Wetland Systems: Guidelines for Creating Diverse and Effective Stormwater Wetland Systems in the Mid-Atlantic Region*, Department of Environmental Programs Metropolitan Washington Council of Governments, Metropolitan Information Center, Washington, D.C.

Schueler, Thomas R. and Claytor, Richard A., 1997. *Design of Stormwater Filtering Systems: Appendix B and C*, Chesapeake Bay Consortium, Silver Spring, MD.

The Pennsylvania State University, College of Agriculture, Cooperative Extension Service, File No. IVC9 10M386, U. Ed. 85-439 and File No. IVC9 10M587 U.Ed. 86-356, *Weed Identification*, The Pennsylvania State University, College of Agriculture, Cooperative Extension Service, University Park, PA.

Tiner, Ralph W. Jr., April 1988. *Field Guide to Non-Tidal Wetland Identification*, U.S. Fish and Wildlife Service, Maryland Department of Natural Resources Maryland Geological

U.S. Army Corps of Engineers, Wetlands Research Program (WRP), 1993. *Baseline Site Assessments for Wetland Vegetation Establishment*. WRP Technical Note VN-EV-2.1, August 1993.

https://www.casqa.org/sites/default/files/downloads/central_coast_bioretention_plant_guidance_print.pdf

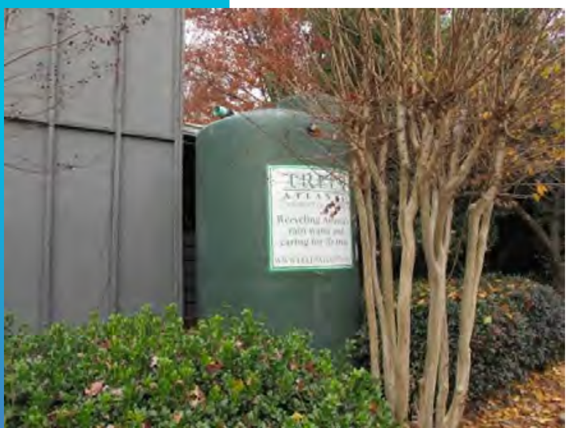
http://www.sandiegocounty.gov/content/dam/sdc/dpw/WATERSHED_PROTECTION_PROGRAM/susmppdf/lid_appendix_g_bioretention_soil_specification.pdf

Special thanks to the N.C. Department of Environment and Natural Resources’ Stormwater Permitting Program for assistance in developing content for this appendix.

BACK TO TOC

Appendix E: Best Management Practice Operations & Maintenance

# Operations & Maintenance Guidance Document



**Georgia Stormwater Management Manual**
**Appendix E**

Sponsored by:
Atlanta Regional Commission
Georgia Environmental Protection Division
Produced by:
AECOM
September 2015

*This page intentionally left blank.*

## Table of Contents

## Introduction

The purpose of this Operation and Maintenance (O&M) Guidance Document is to define a Stormwater Best Management Practice (also called a BMP or a practice), explain the importance of BMPs, show the components of a typical BMP, and offer direction and information to keep the practice operational.

BMPs are structural practices designed to store or treat stormwater runoff to prevent or reduce pollution from entering surface waters in the State of Georgia. They improve water quality by treating, detaining, and retaining stormwater runoff. Detaining water is accomplished by creating a basin that holds the water for a short period of time to allow some of the water to exfiltrate into the ground and the remainder of the water to release slowly over a period of time. Retaining stormwater is similar to detaining stormwater; however, the difference is the length of time the water is held. Retaining the water extends the period of time the water is held.

In order for a BMP to work properly, it must be maintained. BMPs generally require annual inspections, but more frequent routine inspections, such as after major storm events, may be required based on the site conditions, past maintenance issues, or risk associated with safety due to non-performance of a structure. The key to the long-term success of a BMP is routine inspection and maintenance.

## Why are BMPs important?

When an area is being developed, the property or portions of the property often change from grassed or wooded areas to paved areas. Grassed or wooded areas are pervious, which means that rainwater can infiltrate into the ground. Paved areas, on the other hand, are impervious, which means that rainwater cannot infiltrate into the ground. Because impervious areas cannot infiltrate water, increasing the amount of impervious area during development results in higher volumes of stormwater runoff. This can cause flooding and stormwater pollution, if not controlled through BMPs or other stormwater control measures. BMPs designed to retain or infiltrate stormwater help recharge the ground water and create a pervious area for the stormwater to infiltrate in the ground that has otherwise been altered by development. BMPs can also help reduce erosion and habitat loss in streams caused by excessive runoff and can reduce flooding and potentially make areas that are prone to flooding safer.

In addition, BMPs improve the quality of stormwater runoff from developed areas by removing pollutants that can contaminate the surrounding streams, rivers, lakes, etc. which, in turn, may contaminate our drinking water and food. Building BMPs in new developments, small business parks, or an individual residential lot or residential subdivisions, provides opportunities to remove the pollutants generated by the development. Example stormwater pollutants include sediment, excess nutrients, trash, fecal coliform, and metals.

An operational BMP will include some variation of the following components as shown in Figure 1.

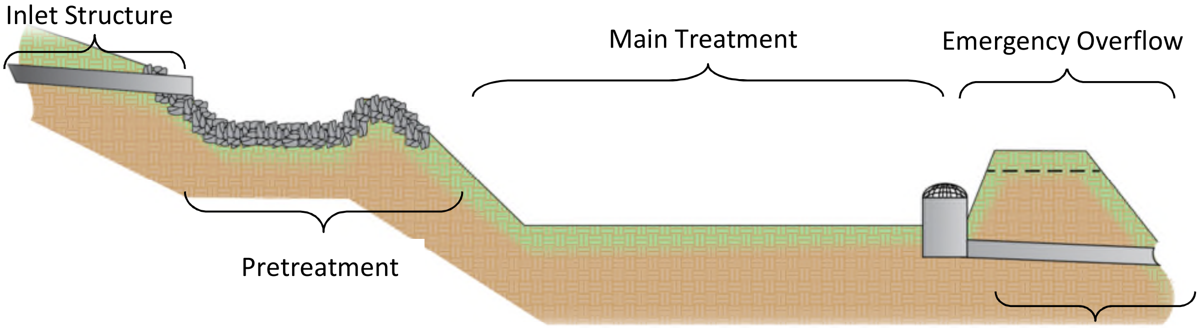



**Figure 1 – Components of a BMP**

The purpose and function of the main components of a BMP are described below:

- **Inlet structure** – This component brings water into the practice. The picture to the right shows an example of an inlet structure. Another example of an inlet structure would be a catch basin.
- **Pretreatment** – Pretreatment is designed to act as the first layer of protection for the main treatment area. Protection is provided by removing debris and coarse sediment, which reduces the frequency of clogging in the main treatment area. The pretreatment area is designed to be somewhat sacrificial so that it can be cleaned (or even replaced) before the main treatment area of the practice. This provides two maintenance benefits: ease of maintenance and less cost to maintain. Because of this, maintenance on this section is critical. The picture to the left shows a forebay, a type of pretreatment device. Other types of pretreatment devices include filter strips or grassy areas, grass channels, or rock lined plunge pools.

- **Main treatment** – The main treatment area is where the majority of the stormwater treatment takes place by removing sediment, nutrients, pollutants, etc. It is also the area where stormwater is contained, either through detention or retention, so that the water can be discharged at a controlled rate. Therefore, it is important that this section is routinely inspected and maintained to ensure the practice is functioning properly. The picture to the right shows an example of the main treatment area of a dry enhanced swale. Main treatment areas treat stormwater runoff through different methods including vegetated conveyance, infiltration, filtration, and settling. For example, the main treatment area of a pond treats stormwater runoff primarily through settling, and the main treatment area of a sand filter treats runoff through filtration. Specific maintenance concerns within a treatment area are based on the method of treatment in a BMP. Examples of specific maintenance concerns for each treatment method include the following:

- Vegetated Conveyance – Erosion
- Infiltration – Media clogging and clogging the underdrain
- Filtration – Media clogging and clogging the underdrain
- Settling – Excessive sediment, embankment failure, and debris or other issues at the outlet structure.

- **Emergency overflow** – An emergency overflow is necessary for rain events that are larger than the practice was designed to treat. This component will keep the area surrounding the practice from flooding by allowing water to continue to flow into a nearby drainage system or water body. Usually an emergency overflow is an elevated grass or paved channel that provides a way for stormwater to leave the BMP in an extreme rain event. It should be noted that other types of emergency overflow exist, and sometimes the emergency overflow and outflow structure can be combined.

- **Outlet structure** –The outlet structure allows treated water to exit the practice. It is important that this component has regular maintenance because if the outlet structure is clogged, flooding will occur within the practice. The picture on the right shows an example of an outlet structure. Other examples of outlet structures include open pipes and underdrains.

## Importance of Inspection

Once the BMP is built, routine inspection is very important to keep the practice working properly and catch potential problems before they become major problems (such as financial problems, legal problems, or both). Another benefit of routine inspection is it allows you to see the area surrounding the site and observe possible pollutants. For example, inspecting a BMP provides the opportunity to discover an unstable or eroding area upstream of the BMP that may be providing excessive sediment to the BMP, which could clog the practice quickly.

Items to check during routine inspections include, but are not limited to, the following:

- Structural problems
- Excessive ponding
- Unhealthy or undesirable vegetation
- Erosion
- Stability of the surrounding ground
- Clogging in the inlet or outlet structures or practice (from sediment, debris, or animals)
- Deterioration of pipes (or observation wells)

- New pollutant sources
- Infiltration rate by completing soil testing
- Monitoring water levels in observation wells

Some BMPs require an underground system, making inspection difficult to conduct. Generally these underground systems can be inspected by looking in the observation well. Sometimes, however, maintenance requires an individual who is certified in Occupational Safety and Health Administration (OSHA) confined space entry. Should there be a situation where a safety concern arises, the inspection should stop and the safety concern addressed. Once the concern is addressed, the inspection can continue. Signs indicating a potential maintenance problem with the underground system include the following:

- Ponding water or water remaining in the observation well longer than the design time
- Excessive sediment built up
- Damage to the structure through compaction or settling

## Maintenance Agreements

Oftentimes BMPs are covered by a maintenance agreement between the owner and local city or county or other jurisdiction.  Be sure to follow these maintenance requirements for the practice.  This guidance document provides helpful maintenance tips; however, the maintenance performed on the practice has to meet the written standards and specifications in the maintenance agreement.

## General Maintenance

Proper maintenance of each BMP is important to make sure the components of the BMP are operating and functioning the way the practice was designed to work. In other words, if the structure is not properly working, this could lead to the release of sediment, debris, and potential pollutants to a receiving water.  Generally, maintenance for each practice includes:

- Removing built up sediment, debris, or trash within the practice
- Removing debris from the inflow and outflow structure of the practice
- Implementing erosion and sediment control practices on portions of the BMP where vegetation is missing or in poor condition, replace vegetation
- Inspecting the BMPs regularly to ensure the structural integrity and functionality of the BMP
- Replacing the filter media (as needed)

Before and after photos are recommended as proof that maintenance has been performed.

## Vegetation Maintenance

Many BMPs include vegetation within or around the practice. Vegetation is an important part of the practice and aids infiltration and filtration. In addition, vegetation keeps the soil from eroding and washing into nearby drainage systems and water bodies. Finally, planting vegetation gives the area an additional aesthetic value. General vegetation maintenance includes:

- Irrigating and weeding during the first few months to establish the vegetation
- Maintaining the vegetation to ensure the health and abundance of native species and plantings
- Mowing, trimming, or pruning annually to prevent unwanted plants from growing in the practice
- Removing grass clippings or dead leaves from the practice to prevent clogging
- Minimize use of fertilizers and herbicides

## When to Call a Professional

It is unlikely that a lawn care (or similar) company will know how to properly inspect or maintain a BMP. Therefore, a qualified licensed professional is recommended to perform inspections and maintain the practice. Sources for potential assistance include the following:

- Local stormwater authority
- Professional Engineer
- Landscape architect
- Extension service office

If it is decided that a licensed professional is not required to perform routine inspection and maintenance, there are times where one will be necessary for major problems. Examples of when to call a licensed professional include, but are not limited to, the following:

- Significant damage to the structure
- Significant sediment build-up
- Excessive ponding of water
- Abnormal odor
- Signs of water seepage on the downstream side of a dam
- Excessive erosion
- Signs of pollutants other than sediment, such as chemical spills

## Additional Resources

Although most BMPs are similar in that they improve water quality or control quantity, they differ through site conditions, purpose, and function. Even two of the same type of BMP may require different maintenance due to site conditions. BMPS are designed to remove pollutants from stormwater which means they must be cleaned out periodically or they will cease to function. Since each BMP is unique there is no "one size fits all" approach to maintaining them. Some BMPs may also have more complicated maintenance requirements that require special skills, tools, or equipment. When in doubt about the proper way to test and maintain a BMP the owner should seek additional assistance.

The following is a list of suggested resources for inspection and maintenance of a BMP:

- Georgia Stormwater Management Manual, Volumes 1 and 2
- Georgia Department of Transportation Stormwater System Inspection and Maintenance Manual
- Louisville Kentucky Metropolitan Sewer District Design Manual (Chapter 18.7)
- City of Philadelphia Green Stormwater Infrastructure Maintenance Manual
- North Carolina Division of Energy, Mineral and Land Resources Stormwater BMP Manual and BMP Forms
- West Virginia Stormwater Manual
- Post Construction Manual Managing Stormwater in Your Community
- Local stormwater authority
- Local Cooperative Extension Service office
- Manufacturer's Guidelines (if applicable)

Within individual sections of this document, there may be references to certain tests such as soil infiltration and pH testing. It is recommended that testing for infiltration or pH be performed periodically on certain BMPs. In those cases it is recommended that a professional is consulted to ensure that the proper test is being performed in accordance with accepted procedures. The following is a list of suggestions for finding a professional:

- Local stormwater authority
- Local Cooperative Extension Service office
- Local Professional Engineer
- US Department of Agriculture Natural Resources Conservation Service

## Bioretention Areas

A bioretention area is a shallow stormwater basin or landscaped area with well-draining soils, generally composed of sand, fines, and organic matter, and vegetation to capture and treat stormwater runoff. The basin or main treatment area of the bioretention area includes plants to aid in the filtration and infiltration of the stormwater flowing through the practice. An underdrain may be placed in the bioretention area to collect runoff that has filtered through the soil layers and pipe it to the storm sewer system or a nearby water body.

There are some common problems to be aware of when maintaining a bioretention area. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the inlet and outlet structure
- Establishing vegetation within the bioretention area
- Clogging the underdrain (if applicable)
- Mosquitoes breeding in the practice
- Ant mounds
- Maintaining the proper pH levels for plants
- Pruning and weeding to maintain appearance

Routine maintenance should be performed on the bioretention areas to ensure that the structure is functioning properly. Note that during the first year the bioretention area is built, maintenance may be required at a higher frequency to ensure the proper establishment of vegetation in the practice.

In addition to routine maintenance, bioretention areas have seasonal and intermittent maintenance requirements. For example, the following are maintenance activities and concerns specific to winter months. Planting material should be trimmed during the winter, when the plants are dormant. In the event of snow, ensure that snow does not pile up in the bioretention area. Accumulated snow adds additional weight and may compact the bioretention area soil, which would reduce its infiltration capacity. In addition, check to make sure that the materials used to de-ice the surrounding areas stay out of the practice to avoid clogging and further pollution.

Bioretention areas should be inspected after a large rainstorm. Keep drainage paths, both to and from the BMP, clean so that the water can properly infiltrate into the ground. Note that it might take longer for the water to infiltrate into the ground during the winter months and early spring. Mulch the practice as needed to keep a thickness of 3-4 inches. Shredded hardwood mulch is preferred, and care should be taken to keep the mulch from piling on the stems of the plants. For more information on vegetation in bioretention areas, see Appendix D: Planting and Soil Guidance.

If the bioretention area is not draining properly, check for clogging of the inflow and outflow structures as well as the infiltration rate of the soil media. If the soil is not draining properly, it could be clogged or over-compacted. In a bioretention area, the media is likely to become clogged at the mulch or upper layer of the soil first. If the media is clogged or over-compacted, then the media should be replaced. Potential sources of excessive sediment that could clog the media include ant mounds and unstable soil upstream of the practice. Possible sources of compaction are vehicles, such as tractors, traveling through the practice. If the practice includes an underdrain, a structural repair or cleanout to unclog the underdrain may be necessary.

In order to keep the water that exits the bioretention area clean, fertilizers should only be used sparingly during the establishment of the practice. Once the vegetation in the practice has been established, fertilizers should not be used. While vegetation in the bioretention area is important, the primary purpose of a bioretention area is to act as a water quality device and introducing fertilizers into the bioretention area introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. In addition, bioretention areas should already be a nutrient rich environment that does not require fertilization. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

If designed correctly, there is no danger of bioretention areas becoming a breeding ground for mosquitoes. A mosquito egg requires 24-48 hours to hatch. In addition, it takes 10-14 more days for the larvae to develop and become an adult. By having a bioretention area that drains properly, it is unlikely that a bioretention area would provide a habitat that could become a breeding area for mosquitoes. Should the bioretention area become a breeding ground for mosquitoes, the problem is likely with the soil media or the overflow structure which may need to be addressed.

The table below shows a schedule for when different maintenance activities should be performed on the bioretention area.

