Georgia Stormwater Management Manual

Volume 2

Technical Handbook

First Edition
August 2001

GEORGIA STORMWATER MANAGEMENT MANUAL

Volume 2:
Technical Handbook

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Atlanta Regional Commission
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Preface

Stormwater management has entered a new phase in the state of Georgia. The requirements for NPDES municipal and industrial permits, TMDLs, watershed assessments and the desire to protect human life, property, aquatic habitats and the quality of life in our communities has brought home the pressing need to manage both stormwater quantity and quality from our developed and developing areas.

This Manual will help Georgia move forward with a comprehensive approach to stormwater management that integrates drainage design, stormwater quantity, and water quality considerations and views stormwater as important resource and opportunity for our communities. The goal of this Manual is to develop and promote a consistent and effective approach and implementation of stormwater management in the state.

Acknowledgements

This Manual is the culmination of a collaborative effort between the Atlanta Regional Commission (ARC), the Georgia Department of Natural Resources-Environmental Protection Division, and 35 cities and counties from across Georgia. These documents reflect the hard work, time and contributions of many individuals.

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Cherokee County
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INTRODUCTION

Objective of the Manual

The objective of the Georgia Stormwater Management Manual is to provide guidance on addressing stormwater runoff. 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 *Manual for Erosion and Sediment Control in Georgia*.

Organization of the Manual

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

Volume One of the Manual, the *Stormwater Policy Guidebook*, is designed to provide guidance for local jurisdictions on the basic principles of effective urban stormwater management. Volume 1 covers the problem of urban stormwater runoff and the need for local communities to address urban stormwater quantity and quality, stormwater management minimum standards, and guidance on local stormwater programs. It also provides an overview of integrated stormwater management, site and watershed level stormwater management, floodplain management, and technology and tools for implementing stormwater management programs.

Volume Two of the Manual, the *Technical Handbook*, provides guidance on the techniques and measures that can be implemented to meet a set of stormwater management minimum 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 required to effectively address and control both water quality and quantity on a development site. This includes guidance on better site design practices, hydrologic techniques, criteria for the selection and design of structural stormwater controls, drainage system design, and construction and maintenance information.

Volume Three, 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.

Users of This Volume

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 minimum 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 advise, and may adopt any part of the guidance and design criteria for structural stormwater controls 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 structural stormwater controls, and drainage (hydraulic) design.

How to Use This Volume

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

- **Chapter 1 (Stormwater Management Planning and Design).** This chapter provides the framework for addressing stormwater runoff on site of new development and redevelopment. This chapter includes the following sections:
  - Section 1.1 – *The Need for Stormwater Management.* This section provides an overview of the impacts of urban stormwater runoff.
  - Section 1.2 – *Stormwater Management Standards*. This section contains the stormwater management minimum standards for new development and redevelopment sites.
  - Section 1.3 – *Unified Stormwater Sizing Criteria*. This section 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 site design practices and structural stormwater controls.
  - Section 1.4 – *Stormwater Better Site Design Practices*. 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 1.5 – *Stormwater Site Planning*. This section outlines the typical contents and procedures for preparing a stormwater site plan.

- **Chapter 2 (Stormwater Hydrology).** This chapter presents engineering topics and methods used in stormwater drainage, conveyance and facility design.
  - Section 2.1 – *Methods for Estimating Stormwater Runoff*. This section provides an overview of the different hydrologic methods and their application.
  - Section 2.2 – *Storage Design*. This section covers the criteria and general procedures for the design and evaluation of stormwater storage (detention and retention) facilities.
  - Section 2.3 – *Outlet Structures*. This section outlines various stormwater facility outlet types and provides criteria and procedures for water quality outlet design.

- **Chapter 3 (Structural Stormwater Controls).** This chapter contains the information and guidance for the selection and design of structural stormwater controls for managing stormwater quantity and quality. It is divided into the following sections:
  - Section 3.1 – *Structural Stormwater Controls Overview.* This section provides an overview of the structural stormwater controls that can be used to treat stormwater runoff and/or mitigate the effects of increased runoff peak rates, volumes, and velocities.
  - Section 3.2 – *General Application Structural Stormwater Controls*. This section contains detailed information and design criteria for structural controls recommended for general use on most sites with a demonstrated ability to meet stormwater management goals.
  - Section 3.3 – *Limited Application Structural Stormwater Controls*. This section contains detailed information and design criteria for structural controls recommended for limited use or for special site or design conditions.
  - Section 3.4 – *Detention Structural Stormwater Controls*. This section contains detailed information and design criteria for detention (water quantity only) structural controls.

- **Chapter 4 (Stormwater Drainage System Design)**. This chapter provides technical guidance on the various elements of stormwater drainage design. This chapter includes the following sections:
  - Section 4.1 – *Stormwater Drainage Design Overview*
  - Section 4.2 – *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 4.3 – *Culvert Design*. This section covers criteria and procedures for the design and evaluation of culverts.
  - Section 4.4 – *Open Channel Design*. This section describes the criteria and calculations for the design of open stormwater drainage channels.
  - Section 4.5 – *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 provides a series of tables for sixteen locations across Georgia providing rainfall intensity information for various stormwater durations and frequencies.
- **Appendix B** – *Soils Information for Georgia*. This appendix contains soils information for Georgia soils, including hydrologic soils group classification and soils permeability.
- **Appendix C** – *Miscellaneous Specifications*. This appendix provides additional miscellaneous design details for the design of stormwater management systems.
- **Appendix D** – *Structural Stormwater Control Design Examples*. This appendix includes design examples for five different general application structural controls: stormwater pond, bioretention area, surface sand filter, infiltration trench, and enhanced (dry) swale.
- **Appendix E** – *Structural Control Maintenance Checklists*. The appendix provides checklists for the inspection and maintenance of structural stormwater controls.
- **Appendix F** – *Landscaping and Aesthetics Guidance*. This appendix provides landscaping criteria and plant selection guidance for stormwater management facilities.
- **Appendix G** – *Stormwater Computer Models.* This appendix provides guidance on various stormwater-related computer models for planning, design and analysis.
- **Appendix H** – *Georgia Safe Dams Rules*. This appendix contains the Georgia DNR-EPD dam safety rules.

Regulatory Status of the Manual

This Manual has been developed to provide guidance on the latest and most relevant stormwater management strategies and practices for the state of Georgia. The Manual itself has no independent regulatory authority. The minimum requirements and technical guidance included in the Manual can only become required through:

(1) Ordinances and rules established by local communities; and

(2) Permits and other authorizations issued by local, state and federal agencies.

Adoption of either the Georgia Stormwater Management Manual – Volume 2 or an equivalent stormwater design manual is required for all municipalities covered under the National Pollutant Discharge Elimination System (NPDES) Municipal Stormwater Permit.

How to Get Printed Copies of the Manual

Printed copies of the Manual or the Manual on CD can be ordered by calling 404-463-3102 or ordered online at the following Internet address:

http://www.atlantaregional.com/bookstore/

How to Find the Manual on the Internet

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

http://www.georgiastormwater.com

Contact Information

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CHAPTER 1

STORMWATER MANAGEMENT PLANNING AND DESIGN

GEORGIA STORMWATER MANAGEMENT MANUAL
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CHAPTER 1

STORMWATER MANAGEMENT PLANNING AND DESIGN

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SECTION

1.1

THE NEED FOR STORMWATER MANAGEMENT

1.1.1 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.

1.1.1.1 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.

Figure 1.1.1-1 Clearing and Grading Alter the Hydrology of the Land

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 1.1.1-2 Impervious Cover Increases Runoff Peak Flows and Volumes While Reducing Recharge

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.

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.

1.1.1.2 Changes to Stream Flow

Urban development alters the hydrology of watersheds and streams by disrupting the natural water cycle. This results in:

- Increased Runoff Volumes – Land surface changes can dramatically increase the total volume of runoff generated in a developed watershed.
- Increased Peak Runoff Discharges – Increased peak discharges for a developed watershed can be two to five times higher than those for an undisturbed watershed.
- Greater Runoff Velocities – Impervious surfaces and compacted soils, as well as improvements to the drainage system such as storm drains, pipes and ditches, increase the speed at which rainfall runs off land surfaces within a watershed.
- Timing – As runoff velocities increase, it takes less time for water to run off the land and reach a stream or other waterbody.
- Increased Frequency of Bankfull and Near Bankfull Events – Increased runoff volumes and peak flows increase the frequency and duration of smaller bankfull and near bankfull events which are the primary channel forming events.
- Increased Flooding – Increased runoff volumes and peaks also increase the frequency, duration and severity of out-of-bank flooding.
- Lower Dry Weather Flows (Baseflow) – Reduced infiltration of stormwater runoff causes streams to have less baseflow during dry weather periods and reduces the amount of rainfall recharging groundwater aquifers.

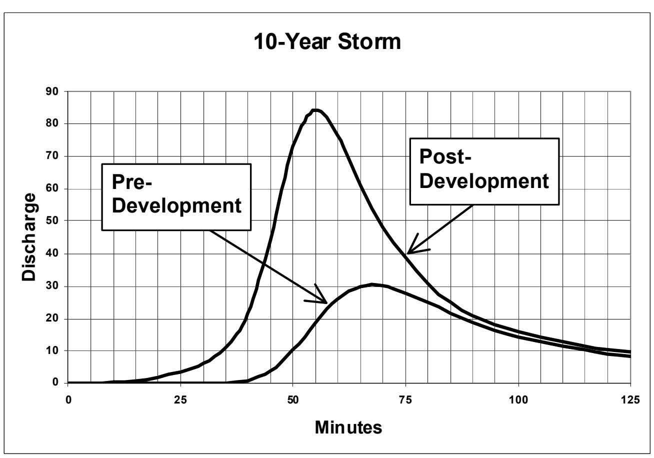

Figure 1.1.1-3 Hydrograph under Pre- and Post Development Conditions

1.1.1.3 Changes to Stream Geometry

The changes in the rates and amounts of runoff from developed watersheds directly affect the morphology, or physical shape and character, of Georgia's streams and rivers. Some of the impacts due to urban development include:

- Stream Widening and Bank Erosion – Stream channels widen to accommodate and convey the increased runoff and higher stream flows from developed areas. More frequent small and moderate runoff events undercut and scour the lower parts of the streambank, causing the steeper banks to slump and collapse during larger storms. Higher flow velocities further increase streambank erosion rates. A stream can widen many times its original size due to post-development runoff.
- Stream Downcutting – Another way that streams accommodate higher flows is by downcutting their streambed. This causes instability in the stream profile, or elevation along a stream's flow path, which increases velocity and triggers further channel erosion both upstream and downstream.
- Loss of Riparian Tree Canopy – As streambanks are gradually undercut and slump into the channel, the trees that had protected the banks are exposed at the roots. This leaves them more likely to be uprooted during major storms, further weakening bank structure.
- Changes in the Channel Bed Due to Sedimentation – Due to channel erosion and other sources upstream, sediments are deposited in the stream as sandbars and other features, covering the channel bed, or substrate, with shifting deposits of mud, silt and sand.
- Increase in the Floodplain Elevation – To accommodate the higher peak flow rate, a stream's floodplain elevation typically increases following development in a watershed due to higher peak flows. This problem is compounded by building and filling in floodplain areas, which cause flood heights to rise even further. Property and structures that had not previously been subject to flooding may now be at risk.

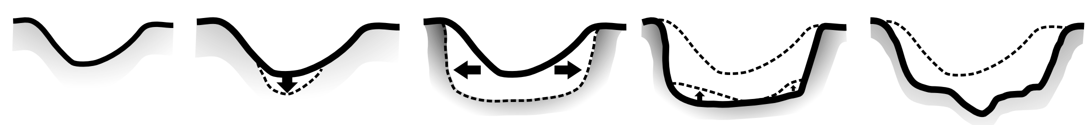

Figure 1.1.1-4 Changes to a Stream's Physical Character Due to Watershed Development

1.1.1.4 Impacts to Aquatic Habitat

Along with changes in stream hydrology and morphology, the habitat value of streams diminishes due to development in a watershed. Impacts on habitat include:

- Degradation of Habitat Structure – Higher and faster flows due to development can scour channels and wash away entire biological communities. Streambank erosion and the loss of riparian vegetation reduce habitat for many fish species and other aquatic life, while sediment deposits can smother bottom-dwelling organisms and aquatic habitat.
- Loss of Pool-Riffle Structure – Streams draining undeveloped watersheds often contain pools of deeper, more slowly flowing water that alternate with "riffles" or shoals of shallower, faster flowing water. These pools and riffles provide valuable habitat for fish and aquatic insects. As a result of the increased flows and sediment loads from urban watersheds, the pools and riffles disappear and are replaced with more uniform, and often shallower, streambeds that provide less varied aquatic habitat.

- Reduce Baseflows -- Reduced baseflows due to increased impervious cover in a watershed and the loss of rainfall infiltration into the soil and water table adversely affect in-stream habitats, especially during periods of drought.
- Increased Stream Temperature – Runoff from warm impervious areas, storage in impoundments, loss of riparian vegetation and shallow channels can all cause an increase in temperature in urban streams. Increased temperatures can reduce dissolved oxygen levels and disrupt the food chain. Certain aquatic species can only survive within a narrow temperature range. Thermal problems are especially critical for many Piedmont streams which straddle the borderline between coldwater and warmwater stream conditions.
- Decline in Abundance and Biodiversity – When there is a reduction in various habitats and habitat quality, both the number and the variety, or diversity, of organisms (wetland plants, fish, macroinvertebrates, etc.) are also reduced. Sensitive fish species and other life forms disappear and are replaced by those organisms that are better adapted to the poorer conditions. The diversity and composition of the benthic, or streambed, community have frequently been used to evaluate the quality of urban streams. Aquatic insects are a useful environmental indicator as they form the base of the stream food chain.

Fish and other aquatic organisms are impacted not only by the habitat changes brought on by increased stormwater runoff quantity, but are often also adversely affected by water quality changes due to development and resultant land use activities in a watershed.

1.1.1.5 Water Quality Impacts

Nonpoint source pollution, which is the primary cause of polluted stormwater runoff and water quality impairment, comes from many diffuse or scattered sources—many of which are the result of human activities within a watershed. Development concentrates and increases the amount of these nonpoint source pollutants. As stormwater runoff moves across the land surface, it picks up and carries away both natural and human-made pollutants, depositing them into Georgia's streams, rivers, lakes, wetlands, coastal waters and marshes, and underground aquifers. Nonpoint source pollution is the leading source of water quality degradation in Georgia.

Water quality degradation in urbanizing watersheds starts when development begins. Erosion from construction sites and other disturbed areas contribute large amounts of sediment to streams. As construction and development proceed, impervious surfaces replace the natural land cover and pollutants from human activities begin to accumulate on these surfaces. During storm events, these pollutants are then washed off into the streams. Stormwater also causes discharges from sewer overflows and leaching from septic tanks. There are a number of other causes of nonpoint source pollution in urban areas that are not specifically related to wet weather events including leaking sewer pipes, sanitary sewage spills, and illicit discharge of commercial/industrial wastewater and wash waters to storm drains.

Due to the magnitude of the problem, it is important to understand the nature and sources of urban stormwater pollution. Table 1.1.1-1 summarizes the major stormwater pollutants and their effects. Some of the most frequently occurring pollution impacts and their sources for urban streams are:

- Reduced Oxygen in Streams – The decomposition process of organic matter uses up dissolved oxygen (DO) in the water, which is essential to fish and other aquatic life. As organic matter is washed off by stormwater, dissolved oxygen levels in receiving waters can be rapidly depleted. If the DO deficit is severe enough, fish kills may occur and stream life can weaken and die. In addition, oxygen depletion can affect the release of toxic chemicals and nutrients from sediments deposited in a waterway.

  All forms of organic matter in urban stormwater runoff such as leaves, grass clippings and pet waste contribute to the problem. In addition, there are a number of non-stormwater discharges of organic matter to surface waters such as sanitary sewer leakage and septic tank leaching.

Table 1.1.1-1 Summary of Urban Stormwater Pollutants

Constituents	Effects
Sediments—Suspended Solids, Dissolved Solids, Turbidity	Stream turbidity Habitat changes Recreation/aesthetic loss Contaminant transport Filling of lakes and reservoirs
Nutrients—Nitrate, Nitrite, Ammonia, Organic Nitrogen, Phosphate, Total Phosphorus	Algae blooms Eutrophication Ammonia and nitrate toxicity Recreation/aesthetic loss
Microbes—Total and Fecal Coliforms, Fecal Streptococci Viruses, E.Coli, Enterocci	Ear/Intestinal infections Shellfish bed closure Recreation/aesthetic loss
Organic Matter—Vegetation, Sewage, Other Oxygen Demanding Materials	Dissolved oxygen depletion Odors Fish kills
Toxic Pollutants—Heavy Metals (cadmium, copper, lead, zinc), Organics, Hydrocarbons, Pesticides/Herbicides	Human & aquatic toxicity Bioaccumulation in the food chain
Thermal Pollution	Dissolved oxygen depletion Habitat changes
Trash and debris	Recreation/aesthetic loss

- Nutrient Enrichment – Runoff from urban watersheds contains increased nutrients such as nitrogen or phosphorus compounds. Increased nutrient levels are a problem as they promote weed and algae growth in lakes, streams and estuaries. Algae blooms block sunlight from reaching underwater grasses and deplete oxygen in bottom waters. In addition, nitrification of ammonia by microorganisms can consume dissolved oxygen, while nitrates can contaminate groundwater supplies. Sources of nutrients in the urban environment include washoff of fertilizers and vegetative litter, animal wastes, sewer overflows and leaks, septic tank seepage, detergents, and the dry and wet fallout of materials in the atmosphere.
- Microbial Contamination – The level of bacteria, viruses and other microbes found in urban stormwater runoff often exceeds public health standards for water contact recreation such as swimming and wading. Microbes can also contaminate shellfish beds, preventing their harvesting and consumption, as well as increasing the cost of treating drinking water. The main sources of these contaminants are sewer overflows, septic tanks, pet waste, and urban wildlife such as pigeons, waterfowl, squirrels, and raccoons.
- Hydrocarbons – Oils, greases and gasoline contain a wide array of hydrocarbon compounds, some of which have shown to be carcinogenic, tumorigenic and mutagenic in certain species of fish. In addition, in large quantities, oil can impact drinking water supplies and affect recreational use of waters. Oils and other hydrocarbons are washed off roads and parking lots, primarily due to engine leakage from vehicles. Other sources include the improper disposal of motor oil in storm drains and streams, spills at fueling stations and restaurant grease traps.
- Toxic Materials – Besides oils and greases, urban stormwater runoff can contain a wide variety of other toxicants and compounds including heavy metals such as lead, zinc, copper, and cadmium, and organic pollutants such as pesticides, PCBs, and phenols. These contaminants are of concern because they are toxic to aquatic organisms and can bioaccumulate in the food chain. In addition, they also impair drinking water sources and human health. Many of these toxicants accumulate in the sediments of streams and lakes.

Sources of these contaminants include industrial and commercial sites, urban surfaces such as rooftops and painted areas, vehicles and other machinery, improperly disposed household chemicals, landfills, hazardous waste sites and atmospheric deposition.

- Sedimentation – Eroded soils are a common component of urban stormwater and are a pollutant in their own right. Excessive sediment can be detrimental to aquatic life by interfering with photosynthesis, respiration, growth and reproduction. Sediment particles transport other pollutants that are attached to their surfaces including nutrients, trace metals and hydrocarbons. High turbidity due to sediment increases the cost of treating drinking water and reduces the value of surface waters for industrial and recreational use. Sediment also fills ditches and small streams and clogs storm sewers and pipes, causing flooding and property damage. Sedimentation can reduce the capacity of reservoirs and lakes, block navigation channels, fill harbors and silt estuaries. Erosion from construction sites, exposed soils, street runoff, and streambank erosion are the primary sources of sediment in urban runoff.
- Higher Water Temperatures – As runoff flows over impervious surfaces such as asphalt and concrete, it increases in temperature before reaching a stream or pond. Water temperatures are also increased due to shallow ponds and impoundments along a watercourse as well as fewer trees along streams to shade the water. Since warm water can hold less dissolved oxygen than cold water, this "thermal pollution" further reduces oxygen levels in depleted urban streams. Temperature changes can severely disrupt certain aquatic species, such as trout and stoneflies, which can survive only within a narrow temperature range.
- Trash and Debris – Considerable quantities of trash and other debris are washed through storm drain systems and into streams, lakes and bays. The primary impact is the creation of an aesthetic "eyesore" in waterways and a reduction in recreational value. In smaller streams, debris can cause blockage of the channel, which can result in localized flooding and erosion.

1.1.1.6 Stormwater Hotspots

Stormwater hotspots are areas of the urban landscape that often produce higher concentrations of certain pollutants, such as hydrocarbons or heavy metals, than are normally found in urban runoff. These areas merit special management and the use of specific pollution prevention activities and/or structural stormwater controls. Examples of stormwater hotspots include:

- Gas / fueling stations
- Vehicle maintenance areas
- Vehicle washing / steam cleaning
- Auto recycling facilities
- Outdoor material storage areas
- Loading and transfer areas
- Landfills
- Construction sites
- Industrial sites
- Industrial rooftops

1.1.1.7 Effects on Lakes, Reservoirs and Estuaries

Stormwater runoff into lakes and reservoirs can have some unique negative effects. A notable impact of urban runoff is the filling in of lakes and embayments with sediment. Another significant water quality impact on lakes related to stormwater runoff is nutrient enrichment. This can result in the undesirable growth of algae and aquatic plants. Lakes do not flush contaminants as quickly as streams and act as sinks for nutrients, metals and sediments. This means that lakes can take longer to recover if contaminated.