**Bioretention Area Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Prune and weed to maintain appearance.<br>• Dissipate flow when erosion is evident.<br>• Remove trash and debris.<br>• Remove sediment and debris from inlets and outlets.<br>• Remove and replace dead or damaged plants.<br>• Mow around the bioretention area as necessary, ensuring grass clippings are not placed in the practice.<br>• Observe infiltration rates after rain events. Bioretention areas should have no standing water within 24 hours of a storm event.<br>• Inspect for evidence of animal activity. | As needed or 4 times during growing season |

| Activity | Schedule |
|---|---|
| • Inspect for erosion, rills, or gullies and repair.<br>• Inspect filter strip/grass channel for erosion or gullying, if applicable. Re-seed or sod as necessary.<br>• Inspect trees and shrubs to evaluate their health, and remove and replace any dead or severely diseased vegetation.<br>• Obtain a mulch depth of at least 3 to 4 inches should be inspected and obtained. Additional mulch should be added as necessary. | Semi-annually in spring and fall |
| • Trim planting material.<br>• Inspect for snow accumulation. | As needed or during winter months |
| • Test the planting soils for pH levels. Consult with a qualified licensed Professional to determine and maintain the proper pH levels. | Annually |
| • Replace/repair inlets, outlets, scour protection or other structures as needed.<br>• Implement plant maintenance plan to trim and divide perennials to prevent overcrowding and stress.<br>• Check soil infiltration rates to ensure the bioretention area soil is draining the water at a proper rate. Re-aerate or replace soil and mulch layers as needed to achieve infiltration rate of at least 0.5 inches per hour. | 2 to 3 years |

*This page intentionally left blank.*

| Bioretention Area | | | | | |
|---|---|---|---|---|---|
| **Maintenance Item** | **Condition** | | | | **Comment** |
| | **Good** | **Marginal** | **Poor** | **N/A*** | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet Structure** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet structure is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet structure. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| Diversion structure (high flow bypass structure or other) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Pretreatment (choose one)** | | | | | |
| Forebay – area is free of trash, debris, and sediment. | | | | | |
| Weir – area is free of trash, debris, and sediment is less than 25% of the total depth of the weir. | | | | | |
| Filter Strip or Grass Channels – area is free of trash debris and sediment. Area has been mowed and grass clippings are removed. No evidence of erosion. | | | | | |
| Rock Lined Plunge Pools – area is free of trash debris and sediment. Rock thickness in pool is adequate. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |

## Bioretention Area

| Maintenance Item | Condition | | | | Comment |
|---|---|---|---|---|---|
| | Good | Marginal | Poor | N/A* | |
| No evidence of long-term ponding or standing water in the ponding area of the practice (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| Structure seems to be working properly. No settling around the structure. Comment on overall condition of structure. | | | | | |
| Vegetation within and around practice is maintained per landscaping plan. Grass clippings are removed. | | | | | |
| Mulching depth of 3-4 inches is maintained. Comment on mulch depth. | | | | | |
| Native plants were used in the practice according to the planting plan. | | | | | |
| No evidence of use of fertilizer on plants (fertilizer crusting on the surface of the soil, tips of leaves turning brown or yellow, blackened roots, etc.). | | | | | |
| Plants seem to be healthy and in good condition. Comment on condition of plants. | | | | | |
| **Emergency Overflow** | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| **Outlet Structure** | | | | | |
| Outlet structure is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| **Results** | | | | | |
| Overall condition of Bioretention Area: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

## Bioslopes

Bioslopes are linear BMPs with a permeable media that allows stormwater runoff to infiltrate and filter through the practice before exiting through an underdrain. High flows bypass the bioslope in the form of sheet flow running over the bioslope. Generally, a filter strip is placed before the bioslope for pretreatment where it captures sediment and debris and prevents premature clogging of the bioslope. If the space available for the bioslope is limited, a grass shoulder or pea gravel diaphragm may be used as an alternate method of pretreatment.

There are some common problems to be aware of when maintaining a bioslope. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the inlet and outlet structure as well as the underdrain
- Undesirable vegetation
- Erosion
- Mowing the grass filter strip
- Compaction

Typically bioslopes are indistinguishable from the surrounding areas, so it is recommended that GPS co-ordinates of the bioslope location be obtained and the BMP be staked with markers. If markers are used, they should be placed at both ends at the toe of the slope and every 50 feet.

Routine inspection and maintenance should be performed on the bioslope to ensure that the practice is functioning properly. Generally maintenance will consist of removing debris and trash that could accumulate on the practice and cause clogging. Other routine maintenance includes mowing the bioslope and removing grass clippings. Mowing and landscaping crews should be alerted not to access the bioslope during wet conditions to avoid damaging or rutting the area.

Inspect the bioslope after a large rainstorm. Keep drainage paths (both to and from the BMP) clean to promote sheet flow and allow stormwater runoff to be routed in the intended direction.

In addition to routine maintenance, bioslopes have seasonal and intermittent maintenance requirements. For example, during the winter months, the bioslope should be inspected after a snow event (this is specific to northern areas of Georgia). Accumulated snow adds additional weight and may compact the media, which would reduce its infiltration capacity. In addition, check to make sure that the materials used to de-ice the surrounding areas stay out of the practice to avoid clogging and further pollution. Also, note that it might take longer for the water to infiltrate into the ground during the winter months and early spring.

If the bioslope is not draining properly, it may be necessary to repair or unclog the underdrain as well as the inflow and outflow structures. Another possible reason the bioslope is not draining properly could be due to clogged or over-compacted bioslope media. If the mix becomes clogged or over-compacted, then it should be replaced. The degree of required media removal and replacement can vary depending on the characteristics of the contributing drainage area and the consistency of regular maintenance. For example, it is likely that removal and replacement of the top two to five inches of media will be necessary every three to five years for low sediment applications. Media replacement may be needed more often for areas of high sediment yield or high oil and grease.

The table below shows a schedule for when different maintenance activities should be performed on the bioslopes.

**Bioslope Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Clear debris in inlets and outlets.<br>• Mow and stabilize the area surrounding the bioslope. Remove grass clippings.<br>• Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system.<br>• Remove trash and debris. | As needed or 4 times during growing season |
| • Stabilize eroded areas on the bioslope.<br>• Ensure that flow is not bypassing the facility.<br>• Ensure that no noticeable odors are detected outside the facility.<br>• Mow the bioslope grass using a retractable arm mower to avoid compaction. Grass height should be mowed to a height of 6 to 15 inches. Remove grass clippings. | Monthly |
| • Ensure that gravel spreader or other structural elements of the bioslope are in good condition and free of debris.<br>• Test the permeability of the bioslope media using a hydraulic conductivity test. Replace the media as needed.<br>• Flow test the cleanouts to look for signs indicating the underdrain system is clogged.<br>• Evaluate sediment accumulation and remove once it reaches or exceeds a depth of 3 inches. | Annually |

## Bioslope

| Maintenance Item | Condition | | | | Comment |
|---|---|---|---|---|---|
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet. | | | | | |
| Water is going through the bioslope (i.e. no evidence of water going around the BMP). | | | | | |
| Diversion structure (high flow bypass structure or other) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Pretreatment** | | | | | |
| Area is free of trash debris and sediment. | | | | | |
| Area has been mowed and grass clippings are removed. Grass seems healthy and there are no bare areas or dying grass. | | | | | |
| No evidence of erosion or gullies. | | | | | |
| Area is free of undesirable vegetation. | | | | | |
| No standing water in the practice. | | | | | |
| No sediment accumulation within in the pretreatment area. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion of gullies within the bioslope. | | | | | |
| No evidence of long-term ponding or standing water in the practice (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| Practice seems to be working properly. No settling around the structure. Comment on overall condition of practice. | | | | | |

| Bioslope | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| No undesirable vegetation within the bioslope. | | | | | |
| Area has been mowed at a height of 6-15 inches. Grass clippings are removed. | | | | | |
| Cleanout caps for underdrain are not damaged or missing. | | | | | |
| Flow testing has been performed on bioslope to determine if underdrain is clogged. | | | | | |
| Observation well has no signs of standing water. | | | | | |
| Emergency Overflow | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| Outlet Structure | | | | | |
| Outlet structure is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| Outlet pipe is not damaged or clogged and is in good condition. | | | | | |
| Results | | | | | |
| Overall condition of Bioslope: | | | | | |
| Additional Comments | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

## Downspout Disconnects

A downspout disconnect is a method to spread rooftop runoff from individual downspouts across lawns and other pervious areas, where it is slowed, filtered and allowed to infiltrate into the native soils. Downspout disconnects can be used in conjunction with other BMPs such as bioretention areas, enhanced swales, and vegetated filter strips. If the downspout disconnect is used in conjunction with another BMP, then that BMP will need to be inspected and maintained as well.

Common problems that can occur when maintaining a downspout disconnect include, but are not limited to, the following:

- Clogged gutters or downspouts
- Loose gutters or downspouts
- Water not draining away from buildings
- Poorly draining soils
- Poorly functioning splash blocks
- Cracks within the downspout extension

Routine maintenance should be performed on the downspout disconnects to ensure that the practice is properly functioning to ensure that a water problem is not created for neighbors. Ensure that there is no erosion occurring at the base of the downspout. Other specific maintenance items depend on where the downspout has been disconnected from the storm or sanitary sewer and where the flow has been redirected.

Inspect the downspout disconnect after a large rainstorm. Make sure that there is no evidence of a leak in the gutters or downspouts, and ensure that rooftop runoff is directed through the system. During the winter months, downspouts should be inspected for cracks caused by water freezing in the downspout.

The table below shows a schedule for when different maintenance activities should be performed routine maintenance activities typically associated with downspout disconnects.

**Downspout Disconnect Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Pervious areas located below simple downspout disconnections should be watered to promote plant growth and survival.<br>• Inspect the pervious areas located below simple downspout disconnections following rainfall events. Plant replacement vegetation in any eroded areas. | As Needed<br>(Following Construction) |

| Activity | Schedule |
|---|---|
| • Inspect pervious area located below simple downspout disconnection. Maintain vegetation (e.g., mow, prune, trim) as needed.<br>• Remove accumulated trash and debris in pervious area located below the simple downspout disconnection. | Regularly (Monthly) |
| • Inspect gutters and downspouts. Remove any accumulated leaves or debris.<br>• Inspect the pervious areas located below simple downspout disconnections for erosion and the formation of rills and gullies. Plant replacement vegetation in any eroded areas.<br>• Inspect the pervious areas located below simple downspout disconnections for dead or dying vegetation. Plant replacement vegetation as needed. | Annually (Semi-Annually During First Year) |

| Downspout Disconnects | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Gutters are clean. No sediment, debris, or trash to clog the system. | | | | | |
| Downspouts are properly fastened to convey water from the roof. | | | | | |
| Downspouts are free of trash, debris, and sediment and conveying water properly. | | | | | |
| No evidence of leaks at joints or other components of downspouts. | | | | | |
| Erosion control mats are present on site to prevent erosion on pervious area below downspout disconnects. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| Vegetation is in place and is healthy. No bare or dying areas. | | | | | |
| Unwanted vegetation is trimmed and removed. | | | | | |
| No evidence of erosion, scour, or flooding. | | | | | |
| Results | | | | | |
| Overall condition of Downspout Disconnects: | | | | | |
| Additional Comments | | | | | |
| | | | | | |
| **Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

*This page intentionally left blank.*

## Dry Detention Basins

A dry detention basin is a storage basin designed to provide water quantity control through detention of stormwater runoff. The purpose of detention is to allow some of the water to exfiltrate into the ground and the remainder of the water to release slowly over a period of time to reduce downstream water quantity impacts. Dry detention basins are designed to completely drain following a storm event and are normally dry between rain events. They provide limited pollutant removal benefits and are not intended for water quality treatment alone.

There are some common problems to be aware of when maintaining a dry detention basin. They include, but are not limited to, the following:

- Sediment build-up
- Trash, litter, and debris accumulation
- Clogging and structural repairs in the inlet and outlet structures
- Establishing vegetation within the dry detention basin
- Erosion
- Mowers compacting and rutting the basin bottom
- Mosquitoes breeding in the practice
- Ant mounds

Routine maintenance should be performed on the dry detention basins to ensure that the structure is properly functioning. Note that during the first year the dry detention basin is built, maintenance may be required at a higher frequency to ensure the proper establishment of vegetation in the practice. In the event of snow, check to make sure that the materials used to de-ice the surrounding areas stay out of the practice to avoid clogging and further pollution.

Dry detention basins should be inspected after a large rainstorm. Keep drainage paths, both to and from the BMP, clean so that the water can properly infiltrate into the ground. Note that it might take longer for the water to infiltrate into the ground during the winter months and early spring. If the dry detention basin is not draining properly, check for clogging of the inflow and outflow structures.

If the forebay or dry detention basin has received a significant amount of sediment over a period of time, then the sediment at the bottom of the forebay or dry detention basin may need to be removed. Accumulated sediment in the practice decreases the available storage volume and affects the basin's ability to function as it was designed.

If designed and maintained correctly, dry detention basins should not become a breeding ground for mosquitoes. A mosquito egg requires 24-48 hours to hatch. In addition, it takes 10-14 more days for the egg to develop and become an adult. By having a dry detention basin that drains properly, it is unlikely that a dry detention basin would provide a habitat that could become a breeding area for mosquitoes. Should the dry detention basin become a breeding ground for mosquitoes, the problem is likely with the overflow structure which may need to be addressed.

The table below shows a schedule for when different maintenance activities should be performed on the dry detention basins.

**Dry Detention Basin Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Remove debris from basin surface to minimize outlet clogging and improve aesthetics.<br>• Note erosion of detention basin banks or bottom<br>• Inspect for damage to the embankment.<br>• Monitor for sediment accumulation in the facility and forebay.<br>• Examine to ensure that inlet and outlet devices are free of debris and operational. | Annually and following significant storm events |
| • Remove sediment buildup.<br>• Repair and revegetate undercut and/or eroded areas.<br>• Perform structural repairs to inlet and outlets.<br>• Repair undercut or eroded areas.<br>• Mow side slopes.<br>• Seed or sod to restore dead or damaged ground cover. | As needed based on inspection |
| • Mow to limit unwanted vegetation.<br>• Litter/ Debris Removal. | Routine |

| Dry Detention Basin | | | | | |
|---|---|---|---|---|---|
| **Maintenance Item** | **Condition** | | | | **Comment** |
| | **Good** | **Marginal** | **Poor** | **N/A*** | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet Structure** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet structure is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet structure. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| Inlet pipe is in good condition and is not clogged. | | | | | |
| Diversion structure (high flow bypass structure or other) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Pretreatment (forebay)** | | | | | |
| Area is free of trash, debris, and sediment. | | | | | |
| Sediment accumulation is less than 50% of the forebay volume. | | | | | |
| No undesirable vegetation within the forebay. Weeds are removed to prevent clogging. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| No evidence of long-term ponding or standing water in the ponding area of the practice (examples include: stains, odors, mosquito larvae, etc.). | | | | | |

| Dry Detention Basin | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| Basin seems to be working properly. No settling around the basin. Comment on overall condition of basin. | | | | | |
| Vegetation within and around practice is maintained. Grass clippings are removed. | | | | | |
| Sediment accumulation within dry detention basin is less than 3 inches. | | | | | |
| No standing water within the basin. | | | | | |
| No evidence of use of fertilizer on grass (fertilizer crusting on the surface of the soil, tips of leaves turning brown or yellow, blackened roots, etc.). | | | | | |
| **Emergency Overflow** | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| No shrubs or trees growing on embankment. | | | | | |
| No signs of seepage on the downstream face. | | | | | |
| No signs of animal activity. | | | | | |
| **Outlet Structure** | | | | | |
| Outlet structure is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| All moveable components are operational. | | | | | |
| **Results** | | | | | |
| Overall condition of Dry Detention Basin: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

## Dry Enhanced Swales/Wet Enhanced Swales

An enhanced swale is a vegetated open channel designed to capture and treat stormwater runoff within dry or wet cells formed by check dams or other means. Enhanced swales are generally shallow, wide, and vegetated to help slow and filter stormwater runoff.

There are two different types of enhanced swales. The first is a dry swale which includes a filter bed of prepared soil that overlays an underdrain system. They are designed to let stormwater be filtered or infiltrated through the bottom of the swale. Because they are dry most of the time, they are often the preferred option in residential settings. The second type of enhanced swale is a wet swale. Wet swales are designed to retain water or marshy conditions that support wetland vegetation. Because this practice is meant to retain water, they are generally used in areas with a high water table or poorly drained soils. Wet swales achieve pollutant removal both from sediment accumulation and biological removal.

There are some common problems to be aware of when maintaining an enhanced swale. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the inlet and outlet structure
- Establishing vegetation
- Clogging in the underdrain (if applicable)
- Mosquitoes breeding in the practice
- Ant mounds
- Maintaining the proper pH levels for plants
- Pruning and weeding to maintain appearance

Routine inspection and maintenance should be performed on the dry or wet enhanced swale to ensure that the practice is properly functioning. Note that during the first year the enhanced swale is built, maintenance may be required at a higher frequency to ensure the proper establishment of vegetation in the practice. For more information on vegetation within a swale, see Appendix D: Planting and Soil Guidance. Enhanced swales should be inspected after a large rainstorm. Keep drainage paths, both to and from the BMP, clean so that the water can properly flow in and out of the practice.

In addition to routine maintenance, dry or wet enhanced swales have seasonal and intermittent maintenance requirements. For example, during the winter months, the enhanced swale should be inspected after a snow event (this is specific to northern areas of Georgia). Accumulated snow adds additional weight and may compact the dry enhanced swale soil, which would reduce its infiltration capacity. In addition, check to make sure that the materials used to de-ice the surrounding areas stay out of the practice to avoid clogging and further pollution. Note that it might take longer for the water to infiltrate into the ground during the winter months and early spring.

If the dry enhanced swale is not draining properly, check for clogging in the inflow and outflow structures. Another consideration would be the permeable soil layer, which could be clogged or over-compacted. In a dry enhanced swale, the media is likely to become clogged at the upper layer of the soil first. Potential sources of excessive sediment that could clog the media include ant mounds and unstable soil upstream of the practice. Possible sources of compaction are vehicles, such as tractors, traveling through the practice. If the media is clogged or over-compacted, then the media should be replaced. If the practice includes an underdrain, a structural repair or cleanout to unclog the underdrain may be necessary.

In order to keep the water that exits the dry or wet enhanced swale clean, fertilizers should only be used sparingly during the establishment of the practice. Once the vegetation in the practice has been established, fertilizers should not be used. While vegetation in the enhanced swale is important, the primary purpose of an enhanced swale is to act as a water quality device, and introducing fertilizers into the enhanced swale introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. In addition, enhanced swales should already be nutrient rich environments that do not require fertilization. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

If designed and maintained correctly, there is no danger of dry enhanced swales becoming a breeding ground for mosquitoes. A mosquito egg requires 24-48 hours to hatch. In addition, it takes 10-14 more days for the egg to develop and become an adult. By having a dry enhanced swale that drains properly (within 24-48 hours), it is unlikely that a dry enhanced swale would provide a habitat that could become a breeding area for mosquitoes. Should the dry enhanced swale become a breeding ground for mosquitoes, the problem is likely with the soil media or the overflow structure which may need to be addressed.

The table below shows a schedule for when different maintenance activities should be performed on an enhanced swale.

**Enhanced Swale Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Prune and weed to maintain appearance.<br>• Dissipate flow when erosion is evident.<br>• Remove trash and debris.<br>• Remove sediment and debris from inlets and outlets.<br>• Remove sediment build-up within the bottom of the swale once it has accumulated to 25% of the original design volume.<br>• Remove and replace dead or damaged plants. | As needed or 4 times during growing season |

| Activity | Schedule |
|---|---|
| • Mow the dry enhanced swale as necessary to maintain a grass height of 4-6 inches, ensuring grass clippings are not placed in the practice.<br>• Observe infiltration rates after rain events. Dry enhanced swales should have no standing water within 48 hours of a storm event (though 24 hours is more desirable).<br>• Inspect for evidence of animal activity. | |
| • Inspect for erosion, rills, or gullies and repair.<br>• Replant wetland species (for wet swale) if not sufficiently established.<br>• Test the planting soils for pH levels. Consult with a qualified licensed Professional to determine and maintain the proper pH levels.<br>• Inspect pea gravel diaphragm for clogging. | Annually (Semi-annually the first year) |
| • Trim planting material.<br>• Inspect for snow accumulation. | As needed or during winter months |
| • Replace/repair inlets, outlets, scour protection or other structures as needed.<br>• Implement plant maintenance plan to trim and divide perennials to prevent overcrowding and stress.<br>• Check soil infiltration rates to ensure the dry enhanced swale soil is draining the water at a proper rate. Roto-till or cultivate the surface of the sand/soil bed of dry swales if the swale does not draw down within 48 hours. | 2 to 3 years |

*This page intentionally left blank.*

| Dry Enhanced Swale/Wet Enhanced Swale | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet Structure** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet structure is mowed and grass clippings are removed (for dry enhanced swale). | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet structure. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| **Pretreatment (choose one)** | | | | | |
| Forebay – area is free of trash, debris, and sediment. | | | | | |
| Weir – area is free of trash, debris, and sediment is less than 25% of the total depth of the weir. | | | | | |
| Filter Strip or Grass Channels – area is free of trash debris and sediment. Area has been mowed and grass clippings are removed. No evidence of erosion. | | | | | |
| Rock Lined Plunge Pools – area is free of trash debris and sediment. Rock thickness in pool is adequate. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| For dry enhanced swale, no evidence of long-term ponding or standing water in the ponding area of the practice (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| Plants were used in the practice according to the planting plan. | | | | | |

| Dry Enhanced Swale/Wet Enhanced Swale | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| Vegetation within and around practice is maintained per landscaping plan. Grass clippings are removed. | | | | | |
| Structure seems to be working properly. No settling around the structure. Comment on overall condition of structure. | | | | | |
| No evidence of undesirable vegetation. | | | | | |
| No evidence of use of fertilizer on plants (fertilizer crusting on the surface of the soil, tips of leaves turning brown or yellow, blackened roots, etc.). | | | | | |
| Plants seem to be healthy and in good condition. Comment on condition of plants. | | | | | |
| No evidence of erosion around the sides of the check dam. | | | | | |
| Cleanout caps are in place and in good condition (for dry enhanced swale). | | | | | |
| The underdrain appears to be unclogged evidenced by water exiting the practice freely (for dry enhanced swale). | | | | | |
| Pea gravel diaphragm or other flow spreader is clean and working properly. | | | | | |
| **Emergency Overflow** | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| **Outlet Structure** | | | | | |
| Outlet structure is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| **Results** | | | | | |
| Overall condition of Enhanced Swale: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please explain why in the appropriate comment box. | | | | | |

## Dry Extended Detention Basins

A dry extended detention basin provides temporary storage of stormwater runoff to control the peak rate of runoff by allowing the stored water to release slowly over a period of time. This practice is mostly used to control water quantity, although some water quality benefits can be obtained by the settling of floatables and sediment. This extended version of a dry detention basin is designed to maximize water quality benefits.

There are some common problems to be aware of when maintaining a dry extended detention basin. They include, but are not limited to, the following:

- Sediment build-up
- Trash, litter, and debris accumulation
- Clogging in the inlet and outlet structures
- Erosion
- Structural repairs to inlets and outlets
- Mowers compacting and rutting the basin bottom
- Clogging in the emergency spillway
- Mosquitoes breeding in the practice

Routine maintenance should be performed on dry extended detention basins to ensure that the structure is properly functioning. Note that during the first year the dry extended detention basin is built, maintenance may be required at a higher frequency to ensure the proper establishment of vegetation in the practice. In the event of snow, check to make sure that the materials used to de-ice the surrounding areas stay out of the practice to avoid clogging and further pollution.

Inspect the dry extended detention basin after a large rainstorm. Keep drainage paths (both to and from the BMP) clean so that the water can properly flow into the basin. If the dry extended detention basin is not draining properly, check for clogging of the inflow and outflow structures.

If the forebay or dry detention basin has received a significant amount of sediment over a period of time, then the sediment at the bottom of the forebay or dry detention basin may need to be removed. Accumulated sediment in the practice decreases the available storage volume and affects the basin's ability to function as it was designed.

The table on the next page shows a schedule for when different maintenance activities should be performed on the dry extended detention basin.

**Dry Extended Detention Basin Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
| --- | --- |
| • Remove trash, sediment, and debris from forebay and inlet and outlet structures.<br>• Mow the embankment and maintenance access. Periodically mow along maintenance rights-of-ways and the embankment. Remove grass clippings. | Monthly or as needed |
| • Repair and re-vegetate eroded areas.<br>• Remove and dispose of vegetation that may hinder the operation of the pond.<br>• Perform structural repairs to pond, outlet structures, embankments, control gates, valves, or other mechanical devices. | As needed |
| • Remove sediment when volume of pond is significantly reduced. | As needed (roughly every 20-50 years, but will vary based on the characteristics of the drainage area and amount of sediment entering the practice) |

| Dry Extended Detention Basin | | | | | |
|---|---|---|---|---|---|
| Inspection Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet/Outlet Structure** | | | | | |
| Drainage ways to and from the practice is free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet/outlet structure is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet/outlet structure. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| No signs of significant sediment accumulation. | | | | | |
| Concrete is in good condition. No signs of cracks. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Vegetation seems healthy. No signs of bare spots or dead vegetation. | | | | | |
| No signs of undesirable vegetation growth. | | | | | |
| No signs of excessive sedimentation. | | | | | |
| No signs of pollution draining into the practice (oil sheens, discolored or unnatural water, odor, etc.). | | | | | |
| **Embankment and Emergency Overflow** | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| No signs of animal activity in embankment. | | | | | |
| No signs of seepage on downstream side of embankment. | | | | | |

| Dry Extended Detention Basin | | | | | |
|---|---|---|---|---|---|
| Inspection Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| No signs structural deformation of embankment. | | | | | |
| No obstructions in spillway. | | | | | |
| Results | | | | | |
| Overall condition of Dry Extended Detention Basin: | | | | | |
| Additional Comments | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please explain why in the appropriate comment box. | | | | | |

## Dry Wells

Dry wells are located below the surface of development sites, and consist of shallow excavations, typically filled with stone, that are designed to intercept and temporarily store post-construction stormwater runoff until it infiltrates into the underlying and surrounding soils. If properly designed, they can provide significant reductions in post-construction stormwater runoff rates, volumes, and pollutant loads on development sites.

There are some common problems to be aware of when maintaining a dry well. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the gutters, pipes, and downspouts

Routine inspection and maintenance should be performed on the dry wells to ensure that the structure is functioning properly. Dry wells should be inspected after a large rainstorm. Keep gutters, pipes, and downspouts draining to the dry well clean and free of trash and debris. Every dry well should include an observation well to observe the draw down time of the dry well following a storm event. This is important to determine if clogging is occurring within the dry well.

If water is not draining to the dry well properly, check for clogging in the gutters, pipes, and downspouts. If the dry well is not draining properly the filter fabric may be clogged. The filter fabric lines the top and sides of the dry well. In addition, if the soil is not draining properly, the soil may be over-compacted. In a dry well, the media is likely to become clogged at the upper layer of the soil first. If the media is clogged or over-compacted, then the filter fabric and media should be replaced.

The table below shows a schedule for when different maintenance activities should be performed on the dry well.

**Dry Well Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
| --- | --- |
| • If applicable, water to promote plant growth and survival within landscaped area over the top of the dry well.<br>• If applicable, inspect vegetative cover on the surface of the dry well following rainfall events. Plant replacement vegetation in any eroded areas. | As Needed |
| • If applicable, inspect gutters and downspouts. Remove any accumulated leaves or debris.<br>• Inspect dry well following rainfall events. Check observation well to | Annually (Semi-Annually During First Year) |

| Activity | Schedule |
|---|---|
| ensure that complete drawdown has occurred within 24 hours after the end of a rainfall event. Failure to drawdown within this timeframe may indicate dry well failure.<br>• If applicable, inspect pretreatment devices for sediment accumulation. Remove accumulated trash and debris.<br>• Inspect top layer of filter fabric for sediment accumulation. Remove and replace if clogged. | |
| • Perform total rehabilitation of the dry well, removing dry well stone and excavating to expose clean soil on the sides and bottom of the well. | Upon Failure |

| Dry Well | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| General Inspection | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean around the practice (trash, debris, grass clippings, etc. removed). | | | | | |
| Gutters, pipes, and downspouts to the dry well are free of trash, debris, leaves, etc. | | | | | |
| No evidence of structural deficiencies or settling around the structure. | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Sediment has not accumulated and clogged filter fabric. | | | | | |
| Preatreatment is in place if dry well does not receive roof top runoff. Pretreatment is in good condition. | | | | | |
| No evidence of long-term ponding or standing water in the ponding area of the practice (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| The observation well is capped and locked when not in use. | | | | | |
| Structure seems to be working properly. No settling around the structure. Comment on overall condition of structure. | | | | | |
| Results | | | | | |
| Overall condition of Dry Well: | | | | | |
| Additional Comments | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please explain why in the appropriate comment box. | | | | | |

*This page intentionally left blank.*

## Grass Channel

Grass channels are vegetated open channels designed to enhance water quality by settling suspended solids through filtration, infiltration, and biofiltration. This practice offers a method to manage pollution while also conveying stormwater runoff. Grass channels are well suited to a number of applications and land uses, including treating runoff from roads and highways and pervious surfaces. Grass channels are broad and shallow channels that are generally positioned parallel to roadways or other impervious areas. They can also be used as a single BMP, a pretreatment to another BMP, or as a link between other BMPs.

There are some common problems to be aware of when maintaining a grass channel. They include, but are not limited to, the following:

- Trash, litter, and debris accumulation
- Watering the practice during dry periods
- Establishing vegetation within the grass channel
- Clogging in the inlet and outlet pipes
- Ant mounds
- Erosion

Routine inspection and maintenance should be performed on the grass channels to ensure that the practice is functioning properly. Routine maintenance tasks include removing trash from the grass channel and ensuring that grass clippings and other debris are removed from the channel.

In order to keep the water that exits the grass channel clean, fertilizers should only be used sparingly during the establishment of the practice. Once the vegetation in the practice has been established, fertilizers should not be used. While vegetation in the grass channel is important, a primary purpose of a grass channel is to act as a water quality device and introducing fertilizers into the grass channel introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

The table on the following page show routine maintenance activities typically associated with grass channels.

**Grass Channel Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Mow grass to maintain a height of 3 to 4 inches. Remove grass clippings.<br>• Repair eroded or bare spots.<br>• Remove accumulated sediment, trash, and debris.<br>• Water the practice during dry condition while vegetation is establishing. | As needed |
| • Inspect grass alongside slopes for erosion and formation of rills or gullies and correct.<br>• Remove sediment from bottom of channel once sediment is 25% of the original design volume.<br>• Remove trash and debris accumulated in the inflow forebay.<br>• Inspect and correct erosion problems in the sand/soil bed of dry swales.<br>• Based on inspection, plant an alternative grass species if the original grass cover has not been successfully established.<br>• Inspect pea gravel diaphragm for clogging and correct the problem. | Annually (Semi-annually the first year and then annually thereafter) |

| Grass Channel | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet. | | | | | |
| No signs of clogging or damage around the inlet. | | | | | |
| **Pretreatment (choose one)** | | | | | |
| Forebay – area is free of trash, debris, and sediment. | | | | | |
| Filter Strip or Grass Channels – area is free of trash debris and sediment. Area has been mowed and grass clippings are removed. No evidence of erosion. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion in the practice. | | | | | |
| No evidence of long-term ponding or standing water in the ponding area of the practice (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| No undesirable vegetation located within the practice. | | | | | |
| No evidence of use of fertilizer on plants (fertilizer crusting on the surface of the soil, blackened roots, etc.). | | | | | |
| Grass within and around practice is maintained at the proper height (3-4 inches). Grass clippings are removed. | | | | | |
| Grass cover seems healthy with no bare spots or dying grass. | | | | | |

| Grass Channel | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| No accumulating sediment within the grass channel. | | | | | |
| Outlet | | | | | |
| Outlet is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding. | | | | | |
| Results | | | | | |
| Overall condition of Grass Channel: | | | | | |
| Additional Comments | | | | | |
| | | | | | |
| **Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

## Gravity (Oil-Grit) Separators


Gravity (oil-grit) separators are designed to treat stormwater runoff by removing settleable solids, oil and grease, debris and floatables from stormwater runoff through gravitational settling and trapping of pollutants. Typically these systems are underground and installed at inlet structures. Gravity (oil-grit) separators come in different shapes and sizes ranging from small to large systems that have multiple chambers that use gravity to separate sediment, floatables, and oil/grease from stormwater runoff.

There are some common problems to be aware of when maintaining gravity (oil-grit) separators. They include, but are not limited to, the following:

- Clogging in the inlet and outlet structure
- Sediment and oil/grease build-up
- Inability to remove dissolved or emulsified oils and pollutants such as coolants, soluble lubricants, glycols and alcohols

Routine inspection and maintenance should be performed on the gravity separator to ensure that the structure is functioning properly. Typical maintenance will include removing accumulated sediment and pressure washing the system to remove blockage. Additional maintenance may be necessary if a spill occurs upstream of the system and drains into the practice. The contributing drainage areas should be maintained to limit the amount of trash and debris that enter the practice.

Gravity (oil-grit) separators should be inspected after a large rainstorm. It may be necessary to make repairs to the inlets, outlets, and other structural components. Check with the manufacturer's guidelines for recommended maintenance on the system. In addition, it is required that a maintenance plan be developed and implemented.

The table below shows a schedule for when different maintenance activities should be performed on gravity (oil-grit) separators.

**Gravity (Oil-Grit) Separators Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Keep contributing drainage area free of trash, chunks of sediment, and debris.<br>• Cleanout if spill occurs and enters the system.<br>• Repair structural components.<br>• Check to make sure practice is draining properly. | As needed (quarterly or after a large rain storm event) |

| Activity | Schedule |
|---|---|
| • Check maintenance plan and/or manufacturer's guidelines for additional maintenance needs.<br>• Check system to make sure no blockage or significant sediment accumulation is occurring in the system. | Quarterly |
| • Cleanout system with vacuum or boom trucks.<br>• Remove sediment and oil from chambers | Annually |

| Gravity (Oil-Grit) Separator | | | | | |
|---|---|---|---|---|---|
| **Maintenance Item** | **Condition** | | | | **Comment** |
| | **Good** | **Marginal** | **Poor** | **N/A*** | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Contributing drainage area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| Inlet and outlet pipes are clean; stormwater can enter and exit the practice without being blocked. | | | | | |
| Overflow structure is in good condition and clean. | | | | | |
| Maintenance is being performed according to manufacturer's guidelines. | | | | | |
| Maintenance is being performed according to maintenance plan. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| Structure seems to be working properly. No settling around the structure. Comment on overall condition of structure. | | | | | |
| **Results** | | | | | |
| Overall condition of Gravity (Oil-Grit) Separator: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

*This page intentionally left blank.*

## Green Roofs

Green roofs represent an alternative to traditional impervious roof surfaces. They typically consist of underlying waterproofing and drainage materials and an overlying engineered growing media that is designed to support plant growth. Stormwater runoff is captured and temporarily stored in the engineered growing media, where it is subjected to evaporation and transpiration before being conveyed back into the storm drain system. This allows green roofs to provide measurable reductions in post-construction stormwater runoff rates, volumes and pollutant loads on development sites. There are two different types of green roof systems, intensive and extensive. Intensive green roofs have a thick layer of soil, can support a diverse plant community, and may include trees. Extensive green roofs have a much thinner layer of soil that is comprised primarily of drought tolerant vegetation. Plants chosen for a green roof should be compatible for warmer temperatures found on rooftops.

There are some common problems to be aware of when maintaining a green roof. They include, but are not limited to, the following:

- Clogging in the outlet structure
- Establishing vegetation within the green roof
- Clogging the drainage layer
- Maintaining the proper pH levels for plants
- Pruning and weeding to maintain appearance and prevent roots from potentially compromising the waterproof membrane

Routine inspection maintenance should be performed on green roofs to ensure that the system is functioning properly. Note that during the first year the green roof is built, inspection and maintenance will be required at a higher frequency to ensure the proper establishment of vegetation in the practice. Frequent watering and weed germination during establishment is key to maintaining a healthy green roof and preventing more long-term maintenance problems. For more information on green roof vegetation, see Appendix D: Planting and Soil Guidance.

Green roofs should be inspected after a large rainstorm. Keep drainage paths, both to and from the BMP, clean so that the water can properly flow to the plants and keep the vegetation healthy. Impaired drainage can cause damage to the roofing system and add structural loads beyond the building's design limits; this could lead to structural failure. Note that it might take longer for the water to infiltrate into the system during the winter months and early spring.