Stormwater runoff can also impact estuaries, especially if runoff events occur in pulses, disrupting the natural salinity of an area and providing large loads of sediment, nutrients and oxygen demanding materials. These rapid pulses or influxes of fresh water into the watershed may be two to ten times greater than normal and may lead to a decrease in the number of aquatic organisms living in the unique estuarine environment. Tidal flow patterns can also effectively trap and concentrate runoff pollutants.

1.1.2 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 both the 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 both 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
- Using the most current and effective erosion and sedimentation control practices during the *construction* phase of development
- 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 1 outlines a technical approach for incorporating all of these stormwater management approaches into the development process. The next section (Section 1.2) discusses the minimum stormwater management standards for new development and redevelopment in Georgia that aim to meet the objectives above.

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SECTION

1.2

STORMWATER MANAGEMENT STANDARDS

1.2.1 Overview

This section presents a comprehensive set of minimum 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 urban development.

The goal of a set of minimum 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 to reduce the generation of runoff and pollutants; (2) managing and treating stormwater runoff though the use of structural stormwater controls; and (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 minimum 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.

1.2.2 Minimum Standards for Development

1.2.2.1 Applicability

The stormwater management standards for new development and redevelopment are intended to apply to any development 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 5,000 square feet 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 a hotspot land use as defined below.

In addition, redevelopment sites that involve land disturbing activity of 5,000 square feet or greater, but less than 1 acre, must meet Minimum Standard 8 (to meet state and NPDES construction erosion and sediment control requirements) and should meet Minimum Standards 2, 9 and 10 to the maximum extent practicable.

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 surface not part of routine maintenance, and land disturbing activities associated with structural or impervious development. Redevelopment does not include such activities as exterior remodeling.

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, salvage yards, material storage sites, garbage transfer facilities, and commercial parking lots with high-intensity use.

Exemptions

The following development activities are suggested to be exempted from the minimum stormwater management standards:

(1) Developments that do not disturb more than 5,000 square feet of land;
(2) Individual single family residential lots. (Single family lots that are part of a subdivision or phased development project should not be exempt from the minimum standards); and
(3) Additions or modifications to existing single-family structures.

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 controls in order to protect a special resource or address certain water quality or drainage problems identified for a drainage area.

1.2.2.2 Minimum Stormwater Management Standards

The following standards are the recommended minimum stormwater management performance requirements for new development or redevelopment sites falling under the applicability criteria in subsection 1.2.2.1. (The word "shall" in brackets is provided for local jurisdictions that wish to adopt these standards as part of their stormwater management ordinances) A more detailed explanation of each minimum standard is provided in the next subsection.

- **Minimum Standard #1 – Use of Better Site Design Practices for Stormwater Management**

  Site designs should preserve the natural drainage and treatment systems and reduce the generation of additional stormwater runoff and pollutants to the fullest extent practicable.

- **Minimum Standard #2 – Stormwater Runoff Quality**

  All stormwater runoff generated from a site should [shall] be adequately treated before discharge. Stormwater management systems (which can include both structural stormwater controls and better site design practices) should [must] be designed to remove 80% of the average annual post-development total suspended solids (TSS) load and be able to meet any other additional watershed- or site-specific water quality requirements.

  It is presumed that a stormwater management system complies with this performance standard if:

- It is sized to capture and treat the prescribed water quality treatment volume, which is defined as the runoff volume resulting from the first 1.2 inches of rainfall from a site; and
- Appropriate structural stormwater controls are selected, designed, constructed, and maintained according to the specific criteria in this Manual.
- Runoff from hotspot land uses and activities is adequately treated and addressed through the use of appropriate structural stormwater controls and pollution prevention practices.

❑ **Minimum Standard #3 – Stream Channel Protection**

Stream channel protection should [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; and (3) preservation of the applicable stream buffer.

❑ **Minimum Standard #4 – Overbank Flood Protection**

Downstream overbank flood protection should [shall] be provided by controlling the post-development peak discharge rate to the predevelopment rate for the 25-year, 24-hour return frequency storm event. If control of the 1-year, 24-hour storm (Minimum Standard #3) is exempted, then overbank flood protection should [shall] be provided by controlling the post-development peak discharge rate to the predevelopment rate for the 2-year through the 25-year return frequency storm events.

❑ **Minimum Standard #5 – Extreme Flood Protection**

Extreme flood protection should [shall] be provided by controlling and/or safely conveying the 100-year, 24-hour return frequency storm event such that flooding is not exacerbated. Existing and future floodplain areas should be preserved as possible.

❑ **Minimum Standard #6 – Downstream Analysis**

A downstream hydrologic analysis should [shall] be performed to determine if there are any additional impacts in terms of peak flow increase or downstream flooding while meeting Minimum Standards #1 through 5. This analysis should [shall] be performed at the outlet(s) of the site, and downstream at each tributary junction to the point(s) in the conveyance system where the area of the portion of the site draining into the system is less than or equal to 10% of the total drainage area above that point.

❑ **Minimum Standard #7 – Groundwater Recharge**

Annual groundwater recharge rates should be maintained to the extent practicable through the use of nonstructural methods.

❑ **Minimum Standard #8 – Construction Erosion and Sedimentation Control**

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

❑ **Minimum Standard #9 – Stormwater Management System Operation and Maintenance**

The stormwater management system, including all structural stormwater controls and conveyances, should [shall] have an operation and maintenance plan to ensure that it continues to function as designed.

❑ **Minimum Standard #10 – Pollution Prevention**

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

❑ **Minimum Standard #11 – Stormwater Management Site Plan**

The development project should [shall] prepare a stormwater management site plan for local government review that addresses Minimum Standards #1 through 10.

1.2.2.3 Explanation of Minimum Standards

Use of Better Site Design Practices for Stormwater Management (Minimum Standard #1)

All site designs should implement a combination of approaches collectively known as *stormwater better site design practices* to the fullest extent possible. 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 structural stormwater controls. Better site design practices are described in Section 1.4.

Stormwater Runoff Quality (Minimum Standard #2)

Stormwater runoff generated on the development site is to be treated by the stormwater management system to remove at least 80% of the calculated average annual post-development TSS loading from the site. This can be achieved through the use of site design practices and structural stormwater controls.

This requirement is quantified and expressed in terms of engineering design criteria through the specification of a *water quality volume* (WQ_v) that must be treated to the 80% TSS removal performance goal. The water quality treatment volume is equal to the runoff generated on a site from 1.2 inches of rainfall. The water quality volume is one of the *unified stormwater sizing criteria,* which are used in conjunction to size and design stormwater management facilities to address stormwater impacts. The unified stormwater sizing criteria and methods to calculate the WQ_v are discussed in Section 1.3.

Structural stormwater controls are sized and designed to treat the WQ_v. Depending on their removal efficiency or site constraints, more than one structural control may need to be used in parallel or in series (treatment train) to meet the water quality treatment requirement. Further, this standard assumes that structural stormwater controls 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 controls and pollution prevention practices. A detailed overview of structural stormwater controls is provided in Section 3.1.

The use of nonstructural 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 1.4.

Stream Channel Protection (Minimum Standard #3)

Protection of stream channels is to be provided to both downstream as well as on-site channels. This is accomplished through three complementary criteria:

The first method of providing streambank protection is the extended detention of the 1-year, 24-hour storm for a period of 24 hours using structural stormwater controls. 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 channel protection volume is one of the *unified stormwater sizing criteria,* which are used in conjunction to size and design stormwater management facilities to address stormwater impacts. The unified stormwater sizing criteria and methods to calculate the storage requirements and routing of Cp_v are discussed in Section 1.3. The use of nonstructural site design practices that reduce the total amount of runoff will also reduce Cp_v by a proportional amount. This requirement may be waived by a local jurisdiction for sites that discharge directly into piped stormwater drainage systems, larger streams, rivers, wetlands, lakes, estuaries, tidal waters, or other situations where the reduction in the smaller flows will not have an impact on streambank or channel integrity.

The second streambank protection method is to implement velocity control, energy dissipation, streambank stabilization, and erosion prevention practices and structures as necessary in the stormwater management system to prevent downstream erosion and streambank damage. Energy dissipation and velocity control methods are discussed in Section 4.5.

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 1.4.

Downstream Overbank Flood Protection (Minimum Standard #4)

Overbank flood protection for downstream channels is to be provided by preventing the post-development 25-year, 24-hour storm peak discharge rate (denoted Q_{p25}) from exceeding the pre-development (or natural conditions) discharge rate using structural stormwater controls. The overbank flood protection peak rate is one of the *unified stormwater sizing criteria,* which are used in conjunction to size and design stormwater management facilities to address stormwater impacts. The unified stormwater sizing criteria and methods to calculate the storage requirements and routing of Q_{p25} are discussed in Section 1.3. The use of nonstructural site design practices that reduce the total amount of runoff will also reduce Q_{p25} by a proportional amount.

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

If the control of the 1-year, 24-hour storm under Minimum Standard #3 is exempted, then for overbank flood protection, peak flow attenuation of the 2-year (Qp_2) through the 25-year (Qp_{25}) return frequency storm events must be provided.

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 causing flood damage.

Extreme Flood Protection (Minimum Standard #5)

Extreme flood protection is to 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 structural stormwater controls 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 at the 100-year level.

The extreme flood protection peak rate is one of the *unified stormwater sizing criteria,* which are used in conjunction to size and design stormwater management facilities to address stormwater impacts. The unified stormwater sizing criteria and methods to calculate the storage requirements and routing of Q_f are discussed in Section 1.3. The use of nonstructural site design practices that reduce the total amount of runoff will also reduce Q_f by a proportional amount.

Downstream Analysis (Minimum Standard #6)

Due to peak flow timing and runoff volume effects, some structural controls fail to reduce discharge peaks to predevelopment levels downstream from the development site.
A downstream peak flow analysis is to 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 structural stormwater controls, 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 2.1.

Groundwater Recharge (Minimum Standard #7)

Recharge to groundwater should be implemented to the extent practicable through the use of nonstructural better site design techniques that allow for recharge of stormwater runoff into the soil. The annual recharge from the post-development site should approximate the annual recharge from the pre-development or existing site conditions, based on soil types. Stormwater runoff from a hotspot site or land use should not be infiltrated without effective pretreatment.

The recommended stormwater runoff volume to be recharged to groundwater should be determined using the existing site (pre-development) soil conditions. The recommended rates of recharge for various hydrologic soil groups are as follows:

Hydrologic Group	Volume to Recharge (x Total Impervious Area)
A	0.40 inches of runoff
B	0.25 inches of runoff
C	0.10 inches of runoff
D	n/a

More information on site design practices that promote infiltration is found in Section 1.4.

Construction Erosion and Sedimentation Control (Minimum Standard #8)

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 *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 structural stormwater controls and conveyance facilities.

Stormwater Management System Operation and Maintenance (Minimum Standard #9)

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, structural stormwater controls, 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; and (4) any necessary legally binding maintenance agreements.

Pollution Prevention (Minimum Standard #10)

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. 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.

Stormwater Management Site Plan (Minimum Standard #11)

All new development and redevelopment sites are to develop a stormwater management site plan. The stormwater site plan is to provide details, including a narrative and technical information and analysis, that indicates how the proposed development meets Minimum Standards #1 through 10 (or the applicable local stormwater regulatory requirements). The preparation of stormwater management site plans is covered in Section 1.5.

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SECTION

1.3

UNIFIED STORMWATER SIZING CRITERIA

1.3.1 Overview

This section presents an integrated approach for meeting the stormwater runoff quality and quantity management requirements in the minimum standards for development (see Section 1.2) 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:

- Remove stormwater runoff pollutants and improve water quality (Minimum Standard #2);
- Prevent downstream streambank and channel erosion (Minimum Standard #3);
- Reduce downstream overbank flooding (Minimum Standard #4); and
- Safely pass or reduce the runoff from extreme storm events (Minimum Standard #5).

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 structural stormwater controls. Table 1.3.1-1 below briefly summarizes the criteria.

Table 1.3.1-1 Summary of the Statewide Stormwater Sizing Criteria for Stormwater Control and Mitigation

Sizing Criteria	Description
Water Quality	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. Reduce average annual post-development total suspended solids loadings by 80%.
Channel Protection	Provide extended detention of the 1-year 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 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 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.

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 storm.

Figure 1.3.1-1 graphically illustrates the relative volume requirements of each of the unified stormwater sizing criteria as well as demonstrates that the criteria are "stacked" upon 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 1.3.1-2 shows how these volumes would be stacked in a typical stormwater wet pond designed to handle all four criteria.

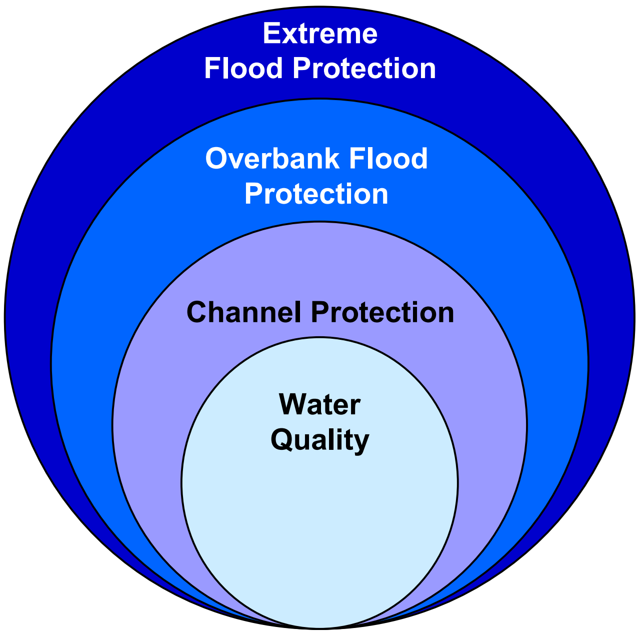


Figure 1.3.1-1 Representation of the Unified Stormwater Sizing Criteria

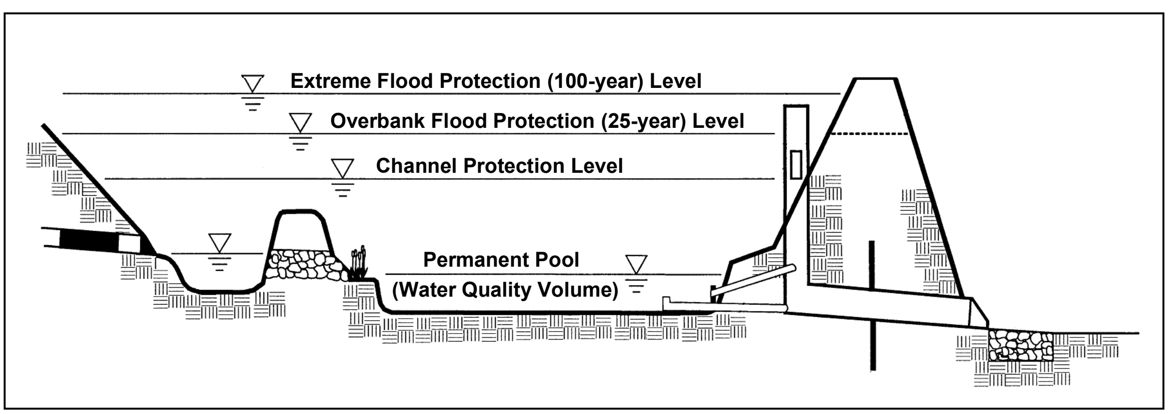


Figure 1.3.1-2 Unified Sizing Criteria Water Surface Elevations in a Stormwater (Wet) Pond

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

1.3.2 Description of Unified Stormwater Sizing Criteria

1.3.2.1 Water Quality (WQ_v)

The Water Quality sizing criterion, denoted WQ_v, specifies a treatment volume required to remove a significant percentage of the total pollution load inherent in stormwater runoff by intercepting and 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:

$$WQ_V = \frac{1.2 R_V A}{12}$$

where: WQ_v = water quality volume (in acre-feet)
R_v = 0.05 + 0.009(I) where I is <u>percent</u> impervious cover
A = site area in acres

<u>Discussion</u>

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 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 treatment volume (WQ_v) is specified to size structural control facilities to 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 structural stormwater controls.

A value of 1.2 inches for the 85th percentile storm was derived from a rainfall analysis for 12 locations across the state of Georgia and is an average value chosen for the entire state. Thus, the statewide water quality treatment volume is equal to the runoff from the first 1.2 inches of rainfall. A stormwater management system designed for the WQ_v will 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.

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 1.3.2-1 shows a plot of the Water Quality Volume versus impervious area percentage.

<u>TSS Reduction Goal</u>

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. This performance goal is based upon U.S. EPA guidance and has been adopted nationwide by many local and statewide agencies.

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 structural stormwater controls (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 structural control practice. For pollutants of specific concern, individual analyses of specific pollutant sources and the appropriate removal mechanisms should be performed.

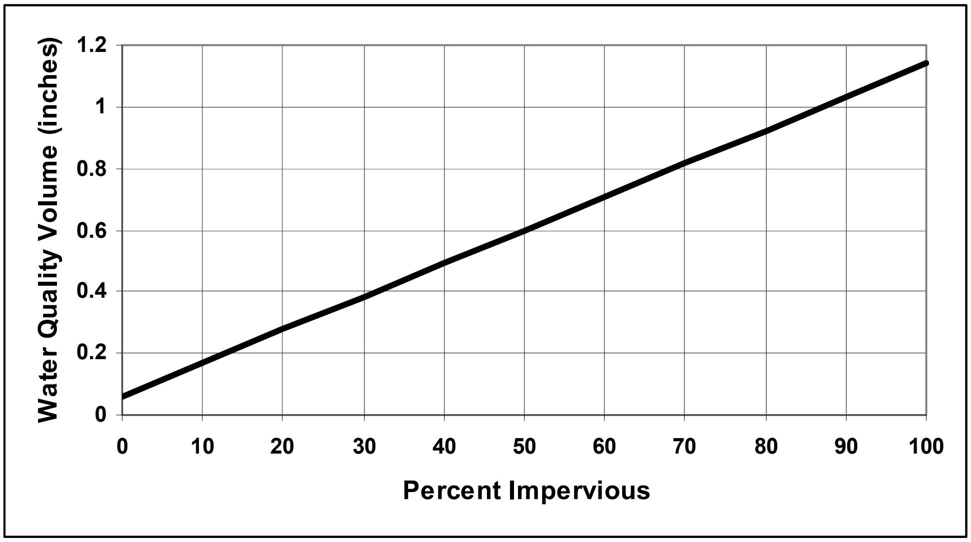


Figure 1.3.2-1 Water Quality Volume versus Percent Impervious Area

Determining the Water Quality Volume (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, WQ_v should be calculated and addressed separately for each drainage area.
- *Off-site Drainage Areas:* Off-site existing impervious areas may be excluded from the calculation of the WQ_v volume.
- *Credits for Site Design Practices*: The use of certain better site design practices may allow the WQ_v volume to be reduced through the subtraction of a site design “credit.” These site design credits are described in Section 1.4.
- *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 2.1.
- *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 stormwater pond or wetland

system (as described in Chapter 3). Referred to as water quality ED (extended detention), it is different than providing extended detention of the 1-year storm for the channel protection volume (CP_v). The ED portion of the WQv may be included when routing the CP_v.

- WQ_v can be expressed in cubic feet by multiplying by 43,560.
- WQ_v can also be expressed in watershed-inches by removing the area (A) and the "12" in the denominator.

1.3.2.2 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.
- The use of nonstructural site design 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 streambank or channel integrity.

Discussion

The increase in the frequency and duration of bankfull flow conditions in stream channels due to urban development is the primary cause of streambank erosion and the widening and downcutting 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 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 2.1.5.8 in Chapter 2 and Appendix D-1 illustrate the recommended average outflow method for volume calculation.
- *Hydrograph Generation:* The SCS TR-55 hydrograph methods provided in Section 2.1 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 tables included in Appendix A for various locations across Georgia.
- *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 structural stormwater control 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 2.3).

1.3.2.3 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 (or undisturbed natural conditions) 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 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 2.1.9). 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 SCS TR-55 or USGS hydrograph methods provided in Section 2.1 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 tables included in Appendix A for various locations across Georgia.
- *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 structural stormwater control.
- *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.

1.3.2.4 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 structural stormwater controls 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 structural stormwater controls 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 structural stormwater controls should be designed appropriately to safely pass the resulting flows.

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

- *Peak-Discharge and Hydrograph Generation:* The SCS TR-55 or USGS hydrograph methods provided in Section 2.1 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 tables included in Appendix A for various locations across Georgia.
- *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.

1.3.3 Meeting the Unified Stormwater Sizing Criteria Requirements

1.3.3.1 Introduction

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

- **The use of better site design practices to reduce the amount of stormwater runoff and pollutants generated and/or provide for natural treatment and control of runoff; and**
- **The use of structural stormwater controls to provide treatment and control of stormwater runoff**

This subsection introduces both of these approaches. Stormwater better site practices are discussed in-depth in Section 1.4, while structural stormwater controls are covered in Chapter 3.

1.3.3.2 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 structural stormwater controls—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 1.4.

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 1.3.3-1 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 1.3.3-1 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 **(WQ_v)**	• Better site design practices that reduce the total amount of runoff will also reduce WQ_v by a proportional amount. • Certain site design practices will allow for a further reduction to the Water Quality Volume. The site design credits are discussed in Section 1.4.
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. • Floodplain preservation may allow waiving of overbank flood and/or extreme flood protection requirements.

1.3.3.3 Recommended Structural Stormwater Control Practices

Structural stormwater controls (sometimes referred to as *structural best management practices* or *BMPs*) are constructed stormwater management facilities designed to 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 structural stormwater controls for meeting unified stormwater sizing criteria. The recommended controls are divided into three categories: *general application*, *limited application*, and *detention* structural controls.