In order to keep the water that exits the green roof clean, fertilizers should be used only be used sparingly and during the establishment of the practice. Once the vegetation in the practice has been established, fertilizers should not be used. While vegetation in the green roof is important, the primary purpose of a green roof is to act as a water quality device and introducing fertilizers into the green roof introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. In addition, after initial vegetation establishment, green roofs should already be a nutrient rich environment that does not require fertilization. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

The table below shows a schedule for when different maintenance activities should be performed on a green roof.

**Green Roofs Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Water to promote plant growth and survival.<br>• Inspect green roof for dead or dying vegetation. Dead vegetation should be removed along with any woody vegetation. Plant replacement vegetation as needed.<br>• Mow and remove grass clippings.<br>• Remove trash, debris, and other pollutants from the rooftop<br>• Observe infiltration rates after rain events, the roof should drain in 24 hours of rain event. | As Needed |
| • Inspect waterproof membrane for leaks. Repair as needed.<br>• Inspect outflow and overflow areas for trash, debris, and sediment accumulation. Remove any accumulated sediment or debris.<br>• Inspect vegetation for signs of stress. If vegetation begins showing signs of stress, including drought, flooding, disease, nutrient deficiency or insect attack, treat the problem or replace the vegetation.<br>• Weed and prune vegetation. | Semi-Annually (Quarterly During First Year) |
| • Test the planting soils for pH levels. Consult with a qualified licensed Professional to determine and maintain the proper pH levels. | Annually |

| Green Roof | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| Inlet and outlet pipes are free of trash, debris, etc. | | | | | |
| Inspect waterproof membrane. | | | | | |
| No signs of structural deficiency or settling. Comment on overall condition of roof. | | | | | |
| Water can flow freely in the drainage routes, no obstructions. | | | | | |
| Native plants were used in the practice according to the landscaping plan. Plants seem to be in good condition. Comment on condition of plants. | | | | | |
| No unwanted vegetation in the practice. | | | | | |
| No evidence of use of fertilizer on plants (fertilizer crusting on the surface of the soil, tips of leaves turning brown or yellow, blackened roots, etc.). | | | | | |
| No evidence of long-term ponding or standing water (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| **Results** | | | | | |
| Overall condition of Green Roof: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

*This page intentionally left blank.*

## Infiltration Practice

An infiltration practice is a shallow excavation, typically filled with stone or an engineered soil mix, which is designed to temporarily hold stormwater runoff until it infiltrates into the surrounding soils. Infiltration practices are able to reduce stormwater quantity, recharge the groundwater, and reduce pollutant loads.

There are some common problems to be aware of when maintaining infiltration practices. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the inlet and outlet structure
- Clogging the underdrain (if applicable)
- Mosquitoes breeding in the practice

Routine maintenance should be performed on infiltration practices to ensure that the practice is functioning properly. Infiltration practices should be inspected after a large rainstorm. Keep drainage paths, both to and from the BMP, clean so that the water can properly infiltrate into the ground. Note that it might take longer for the water to infiltrate into the ground during the winter months and early spring.

In order to limit the sediment that enters the infiltration practice, infiltration practices should always be designed with adequate pretreatment (e.g., vegetated filter strip, sediment forebay). Routine maintenance of the pretreatment device, such as removing accumulated sediment, trash, and debris, decreases the amount of maintenance required on the infiltration practice as well as its likelihood of clogging and failing. Infiltration trenches can have either exposed aggregate at the surface of the practice which provides sediment removal and additional pretreatment upstream of the infiltration trench and can be easily removed and replaced when it becomes clogged.

If the infiltration practice is not draining properly, check for clogging of the inflow structure or underdrain. To help ensure that larger storm events are able to safely bypass the infiltration practice a perforated pipe (e.g., underdrain) is sometimes placed near the top of the stone reservoir or planting bed. This provides additional conveyance of stormwater runoff after the infiltration trench or basin has filled. Another consideration is the infiltration rate of the soil media. If the soil is not draining properly, the filter fabric could be clogged or the soil could be clogged or over-compacted. In an infiltration practice, the filter fabric is likely to be clogged along the top and sides of the infiltration practice. If the filter fabric becomes clogged, the practices will need to be dug up, cleaned, and the fabric replaced. The media is likely to become clogged at the upper layer of the soil first. If the media is clogged or over-compacted, then the media should be replaced. Potential sources of excessive sediment that could clog the media include ant mounds and unstable soil upstream of the practice. Possible sources of compaction are tractors or maintenance vehicles traveling through the practice. If the practice includes an underdrain, a structural repair or cleanout to unclog the underdrain may be necessary.

If designed and maintained correctly, there is no danger of infiltration practices becoming a breeding ground for mosquitoes. A mosquito egg requires 24-48 hours to hatch. In addition, it takes 10-14 more days for the egg to develop and become an adult. By having an infiltration practice that drains properly, it is unlikely that it would provide a habitat that could become a breeding area for mosquitoes. Should the infiltration practices become a breeding ground for mosquitoes, the problem is likely with the soil media or the overflow structure which may need to be addressed.

The table below shows a schedule for when different maintenance activities should be performed on the infiltration practice.

**Infiltration Practice Typical Routine Maintenance Activities and Schedule**

| Maintenance Activity | Schedule |
|---|---|
| • Inspect to ensure that contributing drainage area and infiltration practice are clear of sediment, trash and debris. Remove any accumulated sediment and debris.<br>• Ensure that the contributing drainage area is stabilized. Plant replacement vegetation as needed.<br>• Check observation well to ensure that infiltration practice is properly dewatering after storm events. | Monthly |
| • Inspect pretreatment devices for sediment accumulation. Remove accumulated sediment, trash and debris.<br>• Inspect top layer of filter fabric and pea gravel or landscaping for sediment accumulation. Remove and replace if clogged.<br>• Inspect the practice for damage, paying particular attention to inlets, outlets and overflow spillways. Repair or replace any damaged components as needed.<br>• Inspect the practice following rainfall events (specifically large rainfall events). Check observation well to ensure that complete drawdown has occurred within 72 hours after the end of a rainfall event. Failure to drawdown within this timeframe may indicate infiltration practice failure. | Semi-Annually during first year and Annually thereafter |
| • Remove aggregate and install clean, washed trench aggregate<br>• It may be necessary to replace piping, filter fabric, etc. | Upon Failure |

| Infiltration Practice | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. Drainage ways are in good condition. | | | | | |
| Area around the inlet structure is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet structure. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| Diversion structure (high flow bypass structure or underdrain) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Pretreatment (choose one)** | | | | | |
| Forebay – area is free of trash, debris, and sediment. | | | | | |
| Forebay – No undesirable vegetation. | | | | | |
| Forebay – No signs of erosion, rills, or gullies. Erosion protection is present on site. | | | | | |
| Forebay – No signs of standing water. | | | | | |
| Filter Strip– area is free of trash debris and sediment. Area has been mowed and grass clippings are removed. No evidence of erosion or sediment accumulation. | | | | | |
| Filter Strip – No signs of unhealthy grass, bare or dying grass. Grass height is maintained to a height of 6 – 15 inches. | | | | | |
| Filter Strip– No signs of erosion, rills, or gullies. Erosion protection is present on site. | | | | | |
| Filter Strip – No undesirable vegetation. | | | | | |
| Filter Strip – No signs of standing water (examples include: stains, odors, mosquito larvae, etc). | | | | | |

| Infiltration Practice | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| Structure seems to be working properly. No settling around the structure. Comment on overall condition of structure. | | | | | |
| No signs of ponding water more than 48 hours after a rain storm event (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| No undesirable vegetation growing within the practice. | | | | | |
| Native plants were used in the practice according to the landscaping plan. | | | | | |
| Observation well is capped and locked when not in use | | | | | |
| Flow testing has been performed on infiltration practice to determine if underdrain is clogged. | | | | | |
| **Emergency Overflow and Outlet Structure** | | | | | |
| Area is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| No signs of sediment accumulation. | | | | | |
| Grass height of 6 – 15 inches is maintained. | | | | | |
| **Results** | | | | | |
| Overall condition of Infiltration Practice: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

## Multi-Purpose Detention Basins

Multi-purpose detention basins are facilities designed primarily for another purpose, such as a parking lot or roof top, that also provide water quantity control through detention of stormwater runoff. The temporary storage provided by multi-purpose detention basins reduces downstream water quantity impacts. Multi-purpose detention areas are normally dry between rain events, and by their very nature must be useable for their primary function the majority of the time.

There are some common problems to be aware of when maintaining a multi-purpose detention basin. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the inlet and outlet structures
- Establishing vegetation within the multi-purpose detention basin
- Erosion
- Structural repairs to inlets and outlets
- Clogging in the emergency spillway

Routine inspection and maintenance should be performed on a multi-purpose detention basin to ensure that the structure is properly functioning. In addition to routine maintenance, multi-purpose detention basins may have seasonal and intermittent maintenance requirements. For example, if vegetation is included in the practice, trim the planting material during the winter, when the plants are dormant.

The table below shows routine maintenance activities typically associated with multi-purpose detention basins.

**Multi-Purpose Detention Basin Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Remove debris from ponding area to minimize outlet clogging and improve aesthetics. | Annually and following significant storm events |
| • Remove sediment buildup.<br>• Repair and revegetate eroded areas.<br>• Perform structural repairs to inlet and outlets. | As needed based on inspection |
| • Perform additional maintenance activities specific to the type of facility. | As required |

*This page intentionally left blank.*

| Multi-Purpose Detention Basin | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet Structure and Pretreatment** | | | | | |
| Drainage ways (overland flow, pipes or preteatment) to the practice are free of trash, debris, large branches, etc. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet structure. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| Diversion structure (high flow bypass structure or other) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| No evidence of long-term ponding or standing water in the ponding area of the practice (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| Structure seems to be working properly. No settling around the structure. Comment on overall condition of structure. | | | | | |
| Vegetation within and around practice is maintained per landscaping plan. Grass clippings are removed. | | | | | |
| Plants seem to be healthy and in good condition. Comment on condition of plants. | | | | | |
| **Emergency Overflow and Outlet Structure** | | | | | |
| Area is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |

| Multi-Purpose Detention Basin | | | | | |
|---|---|---|---|---|---|
| **Maintenance Item** | **Condition** | | | | **Comment** |
| | **Good** | **Marginal** | **Poor** | **N/A*** | |
| **Results** | | | | | |
| Overall condition of Multi-Purpose Detention Basin: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please explain why in the appropriate comment box. | | | | | |

## Organic Filter

Organic filters, a design variant of the surface sand filter with organic materials as the filter media, are multi-chamber structures designed to treat stormwater runoff through filtration. An organic filter consists of a pretreatment chamber, and one or more filter cells. Each filter bed contains a layer of leaf compost or a peat/sand mixture, followed by an underdrain system. Maintenance frequency on organic filters is typically high due to clogging.

Common problems to be aware of when maintaining an organic filter include, but are not limited to, the following:

- Sediment build-up
- Clogging in the inlet and outlet structure
- Clogging the underdrain
- Mosquitoes breeding in the practice
- Ant mounds

Routine inspection and maintenance should be performed on the organic filters to ensure that the structure is functioning properly. Note that if the organic filter includes topsoil and vegetation, maintenance may be required at a higher frequency during the first year the organic filter is built to ensure the proper establishment of vegetation in the practice.

Inspect the organic filter after a large rainstorm. Keep drainage paths (both to and from the BMP) clean so that the water can properly infiltrate into the ground. If the organic filter is not draining properly, check for clogging at the inflow and outflow structures as well as the infiltration rate of the filter bed. In an organic filter, the filter bed is likely to become clogged at the upper layer of the filter (top 2-3 inches) and will need to be removed and replaced. If the filter becomes clogged or over-compacted, then the media should be replaced. In order to determine if maintenance is necessary, a record should be kept of the dewatering time for an organic filter. Typically the filter bed is designed to drain in 40 hours or less, if it the practice takes longer to drain, maintenance may be required for the practice.

For organic filters with vegetation, to keep the water that exits the organic filter clean, fertilizers should only be used sparingly during the establishment of the practice. Once the vegetation in the practice has been established, fertilizers should not be used. While vegetation in the organic filter is important, the primary purpose of an organic filter is to act as a water quality device and introducing fertilizers into the organic filter introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. In addition, organic filters should already be a nutrient rich environment that does not require fertilization. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

Potential sources of excessive sediment that could clog the media include ant mounds and unstable soil upstream of the practice. Possible sources of compaction are maintenance vehicles traveling through the practice. If the underdrain does not work properly, a structural repair or cleanout to unclog the underdrain may be necessary.

In the event of snow, ensure that the snow does not pile up in the organic filter. Accumulated snow adds additional weight and may compact the organic filter soil, which would reduce its infiltration capacity. In addition, check to make sure that the materials used to de-ice the surrounding areas stay out of the practice to avoid clogging and further pollution.

If designed and maintained correctly, there is no danger of organic filters becoming a breeding ground for mosquitoes. A mosquito egg requires 24-48 hours to hatch. In addition, it takes 10-14 more days for the egg to develop and become an adult. By having an organic filter that drains properly, it is unlikely that an organic filter would provide a habitat that could become a breeding area for mosquitoes. Should the organic filter become a breeding ground for mosquitoes, the problem is likely with the soil media or the overflow structure which may need to be addressed. This is not applicable to the perimeter organic filter which has a permanent pool.

The table below shows a schedule for when different maintenance activities should be performed on the organic filter.

**Organic Filter Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Check to see that the filter bed is clean of sediment, and the sediment chamber is not more than 50% full or 6 inches, whichever is less, of sediment (also check after moderate and major storms). Remove sediment as necessary.<br>• Make sure that there is no evidence of deterioration, spalling or cracking of concrete.<br>• Inspect grates (perimeter Organic Filter).<br>• Inspect inlets, outlets and overflow spillway to ensure good condition and no evidence of erosion.<br>• Repair or replace any damaged structural parts.<br>• Stabilize any eroded areas.<br>• Ensure that flow is not bypassing the practice.<br>• Ensure that no noticeable odors are detected outside the BMP. | Monthly |
| • Ensure that contributing area, organic filter, inlets and outlets are clear of trash and debris.<br>• Ensure that the contributing area is stabilized and mowed, with clippings removed.<br>• Prune and weed to maintain appearance, if applicable.<br>• Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system.<br>• If permanent water level is present (perimeter Organic Filter), ensure that the chamber does not leak, and normal pool level is retained. | As needed or 4 times during growing season |

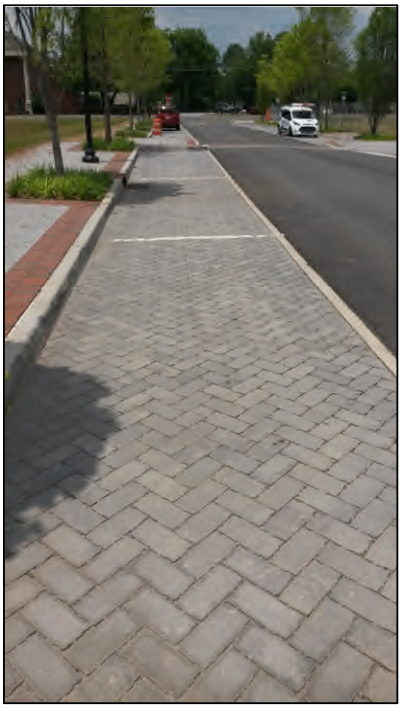
| Activity | Schedule |
|---|---|
| • If filter bed is clogged or partially clogged, manual manipulation of the surface layer of sand may be required. Remove the top few inches of sand, roto-till, or otherwise cultivate the surface, and replace media with sand meeting the design specifications.<br>• Replace any filter fabric that has become clogged. | Annually |
| • Remove and replace the top 2-3 inches of sand in the filter. | Every 3-5 years or as needed |

*This page intentionally left blank.*

| Organic Filter | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| Area is free of signs of erosion. | | | | | |
| **Inlet Structure** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet structure is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet structure. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| Diversion structure (high flow bypass structure or other) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Pretreatment (choose one)** | | | | | |
| Forebay – area is free of trash, debris, and sediment. Area is free of undesirable vegetation. | | | | | |
| Sedimentation Chamber – area is free of trash, debris, and sediment. | | | | | |
| Perforated stand-pipe is free of trash, debris, and sediment. Surrounding vegetation is trimmed back so that there is no potential to restrict flow. Pipe is in good working order. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| Structure seems to be working properly. No settling around the structure. Comment on overall condition of structure. | | | | | |

| Organic Filter | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| Undesirable vegetation within and around practice is trimmed and removed. | | | | | |
| Significant sediment accumulation is not occurring within the filter bed. | | | | | |
| Grass cover is healthy and there are no bare areas or dying grass. | | | | | |
| No evidence of leaks at joints or other components of the practice. | | | | | |
| Underdrain cleanout caps are not missing or damaged. | | | | | |
| Observation well does not have standing water. | | | | | |
| Emergency Overflow | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| No evidence of animal activity. | | | | | |
| No evidence of seepage on the downstream face of the structure. | | | | | |
| Outlet Structure | | | | | |
| Outlet structure is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| Outlet structure does not appear to be blocked. | | | | | |
| Results | | | | | |
| Overall condition of Organic Filter: | | | | | |
| Additional Comments | | | | | |
| | | | | | |
| **Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

## Permeable Bricks/Blocks



Permeable bricks/blocks are pavers with void areas areas between the bricks or blocks that are generally filled with pervious materials such as small pieces of gravel, or top soil if a grid is used. Beneath the bricks/blocks is a base layer of aggregate that acts as a holding area for stormwater runoff while still providing structural support for the road. This practice provides enough structural support so that cars can drive over them or they can be used in parking lots.  Permeable brick/blocks are not recommended in areas with high traffic volume or heavy trucks. These systems provide water quality benefits in addition to groundwater recharge and a reduction in stormwater volume.

There are some common problems to be aware of when maintaining permeable bricks/blocks.  They include, but are not limited to, the following:

- Sediment build-up and clogging between bricks/blocks
- Settling
- Bricks/blocks cracking or splitting

There are four basic types of permeable bricks/blocks that are used. They are bricks, concrete blocks, concrete grid, and articulated concrete block.  The concrete grid can be filled with grass or gravel. Routine maintenance should be performed on the permeable bricks/blocks to ensure that the structure is functioning properly. Permeable bricks/blocks should be cleaned with a street vacuum or low pressure washer to remove debris and sediment monthly, or as needed, and all vegetation between bricks/blocks should be mowed and clippings removed to reduce clogging. Cleaning the bricks will help keep the water permeating through the pavers. After cleaning, the bricks/blocks may need to be filled in with additional aggregate or top soil to replace anything that may have been removed during cleaning.

In addition to routine maintenance, permeable bricks/blocks have seasonal and intermittent maintenance requirements.  In the winter months permeable bricks/blocks can be plowed similarly to any other unpaved road by lifting the blade a few inches above the road or by using a beveled plow. Deicing materials such as sand, ash, or salt should be avoided if possible.  They can potentially harm the bricks/blocks and may cause clogging. Non-toxic, organic deicers are recommended.

Permeable bricks/blocks should be inspected after a large rainstorm. Keep drainage paths, both to and from the BMP, clean so that the water can properly infiltrate into the ground.  Note that it might take longer for the water to permeate into the ground during the winter months and early spring.

If the permeable bricks/blocks are not draining properly, check for clogging between the bricks or blocks or at the upper layer of the aggregate, directly below the bricks/blocks. If clogging occurs, then the stones between the blocks/bricks should be replaced. In addition, the top layer of soil under the bricks/blocks may also need to be cleaned and replaced. Some areas of the blocks/bricks may need additional maintenance due to potential sources of clogging which include unstable soil upstream of the practice, leaves from trees, low points in blocks/bricks, trash, and debris from vehicle traffic. Another reason for the bricks/blocks not draining properly is settling. If major settling occurs, then the bricks/blocks should be removed, cleaned, and replaced.

Permeable bricks/blocks may also include an underdrain. If the practice includes an underdrain, additional maintenance will be required. Periodic testing will need to be done on the system to make sure that the underdrain is not clogged. This is done by pouring water into cleanout and observing how the water exits the practice. The observation well should be checked to make sure water is draining out of the practice.

The table below shows a schedule for when different maintenance activities should be performed on the permeable bricks/blocks.

**Permeable Bricks/Blocks Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Keep the permeable bricks/blocks free of trash, debris, and sediment.<br>• Make sure that there is no standing water in the bricks/blocks between storms.<br>• Remove weeds and grass growing between the bricks/blocks (unless concrete grid pavers are being used).<br>• Mow grass within the bricks/blocks (only for concrete grid with grass)<br>• Mow / trim grass or vegetation near the bricks/blocks and remove clippings from area.<br>• Visually inspect the bricks/blocks after large storms to ensure the overflow drainage system is working.<br>• Inspect the bricks/blocks for damage and repair.<br>• Vacuum sweep permeable brick/block surface to keep free of sediment.<br>• After cleaning, additional aggregate may need to be added between the pavers. Replace aggregate between pavers as necessary. | Monthly during warm weather |
| • Keep the contributing drainage area and surface of the bricks/blocks clear of debris, trash, and sediment.<br>• Ensure that the areas surrounding the practice are stabilized and mowed, remove grass clippings. | As needed, based on inspection |

| Activity | Schedule |
|---|---|
| • If the pavers are installed in an area that is subject to high amounts of sediment, leaves, or low point (i.e. large trucks traveling on the bricks/blocks daily) additional cleaning may be necessary.<br>• Replace any joint material that has eroded or washed away.<br>• Observe the system during a rain event.<br>• Areas should be routinely inspected for settling and loss of water flow through the system. | Semi-annually in Spring and Fall |
| • Organic deicers may be used to melt ice and snow.<br>• Snow plows may be used when necessary under the following conditions:<br>o The edges of the plows are beveled.<br>o The blade of the snow plow is raised 1-2 inches.<br>o The snow plow is equipped with snow shoes which allow the blade to glide across uneven surfaces. | As needed in winter |
| • Inspect the surface for deterioration or breaking into fragments.<br>• Flush the underdrain system to check for clogging (if applicable). | Annually |
| • Remove the permeable bricks/blocks; include the top and base layers of the practice. Clean bricks/blocks and base aggregate, and replace as needed. | Upon failure |

*This page intentionally left blank.*

| Permeable Bricks/Blocks | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| General Inspection | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, leaves, etc. removed). | | | | | |
| Area around the practice is mowed and grass clippings are removed. No signs of bare or dead grass. | | | | | |
| No evidence of gullies, rills, or erosion around the practice. | | | | | |
| Water is permeating the bricks/blocks (i.e. no evidence of water going around the practice). | | | | | |
| Bricks/blocks are structurally sound. No signs of cracks or splitting. | | | | | |
| Aggregate between the bricks/blocks is reasonable. | | | | | |
| No evidence of long-term ponding or standing water in the practice. | | | | | |
| Grass in the concrete grid is healthy, no dead grass or bare spots. | | | | | |
| Grass in the concrete grid is mowed and grass clippings are removed. | | | | | |
| Structure seems to be working properly. No signs of the bricks/blocks settling. Comment on overall condition of bricks/blocks. | | | | | |
| Vegetation within and around practice is maintained. Grass clippings are removed. | | | | | |
| No exposed soil near the bricks/blocks that could cause sediment accumulation within the practice. | | | | | |
| Cleanout caps are present and not missing (if applicable). | | | | | |
| The underdrain system has been flushed properly and there is no sign of clogging (if applicable). | | | | | |
| Results | | | | | |
| Overall condition of Permeable Bricks/Blocks: | | | | | |

<table>
<tr><th colspan="6">Permeable Bricks/Blocks</th></tr>
<tr><th rowspan="2">Maintenance Item</th><th colspan="4">Condition</th><th rowspan="2">Comment</th></tr>
<tr><th>Good</th><th>Marginal</th><th>Poor</th><th>N/A*</th></tr>
<tr><th colspan="6">Additional Comments</th></tr>
<tr><td colspan="6"></td></tr>
<tr><td colspan="6">Notes: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box.</td></tr>
</table>

## Pervious Concrete

Pervious concrete is a special mixture with a high void space that allows water to infiltrate into the subsoil through the pavement surface and base layers. This aggregate base layer acts as both a structural layer and a container to temporarily hold stormwater runoff until it can infiltrate into the subsoil or drainage system.

There are some common problems to be aware of when maintaining pervious concrete. They include, but are not limited to, the following:

- Sediment build-up on surface
- Settling
- Cracking

Routine maintenance should be performed on pervious concrete to ensure that the area is functioning properly. Pervious concrete should be observed monthly to ensure the practice is functioning properly. Maintenance activities including street vacuum or low pressure washer to remove debris and sediment should be conducted at least annually, or as needed.

In addition to routine maintenance, pervious concrete has seasonal and intermittent maintenance requirements. In the winter months pervious concrete can be plowed, however, the snow plow should be equipped with snow shoes which can allow the blade to glide across uneven surfaces. Deicing materials such as sand, ash, salt, or other products should be avoided if possible. They will harm the concrete and other materials and may cause clogging. Organic deicers are recommended.

Pervious concrete should be inspected after a large rainstorm. Keep drainage paths, both to and from the BMP, clean so that the water can properly infiltrate through the concrete and into the ground. Note that it might take longer for the water to infiltrate into the ground during the winter months and early spring.

If the pervious concrete is not draining properly, check for clogging at the top of the concrete. If clogging occurs, then the concrete should be cleaned by vacuuming or jet washing the area. Potential sources of clogging include unstable soil, leaves from trees, trash, and debris from vehicle traffic. The concrete could also not be draining properly due to settling or structural failure. If this happens, then the concrete should be removed and replaced. Settling or structural failure is most likely to occur in areas with high volumes of traffic or in areas with heavy traffic, such as large trucks.

The surface of the concrete should be inspected for deterioration. If the concrete fails, then the concrete should be resurfaced. Pervious concrete is intended for areas of low traffic; constant traffic and heavy equipment will cause the pavement to deteriorate more quickly.

Pervious concrete may also include an underdrain or a trench outlet. If the practice includes an underdrain or a trench, additional maintenance will be required. Periodic testing may be necessary to make sure that the underdrain or trench is not clogged. The underdrain or trench can be tested by pouring water into cleanout and observing how the water exits the practice. The observation well for the underdrain should be checked to make sure water is draining out of the practice.

The table below shows routine maintenance activities typically associated with pervious concrete.

**Pervious Concrete Typical Routine Maintenance Activities and Schedule**

| Maintenance Activity | Schedule |
|---|---|
| • Ensure that contributing area, facility, inlets and outlets are clear of debris.<br>• Ensure that the contributing area is stabilized and mowed, with clippings removed.<br>• Remove trash and debris.<br>• Check to ensure that the pavement surface is not clogging (also check after moderate and major storms).<br>• Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system. | As needed |
| • Make sure that there is no evidence of deterioration or cracking of the concrete.<br>• Inspect inlets, outlets and overflow spillway to ensure good condition and no evidence of erosion.<br>• Repair or replace any damage to the asphalt.<br>• Ensure that flow is not bypassing the facility. | Monthly |
| • Vacuum sweep pervious concrete surface followed by high pressure hosing to keep pores free of sediment.<br>• Flush the underdrain system and check for signs of clogging. | Annually or based on inspection |
| • Utilize organic de-icers on the pavement surface | During temperatures below freezing |

| Pervious Concrete | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. No signs of bare or dead grass. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| No evidence of long-term ponding or standing water in the practice (examples include: stains, odors, etc). | | | | | |
| Structure seems to be working properly. No signs of concrete settling or cracking. Comment on overall condition of concrete. | | | | | |
| Vegetation around practice is maintained. Grass clippings are removed. | | | | | |
| No exposed soil near the concrete. | | | | | |
| Cleanout caps are present and not missing (if applicable). | | | | | |
| The underdrain system or trench has been flushed properly and there is no sign of clogging. | | | | | |
| **Results** | | | | | |
| Overall condition of Pervious Concrete: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

*This page intentionally left blank.*

## Porous Asphalt

In general, porous asphalt is asphalt with reduced sands or fines and larger void spaces to allow water to drain through it. Porous asphalt allows water to infiltrate into the subsoil through the paved surface and base layer. This base, aggregate layer acts as both a structural layer and container to temporarily hold water. Porous asphalt is generally used instead on sidewalks or bicycle paths or roads with low traffic volumes.

There are some common problems to be aware of when maintaining porous asphalt. They include, but are not limited to, the following:

- Sediment build-up on surface
- Settling
- Cracking

Routine maintenance should be performed on porous asphalt to ensure that the area is functioning properly. Porous asphalt should be observed monthly to ensure the practice is functioning properly. Maintenance activities including vacuuming and jet washing should be conducted at least annually, or as needed.

In addition to routine maintenance, porous asphalt has seasonal and intermittent maintenance requirements. In the winter months porous asphalt can be plowed, however, the snow plow should be equipped with snow shoes which allow the blade to glide across uneven surfaces. Deicing materials such as sand, ash, or salt should be avoided if possible because they may harm the asphalt and aggregate and may cause clogging. Non-toxic, organic deicers are recommended.

Porous asphalt should be inspected after a large rainstorm. Keep drainage paths, both to and from the BMP, clean so that the water can properly infiltrate into the ground. Note that it might take longer for the water to infiltrate into the ground during the winter months and early spring.

If the porous asphalt is not draining properly, check for clogging at the top of the asphalt. If clogging occurs, then the asphalt should be cleaned by vacuuming or low pressure washer the area. Potential sources of clogging include upstream unstable soil, leaves from trees, trash, and debris from vehicle traffic. Asphalt could also not be draining properly due to settling or structural failure. If this happens, then the asphalt should be removed and replaced. Settling or structural failure is most likely to occur in areas with high volumes of traffic or in areas with heavy traffic, such as large trucks.

The surface of both types of porous asphalt should be inspected for deterioration. If the pavement fails, the asphalt should be resurfaced. Potholes, though uncommon, can be patched using standard measures. If the damaged area is 10% or more of the total area, consult a qualified licensed Professional Engineer for repair.

Porous asphalt may also include an underdrain. If the practice includes an underdrain, additional maintenance will be required. Periodic testing will be necessary to make sure that the underdrain is not clogged. This is done by pouring water into cleanout and observing how the water exits the practice. The observation well should be checked to make sure water is draining out of the practice.

The table below shows routine maintenance activities typically associated with porous asphalt.

**Porous Asphalt Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Ensure that contributing area, facility, inlets and outlets are clear of debris.<br>• Ensure that the contributing area is stabilized and mowed, with clippings removed.<br>• Remove trash and debris.<br>• Check to ensure that the pavement surface is not clogging (also check after moderate and major storms).<br>• Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system. | As needed |
| • Make sure that there is no evidence of deterioration, spalling or cracking of asphalt.<br>• Inspect inlets, outlets and overflow spillway to ensure good condition and no evidence of erosion.<br>• Repair or replace any damage to the asphalt.<br>• Ensure that flow is not bypassing the facility. | Monthly |
| • Vacuum sweep pervious concrete surface followed by high pressure hosing to keep pores free of sediment.<br>• Flush the underdrain system and check for signs of clogging. | Annually or based on inspection |
| • Utilize organic de-icers on the pavement surface | During temperatures below freezing |

| Porous Asphalt | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| Filter Strip (if applicable) – area is free of trash debris and sediment. Area has been mowed and grass clippings are removed. No evidence of erosion. | | | | | |
| Asphalt is structurally sound. No signs of cracks or raveling (disintegration of material from surface down). | | | | | |
| No evidence of long-term ponding or standing water in the practice. | | | | | |
| Structure seems to be working properly. No settling around the structure. Comment on overall condition of structure. | | | | | |
| Vegetation around practice is maintained. Grass clippings are removed. | | | | | |
| No exposed soil near the asphalt. | | | | | |
| Cleanout caps are present and not missing. | | | | | |
| The underdrain system has been flushed properly and there is no sign of clogging (if applicable). | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| **Results** | | | | | |
| Overall condition of Porous Asphalt: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please explain what and why in the appropriate comment box. | | | | | |

*This page intentionally left blank.*

## Proprietary Systems

Proprietary systems are control systems available from commercial vendors designed to treat stormwater runoff and/or provide water quantity control. Typically these systems are underground and installed at inlet structures. There are many types of proprietary stormwater structural controls to provide water quality treatment and quantity control. These systems come in different shapes and sizes ranging from small systems that use a swirling vortex to a large system that has multiple chambers to separate sediment, floatables, and oil/grease from the stormwater runoff.

There are some common problems to be aware of when maintaining propriety systems. They include, but are not limited to, the following:

- Sediment and oil/grease build-up
- Clogging in the inlet and outlet structure
- Inability to remove dissolved pollutants
- Must be maintained routinely so that system does not become a potential source of pollutants

Routine inspection and maintenance should be performed on the proprietary systems to ensure that the structure is functioning properly. Typical maintenance will include removing accumulated sediment and pressure washing the system to remove blockage. It is important that the accumulated sediment and water from cleaning the proprietary system be collected and disposed of properly. This is important to keep the ground surrounding the system clean to avoid clogging.

Additional maintenance may be necessary if a spill occurs upstream of the system and drains into the practice. The contributing drainage area should be maintained to limit the amount of trash and debris that enters the practice.

Proprietary systems should be inspected after a large rainstorm. It may be necessary to make repairs to the inlets, outlets, and other structural components. Check the manufacturer's guidelines for recommended maintenance on the system. In addition, it is required that a maintenance plan is developed and implemented.

The table below shows a schedule for when different maintenance activities should be performed on proprietary systems.

**Proprietary Systems Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Check to make sure practice is draining properly. | After a large rain storm event or as needed |
| • Keep contributing drainage area free of trash, chunks of sediment, and debris.<br>• Cleanout if spill occurs and enters the system.<br>• Repair structural components. | As needed |

| Activity | Schedule |
| --- | --- |
| • Check maintenance plan and/or manufacturer's guidelines for additional maintenance needs.<br>• Check system to make sure no blockage or significant sediment accumulation is occurring in the system. | Quarterly |
| • Cleanout system with vacuum or boom trucks.<br>• Remove sediment and oil from chambers | Annually |

## Proprietary System

| Maintenance Item | Condition | | | | Comment |
|---|---|---|---|---|---|
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Contributing drainage area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| Inlet and outlet pipes are clean; stormwater can enter and exit the practice without being blocked. | | | | | |
| Overflow structure is in good condition and clean. | | | | | |
| Maintenance is being performed according to manufacturer's guidelines. | | | | | |
| Maintenance is being performed according to the maintenance plan. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| Structure seems to be working properly. No settling around the structure. Comment on overall condition of structure. | | | | | |
| **Results** | | | | | |
| Overall condition of Proprietary System: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

*This page intentionally left blank.*

## Rainwater Harvesting

Rainwater harvesting is a common stormwater management practice used to catch rainfall and store it to be used later. Gutters and downspout systems are typically used to collect the water from roof tops. Rainwater harvesting systems can be either above or below the ground. Once captured in the storage tank, the water may be used for non-potable indoor and outdoor uses. If properly designed, rainwater harvesting systems can significantly reduce post-construction stormwater runoff rates, volumes and pollutant loads on development sites. Rainwater harvesting also helps reduce the demand on public water supplies, which in turn helps protect aquatic resources, such as groundwater aquifers, from drawdown and seawater intrusion.

There are some common problems to be aware of when maintaining a rainwater harvesting system.  They include, but are not limited to, the following:

- Sediment build-up in the system
- Wear and tear on pumping equipment (if applicable)
- Clogging in the gutters or downspout connections
- Algae growing in the rainwater system

Routine maintenance should be performed on the rainwater harvesting system. A well-designed rainwater harvesting system typically consists of five major components, including the collection and conveyance system (e.g., gutter and downspout system), pretreatment devices (e.g., leaf screens, first flush diverters, roof washers), the storage tank or cistern, the overflow pipe (which allows excess stormwater runoff to bypass the storage tank or cistern) and the distribution system (which may or may not require a pump, depending on site characteristics). Each of these components should be inspected and maintained.