General Application Controls

General application structural controls are recommended for use with a wide variety of land uses and development types. These structural controls have a demonstrated ability to effectively treat the Water Quality Volume (WQ_v) and are presumed to be able to remove 80% of the total annual average TSS load in typical post-development urban runoff when designed, constructed and maintained in accordance with recommended specifications. Several of the general application structural controls can also be designed to provide water quantity control; i.e., downstream channel protection (CP_v), overbank flood protection (Q_{p25}) and/or extreme flood protection (Q_f). General application controls are the recommended stormwater management facilities for a site wherever feasible and practical.

There are six types of general application controls, which are summarized below. Detailed descriptions of each structural control along with design criteria and procedures are provided in Section 3.2.

<u>Stormwater Ponds</u>

Stormwater ponds are constructed stormwater retention basins that have a permanent pool (or micropool) of water. Runoff from each rain event is detained and treated in the pool. Pond design variants include:

- Wet Pond
- Wet Extended Detention Pond
- Micropool Extended Detention Pond
- Multiple Pond Systems

<u>Stormwater Wetlands</u>

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. Wetland design variants include:

- Shallow Wetland
- Extended Detention Shallow Wetland
- Pond/Wetland Systems
- Pocket Wetland

<u>Bioretention Areas</u>

Bioretention areas are shallow stormwater basins or landscaped areas that utilize engineered soils and vegetation to capture and treat stormwater runoff. Runoff may be returned to the conveyance system, or allowed to fully or partially exfiltrate into the soil.

<u>Sand Filters</u>

Sand filters are multi-chamber structures designed to treat stormwater runoff through filtration, using a sand bed as the primary filter media. Filtered runoff may be returned to the conveyance system, or allowed to fully or partially exfiltrate into the soil. The two sand filter design variants are:

- Surface Sand Filter
- Perimeter Sand Filter

Infiltration Trenches

An infiltration trench is an excavated trench filled with stone aggregate used to capture and allow infiltration of stormwater runoff into the surrounding soils from the bottom and sides of the trench.

Enhanced Swales

Enhanced swales are 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. The two types of enhanced swales are:

- Dry Swale
- Wet Swale/Wetland Channel

Limited Application Controls

Limited application structural controls are those that are recommended only for limited use or for special site or design conditions. Generally, these practices: (1) cannot alone achieve the 80% TSS removal target, (2) are intended to address hotspot or specific land use constraints or conditions, and/or (3) may have high or special maintenance requirements that may preclude their use. Limited application controls are typically used for *water quality treatment only*. Some of these controls can be used as a pretreatment measure or in series with other structural controls to meet pollutant removal goals. Limited application structural controls should be considered primarily for commercial, industrial or institutional developments.

The following limited application controls are provided for consideration in this Manual. Each is discussed in detail with appropriate application guidance in Section 3.3.

Biofilters

- Filter Strip
- Grass Channel

Filtering Practices

- Organic Filter
- Underground Sand Filter

Wetland Systems

- Submerged Gravel Wetland

Hydrodynamic Devices

- Gravity (Oil-Grit) Separator

Porous Surfaces

- Modular Porous Paver Systems
- Porous Concrete

Chemical Treatment

- Alum Treatment System

Proprietary Systems

- Commercial Stormwater Controls

Detention Controls

Detention structural controls provide only water quantity control (CP_v, Q_{p25}, and/or Q_f), and are typically used downstream of a general application or limited application structural control. Types of detention controls include:

- Dry Detention and Dry Extended Detention Basins
- Multi-purpose Detention Areas
- Underground Detention

A detailed discussion of each of the detention controls, as well as design criteria and procedures can be found in Section 3.4.

1.3.3.4 Using Structural Stormwater Controls to Meet Unified Stormwater Sizing Criteria Requirements

Structural stormwater controls 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 controls has been established, one or more appropriate controls 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 structural stormwater control(s) for a site is provided in Section 3.1.

Table 1.3.3-2 summarizes the stormwater management suitability of the various structural controls in addressing the unified stormwater sizing criteria. Given that many structural controls cannot meet all of the sizing criteria, typically two or more controls are used in series to form what is known as a stormwater "treatment train." Section 3.1 provides guidance on the use of a treatment train as well as calculating the pollutant removal efficiency for structural controls in series.

Table 1.3.3-2 Suitability of Structural Stormwater Controls to Meet Unified Stormwater Sizing Criteria

Structural Stormwater Control	Water Quality Volume (WQ_v)	Channel Protection (CP_v)	Overbank Flood Protection (Q_{p25})	Extreme Flood Protection (Q_f)
General Application				
Stormwater Ponds	✓	✓	✓	✓
Stormwater Wetlands	✓	✓	✓	✓
Bioretention Areas	✓	✪	●	●
Sand Filters	✓	✪	●	●
Infiltration Trenches	✓	✪	●	●
Enhanced Swales	✓	✪	✪	●
Limited Application				
Biofilters	○	●	●	●
Filtering Practices	✓	●	●	●
Wetland Systems	✓	●	●	●
Hydrodynamic Devices	○	●	●	●
Porous Surfaces	✓	✪	●	●
Chemical Treatment	✓	●	●	●
Proprietary Systems	*	*	*	*
Detention Controls	●	✓	✓	✓

✓ = Able to meet stormwater sizing criterion (for water quality, this control is presumed to meet the 80% TSS reduction goal when sized to treat the WQ_v and designed, constructed and maintained properly)

○ = Typically provides partial treatment of WQ_v. May be used in pretreatment and as part of a "treatment train"

✪ = Can be incorporated into the structural control in certain situations

● = Not typically able or used to meet stormwater sizing criterion

* = The application and performance of specific commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data

1.3.3.5 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 minimum stormwater management standards and unified stormwater sizing criteria are addressed.

Figure 1.3.3-1 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 1.5.

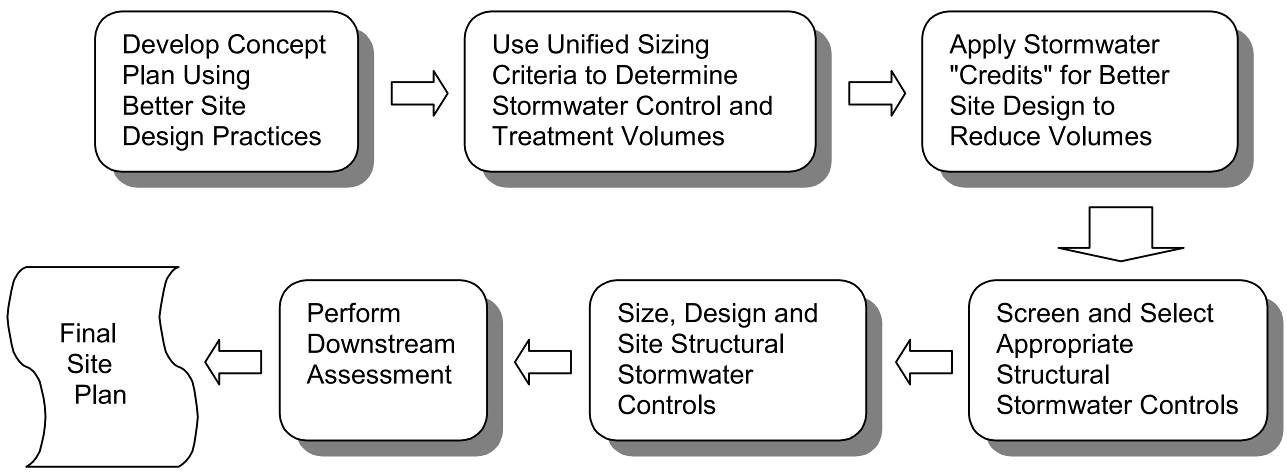


Figure 1.3.3-1 Typical Stormwater Management System Design Process

SECTION

1.4

STORMWATER BETTER SITE DESIGN

1.4.1 Overview

1.4.1.1 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
- 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, all opportunities for using these methods should be explored and all options exhausted before considering structural stormwater controls.

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 structural stormwater controls. 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.

Several 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. Subsection 1.4.4 will discuss 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

1.4.1.2 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 Floodplains
  - Avoid 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
  - 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 1.4.2. These summaries provide the key benefits of each practice, examples and details on how to apply them in site design.

1.4.1.3 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 1.4.1-1 illustrates the stormwater better site design process that utilizes the four better site design categories.

Figure 1.4.1-1 Stormwater Better Site Design Process

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.

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.

1.4.2 Better Site Design Practices

1.4.2.1 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

Delineation of natural features is typically done through a comprehensive site analysis and inventory before any site layout design is performed (see Section 1.5). From this site analysis, a concept plan for a site can be prepared that provides for the conservation and protection of natural features. Figure 1.4.2-1 shows an example of the delineation of natural features on a base map of a development parcel.

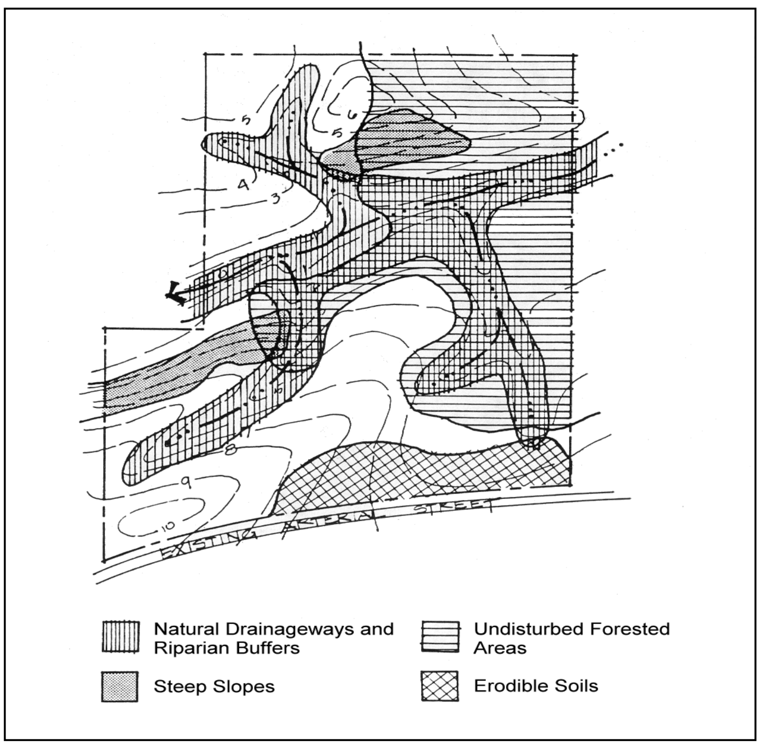


Figure 1.4.2-1 Example of Natural Feature Delineation
(Source: MPCA, 1989)

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	USING THIS PRACTICE
• Conserving undisturbed natural areas helps to preserve a portion of the site's natural predevelopment 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 1.4.4)	☑ 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 1.4.2-2 shows a site map with undisturbed natural areas delineated.

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.

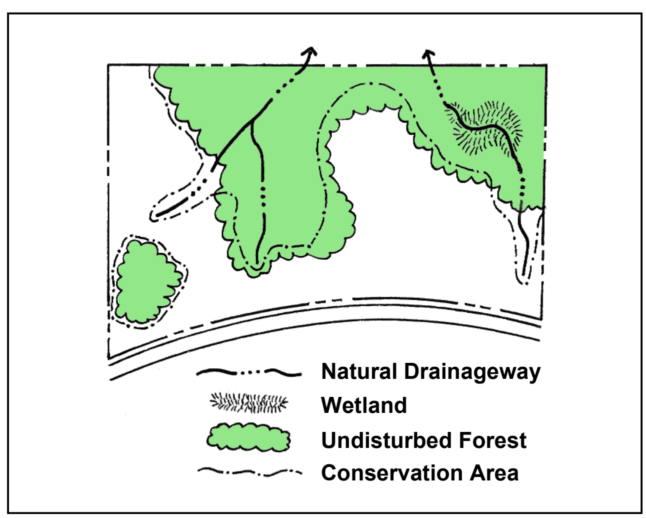


Figure 1.4.2-2 Delineation of Natural Conservation Areas

Better Site Design Practice #2:

Preserve Riparian Buffers

Conservation of Natural Features and Resources

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

KEY BENEFITS	**USING THIS PRACTICE**
• 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 • A stormwater site design credit can be taken if allowed by the local review authority (see subsection 1.4.4)	☑ 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 1.4.2-3.

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.

Figure 1.4.2-3 Riparian Stream Buffer

Ideally, riparian buffers should be sized to include the 100-year floodplain as well as steep banks and freshwater wetlands. The buffer depth 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 depth are shown in Figure 1.4.2-4. The function, vegetative target and allowable uses vary by zone as described in Table 1.4.2-1.

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 supercede deeper or more restrictive buffer requirements that are already in place.

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.

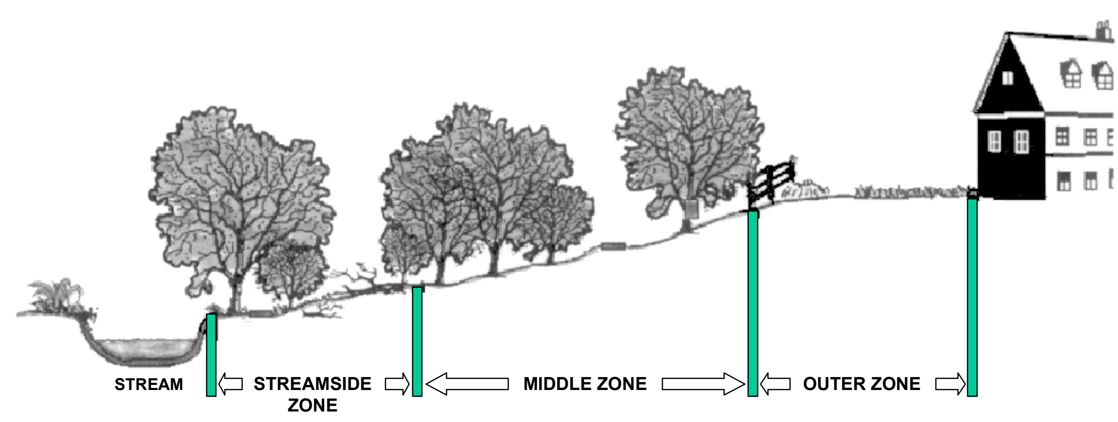


Figure 1.4.2-4 Three-Zone Stream Buffer System

Table 1.4.2-1 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 turfgrass.
Allowable Uses	**Very Restricted** e.g., flood control, utility easements, footpaths	**Restricted** e.g., some recreational uses, some stormwater controls, bike paths	**Unrestricted** e.g., residential uses including lawn, garden, most stormwater controls

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	USING THIS PRACTICE
• 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	☑ 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.

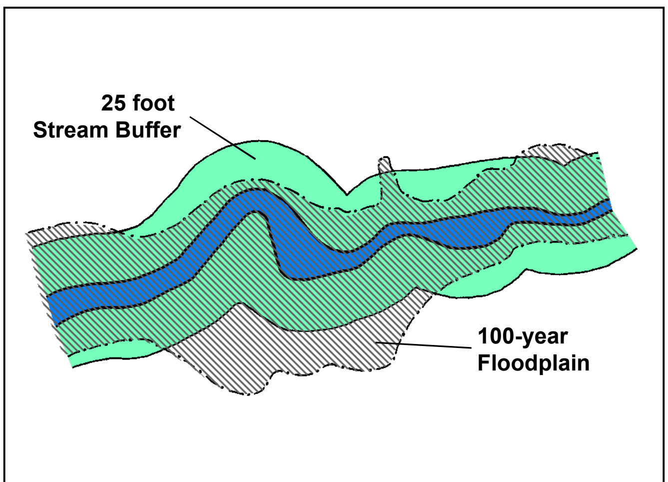


Figure 1.4.2-5 Floodplain Boundaries in Relation to a Riparian Buffer

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 1.4.2-5.

Maps of the 100-year floodplain can typically be obtained through the local review authority. Developers and builders should also ensure that their site design comply will any other relevant local floodplain and FEMA requirements.

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	USING THIS PRACTICE
• 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	☑ 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 SCS (now 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 1.4.2-6.

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.

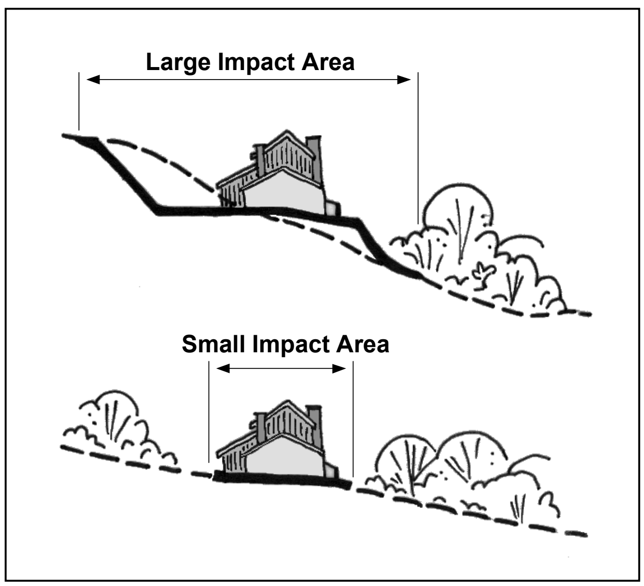


Figure 1.4.2-6 Flattening Steep Slopes for Building Sites Uses More Land Area Than Building on Flatter Slopes
(Source: MPCA, 1989)

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	**USING THIS PRACTICE**
• Areas with highly permeable soils can be used as nonstructural stormwater infiltration zones. A stormwater site design credit can be taken if allowed by the local review authority (see subsection 1.4.4) • Avoiding high erodible or unstable soils can prevent erosion and sedimentation problems and water quality degradation	☑ 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 1.4.2-7. Soil surveys can provide a considerable amount of information relating to all relevant aspects of soils. Appendix B 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.

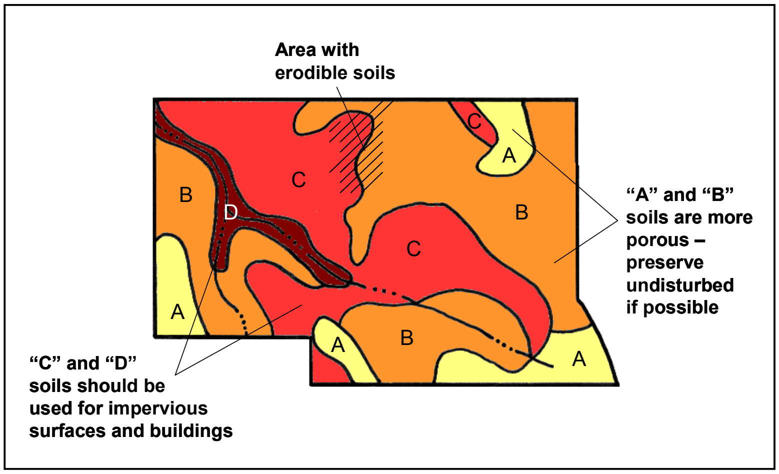


Figure 1.4.2-7 Soil Mapping Information Can Be Used to Guide Development

1.4.2.2 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 1.4.2-8 shows a development that has utilized several lower impact site design techniques

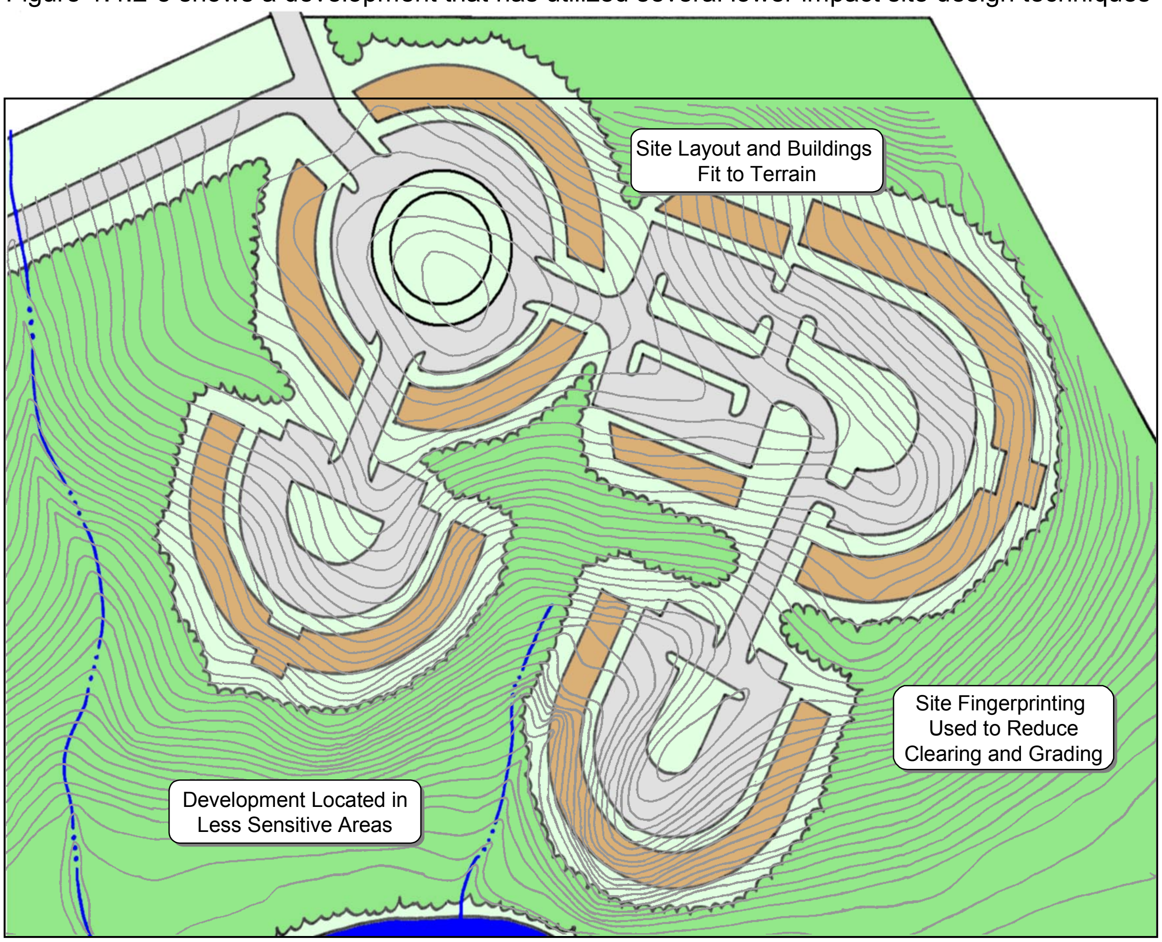


Figure 1.4.2-8 Development Design Utilizing Several Lower Impact Site Design Techniques

Better Site Design Practice #6:

Fit Design to the Terrain

Lower Impact
Site Design Techniques

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	USING THIS PRACTICE
• 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	☑ 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 1.4.2-9 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 1.4.2-10. 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 1.4.2-11). 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.