Generally, maintenance should be performed in the spring and fall. Before the first significant freeze, downspouts should be disconnected, and the rainwater harvesting system should be completely drained. Other maintenance includes checking the system to make sure algae is not growing in the system.  Check the elements of the unit and make repairs or replace broken parts as necessary. Any vegetation that receives accumulated water from the system should be checked for signs of stress. Replace the plants as necessary.

The table below shows a schedule for when different maintenance activities should be performed on the rainwater harvesting system.

**Rainwater Harvesting System Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Disconnect rainwater harvesting system from roof downspouts (this may not be necessary for all areas of the state of Georgia).<br>• Drain aboveground cisterns and clean for winter. | Late Fall (Before major freeze) |
| • Connect rainwater harvesting system to roof downspouts. | Early Spring (After last major freeze) |
| • Empty harvesting rainwater system periodically by watering vegetation.<br>• Examine vegetation for health/distress and determine if additional watering needs are necessary.<br>• Inspect storage tank screens and pretreatment devices. Clean as needed. | Regularly during above freezing temperatures |
| • Inspect gutters and downspouts. Remove any accumulated leaves or debris.<br>• Clean storage tank screens.<br>• Inspect pretreatment devices for sediment accumulation. Remove accumulated trash and debris.<br>• Inspect for tight connection at inlet and drain valve.<br>• Verify pumping system is properly working.<br>• Keep pipe clear of obstructions.<br>• Inspect storage tank for algal blooms. Treat as necessary.<br>• Inspect overflow areas for erosion and the formation of rills and gullies. Plant replacement vegetation in any eroded areas. | Semi-annually in spring and fall |
| • Check system for sediment. Clean out the tank when the sediment is more than 5% of the volume in the cistern. | Annually |

| Rainwater Harvesting | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| Gutters and downspouts are free of trash, debris, etc. | | | | | |
| Leaf screens are clean and in good condition. | | | | | |
| First flush diverter is working properly and in good condition (if applicable). | | | | | |
| Roof washer is working properly and in good condition (if applicable). | | | | | |
| Cistern inlet and downspout fits tightly. | | | | | |
| Cistern tank is clean and free of sediment. | | | | | |
| Cistern is free of indication of algal blooms. | | | | | |
| Plants being watered from the rainwater harvesting system seem to be healthy and in good condition. Comment on condition of plants. | | | | | |
| No signs of the overflow valve leaking (stains, dampness). | | | | | |
| Cistern is in good condition structurally, no signs of cracking or leaking. | | | | | |
| Performance of pump matches pumping details (if applicable). | | | | | |
| **Results** | | | | | |
| Overall condition of Rainwater Harvesting: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please explain why in the appropriate comment box. | | | | | |

*This page intentionally left blank.*

## Regenerative Stormwater Conveyance

A regenerative stormwater conveyance (RSC) is a practice that provides treatment, infiltration, and conveyance to stormwater runoff through a combination of pools, riffles (with either cobble rocks or boulders), native vegetation, an underlying sand layer, and wood chips.

There are some common problems to be aware of when maintaining a RSC. They include, but are not limited to, the following:

- Establishing vegetation within the RSC area
- Ant mounds
- Pruning and weeding to maintain appearance
- Deterioration of riprap

Routine maintenance should be performed on RSC to ensure that the system is functioning properly. Note that during the first year the RSC is built, maintenance may be required at a higher frequency to ensure the proper establishment of vegetation in the practice.

A RSC should be inspected after a large rainstorm, especially during the first six months after establishment. Keep drainage paths, both to and from the BMP, clean so that the water can properly infiltrate into the ground.

Spot fertilization may be required during the first two months to establish vegetation. After that period, however, fertilizers should not be used. While vegetation in the RSC is important, a primary purpose of a RSC is to act as a water quality device, and introducing fertilizers into the RSC introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

The table on the following page shows a schedule for when different maintenance activities should be performed on the RSC area.

**Regenerative Storm Conveyance Typical Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Inspect two times after establishment for first 6 months after storms that exceed ½ inch of rain.<br>• Repair any erosion, rills, or gullies that may form in the practice<br>• Conduct any needed repairs or stabilization<br>• Repair areas with bare or dead grass in the contributing drainage area or around the RSC.<br>• Watering and spot fertilization may be necessary during first 2 months to establish vegetation.<br>• Remove and replace dead, damaged, or diseased plants. | Upon establishment |
| • Prune and weed vegetation.<br>• Remove trash, sediment, and debris. | Four times per year |
| • Add additional plants to maintain needed vegetation density.<br>• Remove and replace any dead, damaged, or diseased plants.<br>• Repair any eroded areas. | As Needed |
| • Make sure weirs, riffles, and pools are in structurally good condition and that the practice has stable water levels.<br>• Prune trees and shrubs (when they are dormant).<br>• Remove any invasive species.<br>• Remove any sediment accumulation in pretreatment area and inflow points. | Annually |
| • Remove accumulated sediment in pools<br>• Repair damage to weirs, riffles, pools, or other structural components. | Once every 2 to 3 years |

| Regenerative Stormwater Conveyance | | | | | |
|---|---|---|---|---|---|
| Inspection Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet Structure** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet structure is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet structure. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| Vegetation within and around practice is maintained per landscaping plan. Grass clippings are removed. | | | | | |
| Native plants were used in the practice according to the planning plan. | | | | | |
| No evidence of use of fertilizer on plants (fertilizer crusting on the surface of the soil, tips of leaves turning brown or yellow, blackened roots, etc.). | | | | | |
| Plants seem to be healthy and in good condition. Comment on condition of plants. | | | | | |
| **Emergency Overflow** | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| **Outlet Structure** | | | | | |
| Outlet structure is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |

| Regenerative Stormwater Conveyance | | | | | |
|---|---|---|---|---|---|
| Inspection Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| Results | | | | | |
| Overall condition of RSC: | | | | | |
| Additional Comments | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please explain why in the appropriate comment box. | | | | | |

## Sand Filters

Sand filters are multi-chamber structures designed to treat stormwater runoff through filtration, using a sediment forebay, a sand bed as its primary filter media, and typically an underdrain system. Sand filters can be designed in many ways; however, there are three primary sand filter system designs, surface sand filter, perimeter sand filter, and underground sand filter. A surface sand filter is a ground-level open air structure that consists of a pretreatment sediment forebay and a filter bed chamber. A perimeter sand filter is an enclosed system typically just below the ground in a vault along the edge of an impervious area such as a parking lot. Finally, an underground sand filter is for areas with limited space and high density areas and should only be considered where local communities allow this practice. Because underground sand filters require additional planning, maintenance and incorporation with the stormwater management plan, coordinate with the local community for specific maintenance concerns. Maintenance frequency on sand filters is typically high due to clogging.

There are some common problems to be aware of when maintaining a sand filter. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the inlet and outlet structure
- Clogging the underdrain
- Mosquitoes breeding in the practice
- Ant mounds

Routine inspection and maintenance should be performed on the sand filters to ensure that the structure is functioning properly. Note that if the sand filter include a grass cover or vegetation, maintenance may be required at a higher frequency during the first year the sand filter is built to ensure the proper establishment of grass cover or vegetation in the practice. For more information on vegetation in a sand filter, see Appendix D: Planting and Soil Guidance.

Inspect the sand filter after a large rainstorm. Keep drainage paths (both to and from the BMP) clean so that the water can properly infiltrate into the ground. If the sand filter is not draining properly, check for clogging at the inflow and outflow structures as well as the infiltration rate of the filter bed. In a sand filter, the filter bed is likely to become clogged at the upper layer of the filter (top 2-3 inches) and will need to be removed and replaced. If the filter becomes clogged or over-compacted, then the media should be replaced. In order to determine if maintenance is necessary, a record should be kept of the dewatering time for a sand filter. Note that sand filters are typically designed to completely drain over 40 hours.

Potential sources of excessive sediment that could clog the media include ant mounds and unstable soil upstream of the practice. Possible sources of compaction are maintenance vehicles traveling through the practice. If the underdrain does not work properly, a structural repair or cleanout to unclog the underdrain may be necessary.

In the event of snow, ensure that the snow does not pile up in the sand filter. Accumulated snow adds additional weight and may compact the sand filter, which would reduce its infiltration capacity. In addition, check to make sure that the materials used to de-ice the surrounding areas stay out of the practice to avoid clogging and further pollution.

If designed and maintained correctly, there is no danger of sand filters becoming a breeding ground for mosquitoes. A mosquito egg requires 24-48 hours to hatch. In addition, it takes 10-14 more days for the larvae to develop and become an adult. By having a sand filter that drains properly, it is unlikely that a sand filter would provide a habitat that could become a breeding area for mosquitoes. Should the sand filter become a breeding ground for mosquitoes, the problem is likely with the sand media or the overflow structure which may need to be addressed. This is for surface sand filters, where there is open water.

The table below shows a schedule for when different maintenance activities should be performed on the sand filter.

**Sand Filter Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Check to see that the filter bed is clean of sediment, and the sediment chamber is not more than 50% full or 6 inches, whichever is less, of sediment. Remove sediment as necessary.<br>• Make sure that there is no evidence of deterioration, spalling or cracking of concrete.<br>• Inspect grates (perimeter sand filter).<br>• Inspect inlets, outlets and overflow spillway to ensure good condition and no evidence of erosion.<br>• Repair or replace any damaged structural parts.<br>• Stabilize any eroded areas.<br>• Ensure that flow is not bypassing the BMP.<br>• Ensure that no noticeable odors are detected outside the practice. | Monthly |
| • Ensure that contributing area, sand filter, inlets and outlets are clear of debris.<br>• Prune and weed to maintain appearance (if applicable).<br>• Ensure that the contributing area is stabilized and mowed, with clippings removed.<br>• Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system.<br>• If permanent water level is present (perimeter sand filter), ensure that the chamber does not leak, and normal pool level is retained. | As needed or 4 times during growing season |

| Activity | Schedule |
|---|---|
| • If filter bed is clogged or partially clogged, manual manipulation of the surface layer of sand may be required. Remove the top few inches of sand, roto-till or otherwise cultivate the surface, and replace media with sand meeting the design specifications.<br>• Replace any filter fabric that has become clogged. | Annually |
| • Remove and replace the top 2-3 inches of sand in the filter. | Every 3-5 years or as needed |

*This page intentionally left blank.*

| Sand Filter | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| Area is free of signs of erosion. | | | | | |
| **Inlet Structure** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet structure is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet structure. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| Diversion structure (high flow bypass structure or other) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Pretreatment (choose one)** | | | | | |
| Forebay – area is free of trash, debris, and sediment. Area is free of undesirable vegetation. | | | | | |
| Sedimentation Chamber – area is free of trash, debris, and sediment. | | | | | |
| Perforated stand-pipe is free of trash, debris, and sediment. Surrounding vegetation is trimmed back so that there is no potential to restrict flow. Pipe is in good working order. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| No evidence of long-term ponding or standing water in the ponding area of the practice (examples: stains, odors, mosquito larvae, etc). | | | | | |

## Sand Filter

| Maintenance Item | Condition | | | | Comment |
|---|---|---|---|---|---|
| | Good | Marginal | Poor | N/A* | |
| Sand Filter seems to be working properly. No settling around the practice. Comment on overall condition of structure. | | | | | |
| Undesirable vegetation within and around practice is trimmed and removed. | | | | | |
| Significant sediment accumulation is not occurring within the filter bed. | | | | | |
| Grass cover is healthy and there are no bare areas or dying grass. | | | | | |
| No evidence of leaks at joints or other components of the practice. | | | | | |
| Underdrain cleanout caps are not missing or damaged. | | | | | |
| Observation well does not have standing water. | | | | | |
| **Emergency Overflow** | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| No evidence of animal activity. | | | | | |
| No evidence of seepage on the downstream face of the structure. | | | | | |
| **Outlet Structure** | | | | | |
| Outlet structure is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| Outlet structure does not appear to be blocked. | | | | | |
| **Results** | | | | | |
| Overall condition of Sand Filter: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |

**Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box.

## Site Reforestation/Revegetation

Site reforestation/revegetation is a process of planting trees, shrubs and other native vegetation in disturbed pervious areas to restore the area to pre-development or better conditions. The process can be used to establish mature native plant communities, such as forests, in pervious areas that have been disturbed by clearing, grading and other land disturbing activities. These plant communities intercept rainfall and slow and filter the stormwater runoff to improve infiltration in the ground. This in turn can reduce the total amount of stormwater runoff and pollutant loads leaving the site. Areas that have been reforested or revegetated should be maintained in an undisturbed, natural state over time. These areas must be designated as conservation areas and protected in perpetuity through a legally enforceable conservation instrument (e.g., conservation easement, deed restriction).

There are some common problems to be aware of when maintaining a site reforestation/revegetation area. They include, but are not limited to, the following:

- Establishing vegetation within the area
- Watering the practice
- Erosion

Routine inspection and maintenance should be performed on the reforestation/revegetation site to ensure that the practice is functioning properly. Note that during the first year this process is implemented, maintenance may be required at a higher frequency to ensure the vegetation is properly established. Once the vegetation is established, very little maintenance is typically needed. For more information on vegetation, see Appendix D: Planting and Soil Guidance.

In order to keep the stormwater runoff that exits the site reforestation/revegetation area clean, fertilizers should only be used sparingly during the establishment of the practice. Once the vegetation in the practice has been established, fertilizers should not be used. While vegetation growth in the site reforestation/revegetation area is important, the primary purpose of this process is to act as both a way to filter and infiltrate stormwater. Introducing fertilizers into the site reforestation/revegetation area introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

The table on the following page shows routine maintenance activities typically associated with site reforestation/revegetation areas.

**Site Reforestation/Revegetation Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Water to promote plant establishment, growth, and survival.<br>• Inspect reforested/revegetated area following rainfall events. Plant replacement vegetation in any eroded areas.<br>• Remove sediment from practice.<br>• Revegetate if eroded. | As needed<br>(Following Construction) |
| • Inspect reforested/revegetated area for erosion. Plant replacement vegetation in any eroded areas.<br>• Inspect reforested/revegetated area for dead or dying vegetation. Plant replacement vegetation as needed.<br>• Prune and care for individual trees and shrubs as needed. | Annually<br>(Semi-Annually During First Year) |

## Site Reforestation/Revegetation

| Maintenance Item | Condition | | | | Comment |
|---|---|---|---|---|---|
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| No evidence of gullies, rills, or excessive erosion. | | | | | |
| Area is free of trash, debris, and sediment. | | | | | |
| No evidence of long-term ponding or standing water (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| Vegetation within and around practice is maintained per landscaping plan. Grass clippings are removed. | | | | | |
| Native plants were used in the practice according to the planting plan. | | | | | |
| No evidence of excessive use of fertilizer on plants (fertilizer crusting on the surface of the soil, tips of leaves turning brown or yellow, blackened roots, etc.). | | | | | |
| Plants seem to be healthy and in good condition. Comment on condition of plants. | | | | | |
| **Results** | | | | | |
| Overall condition of Site Reforestation/Revegetation area: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

*This page intentionally left blank.*

## Soil Restoration

Soil restoration is the process of tilling and adding compost and other amendments to soils to restore them to their pre-development conditions. This improves the soil's ability to reduce post-construction stormwater runoff rates, volumes and pollutant loads. This process is ideal for areas that have been disturbed by clearing, grading and other land disturbing activities. This process is generally used in conjunction with other practices including, but not limited to, vegetated filter strips, grass channels, and simple downspout disconnections.

Restored pervious areas require some maintenance during the first few months following construction, but typically require very little maintenance after that.

In order to keep the water that exits the soil restoration area clean, fertilizers should be used sparingly during the establishment of the practice. Once the vegetation in the practice has been established, fertilizers should not be used. While vegetation growth in the soil restoration area is important, introducing fertilizers into the soil restoration area introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

The table below shows routine maintenance activities typically associated with soil restoration areas.

**Soil Restoration Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Water to promote plant growth and survival.<br>• Inspect restored pervious area following rainfall events. Plant replacement vegetation in any eroded areas. | As Needed<br>(Following Construction) |
| • Inspect restored pervious area for erosion. Plant replacement vegetation in any eroded areas.<br>• Inspect restored pervious area for dead or dying vegetation. Plant replacement vegetation as needed. | Annually<br>(Semi-Annually During First Year) |

*This page intentionally left blank.*

| Soil Restoration | | | | | |
|---|---|---|---|---|---|
| **Maintenance Item** | **Condition** | | | | **Comment** |
| | **Good** | **Marginal** | **Poor** | **N/A*** | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| No evidence of gullies, rills, or excessive erosion. | | | | | |
| No evidence of long-term ponding or standing water (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| Vegetation within and around practice is maintained per landscaping plan. Grass clippings are removed. | | | | | |
| Native plants were used in the practice according to the landscaping plan. | | | | | |
| Plants seem to be healthy and in good condition. Comment on condition of plants. | | | | | |
| **Results** | | | | | |
| Overall condition of Soil Restoration: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please explain why in the appropriate comment box. | | | | | |

*This page intentionally left blank.*

## Stormwater Planters/Tree Boxes

Stormwater planters are similar to bioretention areas in their design purpose to detain, filter, and infiltrate stormwater. In addition stormwater planters utilize native or non-invasive flowers, shrubs, and trees to provide aesthetic qualities to the site. Planters and tree boxes receive stormwater from a variety of sources such as, roof tops and downspouts and runoff from streets.

There are some common problems when maintaining a stormwater planter. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the downspouts (only applicable of stormwater planters receive water from downspouts)
- Establishing vegetation within the stormwater planter
- Clogging the underdrain
- Maintaining the proper pH levels for plants

Routine maintenance should be performed on stormwater planters to ensure that the structure is functioning properly. Note that during the first year the stormwater planter is built, maintenance may be required at a higher frequency to ensure the proper establishment of vegetation in the practice.