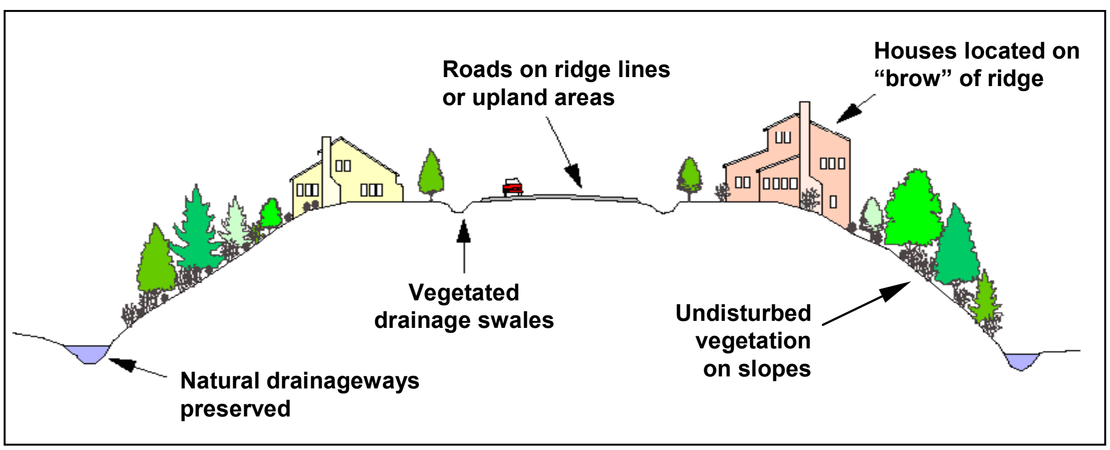


Figure 1.4.2-9 Preserving the Natural Topography of the Site
(Adapted from Sykes, 1989)

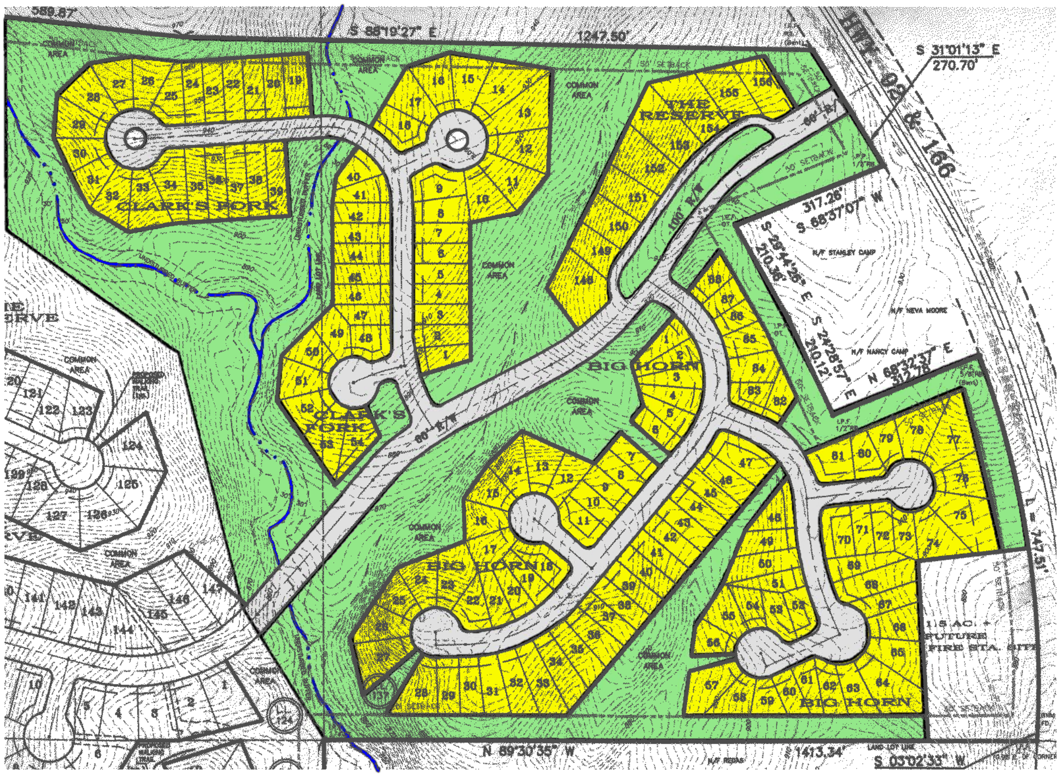


Figure 1.4.2-10 Subdivision Design for Hilly or Steep Terrain Utilizes Branching Streets From Collectors that Preserves Natural Drainageways and Stream Corridors

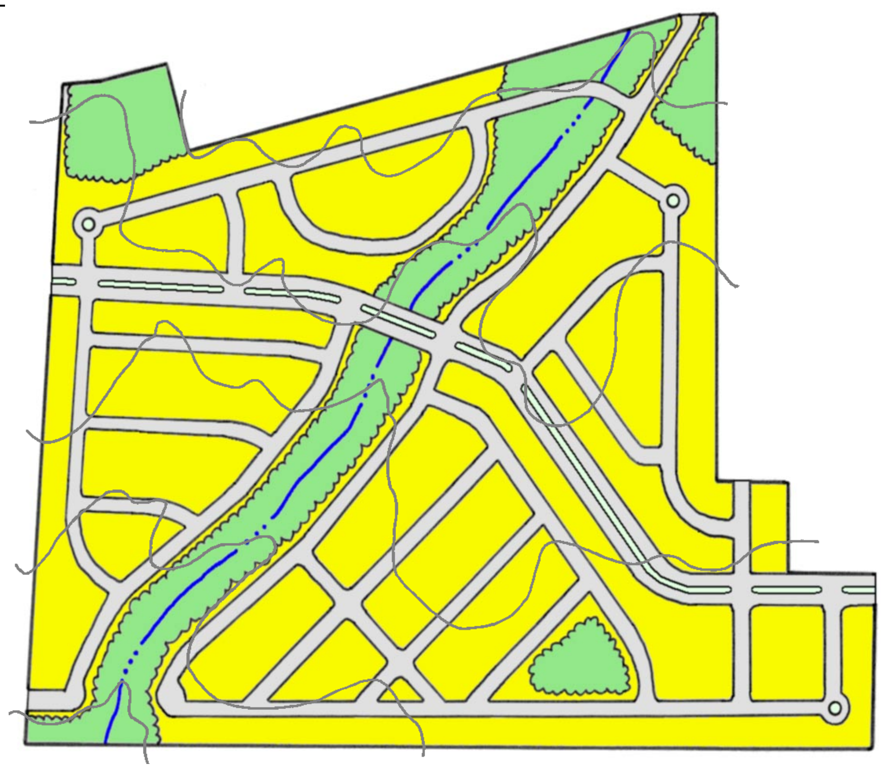


Figure 1.4.2-11 A Subdivision Design for Flat Terrain Uses a Fluid Grid Layout that is Interrupted by the Stream Corridor

Better Site Design Practice #7:

Locate Development in Less Sensitive Areas

Lower Impact
Site Design Techniques

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	USING THIS PRACTICE
• 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	☑ 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.

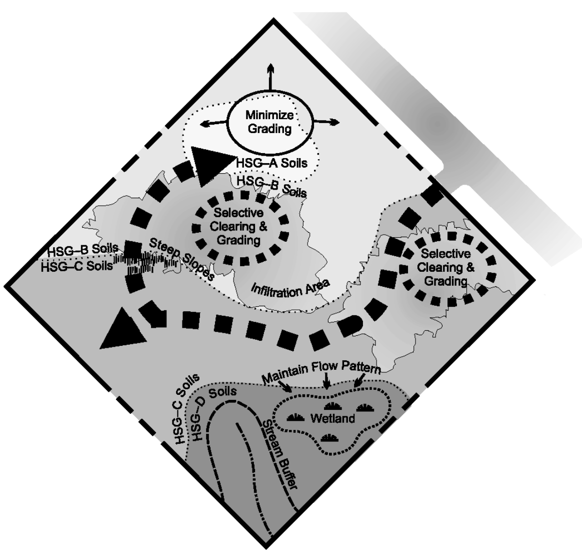


Figure 1.4.2-12 Guiding Development to Less Sensitive Areas of a Site
(Source: Prince George's County, MD, 1999)

Figure 1.4.2-12 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.

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	USING THIS PRACTICE
• Preserves more undisturbed natural areas on a development site • Techniques can be used to help protect natural conservation areas and other site features	☑ 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 1.4.2-13 and 1.4.2-14.
- Fitting the site design to the terrain.
- Using special procedures and equipment which reduce land disturbance.

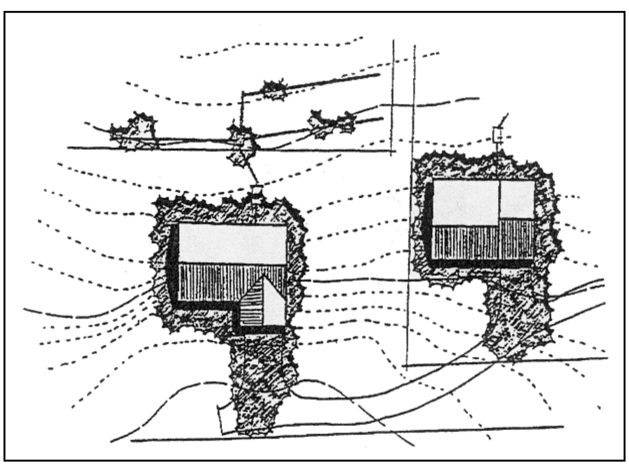


Figure 1.4.2-13 Establishing Limits of Clearing (Source: DDNREC, 1997)

Figure 1.4.2-14 Example of Site Footprinting

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	USING THIS PRACTICE
• 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	☑ 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 1.4.2-15 and 1.4.2-16 show examples of open space developments.

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. 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.

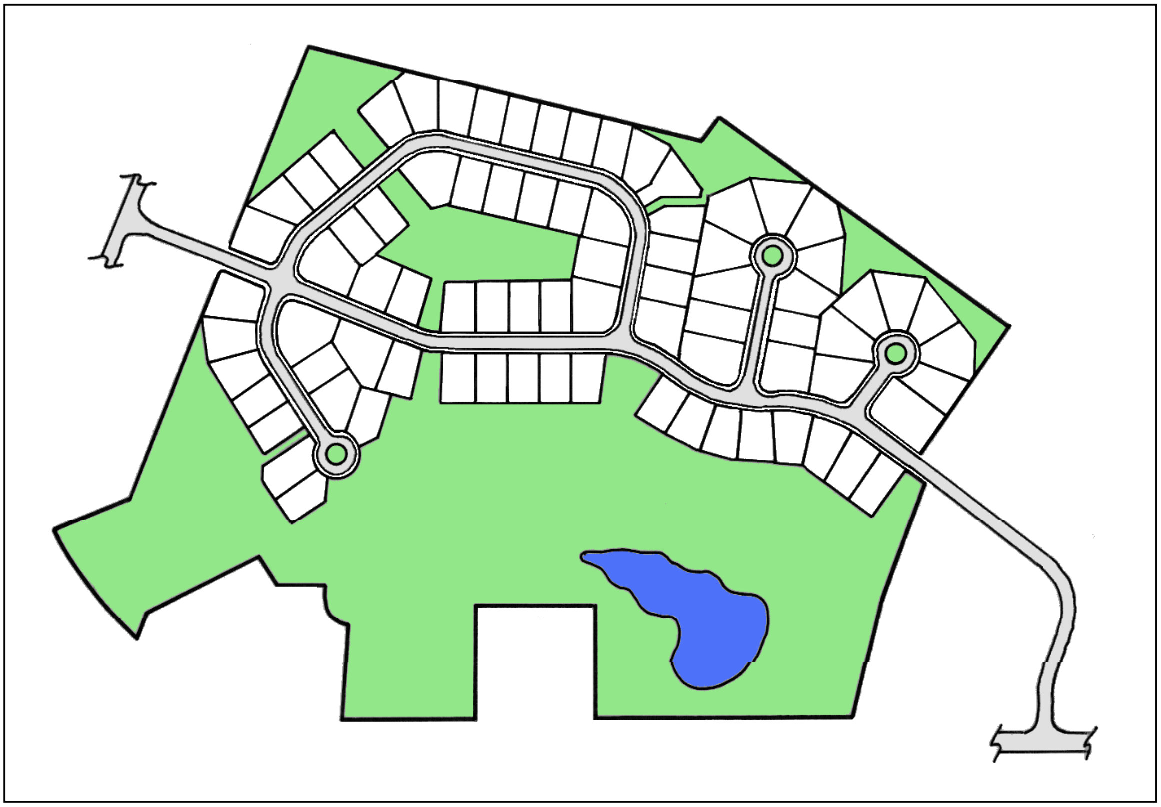


Figure 1.4.2-15 Open Space Subdivision Site Design Example

Figure 1.4.2-16 Aerial View of an Open Space Subdivision

Better Site Design Practice #10:

Consider Creative Development Design

Lower Impact
Site Design Techniques

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	USING THIS PRACTICE
• 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	☑ 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

Many of these changes are useful in reducing the amount of impervious cover on a development site (see Better Site Design Practices #11 through #16).

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.

1.4.2.3 Reduction of Impervious Cover

The level of impervious cover, i.e. rooftops, parking lots, roadways, sidewalks and other surfaces that do not allow rainfall to infiltrate into the soil, is an essential factor to consider in better site design for stormwater management. Increased impervious cover means increased stormwater generation and increased pollutant loadings.

Thus by reducing the area of total impervious surface on a site, a site designer can directly reduce the volume of stormwater runoff and associated pollutants that are generated. It can also reduce the size and cost of necessary infrastructure for stormwater drainage, conveyance, and control and treatment. Some of the ways that impervious cover can be reduced in a development include:

- 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

Figure 1.4.2-17 shows an example of a residential subdivision that employed several of these principles to reduce the overall imperviousness of the development. The next several pages cover these methods in more detail.

Figure 1.4.2-17 Example of Reducing Impervious Cover (clockwise from upper left): (a) Cul-de-sac with Landscaped Island; (b) Narrower Residential Street; (c) Landscape Median in Roadway; and (d) "Green" Parking Lot with Landscaped Islands

Better Site Design Practice #11:

Reduce Roadway Lengths and Widths

Reduction of Impervious Cover

Description: Roadway lengths and widths should be minimized on a development site where possible to reduce overall imperviousness.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated • Reduces the costs associated with road construction and maintenance	☑ Consider different site and road layouts that reduce overall street length ☑ Minimize street width by using narrower street designs

Discussion

The use of alternative road layouts that reduce the total linear length of roadways can significantly reduce overall imperviousness of a development site. Site designers are encouraged to analyze different site and roadway layouts to see if they can reduce overall street length. The length of local cul-de-sacs and cross streets should be shortened to a maximum of 200 ADT (average trips per day) to minimize traffic and road noise so that shorter setbacks may be employed.

In addition, residential streets and private streets within commercial and other development should be designed for the minimum required pavement width needed to support travel lanes, on-street parking, and emergency access. Figure 1.4.2-18 shows a number of different options for narrower street designs. Many times on-street parking can be reduced to one lane or eliminated on local access roads with less than 200 ADT on cul-de-sac streets and 400 ADT on two-way loops. One-way single-lane loop roads are another way to reduce the width of lower traffic streets.

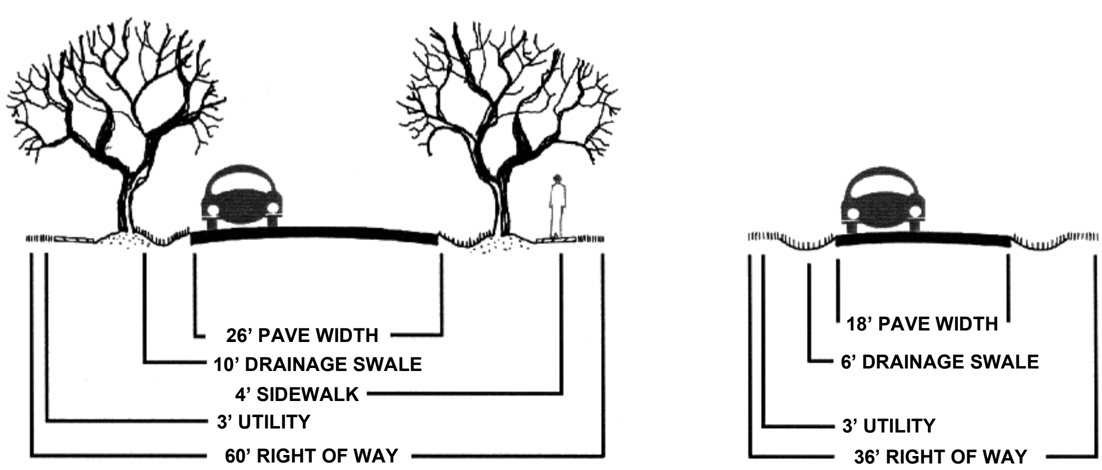


Figure 1.4.2-18 Potential Design Options for Narrower Roadway Widths
(Source: VPISU, 2000)

Better Site Design Practice #12:

Reduce Building Footprints

Description: The impervious footprint of commercial buildings and residences can be reduced by using alternate or taller buildings while maintaining the same floor to area ratio.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated	☑ Use alternate or taller building designs to reduce the impervious footprint of buildings

Discussion

In order to reduce the imperviousness associated with the footprint and rooftops of buildings and other structures, alternative and/or vertical (taller) building designs should be considered. Consolidate functions and buildings, as required, or segment facilities to reduce the footprint of individual structures. Figure 1.4.2-19 shows the reduction in impervious footprint by using a taller building design.

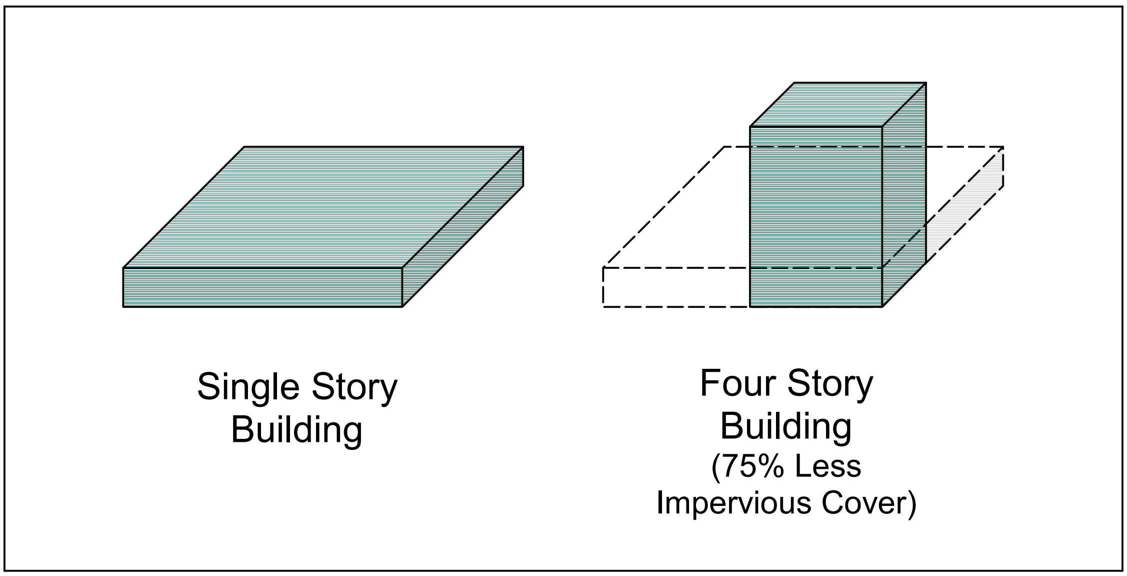


Figure 1.4.2-19 Building Up Rather Than Out Can Reduce the Amount of Impervious Cover

Better Site Design Practice #13:
Reduce the Parking Footprint

Reduction of Impervious Cover

Description: Reduce the overall imperviousness associated with parking lots by providing compact car spaces, minimizing stall dimensions, incorporating efficient parking lanes, parking decks, and using porous paver surfaces or porous concrete in overflow parking areas where feasible and possible.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated	☑ Reduce the number of parking spaces ☑ Minimize stall dimensions ☑ Consider parking structures and shared parking ☑ Use alternative porous surface for overflow areas

Discussion

Setting maximums for parking spaces, minimizing stall dimensions, using structured parking, encouraging shared parking and using alternative porous surfaces can all reduce the overall parking footprint and site imperviousness.