In addition to routine maintenance, stormwater planters have seasonal and intermittent maintenance requirements. For example, the following are maintenance activities and concerns specific to winter months. Planting material should be trimmed during the winter, when the plants are dormant. In the event of snow, ensure that snow does not pile up in the stormwater planter. Accumulated snow adds additional weight and may compact the stormwater planter soil, which would reduce its infiltration capacity. In addition, check to make sure that the materials used to de-ice the surrounding areas stay out of the practice to avoid clogging and further pollution.

Stormwater planters should be inspected after a large rainstorm. Mulch the practice as needed to keep a thickness of 2-4 inches. Shredded hardwood mulch is preferred, and care should be taken to keep the mulch from piling on the stems of the plants. For more information on vegetation in stormwater planters, see Appendix D: Planting and Soil Guidance.

In order to keep the water that exits the stormwater planter clean, fertilizers should only be used sparingly during the establishment of the practice. Once the vegetation in the practice has been established, fertilizers should not be used. While vegetation in the stormwater planter is important, the primary purpose of a stormwater planter is to act as a water quality device and introducing fertilizers into the stormwater planter introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. In addition, stormwater planters should already be a nutrient rich environment that does not require fertilization. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

Invasive species should be kept out of the stormwater planters and the overall health of the plants should be maintained. If periodic observations indicate the presence of contaminants, the soil and mulch in the plants should be tested to avoid the build-up of pollutants that may harm the vegetation.

The table below shows a schedule for when different maintenance activities should be performed on a stormwater planter.

**Stormwater Planters/Tree Boxes Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Water to promote plant growth and survival.<br>• Inspect stormwater planter following rainfall events to ensure the planter is working properly. | As Needed<br>(Following Construction) |
| • Prune and weed stormwater planter.<br>• Remove accumulated trash and debris.<br>• Remove and replace dead or damaged plants. Plant replacement vegetation as needed.<br>• Observe infiltration rates after rain events. Planters should have no standing water within 24 hours. | As needed, or 4 times during growing season |
| • Inspect inflow and outflow areas for sediment accumulation. Remove any accumulated sediment or debris.<br>• Inspect stormwater planter for erosion and the formation of rills and gullies. Plant replacement vegetation in any eroded areas.<br>• Replace mulch.<br>• Test the planting soils for pH levels. Consult with a qualified licensed Professional to determine and maintain the proper pH levels. | Semi-annually in spring and fall |
| • Implement plant maintenance plan to trim and divide perennials to prevent overcrowding and stress.<br>• Check soil infiltration rates to ensure the soil is draining the water at a proper rate. Re-aerate or replace soil and mulch layers as needed to achieve infiltration rate of at least 0.25 inches per hour (1-2 in/hr preferred). | 2 to 3 years |

| Stormwater Planter (Tree Box) | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comments |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet Structure** | | | | | |
| Pipes to the practice are free of trash, debris, large branches, etc. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| Level spreader is in good condition with no trash, debris, or sediment accumulation (applicable if planters do not receive rooftop runoff). | | | | | |
| Diversion structure (high flow bypass structure or other) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| No evidence of long-term ponding or standing water in the ponding area of the practice (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| Structure seems to be working properly. No settling around the structure. Comment on overall condition of structure. | | | | | |
| Vegetation within the practice is maintained per landscaping plan. | | | | | |
| Mulching depth of 2-4 inches is maintained. Comment on mulch depth. | | | | | |
| Plants used in the practice are consistent with the requirements in the landscaping plan. | | | | | |
| No evidence of use of fertilizer on plants (fertilizer crusting on the surface of the soil, tips of leaves turning brown or yellow, blackened roots, etc.). | | | | | |

| Stormwater Planter (Tree Box) | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comments |
| | Good | Marginal | Poor | N/A* | |
| Plants seem to be healthy and in good condition. Comment on condition of plants. | | | | | |
| The underdrain has been flushed and there is no indication the underdrain system is clogged. | | | | | |
| Cleanout caps for underdrain is present and in good condition. | | | | | |
| Emergency Overflow | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| Outlet Structure | | | | | |
| Outlet structure is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| Results | | | | | |
| Overall condition of Stormwater Planter: | | | | | |
| Additional Comments | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please explain why in the appropriate comment box. | | | | | |

## Stormwater Ponds

A stormwater pond is a constructed, shallow stormwater retention basin or landscaped area with a permanent pool of water. Stormwater runoff collected in the pool is treated through settling. In addition, the aquatic bench (fringe wetlands), safety bench, side slopes, and shallow areas of the pond include plants to aid in the filtration and infiltration of the stormwater runoff flowing through the practice.

There are some common problems to be aware of when maintaining a stormwater pond. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the inlet and outlet structure
- Establishing vegetation within the stormwater pond
- Pruning and weeding to maintain appearance
- Eutrophic conditions indicated by excessive algae growth or fish kills
- Creating a mosquito habitat

Routine inspection and maintenance should be performed on stormwater ponds to ensure that the structure is functioning properly.  Note that during the first year the stormwater pond is built, maintenance may be required at a higher frequency to ensure the proper establishment of vegetation in the practice. For more information on vegetation in stormwater ponds, see Appendix D: Planting and Soil Guidance.

In addition to routine maintenance, stormwater ponds have seasonal and intermittent maintenance requirements. During the winter months, the stormwater pond should be inspected after a snow event (this is specific to northern areas of Georgia) to make sure that the materials used to de-ice the surrounding areas stay out of the practice to avoid further pollution. In addition, planting material should be trimmed during the winter, when the plants are dormant.

Inspect the stormwater pond after a large rainstorm. Keep drainage paths (both to and from the BMP) clean so that the water can properly flow into the stormwater pond. If the stormwater pond is not draining properly, check for clogging in the inflow and outflow structures.

If the forebay or stormwater pond has received a significant amount of sediment over a period of time, then the sediment at the bottom of the forebay or pond may need to be removed. Accumulated sediment in the practice decreases the available storage volume and affects the pond's ability to function as it was designed. A sediment marker should be placed in the forebay to determine when sediment removal is required. It important to note that sediment excavated from stormwater ponds that does not receive stormwater runoff from stormwater hotspots are typically not considered to be toxic and can be safely disposed through either land application or landfilling. Stormwater hotspots are areas that produce higher concentrations of metals, hydrocarbons, or other pollutants than normally found in urban runoff. Examples of operations performed in potential stormwater hotspots include vehicle maintenance and repair, vehicle washing, landscaping/grounds care, and outdoor material and product storage. Check with the local development review authority to identify any additional constraints on the disposal of sediments excavated from stormwater ponds.

Periodic mowing of the pond buffer is only required along maintenance right-of-way and the embankment. The remaining buffer can be managed as a meadow (mowing every other year) or a forest.

In order to keep the water that exits the stormwater pond clean, fertilizers should be used sparingly during establishment. Once the vegetation in the practice has been established, fertlizers should not be used. While vegetation in the stormwater pond is important, the primary purpose of a stormwater pond is to act as a water quantity and quality device, and introducing fertilizers into the stormwater pond introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. In addition, stormwater ponds should already be nutrient rich environments that do not require fertilization. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

Stormwater ponds create a challenge for controlling mosquitos, because some types of vegetation, such as cattails, can create an environment that allows mosquitoes to breed both in the pond and along the shoreline. Keeping the practice free of trash will help the practice from becoming a mosquito habitat. Another method to control mosquitoes is to place fish, such as the mosquitofish (Gambusia affinis), in the pond to help with controlling the mosquitoes. Animals such as dragonflies, diving beetles, birds, and bats may aid on controlling mosquitoes, however it is likely that additional measures, such as chemicals, may be required to control the mosquitoes (using chemicals should be a last resort). Keeping the pond at a depth of four feet or greater can aid in mosquito control by limiting vegetation growing around the pond. If mosquitoes begin to pose a problem, consult a qualified professional.

Pond dam inspection and maintenance is also very important. The pond dam should be inspected for seepage and structural integrity. Look for saturated soil, sediment deposits, and flowing water at the base of an earthen dam and on the rear face of the dam. On concrete dams, look for seepage, cracks, leaks and rust stains, or bulges. If any signs of seepage are found, consult a Professional Engineer. Pests such as burrowing animals and fire ants can pose a major threat to dam safety. Fire ant tunnels and animal burrows can weaken the dam structure and create an undesired water pathway through the dam. In addition, tree roots are another source of potential damage and failure. Woody vegetation may not be planted on the embankment or allowed to grow within 15 feet of the toe of the embankment and 25 feet from the principal spillway structure. If you have a large dam that is subject to regulations by the state, other maintenance items may be required. Please consult a Professional Engineer for additional guidance.

Ponds can be an attractive nuisance, so security and safety should be taken into consideration. Fencing requirements are at the discretion of the local government. If security measures such as a fence and gate are present, ensure that they are functional and locked.

It is important that the embankment for a pond be inspected regularly for trees and animal activity. Trees growing on the top or sides of the embankment should be removed. The roots of trees grow into the embankment and will weaken the structure of the embankment by creating passage ways that allow water to flow through the embankment. Trees that are blown over or damaged by storms can loosen or remove soil which weakens the strength of the embankment. In the same way animals can burrow holes weakening the structure of the embankment. These holes act as a passage way for the water to travel through the embankment, increasing the potential for the embankment to fail.

Geese are attracted to open water, clean lines of sight, and grass. They can become a nuisance to stormwater ponds if they are causing damage to plants or the banks, or if they are 'loading' the pond with nutrients and bacteria. Geese can be discouraged from using a stormwater pond by planting the buffer with shrubs and native ground covers or installing an aquatic shelf, but ensure that access points are maintained.

The table below shows a schedule for when different maintenance activities should be performed on a stormwater pond.

**Stormwater Ponds Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Inspect inlets, outlets and overflow spillway to ensure good condition and no evidence of erosion.<br>• Clean and remove debris from inlet and outlet structures.<br>• Mow side slopes.<br>• Inspect pond dam for structural integrity.<br>• Remove trash from the area around the pond. | Monthly |
| • If wetland components are included, inspect for invasive vegetation. | Semiannual Inspection |
| • Inspect for damage, paying particular attention to the control structure.<br>• Check for signs of eutrophic conditions (e.g., algal blooms and fish kills).<br>• Note signs of hydrocarbon build-up (e.g., an oil sheen), and remove appropriately.<br>• Monitor for sediment accumulation in the facility and forebay.<br>• Check all control gates, valves, or other mechanical devices. | Annual Inspection |
| • Repair undercut or eroded areas. | As Needed |
| • Perform wetland plant management and harvesting. | Annually (if needed) |
| • Remove sediment from the forebay. | 5 to 7 years or after 50% of the total forebay capacity has been lost |

| Activity | Schedule |
| --- | --- |
| • Monitor sediment accumulations, and remove sediment when the pool volume has become reduced significantly, or the pond becomes eutrophic. | 10 to 20 years or after 25% of the permanent pool volume has been lost |

(Source: WMI, 1997)

## Stormwater Pond

| Maintenance Item | Condition | | | | Comment |
|---|---|---|---|---|---|
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet Structure** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet structure is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet structure. | | | | | |
| Inlet pipe is in good condition, and water is going through the structure (i.e. no evidence of water going around the structure). | | | | | |
| Diversion structure (high flow bypass structure or other) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Pretreatment (choose one)** | | | | | |
| Forebay – area is free of trash, debris, and sediment. | | | | | |
| Filter Strip or Grass Channels – area is free of trash debris and sediment. Area has been mowed and grass clippings are removed. No evidence of erosion. | | | | | |
| Rock Lined Plunge Pools – area is free of trash debris and sediment. Rock thickness in pool is adequate. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| No algal growth along or within the pond. | | | | | |
| Native plants were used in the practice according to the planting plan. No undesirable vegetation. | | | | | |
| Practice seems to be working properly. No settling around the stormwater pond. | | | | | |

## Stormwater Pond

| Maintenance Item | Condition | | | | Comment |
|---|---|---|---|---|---|
| | Good | Marginal | Poor | N/A* | |
| Comment on overall condition of stormwater pond. | | | | | |
| Vegetation within and around practice is maintained per landscaping plan. Grass clippings are removed. | | | | | |
| No significant sediment accumulation within the practice. | | | | | |
| No evidence of use of fertilizer on plants (fertilizer crusting on the surface of the soil, tips of leaves turning brown or yellow, blackened roots, etc.). | | | | | |
| Plants seem to be healthy and in good condition. Comment on condition of plants. | | | | | |
| **Emergency Overflow** | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, flooding, or animal activity around the structure. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| **Outlet Structure** | | | | | |
| Outlet structure is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| Outlet structure does not appear to be blocked. | | | | | |
| No evidence of animal activity. | | | | | |
| No evidence of seepage on the downstream face. | | | | | |
| **Results** | | | | | |
| Overall condition of Stormwater Pond: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

## Stormwater Wetland

Stormwater wetlands are constructed wetland systems built for stormwater management purposes. They typically consist of a combination of open water, shallow marsh and semi-wet areas that are located just above the permanent water surface. As stormwater runoff flows through a wetland, it is treated, primarily through gravitational settling and biological uptake.

There are some common problems to be aware of when maintaining a stormwater wetland. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the inlet and outlet structure
- Establishing vegetation within the wetland area
- Maintaining the proper pH levels for plants
- Pruning and weeding to maintain appearance
- Mosquitoes breeding in the practice

Routine maintenance should be performed on the stormwater wetlands to ensure that the structure is properly functioning. Note that during the first year the stormwater wetland is built, maintenance may be required at a higher frequency to ensure the proper establishment of vegetation in the practice. For more information on stormwater wetland vegetation, see Appendix D: Planting and Soil Guidance. Regular inspection and maintenance is crucial to the success of the wetland as an effective stormwater management practice.

In addition to routine maintenance, stormwater wetlands have seasonal and intermittent maintenance requirements. During the winter months, the stormwater pond should be inspected after a snow event (this is specific to northern areas of Georgia) to make sure that the materials used to de-ice the surrounding areas stay out of the practice to avoid further pollution. In addition, planting material should be trimmed during the winter, when the plants are dormant.

Inspect the stormwater wetland after large rainstorm events. Keep drainage paths (both to and from the BMP) clean so that the water can properly flow into the stormwater wetland. If the stormwater wetland is not draining properly, check for clogging in the inflow and outflow structures.

If the forebay or stormwater wetland has received a significant amount of sediment over a period of time, then the sediment at the bottom of the forebay or wetland may need to be removed. Accumulated sediment in the practice decreases the available storage volume and affects the wetland's ability to function as it was designed. It important to note that sediment excavated from stormwater wetlands that do not receive stormwater runoff from stormwater hotspots are typically not considered to be toxic and can be safely disposed through either land application or landfilling. Stormwater hotspots are areas that produce higher concentrations of metals, hydrocarbons, or other pollutants than normally found in urban runoff. Examples of operations performed in potential stormwater hotspots include vehicle maintenance and repair, vehicle washing, landscaping/grounds care, and outdoor material and product storage. Check with the local development review authority to identify any additional constraints on the disposal of sediments excavated from stormwater wetlands.

In order to keep the water that exits the stormwater wetland clean, fertilizers should be used sparingly around the wetland. Once the vegetation in the practice has been established, fertilizers should not be used. While vegetation in the stormwater wetland is important, the primary purpose of a stormwater wetland is to act as a water quantity and quality device and introducing fertilizers into the stormwater wetland introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. In addition, stormwater wetlands should already be a nutrient rich environment that does not require fertilization. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

It is important that the embankment for a wetland be inspected regularly for trees and animal activity. Trees growing on the top or sides of the embankment should be removed. The roots of trees grow into the embankment and will weaken the structure of the embankment by creating passage ways that allow water to flow through the embankment. Trees that are blown over or damaged by storms can loosen or remove soil which weakens the strength of the embankment. In the same way animals can burrow holes weakening the structure of the embankment. These holes act as a passage way for the water to travel through the embankment, increasing the potential for the embankment to fail.

Stormwater wetlands create a challenge for controlling mosquitos, because some types of vegetation, such as cattails, can create an environment that allows mosquitoes to breed both in the pond and along the shoreline. Keeping the practice free of trash will help the practice from becoming a mosquito habitat. Another method to control mosquitoes is to place fish, such as the mosquitofish (Gambusia affinis), in the wetland to help with controlling the mosquitoes. Animals such as dragonflies, diving beetles, birds, and bats may aid on controlling mosquitoes, however it is likely that additional measures, such as chemicals, may be required to control the mosquitoes (using chemicals should be a last resort). Keeping the wetland at a depth of four feet or greater can aid in controlling mosquitoes by limiting vegetation growing around the wetland. If mosquitoes begin to pose a problem, consult a qualified professional.

The table below shows a schedule for when different maintenance activities should be performed on a stormwater wetland.