Many parking lot designs result in far more spaces than actually required. This problem is exacerbated by a common practice of setting parking ratios to accommodate the highest hourly parking during the peak season. By determining average parking demand instead, a lower maximum number of parking spaces can be set to accommodate most of the demand. Table 1.4.2-2 provides examples of conventional parking requirements and compares them to average parking demand.

Table 1.4.2-2 Conventional Minimum Parking Ratios (Source: ITE, 1987; Smith, 1984; Wells, 1994)

Land Use	Parking Requirement		Actual Average Parking Demand
	Parking Ratio	Typical Range	
Single family homes	2 spaces per dwelling unit	1.5–2.5	1.11 spaces per dwelling unit
Shopping center	5 spaces per 1000 ft^2 GFA	4.0–6.5	3.97 per 1000 ft^2 GFA
Convenience store	3.3 spaces per 1000 ft^2 GFA	2.0–10.0	--
Industrial	1 space per 1000 ft^2 GFA	0.5–2.0	1.48 per 1000 ft^2 GFA
Medical/ dental office	5.7 spaces per 1000 ft^2 GFA	4.5–10.0	4.11 per 1000 ft^2 GFA
GFA = Gross floor area of a building without storage or utility spaces.			

Another technique to reduce the parking footprint is to minimize the dimensions of the parking spaces. This can be accomplished by reducing both the length and width of the parking stall. Parking stall dimensions can be further reduced if compact spaces are provided. While the trend toward larger sport utility vehicles (SUVs) is often cited as a barrier to implementing stall minimization techniques, stall width requirements in most local parking codes are much larger than the widest SUVs.

Structured parking decks are one method to significantly reduce the overall parking footprint by minimizing surface parking. Figure 1.4.2-20 shows a parking deck used for a commercial development.

Figure 1.4.2-20 Structured Parking at an Office Park Development

Shared parking in mixed-use areas and structured parking are techniques that can further reduce the conversion of land to impervious cover. A shared parking arrangement could include usage of the same parking lot by an office space that experiences peak parking demand during the weekday with a church that experiences parking demands during the weekends and evenings.

Utilizing alternative surfaces such as porous pavers or porous concert is an effective way to reduce the amount of runoff generated by parking lots. They can replace conventional asphalt or concrete in both new developments and redevelopment projects. Figure 1.4.2-21 is an example of porous paver used at an overflow lot. Alternative pavers can also capture and treat runoff from other site areas. However, porous pavement surfaces generally require proper installation and more maintenance than conventional asphalt or concrete. For more specific information using these alternative surfaces, see subsections 3.3.8 (Modular Porous Paver Systems) and 3.3.9 (Porous Concrete).

Figure 1.4.2-21 Grass Paver Surface Used for Parking

Better Site Design Practice #14:

Reduce Setbacks and Frontages

Reduction of Impervious Cover

Description: Use smaller front and side setbacks and narrower frontages to reduce total road length and driveway lengths.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated	☑ Reduce building and home front and side setbacks ☑ Consider narrower frontages

Discussion

Building and home setbacks should be shortened to reduce the amount of impervious cover from driveways and entry walks. A setback of 20 feet is more than sufficient to allow a car to park in a driveway without encroaching into the public right of way, and reduces driveway and walk pavement by more than 30% compared with a setback of 30 feet (see Figure 1.4.2-22).

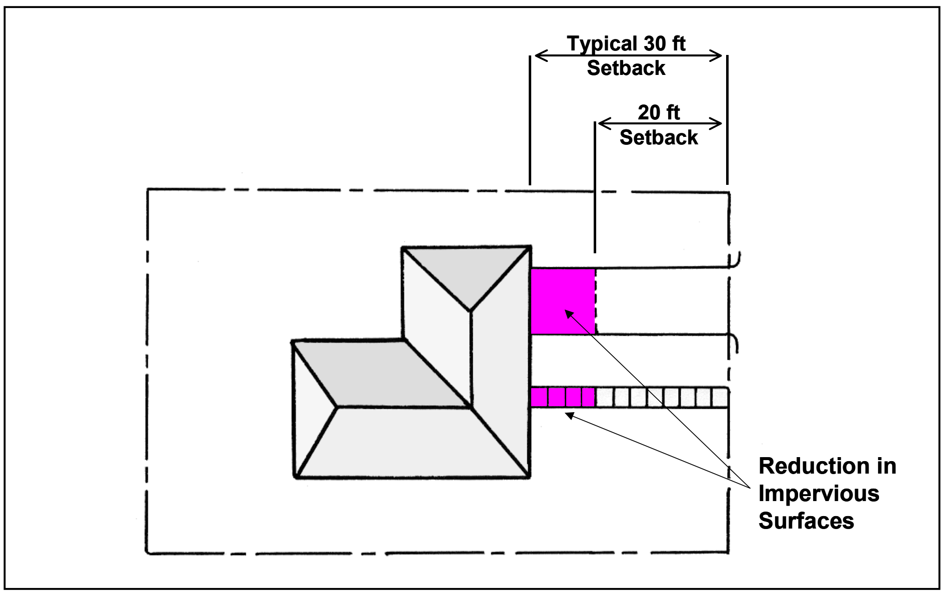
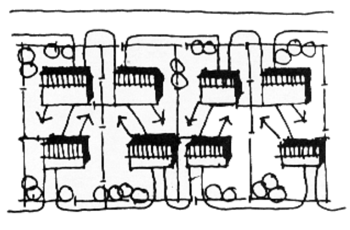


Figure 1.4.2-22 Reduced Impervious Cover by Using Smaller Setbacks
(Adapted from: MPCA, 1989)

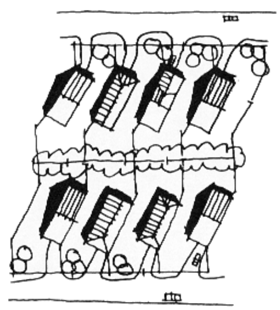
Further, reducing side yard setbacks and using narrower frontages can reduce total street length, especially important in cluster and open space designs. Figure 1.4.2-23 shows residential examples of reduced front and side yard setbacks and narrow frontages.

Flexible lot shapes and setback and frontage distances allow site designers to create attractive and unique lots that provide homeowners with enough space while allowing for the preservation of natural areas in a residential subdivison. Figure 1.4.2-24 illustrates various nontraditional lot designs.

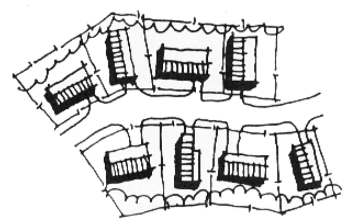
Figure 1.4.2-23 Examples of Reduced Frontages and Side Yard Setbacks



Zipper Lots



Angled Z-Lots



Alternative Lot Widths

Figure 1.4.2-24 Nontraditional Lot Designs
(Source: ULI, 1992)

Better Site Design Practice #15:

Use Fewer or Alternative Cul-de-Sacs

Reduction of Impervious Cover

Description: Minimize the number of residential street cul-de-sacs and incorporate landscaped areas to reduce their impervious cover. The radius of cul-de-sacs should be the minimum required to accommodate emergency and maintenance vehicles. Alternative turnarounds should also be considered.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated	☑ Consider alternative cul-de-sac designs

Discussion

Alternative turnarounds are designs for end-of-street vehicle turnarounds that replace cul-de-sacs and reduce the amount of impervious cover created in developments. Cul-de-sacs are local access streets with a closed circular end that allows for vehicle turnarounds. Many of these cul-de-sacs can have a radius of more than 40 feet. From a stormwater perspective, cul-de-sacs create a huge bulb of impervious cover, increasing the amount of runoff. For this reason, reducing the size of cul-de-sacs through the use of alternative turnarounds or eliminating them altogether can reduce the amount of impervious cover created at a site.

Numerous alternatives create less impervious cover than the traditional 40-foot cul-de-sac. These alternatives include reducing cul-de-sacs to a 30-foot radius and creating hammerheads, loop roads, and pervious islands in the cul-de-sac center (see Figure 1.4.2-25).

Sufficient turnaround area is a significant factor to consider in the design of cul-de-sacs.
In particular, the types of vehicles entering into the cul-de-sac should be considered. Fire trucks, service vehicles and school buses are often cited as needing large turning radii. However, some fire trucks are designed for smaller turning radii. In addition, many newer large service vehicles are designed with a tri-axle (requiring a smaller turning radius) and many school buses usually do not enter individual cul-de-sacs.

Implementing alternative turnarounds will require addressing local regulations and marketing issues. Communities may have specific design criteria for cul-de-sacs and other alternative turnarounds that need to be modified.

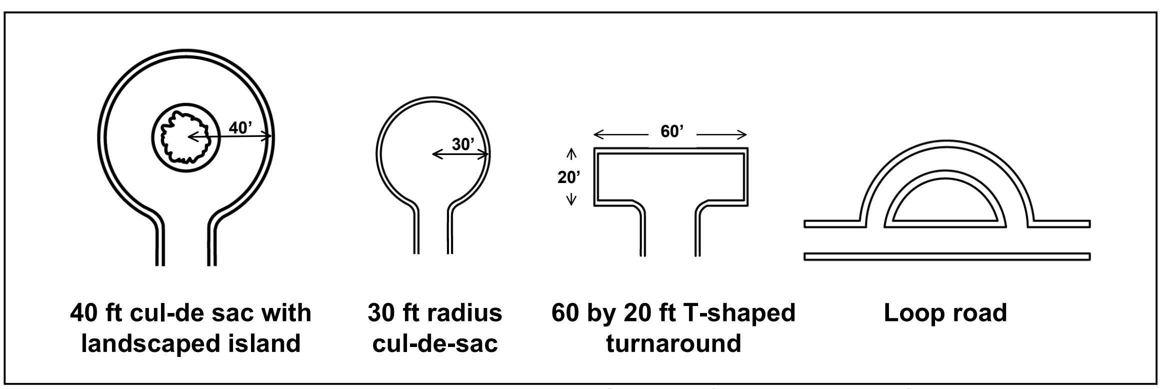


Figure 1.4.2-25 Four Turnaround Options for Residential Streets
(Source: Schueler, 1995)

Better Site Design Practice #16:
Create Parking Lot Stormwater "Islands"

Reduction of Impervious Cover

Description: Provide stormwater treatment for parking lot runoff using bioretention areas, filter strips, and/or other practices that can be integrated into required landscaping areas and traffic islands.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated • Provides an opportunity for the siting of structural control facilities • Trees in parking lots provide shading for cars and are more visually appealing	☑ Integrate porous areas such as landscaped islands, swales, filter strips and bioretention areas in a parking lot design.

Discussion

Parking lots should be designed with landscaped stormwater management "islands" which reduce the overall impervious cover of the lot as well as provide for runoff treatment and control in stormwater facilities.

When possible, expanses of parking should be broken up with landscaped islands which include shade trees and shrubs. Fewer large islands will sustain healthy trees better than more numerous very small islands. The most effective solutions in designing for tree roots in parking lots use a long planting strip at least 8 feet wide, constructed with sub-surface drainage and compaction resistant soil.

Structural control facilities such as filter strips, dry swales and bioretention areas can be incorporated into parking lot islands. Stormwater is directed into these landscaped areas and temporarily detained. The runoff then flows through or filters down through the bed of the facility and is infiltrated into the subsurface or collected for discharge into a stream or another stormwater facility. These facilities can be attractively integrated into landscaped areas and can be maintained by commercial landscaping firms. For detailed design specifications of filter strips, enhanced swales and bioretention areas, refer to Chapter 3.

Figure 1.4.2-26 Parking Lot Stormwater "Island"

1.4.2.4 Utilization of Natural Features for Stormwater Management

Traditional stormwater drainage design tends to ignore and replace natural drainage patterns and often results in overly efficient hydraulic conveyance systems. Structural stormwater controls are costly and often can require high levels of maintenance for optimal operation. Through use of natural site features and drainage systems, careful site design can reduce the need and size of structural conveyance systems and controls.

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 utilize the natural and/or nonstructural drainage system and improve the effectiveness of natural systems rather than to ignore or replace them. These natural systems typically require low or no maintenance and will continue to function many years into the future.

Some of the methods of incorporating natural features into an overall stormwater management site plan include the following practices:

- Use Buffers and Undisturbed Areas
- Use Natural Drainageways Instead of Storm Sewers
- Use Vegetated Swales Instead of Curb and Gutter
- Drain Runoff to Pervious Areas

The following pages cover each practice in more detail.

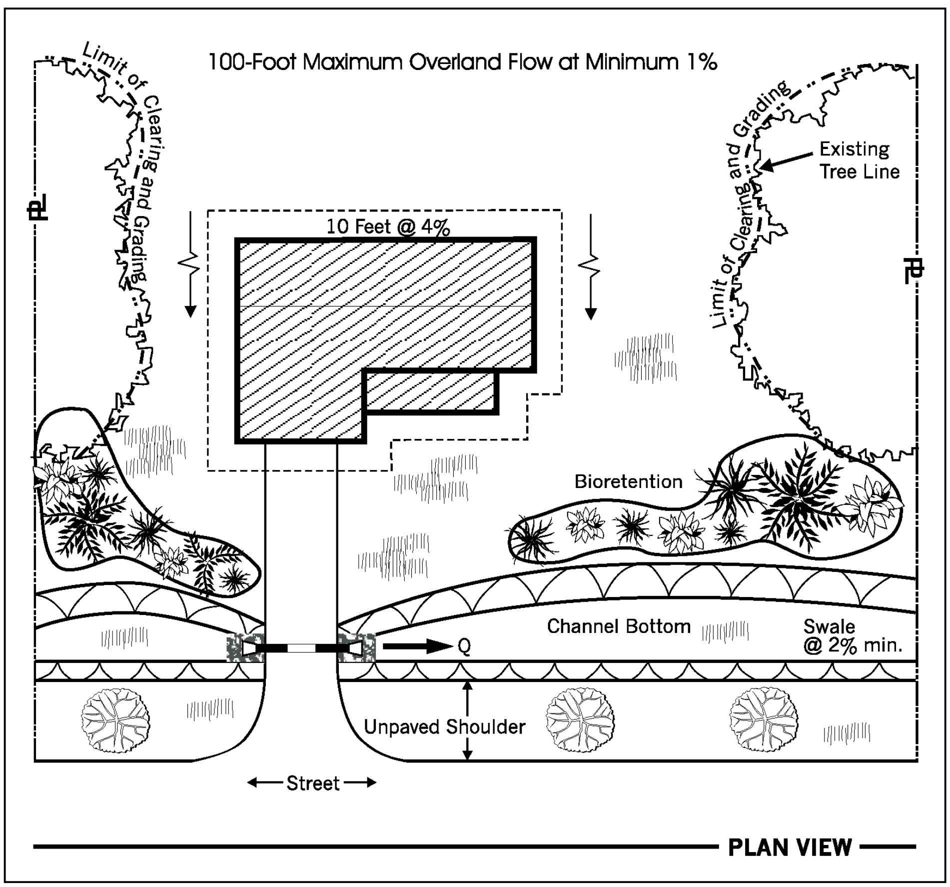


Figure 1.4.2-27 Residential Site Design Using Natural Features for Stormwater Mangement
(Source: Prince George's County, MD, 1999)

Better Site Design Practice #17:

Use Buffers and Undisturbed Areas

Description: Undisturbed natural areas such as forested conservation areas and stream buffers can be used to treat and control stormwater runoff from other areas of the site with proper design.

KEY BENEFITS	USING THIS PRACTICE
• Riparian buffers and undisturbed vegetated areas can be used to filter and infiltrate stormwater runoff • Natural depressions can provide inexpensive storage and detention of stormwater flows • A stormwater site design credit can be taken if allowed by the local review authority (see subsection 1.4.4)	☑ Direct runoff towards buffers and undisturbed areas using a level spreader to ensure sheet flow ☑ Utilize natural depressions for runoff storage

Discussion

Runoff can be directed towards riparian buffers and other undisturbed natural areas delineated in the initial stages of site planning to infiltrate runoff, reduce runoff velocity and remove pollutants. Natural depressions can be used to temporarily store (detain) and infiltrate water, particularly in areas with porous (hydrologic soil group A and B) soils.

The objective in utilizing natural areas for stormwater infiltration is to intercept runoff before it has become substantially concentrated and then distribute this flow evenly (as sheet flow) to the buffer or natural area. This can typically be accomplished using a level spreader, as seen in Figure 1.4.2-28. A mechanism for the bypass of higher flow events should be provided to reduce erosion or damage to a buffer or undisturbed natural area.

Carefully constructed berms can be placed around natural depressions and below undisturbed vegetated areas with porous soils to provide for additional runoff storage and/or infiltration of flows.

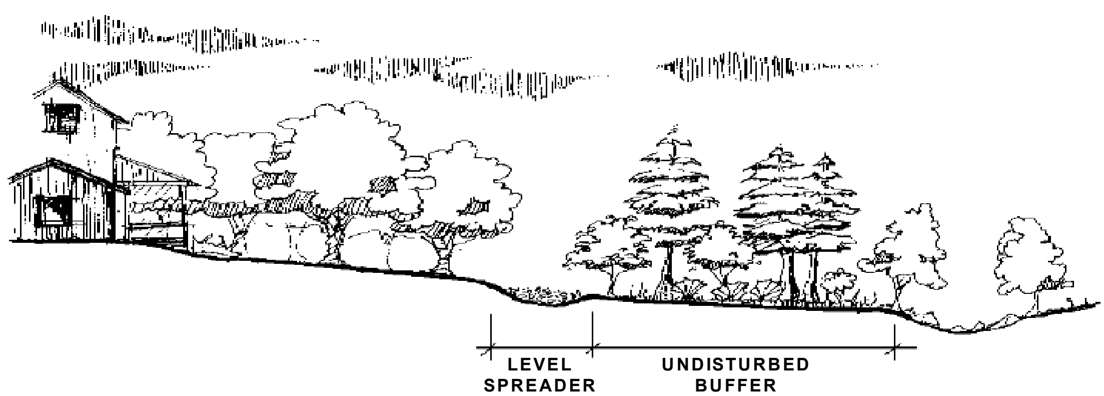


Figure 1.4.2-28 Use of a Level Spreader with a Riparian Buffer
(Adapted from NCDENR, 1998)

Better Site Design Practice #18:

Use Natural Drainageways Instead of Storm Sewers

Utilization of Natural Features for Stormwater Management

Description: The natural drainage paths of a site can be used instead of constructing underground storm sewers or concrete open channels.

KEY BENEFITS	USING THIS PRACTICE
• Use of natural drainageways reduces the cost of constructing storm sewers or other conveyances, and may reduce the need for land disturbance and grading • Natural drainage paths are less hydraulically efficient than man-made conveyances, resulting in longer travel times and lower peak discharges • Can be combined with buffer systems to allow for stormwater filtration and infilration	☑ Preserve natural flow paths in the site design ☑ Direct runoff to natural drainageways, ensuring that peak flows and velocities will not cause channel erosion

Discussion

Structural drainage systems and storm sewers are designed to be hydraulically efficient in removing stormwater from a site. However, in doing so these systems tend to increase peak runoff discharges, flow velocities and the delivery of pollutants to downstream waters. An alternative is the use of natural drainageways and vegetated swales (where slopes and soils permit) to carry stormwater flows to their natural outlets, particularly for low-density development and residential subdivisions.

The use of natural open channels allows for more storage of stormwater flows on-site, lower stormwater peak flows, a reduction in erosive runoff velocities, infiltration of a portion of the runoff volume, and the capture and treatment of stormwater pollutants. It is critical that natural drainageways be protected from higher post-development flows by applying downstream channel protection methods (including the CP_v criteria) to prevent erosion and degradation.

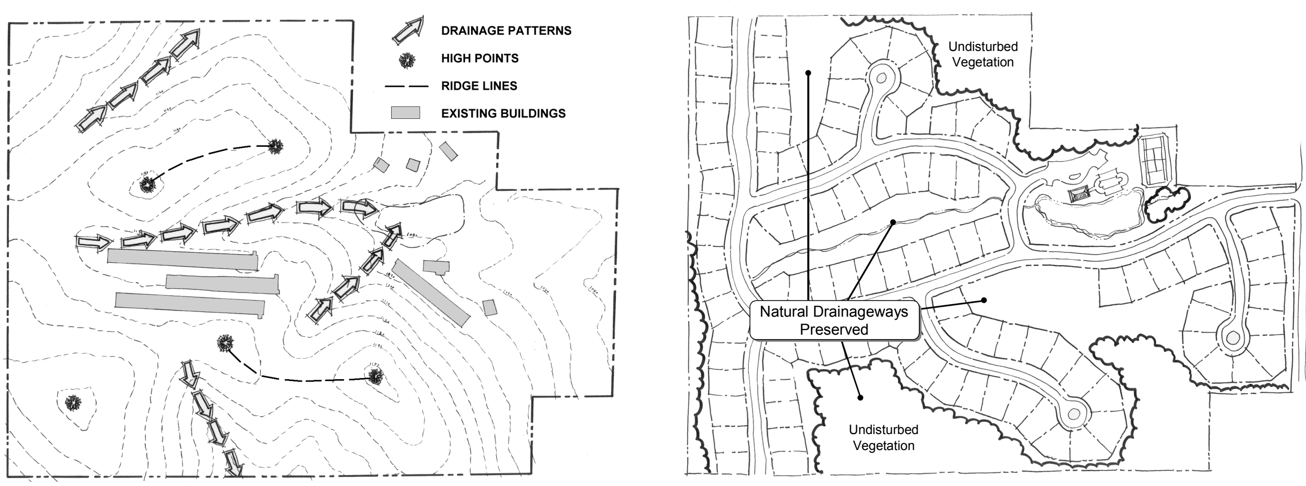


Figure 1.4.2-29 Example of a Subdivision Using Natural Drainageways for Stormwater Conveyance and Management

Better Site Design Practice #19:

Use Vegetated Swales Instead of Curb and Gutter

Description: Where density, topography, soils, slope, and safety issues permit, vegetated open channels can be used in the street right-of-way to convey and treat stormwater runoff from roadways.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the cost of road and storm sewer construction • Provides for some runoff storage and infiltration, as well as treatment of stormwater • A stormwater site design credit can be taken if allowed by the local review authority (see subsection 1.4.4)	☑ Use vegetated open channels (enhanced wet or dry swales or grass channels) in place of curb and gutter to convey and treat stormwater runoff

Discussion

Curb and gutter and storm drain systems allow for the quick transport of stormwater, which results in increased peak flow and flood volumes and reduced runoff infiltration. Curb and gutter systems also do not provide treatment of stormwater that is often polluted from vehicle emissions, pet waste, lawn runoff and litter.

Open vegetated channels along a roadway (see Figure 1.4.2-30) remove pollutants by allowing infiltration and filtering to occur, unlike curb and gutter systems which move water with virtually no treatment. Engineering techniques have advanced the roadside ditches of the past, which suffered from erosion, standing water and break up of the road edge. Grass channels and enhanced dry swales are two such alternatives and with proper installation under the right site conditions, they are excellent methods for treating stormwater on-site. In addition, open vegetated channels can be less expensive to install than curb and gutter systems. Further design information and specifications for grass channels and enhanced swales can be found in Chapter 3.

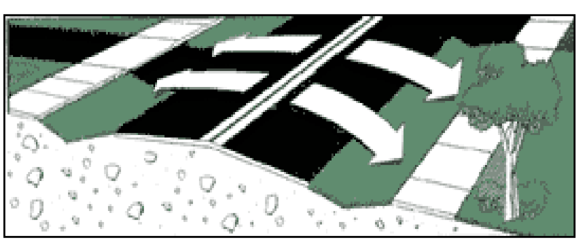


Figure 1.4.2-30 Using Vegetated Swales Instead of Curb and Gutter

Better Site Design Practice #20:

Drain Runoff to Pervious Areas

Description: Where possible, direct runoff from impervious areas such as rooftops, roadways and parking lots to pervious areas, open channels or vegetated areas to provide for water quality treatment and infiltration. Avoid routing runoff directly to the structural stormwater conveyance system.

KEY BENEFITS	USING THIS PRACTICE
• Sending runoff to pervious vegetated areas increases overland flow time and reduces peak flows • Vegetated areas can often filter and infiltrate stormwater runoff • A stormwater site design credit can be taken if allowed by the local review authority (see subsection 1.4.4)	☑ Minimize directly connected impervious areas and drain runoff as sheet flow to pervious vegetated areas

Discussion

Stormwater quantity and quality benefits can be achieved by routing the runoff from impervious areas to pervious areas such as lawns, landscaping, filter strips and vegetated channels. Much like the use of undisturbed buffers and natural areas (Better Site Design Practice #17), revegetated areas such as lawns and engineered filter strips and vegetated channels can act as biofilters for stormwater runoff and provide for infiltration in porous (hydrologic group A and B) soils. In this way, the runoff is “disconnected” from a hydraulically efficient structural conveyance such as a curb and gutter or storm drain system.

Some of the methods for disconnecting impervious areas include:

- Designing roof drains to flow to vegetated areas
- Directing flow from paved areas such as driveways to stabilized vegetated areas
- Breaking up flow directions from large paved surfaces (see Figure 1.4.2-31)
- Carefully locating impervious areas and grading landscaped areas to achieve sheet flow runoff to the vegetated pervious areas

For maximum benefit, runoff from impervious areas to vegetated areas must occur as sheet flow and vegetation must be stabilized. See Chapter 3 for more design information and specifications on filter strips and vegetated channels.

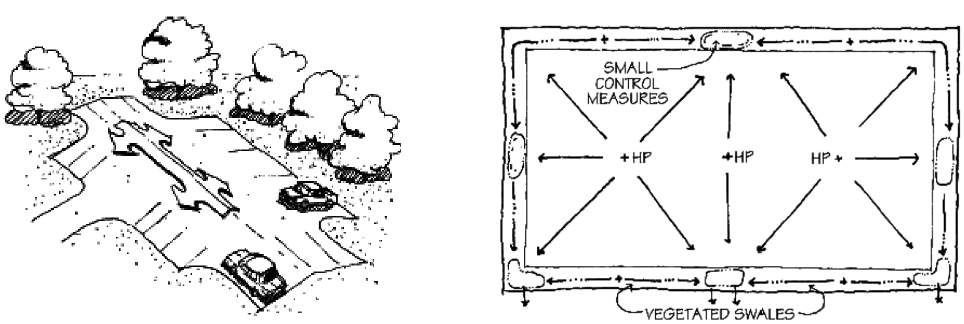


Figure 1.4.2-31 Design Paved Surfaces to Disperse Flow to Vegetated Areas
Source: NCDENR, 1998

1.4.3 Better Site Design Examples

1.4.3.1 Residential Subdivision Example 1

A typical residential subdivision design on a parcel is shown in Figure 1.4.3-1 (a). The entire parcel except for the subdivision amenity area (clubhouse and tennis courts) is used for lots. The entire site is cleared and mass graded, and no attempt is made to fit the road layout to the existing topography. Because of the clearing and grading, all of the existing tree cover and vegetation and topsoil are removed dramatically altering both the natural hydrology and drainage of the site. The wide residential streets create unnecessary impervious cover and a curb and gutter system that carries stormwater flows to the storm sewer system. No provision for non-structural stormwater treatment is provided on the subdivision site.

A residential subdivision employing stormwater better site design practices is presented in Figure 1.4.3-1 (b). This subdivision configuration preserves a quarter of the property as undisturbed open space and vegetation. The road layout is designed to fit the topography of the parcel, following the high points and ridgelines. The natural drainage patterns of the site are preserved and are utilized to provide natural stormwater treatment and conveyance. Narrower streets reduce impervious cover and grass channels provide for treatment and conveyance of roadway and driveway runoff. Landscaped islands at the ends of cul-de-sacs also reduce impervious cover and provide stormwater treatment functions. When constructing and building homes, only the building envelopes of the individual lots are cleared and graded, further preserving the natural hydrology of the site.

1.4.3.2 Residential Subdivision Example 2

Another typical residential subdivision design is shown in Figure 1.4.3-2 (a). Most of this site is cleared and mass graded, with the exception of a small riparian buffer along the large stream at the right boundary of the property. Almost no buffer was provided along the small stream that runs through the middle of the property. In fact, areas within the 100-year floodplain were cleared and filled for home sites. As is typical in many subdivision designs, this one has wide streets for on-street parking and large cul-de-sacs.

The better site design subdivision can be seen in Figure 1.4.3-2 (b). This subdivision layout was designed to conform to the natural terrain. The street pattern consists of a wider main thoroughfare that winds through the subdivision along the ridgeline. Narrower loop roads branch off of the main road and utilize landscaped islands. Large riparian buffers are preserved along both the small and large streams. The total undisturbed conservation area is close to one-third of the site.

1.4.3.3 Commercial Development Example

Figure 1.4.3-3 (a) shows a typical commercial development containing a supermarket, drugstore, smaller shops and a restaurant on an outlot. The majority of the parcel is a concentrated parking lot area. The only pervious area is a small replanted vegetation area acting as a buffer between the shopping center and adjacent land uses. Stormwater quality and quantity control are provided by a wet extended detention pond in the corner of the parcel.

A better site design commercial development can be seen in Figure 1.4.3-3 (b). Here the retail buildings are dispersed on the property, providing more of an “urban village” feel with pedestrian access between the buildings. The parking is broken up, and bioretention areas for stormwater treatment are built into parking lot islands. A large bioretention area which serves as open green space is located at the main entrance to the shopping center. A larger undisturbed buffer has been preserved on the site. Because of the bioretention areas and buffer provide water quality treatment, only a dry extended detention basin is needed for water quantity control.

1.4.3.4 Office Park Example

An office park with a conventional design is shown in Figure 1.4.3-4 (a). Here the site has been graded to fit the building layout and parking area. All of the vegetated areas of this site are replanted areas.

The better site design layout, presented in Figure 1.4.3-4 (b), preserves undisturbed vegetated buffers and open space areas on the site. Both the parking areas and buildings have been designed to fit the natural terrain of the site. In addition, a modular porous paver system is used for the overflow parking areas.

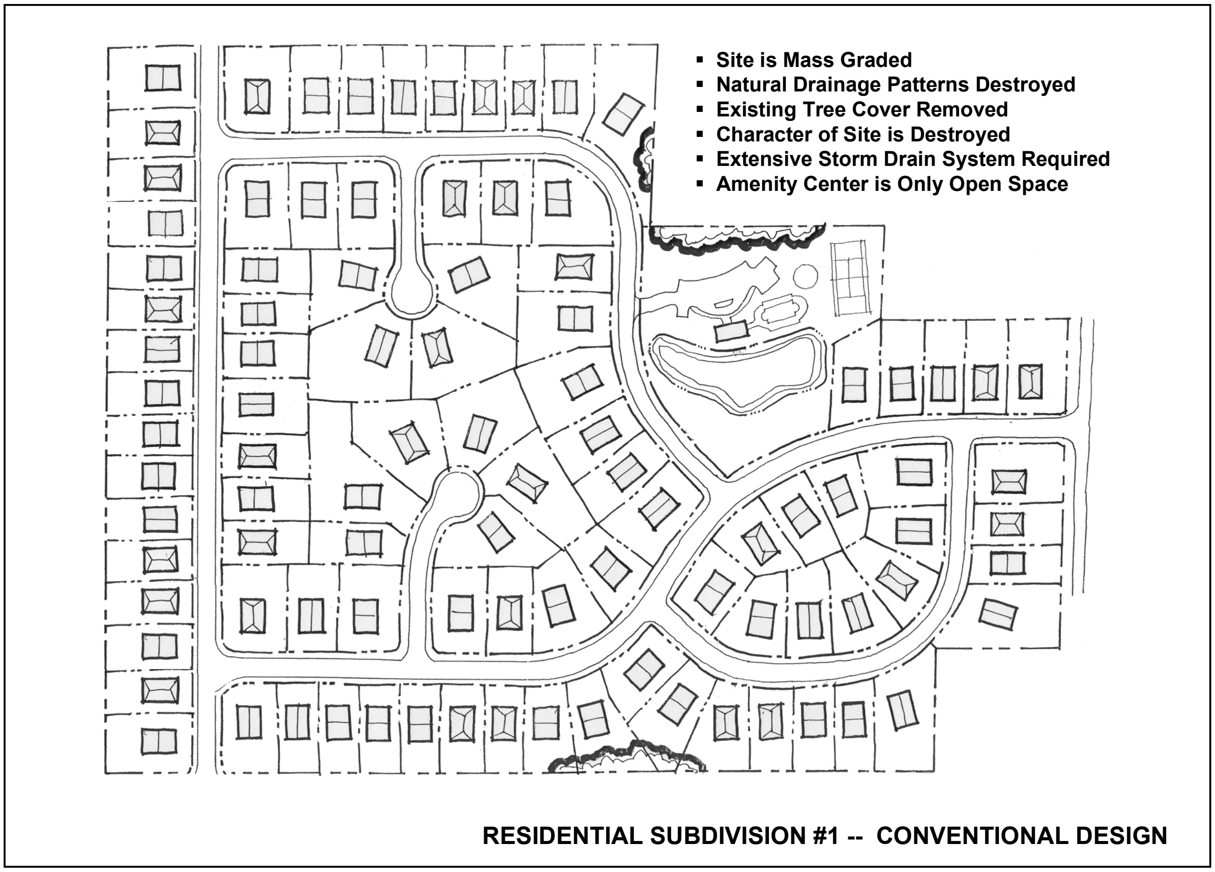


Figure 1.4.3-1 Comparison of a Traditional Residential Subdivision Design (above) with an Innovative Site Plan Developed Using Better Site Design Practices (below).

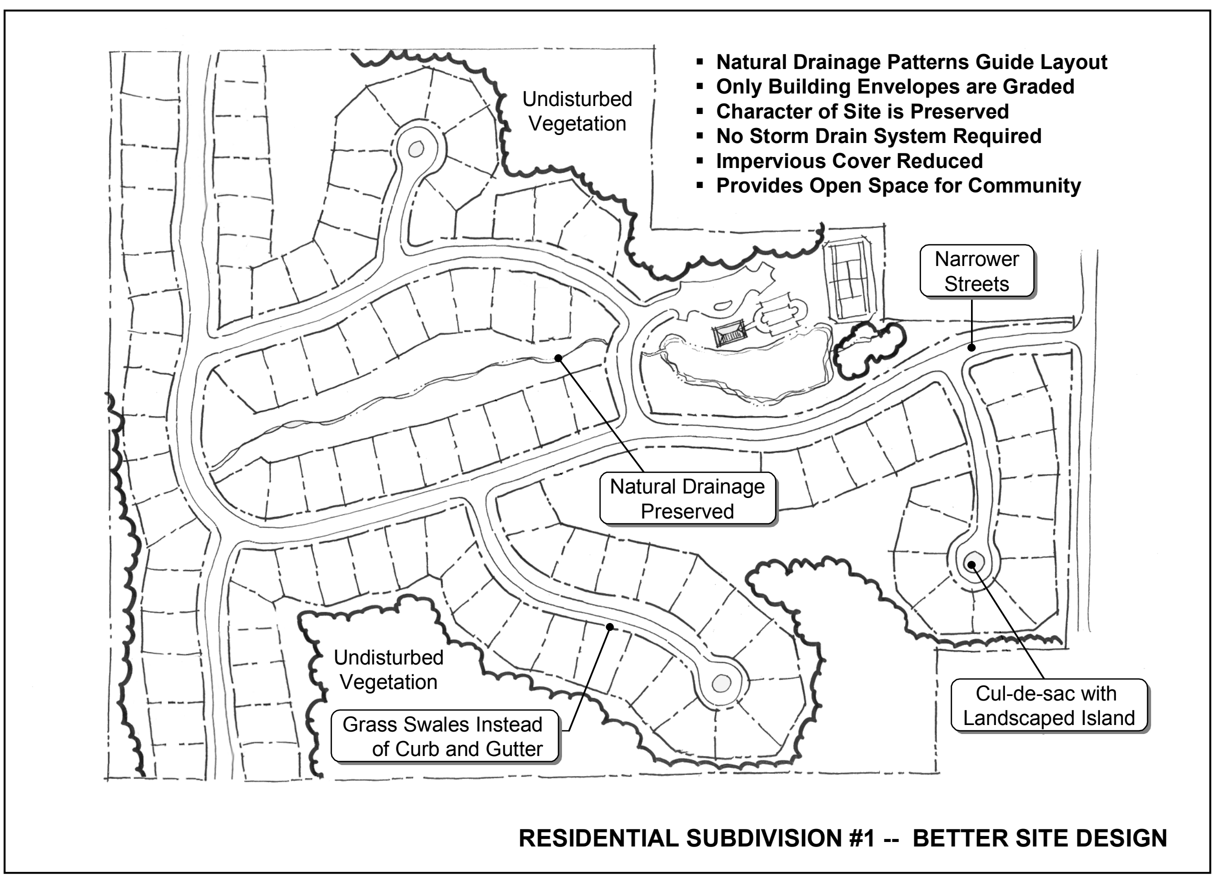
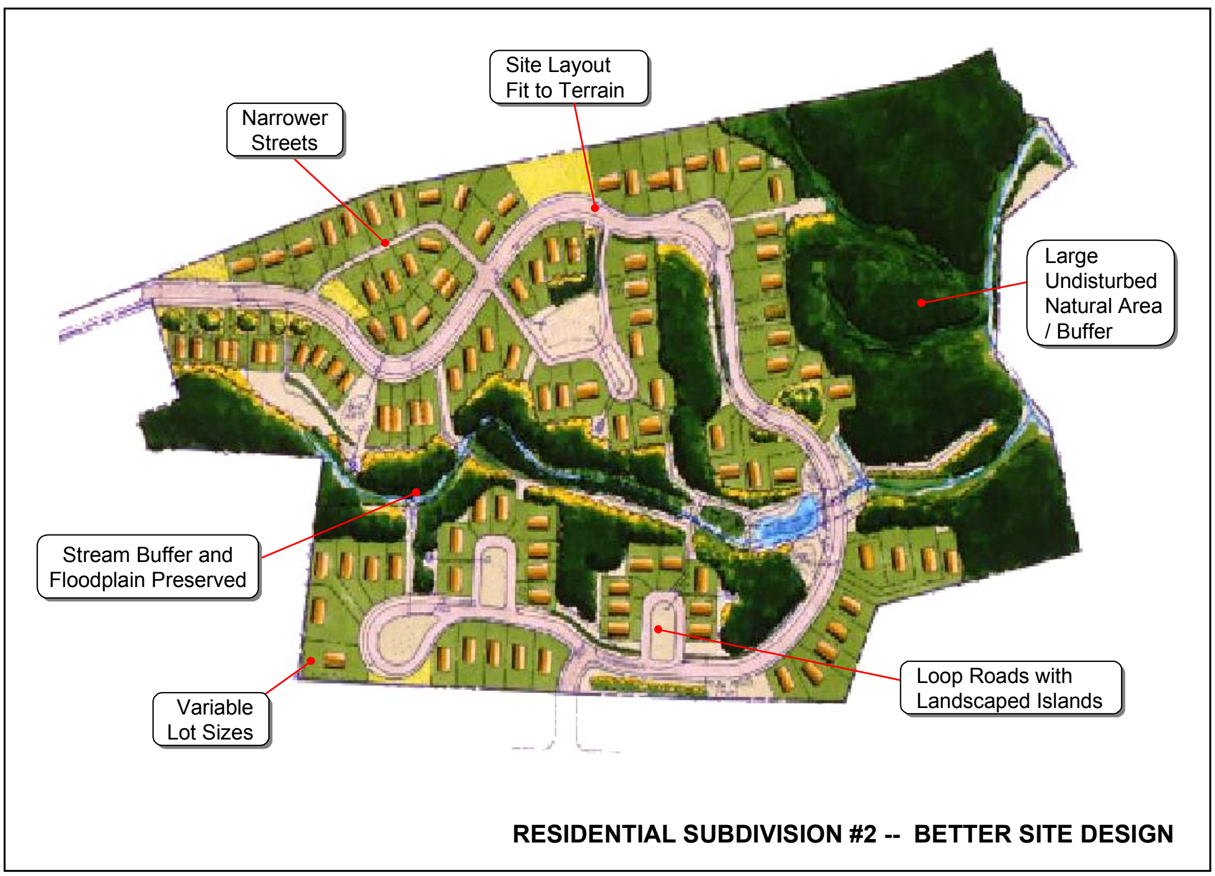


Figure 1.4.3-2 Comparison of a Traditional Residential Subdivision Design (above) with an Innovative Site Plan Developed Using Better Site Design Practices (below).