**Stormwater Wetland Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Water side slopes and buffers to promote plant growth and survival.<br>• Inspect wetland, side slopes and buffers following major storm events. Plant replacement vegetation in any eroded areas. | As Needed<br>(Following Construction) |

| Activity | Schedule |
|---|---|
| • Examine to ensure that inlet and outlet devices are free of sediment and debris and are operational.<br>• Inspect wetland, side slopes and buffers for dead or dying vegetation. Plant replacement vegetation as needed.<br>• Inspect wetland, side slopes and buffers for invasive vegetation and remove as needed. | Monthly |
| • Inspect wetland, side slopes and buffers for erosion. Plant replacement vegetation in any eroded areas.<br>• Monitor wetland vegetation and perform replacement planting as necessary.<br>• Harvest wetland plants that have been "choked out" by sediment build-up. | Semi-Annually<br>(Quarterly During First Year) |
| • Inspect for damage, paying particular attention to the control structure and side slopes. Repair as necessary.<br>• Examine stability of the original depth zones and microtopographical features (i.e., shallow areas with minor ridges that increase water quality, provide flood storage, and enhance the development of a more diverse vegetative community).<br>• Inspect side slopes for erosion and undercutting and repair as needed.<br>• Check for signs of eutrophic conditions (e.g., excessive algal growth).<br>• Check for signs of hydrocarbon accumulation (e.g., oil sheens) and remove appropriately.<br>• Monitor sediment markers for sediment accumulation in forebays and permanent pools.<br>• Check all control gates, valves and other mechanical devices. | Annually |
| • Remove sediment, trash, and debris from inlets/forebay. | 5 years or after 50% of the total forebay storage capacity has been lost |
| • Monitor sediment accumulation in the wetland and remove sediment when the permanent pool volume has become reduced significantly, plants are "choked" with sediment, or the wetland becomes eutrophic. | 10 plus years or after 25% of the wetland storage volume has been lost |

*This page intentionally left blank.*

| Stormwater Wetland | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet Structure** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet structure is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet structure. | | | | | |
| Inlet pipe is in good condition, and water is going through the structure (i.e. no evidence of water going around the structure). | | | | | |
| Diversion structure (high flow bypass structure or other) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Pretreatment (choose one)** | | | | | |
| Forebay – area is free of trash, debris, and sediment. Sediment accumulation in forebay is less than 50% of the storage capacity. | | | | | |
| Filter Strip or Grass Channels – area is free of trash debris and sediment. Area has been mowed and grass clippings are removed. No evidence of erosion. | | | | | |
| Rock Lined Plunge Pools – area is free of trash debris and sediment. Rock thickness in pool is adequate. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| No algal growth along or within the wetland. | | | | | |
| Native plants were used in the practice according to the planting plan. No undesirable vegetation. | | | | | |

| Stormwater Wetland | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| Vegetation within and around practice is maintained per landscaping plan. Grass clippings are removed. | | | | | |
| Wetland seems to be working properly. No settling around the practice. Comment on overall condition. | | | | | |
| No significant sediment accumulation within the practice. | | | | | |
| No evidence of use of fertilizer on plants (fertilizer crusting on the surface of the soil, tips of leaves turning brown or yellow, blackened roots, etc.). | | | | | |
| Plants seem to be healthy and in good condition. Comment on condition of plants. | | | | | |
| **Emergency Overflow** | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, flooding, or animal activity around the structure. No evidence of seepage on the downstream face. | | | | | |
| No evidence of unwanted vegetation and vegetation is in good condition. | | | | | |
| **Outlet Structure** | | | | | |
| Outlet structure is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| Outlet structure does not appear to be blocked. | | | | | |
| **Results** | | | | | |
| Overall condition of Stormwater Wetland: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

## Submerged Gravel Wetlands

The submerged gravel wetland system is similar to a regular stormwater wetland; however, it is filled with crushed rock or gravel and designed to allow stormwater to flow through the root zone of the constructed wetland. The outlet from each cell is set at an elevation to keep the rock or gravel submerged. Wetland plants are rooted in the media, where they can directly take up pollutants. In addition, algae and microbes thrive on the surface area of the rocks. Mimicking the pollutant removal ability of nature, this structural control relies on the pollutant-stripping ability of plants and soils to remove pollutants from runoff.

There are some common problems to be aware of when maintaining a submerged gravel wetland. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the inlet and outlet structure
- Establishing vegetation within the wetland area
- Maintaining the proper pH levels for plants
- Pruning and weeding to maintain appearance
- Mosquitoes breeding in the practice

Routine maintenance should be performed on the submerged gravel wetlands to ensure that the structure is properly functioning. Note that during the first year the submerged gravel wetland is built, maintenance may be required at a higher frequency to ensure the proper establishment of vegetation in the practice. For more information on vegetation in submerged gravel wetlands, see Appendix D: Planting and Soil Guidance. Regular inspection and maintenance is crucial to the success of the wetland as an effective stormwater management practice.

In addition to routine maintenance, submerged gravel wetlands have seasonal and intermittent maintenance requirements. During the winter months, the stormwater pond should be inspected after a snow event (this is specific to northern areas of Georgia) to make sure that the materials used to de-ice the surrounding areas stay out of the practice to avoid further pollution. In addition, planting material should be trimmed during the winter, when the plants are dormant.

Inspect the submerged gravel wetland after large rainstorm events. Keep drainage paths (both to and from the BMP) clean so that the water can properly flow into the submerged gravel wetland. If the submerged gravel wetland is not draining properly, check for clogging in the inflow and outflow structures. If sediment buildup is preventing flow through the wetland, remove gravel and sediment from cell. Replace with clean gravel and replant vegetation.

If the forebay or submerged gravel wetland has received a significant amount of sediment over a period of time, then the sediment at the bottom of the forebay or gravel wetland may need to be removed. It is important to note that sediment excavated from submerged gravel wetlands that do not receive stormwater runoff from stormwater hotspots are typically not considered toxic and can be safely disposed through either land application or landfilling. Stormwater hotspots are areas that produce higher concentrations of metals, hydrocarbons, or other pollutants than normally found in urban runoff. Examples of operations performed in potential stormwater hotspots include vehicle maintenance and repair, vehicle washing, landscaping/grounds care, and outdoor material and product storage. Check with the local development review authority to identify any additional constraints on the disposal of sediments excavated from submerged gravel wetlands.

In order to keep the water that exits the submerged gravel wetland clean, fertilizers should be used sparingly during the establishment of the practice. Once the vegetation in the practice has been established, fertilizer should not be used. While vegetation in the submerged gravel wetland is important, the primary purpose of a submerged gravel wetland is to act as a water quantity and quality device and introducing fertilizers into the submerged gravel wetland introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. In addition, submerged gravel wetlands should already be a nutrient rich environment that does not require fertilization. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

It is important that the embankment of a submerged gravel wetland be inspected regularly for trees and animal activity. Trees growing on the top or sides of the embankment should be removed. The roots of trees grow into the embankment and will weaken the structure of the embankment by creating passage ways that allow water to flow through the embankment. Trees that are blown over or damaged by storms can loosen or remove soil which weakens the strength of the embankment. In the same way animals can burrow holes weakening the structure of the embankment. These holes act as a passage way for the water to travel through the embankment, increasing the potential for the embankment to fail.

The table below shows a schedule for when different maintenance activities should be performed on a submerged gravel wetland.

**Submerged Gravel Wetlands Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Ensure that inlets and outlets to each submerged gravel wetland cell are free from debris and not clogged.<br>• Remove any accumulated sediment and debris from inlet and outlet structures. | Monthly |

| Activity | Schedule |
|---|---|
| • Inspect wetland, side slopes and buffers for erosion. Replace vegetation in eroded areas.<br>• Inspect wetland, side slopes and buffers for dead or dying vegetation. Replace vegetation as needed.<br>• Inspect wetland, side slopes and buffers for invasive vegetation and remove as needed. | Semi-Annually (Quarterly During First Year) |
| • Inspect for damage to the embankment and inlet/outlet structures. Repair as necessary.<br>• Monitor for sediment accumulation in the facility.<br>• Examine to ensure that inlet and outlet devices are free of sediment and debris and operational.<br>• Inspect side slopes for erosion and undercutting and repair as needed.<br>• Check for signs of eutrophic conditions (e.g., excessive algal growth).<br>• Check for signs of hydrocarbon accumulation and remove appropriately.<br>• Monitor sediment markers for sediment accumulation in forebays and permanent pools.<br>• Check all control gates, valves and other mechanical devices. | Annually |
| • Water side slopes and buffers to promote plant growth and survival.<br>• Inspect wetland, side slopes, strucutres, and buffers following rainfall events. Plant replacement vegetation in any eroded areas. | As Needed |
| • Remove sediment, trash, and debris from inlets/forebay. | 5 years or after 50% of the total forebay storage capacity has been lost |
| • Monitor sediment accumulation in the wetland and remove sediment when the permanent pool volume has become reduced significantly, plants are “choked” with sediment, sediment buildup is preventing flow through the wetland, or the wetland becomes eutrophic. Replace with clean gravel and replant vegetation. | 10 plus years or after 25% of the wetland storage volume has been lost |

*This page intentionally left blank.*

| Submerged Gravel Wetlands | | | | | |
|---|---|---|---|---|---|
| Inspection Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet Structure** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet structure is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet structure. | | | | | |
| Water is going through structure (i.e. no evidence of water going around the structure). | | | | | |
| Diversion structure (high flow bypass structure or other) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Pretreatment (choose one)** | | | | | |
| Forebay – area is free of trash, debris, and sediment. | | | | | |
| Filter Strip or Grass Channels – area is free of trash debris and sediment. Area has been mowed and grass clippings are removed. No evidence of erosion. | | | | | |
| Rock Lined Plunge Pools – area is free of trash debris and sediment. Rock thickness in pool is adequate. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Erosion protection is present on site (i.e. turf reinforcement mats). Comment on types of erosion protection and evaluate condition. | | | | | |
| No algal growth along or within the wetland. | | | | | |
| Native plants were used in the practice according to the landscaping plan. No undesirable vegetation. | | | | | |

| Submerged Gravel Wetlands | | | | | |
|---|---|---|---|---|---|
| Inspection Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| Vegetation within and around practice is maintained per landscaping plan. Grass clippings are removed. | | | | | |
| Wetland seems to be working properly. No settling around the practice. Comment on overall condition. | | | | | |
| No significant sediment accumulation within the practice. | | | | | |
| No evidence of use of fertilizer on plants (fertilizer crusting on the surface of the soil, tips of leaves turning brown or yellow, blackened roots, etc.). | | | | | |
| Plants seem to be healthy and in good condition. Comment on condition of plants. | | | | | |
| **Emergency Overflow** | | | | | |
| Emergency overflow is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| **Outlet Structure** | | | | | |
| Outlet structure is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding around the structure. | | | | | |
| Outlet structure does not appear to be blocked. | | | | | |
| **Results** | | | | | |
| Overall condition of Submerged Gravel Wetland: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: * If a specific maintenance item was not checked, please check N/A and explain why in the appropriate comment box. | | | | | |

## Underground Detention

Underground detention is detention storage located in underground tanks or vaults designed to provide water quantity control through temporary storage of stormwater runoff. In addition they can improve water quality by removing heavy amounts of sediment.

There are some common problems to be aware of when maintaining an underground detention area. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the inlet and outlet structure
- Requirement to have Occupational Safety and Health Administration (OSHA) confined space entry training

Routine maintenance should be performed on the underground detention areas to ensure that the structure is properly functioning. Routine maintenance includes the removal of debris from inlet and outlet structures and cleaning sediment built up inside the structure. Because this is an underground system, inspection and maintenance may be difficult to conduct. Generally these underground systems can be inspected by looking in an access opening. Sometimes, however, maintenance requires an individual who is certified in OSHA confined space entry. Should there be a situation where a safety concern arises, the inspection should stop and the safety concern addressed. Once the concern is addressed, the inspection can continue.

Inspect the underground detention area after a large rainstorm. If the underground detention area is not draining properly, check the inlet and outlet structures to make sure they are not clogged.

Sediment should be removed from the practice by either a vacuum or boom. If the system is accepting water that flowed from a hazardous facility, the sediment may need to be disposed of by other means. Check with the local government to identify any additional constraints on the disposal of sediments excavated from underground detention.

The table on the following page shows a schedule for when different maintenance activities should be performed on a submerged gravel wetland.

**Underground Detention Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Remove any trash/debris and sediment buildup in the underground trash racks, vaults or tanks.<br>• Check drainage areas for trash, erosion, and debris.<br>• Clean underground detention if hazardous or foreign substances are spilled in the contributing drainage area.<br>• Perform structural repairs to inlet and outlets. | As needed |
| • Follow manufacturer's guidelines and develop/adjust plan for the underground detention.<br>• Clean out underground detentions with vacuum or boom trucks.<br>• Clean sediment or oil chambers | Annually |

| Underground Detention | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet Structure and Pretreatment** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Inlet structure is in good condition. No signs of cracks or leaks. | | | | | |
| Diversion structure (high flow bypass structure or other) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| Inlet pipe fits tightly to the underground detention. | | | | | |
| Inlet has protection to prevent clogging with leaves or other debris and has fine mesh for mosquito control. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| Structure seems to be working properly. No signs of settling, leaking, or cracking. Comment on overall condition of structure. | | | | | |
| **Emergency Overflow and Outlet Structure** | | | | | |
| Area is free of trash, debris, and sediment. | | | | | |
| Overflow valve appears to be in good condition and show no signs of leaking. | | | | | |
| **Results** | | | | | |
| Overall condition of Underground Detention: | | | | | |
| **Additional Comments** | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please explain why in the appropriate comment box. | | | | | |

*This page intentionally left blank.*

## Vegetated Filter Strips

Vegetated filter strips are uniformly graded and densely vegetated sections of land, designed to treat runoff and remove pollutants through vegetative filtering and infiltration. Vegetated filter strips are best suited to treating runoff from roads and highways, roof downspouts, very small parking lots, and pervious surfaces. These filter strips may be constructed with turf, meadow grasses, or other dense vegetation. They are also ideal components of the "outer zone" of a stream buffer, or as pretreatment for another structural stormwater control. Filter strips can serve as a buffer between incompatible land uses, be landscaped to be aesthetically pleasing, and provide groundwater recharge in areas with pervious soils.

There are some common problems to be aware of when maintaining a vegetated filter strip. They include, but are not limited to, the following:

- Sediment build-up
- Clogging in the pea gravel diaphragm or other flow spreader
- Establishing vegetation within the vegetated filter strip
- Ant mounds
- Erosion
- Concentrated flow

Routine maintenance should be performed on the vegetated filter strips to ensure that the practice is functioning properly. Note that during the first year the vegetated filter strip is built, maintenance may be required at a higher frequency to ensure the proper establishment of grass and vegetation in the practice. Upon establishment, grass should be routinely cut and vegetation trimmed, as necessary, to maintain a grass height of 3-12 inches or 6-15 inches along a roadway. Other routine maintenance includes removing trash from the vegetated filter strip and ensuring that grass clippings and other debris are removed from the filter strip.

Vegetated filter strips should be inspected after a large rainstorm. Keep drainage paths, both to and from the BMP, clean to promote sheet flow so that the water can be filtered by the BMP.

If the vegetated filter strip is not draining properly, check for clogging in the inlet and outlet structures. Also, consider if the filter strip has a sufficient slope or if there are obstructions within the filter strip that may cause inhibit the flow of water. If the practice includes a permeable berm, a structural repair or cleanout to unclog the outlet pipe may be necessary.

In order to keep the water that exits the vegetated filter strip clean, fertilizers should be used sparingly during the establishment of the practice. Once the vegetation in the practice has been established, fertilizers should not be used. While vegetation in the vegetated filter strip is important, a primary purpose of a vegetated filter strip is to act as a water quality device and introducing fertilizers into the vegetated filter strip introduces nutrients such as phosphorus and nitrogen that can pollute downstream waters. To control animal nuisances and invasive species, pesticides (including herbicides, fungicides, insecticides, or nematode control agents) should be used sparingly and only if necessary.

The table below shows a schedule for when different maintenance activities should be performed on a vegetated filter strip.

**Vegetated Filter Strips Typical Routine Maintenance Activities and Schedule**

| Activity | Schedule |
|---|---|
| • Mow grass to a height to maintain a dense vegetative cover. It is recommended that the height of grass is 3-12 inches and 6-15 inches along a roadway. Remove any grass clippings<br>• Keep the practice clean and remove all trash, sediment, and debris.<br>• Reseed any eroded or bare spots.<br>• Water the practice during dry conditions of vegetation establishment. | As needed |
| • Inspect vegetated filter strip for signs of erosion, and repair the strip as needed.<br>• Inspect for invasive species and remove as needed.<br>• Inspect pea gravel diaphragm for clogging and remove built-up sediment.<br>• Inspect vegetation for rills and gullies. Seed or sod bare areas.<br>• Inspect to ensure that grass has established. If not, replace with an alternative species. | Annual Inspection |

## Vegetated Filter Strip

| Maintenance Item | Condition | | | | Comment |
|---|---|---|---|---|---|
| | Good | Marginal | Poor | N/A* | |
| **General Inspection** | | | | | |
| Access to the site is adequately maintained for inspection and maintenance. | | | | | |
| Area is clean (trash, debris, grass clippings, etc. removed). | | | | | |
| **Inlet** | | | | | |
| Drainage ways (overland flow or pipes) to the practice are free of trash, debris, large branches, etc. | | | | | |
| Area around the inlet is mowed and grass clippings are removed. | | | | | |
| No evidence of gullies, rills, or excessive erosion around the inlet. | | | | | |
| Water is going through the filter (i.e. no evidence of water going around the filter). | | | | | |
| Diversion structure (high flow bypass channel or overflow spillway) is free of trash, debris, or sediment. Comment on overall condition of diversion structure and list type. | | | | | |
| **Pretreatment (choose one)** | | | | | |
| Area is free of trash, debris, and sediment. | | | | | |
| No signs of erosion, rills, or gullies. | | | | | |
| Pea gravel diaphragm or other level or flow spreader – No cracks or structural damage in concrete trough. | | | | | |
| **Main Treatment** | | | | | |
| Main treatment area is free of trash, debris, and sediment. | | | | | |
| No signs of erosion, rills, or gullies. | | | | | |
| No evidence of long-term ponding or standing water in the ponding area of the practice (examples include: stains, odors, mosquito larvae, etc). | | | | | |
| Practice seems to be working properly. | | | | | |
| No areas of unhealthy grass or bare areas. | | | | | |
| No unwanted or invasive vegetation. | | | | | |
| No evidence of use of fertilizer on plants (fertilizer crusting on the surface of the soil, tips of leaves turning brown or yellow, blackened roots, etc.). | | | | | |

| Vegetated Filter Strip | | | | | |
|---|---|---|---|---|---|
| Maintenance Item | Condition | | | | Comment |
| | Good | Marginal | Poor | N/A* | |
| Grass is kept at the proper mowing height, 3-12 inches and 6-15 inches along the roadway. Grass clippings are removed. | | | | | |
| No signs of accumulated sediment. | | | | | |
| Outlet Structure | | | | | |
| Outlet is free of trash, debris, and sediment. | | | | | |
| No evidence of erosion, scour, or flooding. | | | | | |
| Results | | | | | |
| Overall condition of Vegetated Filter Strip: | | | | | |
| Additional Comments | | | | | |
| | | | | | |
| **Notes**: *If a specific maintenance item was not checked, please explain why in the appropriate comment box. | | | | | |