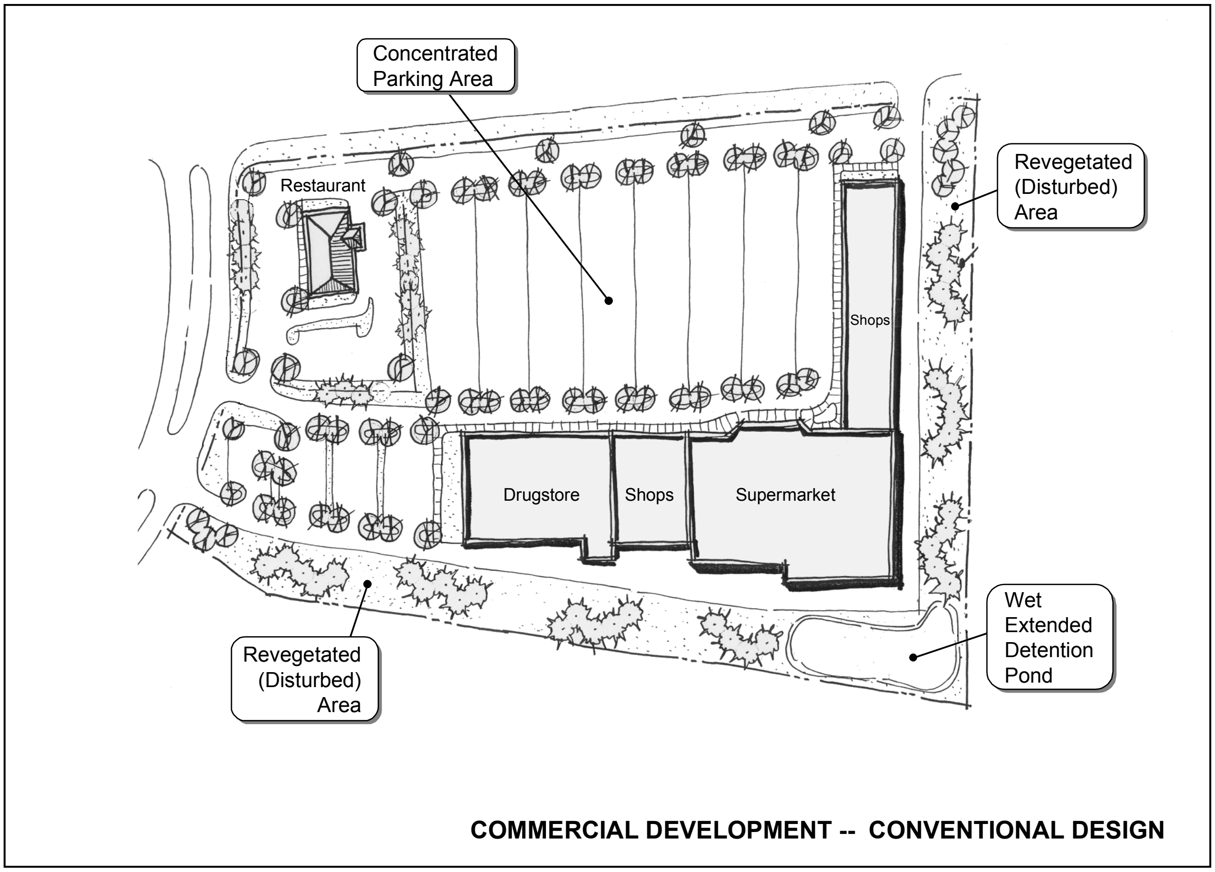
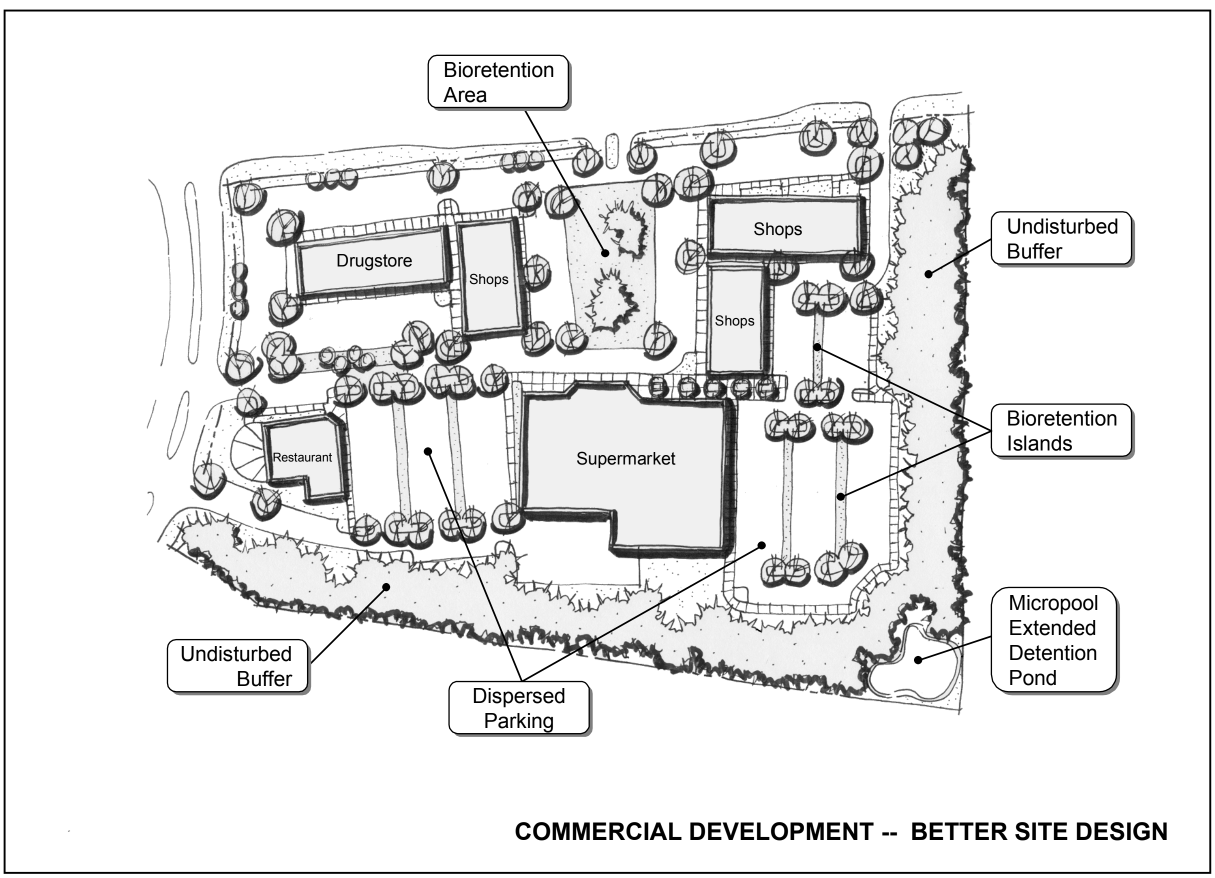


Figure 1.4.3-3 Comparison of a Traditional Commercial Development (above) with an Innovative Site Plan Developed Using Better Site Design Practices (below).



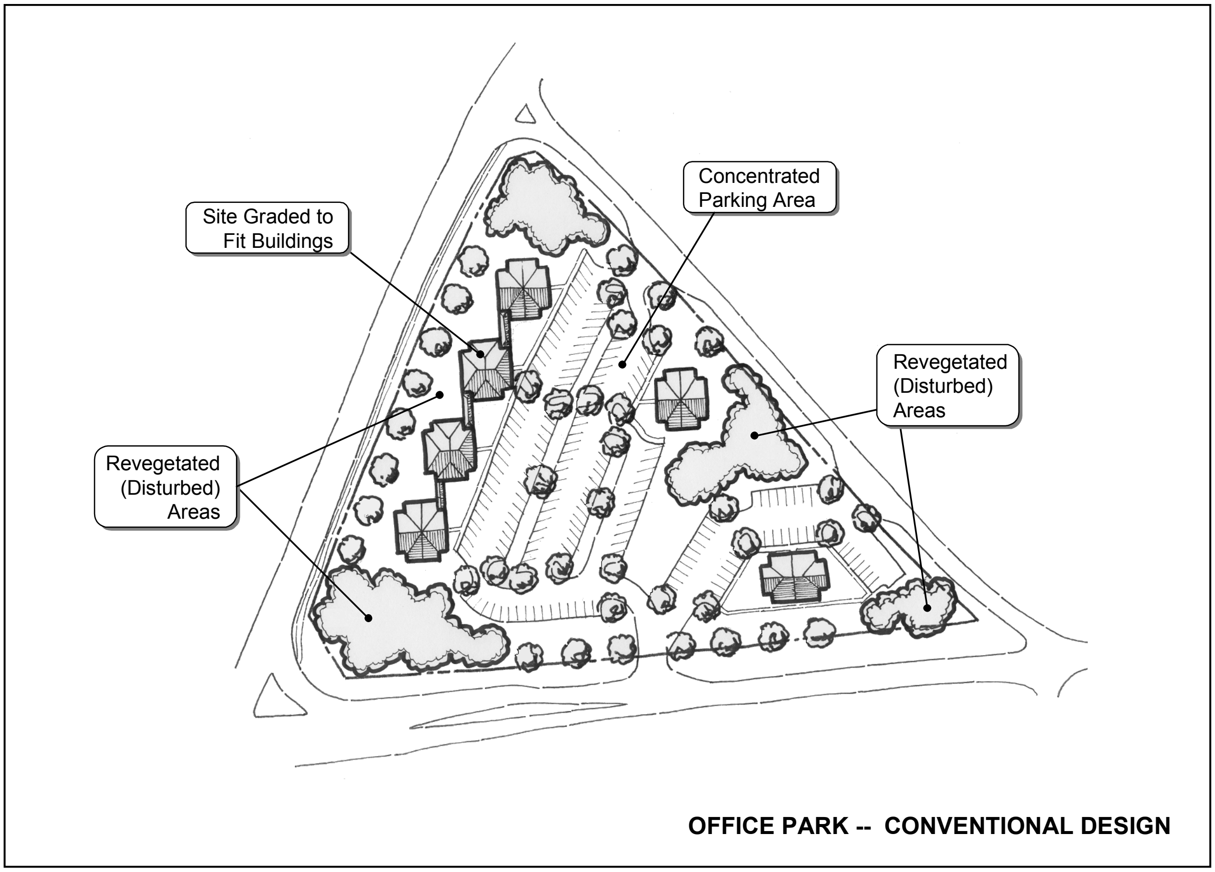
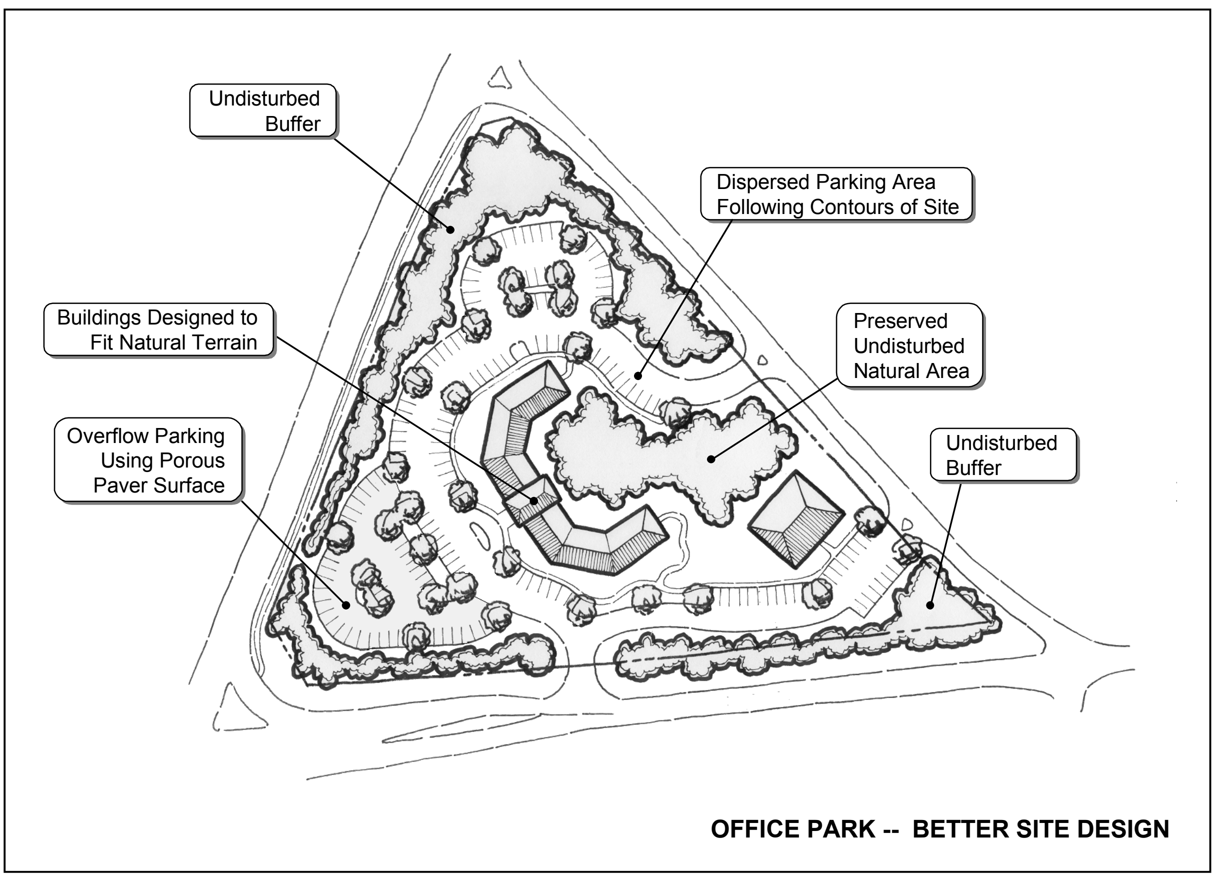


Figure 1.4.3-4 Comparison of a Traditional Office Park Design (above) with an Innovative Site Plan Developed Using Better Site Design Practices (below).



1.4.4 Site Design Stormwater Credits

1.4.4.1 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 structural stormwater control 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 better site design practices that provide stormwater credits are listed in Table 1.4.4-1. Site-specific conditions will determine the applicability of each credit. For example, stream buffer credits cannot be taken on upland sites that do not contain perennial or intermittent streams.

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.

Table 1.4.4-1 Summary of Better Site Design Practices That Provide for Site Design Stormwater Credits

Practice	Description
Natural area conservation	Undisturbed natural areas are conserved on a site, thereby retaining their pre-development hydrologic and water quality characteristics.
Stream buffers	Stormwater runoff is treated by directing sheet flow runoff through a naturally vegetated or forested buffer as overland flow.
Use of vegetated channels	Vegetated channels are used to provide stormwater treatment.
Overland flow filtration/infiltration zones	Overland flow filtration/infiltration zones are incorporated into the site design to receive runoff from rooftops and other small impervious areas.
Environmentally sensitive large lot subdivisions	A group of site design techniques are applied to low and very low density residential development.

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 as many credits as they can 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 stream buffers 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.

1.4.4.2 Stormwater Credits and the Site Planning Process

During the site planning process described in Section 1.5 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 1.4.4-2.

Table 1.4.4-2 Integration of Site Design Credits with Site Development Process

Site Development Phase	Site Design Credit Activity
Feasibility Study	• Determine stormwater management requirements • 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 • Reduce impervious surface area through various techniques • Identify locations for use of vegetated channels and groundwater recharge • Look for areas to disconnect impervious surfaces • Document the use of site design credits.
Preliminary and Final Plan	• Perform layout and design of credit areas – integrating them into treatment trains • Ensure unified stormwater sizing criteria are satisfied • Ensure appropriate documentation of site design credits according to local requirements.
Construction	• Ensure protection of key areas • Ensure correct final construction of areas needed for credits
Final Inspection	• Develop maintenance requirements and documents • Ensure long term protection and maintenance • Ensure credit areas are identified on final plan and plat if applicable

1.4.4.3 Site Design Credit #1: 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
- 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], and
- Shall have a minimum contiguous area requirement of 10,000 square feet
- R_v is kept constant when calculating WQ_v

Example:

Residential Subdivision
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 ac-ft

With credit:
WQ_v = (1.2)(0.37)(31)/12 = 1.15 ac-ft

(18% reduction in water quality volume)

1.4.4.4 Site Design Credit #2: Stream Buffers

This credit can be taken when stormwater runoff is effectively treated by a stream buffer. Effective treatment constitutes treating runoff through overland flow in a naturally vegetated or forested buffer. Under the proposed credit, a designer would be able to subtract areas draining via overland flow to the buffer from total site area when computing water quality volume requirements. In addition, the volume of runoff draining to the buffer can be subtracted from the channel protection volume. The design of the stream buffer treatment system must use appropriate methods for conveying flows above the annual recurrence (1-yr storm) event.

Rule: Subtract areas draining via overland flow to the buffer from total site area when computing water quality volume requirements.

Criteria:

- The minimum undisturbed buffer width shall be 50 feet
- The maximum contributing length shall be 150 feet for pervious surfaces and 75 feet for impervious surfaces
- The average contributing slope shall be 3% maximum unless a flow spreader is used
- Runoff shall enter the buffer as overland sheet flow. A flow spreader can be supplied to ensure this, or if average contributing slope criteria cannot be met
- Not applicable if overland flow filtration/groundwater recharge credit is already being taken
- Buffers shall remain unmanaged other than routine debris removal
- R_v is kept constant when calculating WQ_v

Example:

Residential Subdivision
Area = 38 acres
Impervious Area = 13.8 acres
Area Draining to Buffer = 5 acres

R_v = 0.05 + 0.009 (I) = 0.05 + 0.009 (36.3%) = 0.37

Credit:

5.0 acres draining to buffer

New drainage area = 38 – 5 = 33 acres

Before credit:
WQ_v = (1.2)(0.37)(38)/12 = 1.40 ac-ft

With credit:
WQ_v = (1.2)(0.37)(33)/12 = 1.22 ac-ft

(13% reduction in water quality volume)

1.4.4.5 Site Design Credit #3: Vegetated Channels

This credit may be taken when vegetated (grass) channels are used for water quality treatment. Under the proposed credit, a designer would be able to subtract the areas draining to a grass channel from total site area when computing water quality volume requirements. A vegetated channel can fully meet the water quality volume requirements for certain kinds of low-density residential development (see low impact development credit). An added benefit will be that the post-development peak discharges will likely be lower due to a longer time of concentration for the site.

This credit cannot be taken if grass channels are being used as a limited application structural stormwater control towards meeting the 80% TSS removal goal for WQ_v treatment.

Rule: Subtract the areas draining to a grass channel from total site area when computing water quality volume requirements.

Criteria:

- The credit shall only be applied to moderate or low density residential land uses (3 dwelling units per acre maximum)
- The maximum flow velocity for water quality design storm shall be less than or equal to 1.0 feet per second
- The minimum residence time for the water quality storm shall be 5 minutes
- The bottom width shall be a maximum of 6 feet. If a larger channel is needed use of a compound cross section is required
- The side slopes shall be 3:1 (horizontal:vertical) or flatter
- The channel slope shall be 3 percent or less
- R_v is kept constant when calculating WQ_v

Example:

Residential Subdivision
Area = 38 acres
Impervious Area = 13.8 acres

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

Credit:

12.5 acres meet grass channel criteria

New drainage area = 38 – 5 = 25.5 acres

Before credit:
WQ_v = (1.2)(0.37)(38)/12 = 1.40 ac-ft

With credit:
WQ_v = (1.2)(0.37)(25.5)/12 = 0.94 ac-ft

(33% reduction in water quality volume)

1.4.4.6 Site Design Credit #4: Overland Flow Filtration/Groundwater Recharge Zones

This credit can be taken when "overland flow filtration/infiltration zones" are incorporated into the site design to receive runoff from rooftops or other small impervious areas (e.g., driveways, small parking lots, etc). This can be achieved by grading the site to promote overland vegetative filtering or by providing infiltration or "rain garden" areas. If impervious areas are adequately disconnected, they can be deducted from total site area when computing the water quality volume requirements. An added benefit will be that the post-development peak discharges will likely be lower due to a longer time of concentration for the site.

Rule: If impervious areas are adequately disconnected, they can be deducted from total site area when computing the water quality volume requirements.

<u>Criteria:</u>

- Relatively permeable soils (hydrologic soil groups A and B) should be present
- Runoff shall not come from a designated hotspot
- The maximum contributing impervious flow path length shall be 75 feet
- Downspouts shall be at least 10 feet away from the nearest impervious surface to discourage "re-connections"
- The disconnection shall drain continuously through a vegetated channel, swale, or filter strip to the property line or structural stormwater control
- The length of the "disconnection" shall be equal to or greater than the contributing length
- The entire vegetative "disconnection" shall be on a slope less than or equal to 3 percent
- The surface imperviousness area to any one discharge location shall not exceed 5,000 square feet
- For those areas draining directly to a buffer, either the overland flow filtration credit *-or-* the stream buffer credit can be used
- R_v is kept constant when calculating WQ_v

Example:

Site Area = 3.0
Impervious Area = 1.9 acres (or 63.3% impervious cover)
"Disconnected" Impervious Area = 0.5 acres

R_v = 0.05 + 0.009 (I) = 0.05 + 0.009 (63.3%) = 0.62

Credit:

0.5 acres of surface imperviousness hydrologically disconnected

New drainage area = 3 – 0.5 = 2.5 acres

Before credit:
WQ_v = (1.2)(0.62)(3)/12 = 0.19 ac-ft

With credit:
WQ_v = (1.2)(0.62)(2.5)/12 = 0.15 ac-ft

(21% reduction in water quality volume)

1.4.4.7 Site Design Credit #5: Environmentally Sensitive Large Lot Subdivisions

This credit can be taken when a group of environmental site design techniques are applied to low and very low density residential development (e.g., 1 dwelling unit per 2 acres [du/ac] or lower). The credit can eliminate the need for structural stormwater controls to treat water quality volume requirements. This credit is targeted towards large lot subdivisions and will likely have limited application.

Rule: Targeted towards large lot subdivisions (e.g. 2 acre lots and greater). The requirement for structural practices to treat the water quality volume treatment requirements shall be waived.

<u>Criteria:</u>

For Single Lot Development:

- Total site impervious cover is less than 15%
- Lot size shall be at least two acres
- Rooftop runoff is disconnected in accordance with the criteria in Credit #4
- Grass channels are used to convey runoff versus curb and gutter

For Multiple Lots:

- Total impervious cover footprint shall be less than 15% of the area
- Lot areas should be at least 2 acres, unless clustering is implemented. Open space developments should have a minimum of 25% of the site protected as natural conservation areas and shall be at least a half-acre average individual lot size
- Grass channels should be used to convey runoff versus curb and gutter (see Credit #3)
- Overland flow filtration/infiltration zones should be established (see Credit #4)

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SECTION

1.5

STORMWATER SITE PLANNING

1.5.1 Stormwater Management and Site Planning

1.5.1.1 Introduction

In order to most effectively address stormwater management objectives, consideration of stormwater 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 framework 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 minimum standards (and any additional needs or objectives)?
- What are the opportunities for utilizing better site design practices to minimize the need for structural stormwater controls?
- What are the development site constraints that preclude the use of certain structural controls?
- What structural controls are most suitable and cost-effective for the site?

1.5.1.2 Principles of Stormwater Management 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. **Structural stormwater controls 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 structural stormwater controls. Therefore, all opportunities for utilizing these methods should be explored before implementing structural stormwater controls 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.

1.5.2 Preparation of Stormwater Management Site Plans

1.5.2.1 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 minimum stormwater management standards contained in this Manual.

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:

(1) Pre-consultation Meeting and Joint Site Visit

(2) Review of Local Requirements

(3) Perform Site Analysis

(4) Prepare Stormwater Concept Plan

(5) Prepare Preliminary Stormwater Site Plan

(6) Complete Final Stormwater Site Plan

1.5.2.2 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 his 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.

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.

1.5.2.3 Review of Local Requirements

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 minimum standards for stormwater management included in this Manual (see Section 1.2)
- 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.

1.5.2.4 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 1.5.2-1). This composite layer can be a useful tool for defining the best buildable areas and delineating and preserving natural feature conservation areas.

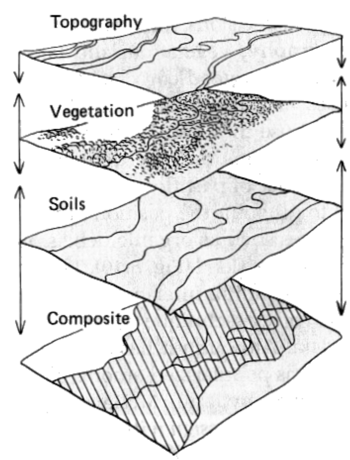

Figure 1.5.2-1
Composite Analysis
(Source: Marsh, 1983)

1.5.2.5 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:

(1) Use better site design approaches (see Section 1.4) 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

(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 1.3)

(3) Determine the site design stormwater credits to be accounted for in the design of structural stormwater controls handling the water quality volume (Section 1.4)

(4) Perform screening and preliminary selection of appropriate structural stormwater controls and identification of potential siting locations (Section 3.1).

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 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 structural stormwater controls
- 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

1.5.2.6 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) Existing Conditions Hydrologic Analysis

Provide an existing condition hydrologic analysis for stormwater runoff rates, volumes, and velocities, which includes:

- 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 subbasin 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
- Methodologies, assumptions, site parameters and supporting design calculations used in analyzing the existing conditions site hydrology

(2) Post-Development Hydrologic Analysis

Provide a post-development hydrologic analysis for stormwater runoff rates, volumes, and velocities, which includes:

- 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 subbasin affected by the project
- Unified stormwater sizing criteria runoff calculations for water quality, channel protection, overbank flooding protection and extreme flood protection for each subbasin
- Location and boundaries of proposed natural feature protection and conservation 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

(3) Stormwater Management System

Provide drawings and design calculations for the proposed stormwater management system, including:

- A 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 inlet 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.
- 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

(4) Downstream Analysis

Provide the assumptions and calculations from a downstream analysis (when required)

- 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 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 controls.

1.5.2.7 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 stormwater controls

(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.

1.5.2.8 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.

1.5.3 Stormwater Planning in the Development Process

1.5.3.1 General Site Development Process

Figure 1.5.3-1 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.

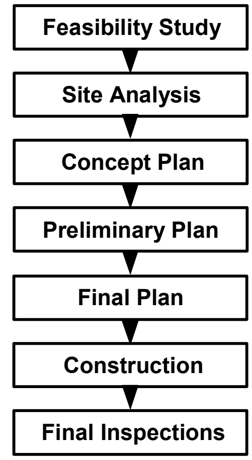

Figure 1.5.3-1 Typical Site Development Flowchart

1.5.3.2 Stormwater Site Planning and Design

Stormwater site planing and design is a subset of overall site development and must fit into the overall process if it is to be successful. Table 1.5.3-1 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 1.5.3-1 Stormwater Planning in the Site Development Process

Feasibility Study

Description:

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.

Stormwater-Related Objectives:

- Understand major site constraints and opportunities
- Understand local and other requirements

Key Actions:

- Initiate discussions with local review authority
- Pre-consultation between developer and plan reviewer
- Determine local stormwater management requirements

Major Activities:

- Base map development
- Review of project requirements
- Review of local development and stormwater management requirements
- Review of local stormwater master plans or comprehensive plans
- Joint site visit with local review authority
- Collection of secondary source information
- Determination of other factors or constraints impacting feasibility

Table 1.5.3-1 continued

Site Analysis

Description:

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.

Stormwater-Related Objectives:

- Identify key site physical, environmental, and other significant resources
- Develop preliminary vision for stormwater management system

Key Actions:

- Site evaluation and delineation of natural feature protection areas

Major Activities:

- Mapping of natural resources: soils, vegetation, streams, topography, slope, wetlands, floodplains, aquifers
- Identification of other key cultural, historic, archaeological, or scenic features, orientation and exposure
- Identification of adjacent land uses
- Identification of adjacent transportation and utility access
- Identification of natural feature protection and conservation areas
- Mapping of easements and utilities
- Integration of all layers – map overlay
- Other constraints and opportunities

Concept Plan

Description:

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.

Stormwater-Related Objectives:

- Develop concept for stormwater management system
- Gain approval from developer and local review authority of concept plan

Key Actions:

- Develop site layout concept using better site design techniques where possible
- Perform initial runoff characterization based on site layout concept
- Determine necessary site design and/or structural controls needed to meet stormwater management requirements

Major Activities:

- Prepare sketches of functional land uses including conservation areas
- "What if" analysis of different design concepts
- Unified stormwater sizing criteria preliminary calculations
- Utilization of better site design concepts and crediting mechanisms in layout concept
- Preliminary selection and siting of structural stormwater controls
- Location of drainage/conveyance facilities

Table 1.5.3-1 continued

Preliminary and Final Plan

Description:

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.

Stormwater-Related Objectives:

- Prepare preliminary and final stormwater management site plan
- Secure local and non-local permits

Key Actions:

- Perform runoff characterization based on preliminary/final site plan
- Design structural stormwater controls and conveyance systems
- Perform downstream analysis

Major Activities:

- Preliminary/final site layout plan
- Unified stormwater sizing criteria calculations
- Calculation of site design credit
- Selection, siting and design of structural stormwater controls
- Design of drainage and conveyance facilities
- Development of erosion and sedimentation control plan and landscaping plan
- Applications for needed permits and waivers

Construction

Summary:

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.

Stormwater Objectives:

- Ensure that stormwater management facilities and site design practices are built as designed

Key Actions:

- Pre-construction meeting
- Inspection during construction

Major Activities:

- Execution of bonds
- Inspection during key phases or key installations
- Protection of structural stormwater controls
- Protection of conservation areas
- Erosion and sedimentation control
- Proper sequencing

Table 1.5.3-1 continued

Final Inspection

Summary:

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:

- Ensure that stormwater management facilities and site design practices are built and operating as designed
- Ensure long-term maintenance of structural stormwater controls and conveyances
- Ensure long-term protection of conservation and resource protection areas

Key Actions:

- Final inspection and submission of record drawings
- Maintenance inspections

Major Activities:

- Final stabilization
- As-built survey
- Execution of maintenance agreements
- Final inspection and use permit

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CHAPTER 2

STORMWATER HYDROLOGY

GEORGIA STORMWATER MANAGEMENT MANUAL
FIRST EDITION – AUGUST 2001

CHAPTER 2

STORMWATER HYDROLOGY

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SECTION 2.2 STORAGE DESIGN

SECTION 2.3 OUTLET STRUCTURES

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SECTION

2.1

METHODS FOR ESTIMATING STORMWATER RUNOFF

2.1.1 Introduction to Hydrologic Methods

Hydrology deals with estimating flow peaks, 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 structural stormwater controls. In the hydrologic analysis of a development site, there are a number of variable 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
- ❑ SCS Unit Hydrograph Method
- ❑ U.S. Geological Survey (USGS) Regression Equations
- ❑ Water Quality Treatment Volume Calculation
- ❑ 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 2.1.1-1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications. Table 2.1.1-2 provides some limitations on the use of several methods.

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.

Table 2.1.1-1 Applications of the Recommended Hydrologic Methods					
Method	**Manual Section**	**Rational Method**	**SCS Method**	**USGS Equations**	**Water Quality Volume**
Water Quality Volume (WQ_v)	1.3				✓
Channel Protection Volume (Cp_v)	1.3		✓		
Overbank Flood Protection (Q_{p25})	1.3		✓	✓	
Extreme Flood Protection (Q_f)	1.3		✓	✓	
Storage Facilities	2.2		✓	✓	
Outlet Structures	2.3		✓	✓	
Gutter Flow and Inlets	4.2	✓			
Storm Drain Pipes	4.2	✓	✓	✓	
Culverts	4.3	✓	✓	✓	
Small Ditches	4.4	✓	✓	✓	
Open Channels	4.4		✓	✓	
Energy Dissipation	4.5		✓	✓	

Table 2.1.1-2 Constraints on Using Recommended Hydrologic Methods

Method	Size Limitations[1]	Comments
Rational	0 – 25 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems. Not to be used for storage design.
SCS[2]	0 – 2000 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 Structural Control	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.

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 stream gage 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.

2.1.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.1.2-1 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 2.1.2-1 Symbols and Definitions

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

t	Time	min
t_c	Time of concentration	min
TIA	Total impervious area	%
V	Velocity	ft/s
V	Pond volume	acre-feet
V_r	Runoff volume	acre-feet
V_s	Storage volume	acre-feet
WQ_v	Water Quality Volume	acre-feet

2.1.3 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 occuring 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.

Rainfall intensities for 16 locations across Georgia (see Figure 2.1.3-1) are provided in Appendix A and should be used for all hydrologic analysis at the given locations. The values in these tables were derived in the following way:

- Initial values were derived from TP40 (Hershfield,1961) and HYDRO 35 (NOAA,1977) with the 60-minute and shorter values coming from HYDRO 35.
- Intensity values for smaller than the 2-year storm were extrapolated through a series of plots.
- All values were plotted and smoothed to ensure continuity between the two different sources and to catch any errors. The values for 60 minutes and less were fit using an equation of the form:

$$\mathbf{i = a/(t+b)^n} \qquad \mathbf{(2.1.1)}$$

where *i* is inches per hour and *t* is in minutes. *a* and *b* are fitting parameters found at the top of each of the tables in Appendix A. The tables are applicable to storm durations up to and including 1 hour. This equation allows for automated calculation of rainfall values for the Rational Method without having to look values up in tables or interpolate them from charts. The time of concentration is then substituted for **t** in Equation 2.1.1. The user can either use the values given in the tables or use the equations to calculate rainfall intensity values for times up to and including 1 hour.

Figure 2.1.3-2 shows an example Intensity-Duration-Frequency (IDF) Curve for Athens, Georgia, for the seven storms (1-year – 100-year). These curves are plots of the tabular values.
No values are given for times less than 5 minutes.

Values for areas other than those cities provided (Figure 2.1.3-2) can be interpolated. Figure 2.1.3-3 (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. This trend is accurate except in the far northeast corner of the state where higher elevations create an anomaly due to the orographic lifting. The anomaly does not extend south from the far northeast counties; therefore it is not correct to interpolate from this area and it should be ignored in areas outside of northeast counties. For these counties local values should be used.

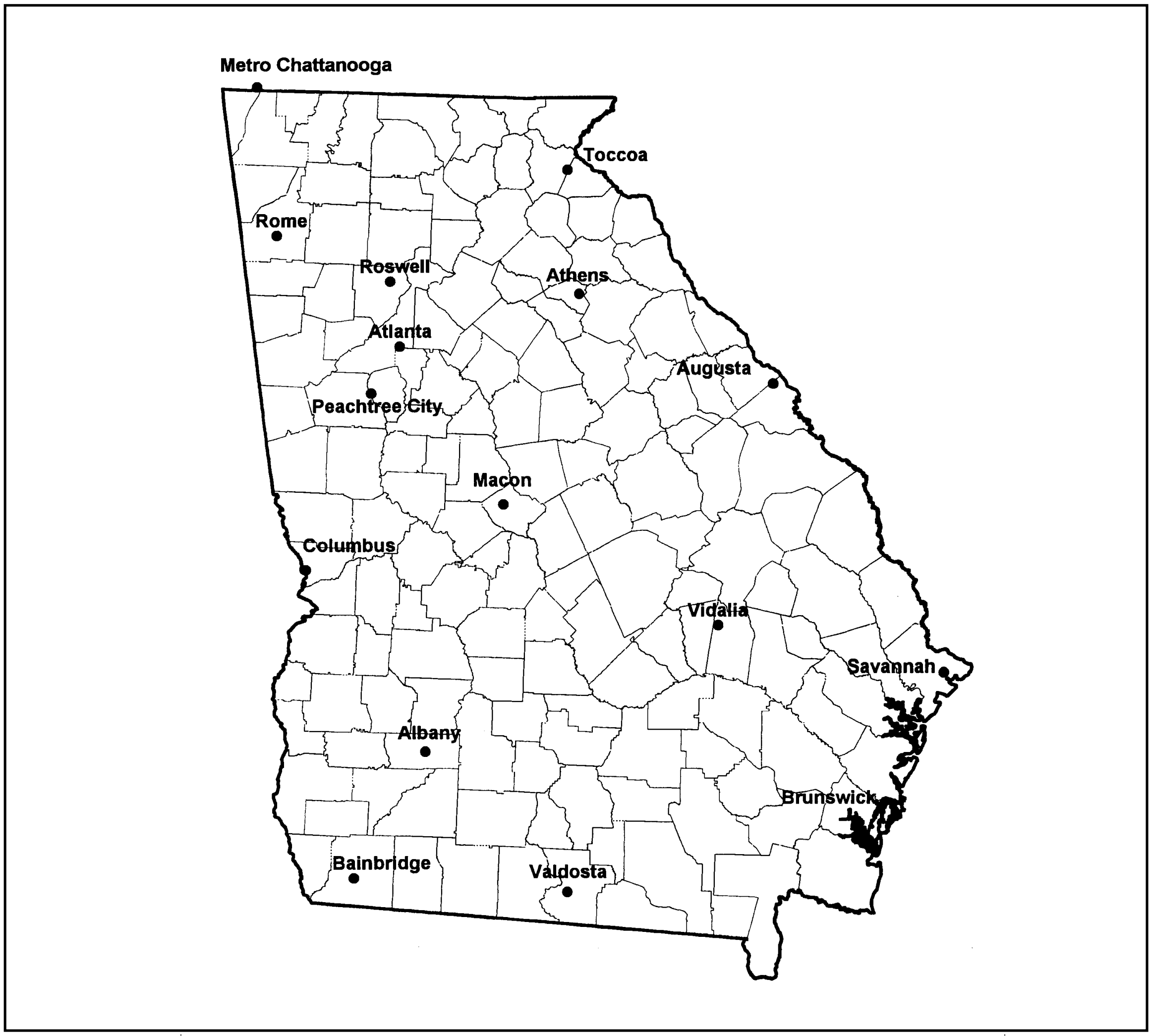


Figure 2.1.3-1 Location of Rainfall Data Sites

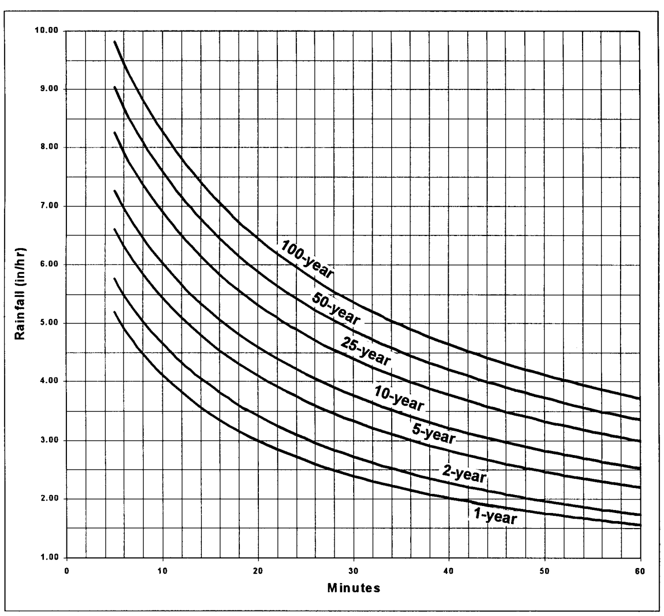


Figure 2.1.3-2 Example IDF Curve (Athens, Georgia)

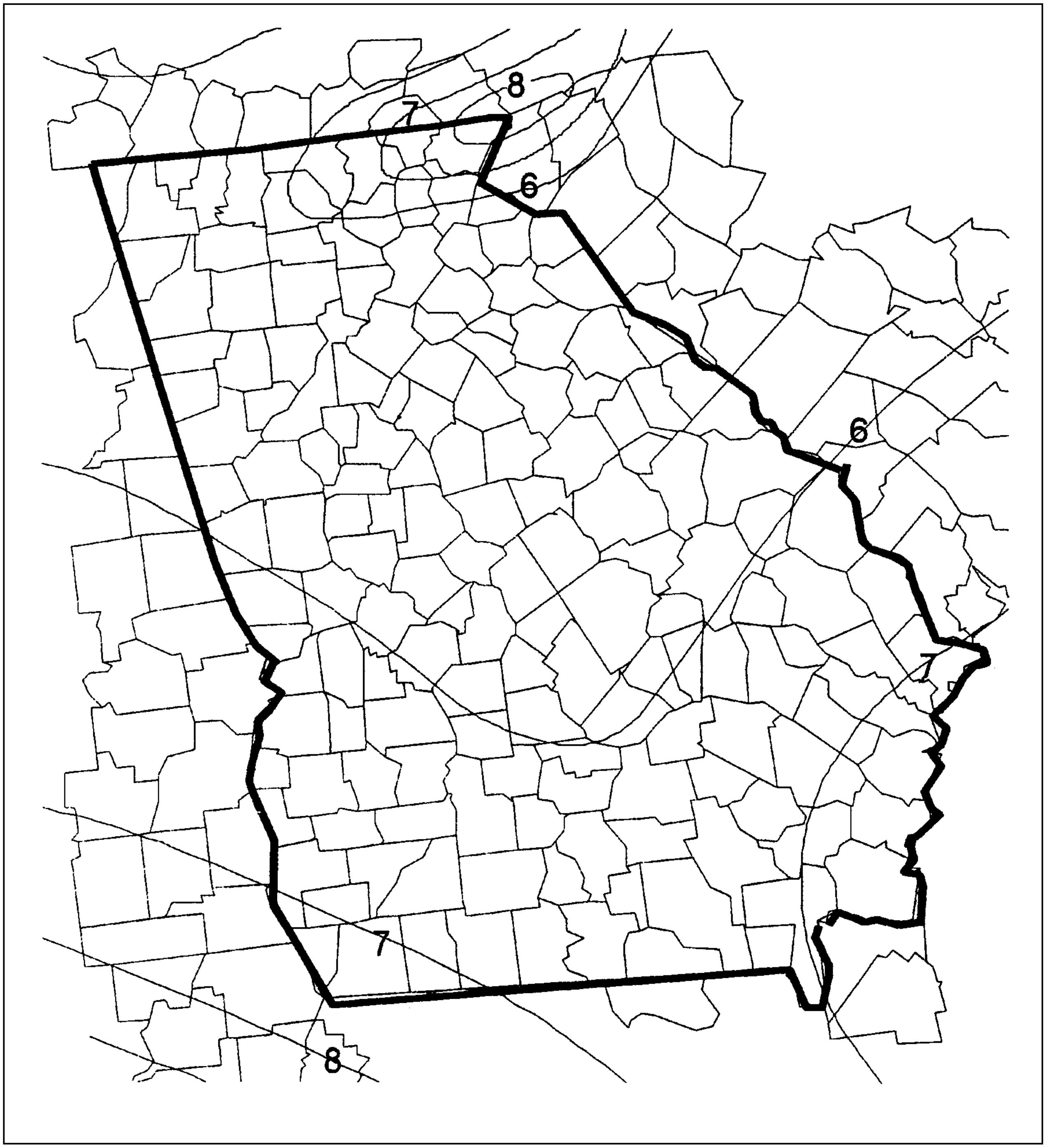


Figure 2.1.3-3 Rainfall Isohyetal Lines (10-year, 24-hour values)

2.1.4 Rational Method

2.1.4.1 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, hydrologic 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 engineer in applying the Rational Method. The 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.

2.1.4.2 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 25 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 2.2. The normal use of the Modified Rational method significantly underpredicts detention volumes, but the improved method in Section 2.2 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.

2.1.4.3 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 analyzed).

The Rational Formula is expressed as follows:

$$Q = CIA \qquad (2.1.2)$$

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 2.1.4-2 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 proportionally 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:

$$Q = C_f CIA \qquad (2.1.3)$$

The C_f values that can be used are listed in Table 2.1.4-1. The product of C_f times C shall not exceed 1.0.

Table 2.1.4-1 Frequency Factors for Rational Formula

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

2.1.4.4 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 2.1.4-1 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 2.1.4-2. 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 2.1.4-3). The kinematic wave method incorporates several variables including rainfall intensity and Manning’s “n”. In using the nomograph, the 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 2.1.4-3 are as follows:

(Step 1) Assume a rainfall intensity.

(Step 2) Use Figure 2.1.4-3 (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 table in Appendix A 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.

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.

2.1.4.5 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 Rainfall-Intensity-Duration data given in the rainfall tables at the end of this section, or through the use of equation 2.1.1.

2.1.4.6 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 2.1.4-2 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 2.1.4-2 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.

2.1.4.7 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.

<u>Site Data</u>

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
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

<u>Overland Flow</u>

A runoff coefficient (C) for the overland flow area is determined from Table 2.1.4-2 to be 0.10.

<u>Time of Concentration</u>

From Figure 2.1.4-1 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 2.1.4-2 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)

<u>Rainfall Intensity</u>

From Table A-12 in Appendix A, using a duration equal to 22 minutes,

I_{25} (25-yr return period) = 5.06 in/hr

Runoff Coefficient

A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the values from Table 2.1.4-2.

Land Use	**Percent of Total Land Area**	**Runoff Coefficient**	**Weighted Runoff Coefficient***
Residential (single family)	.80	.50	.40
Graded area	.20	.30	.06

Total Weighted Runoff Coefficient = .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:

$$Q_{25} = C_f CIA = (1.10)(.46)(5.06)(23) = 59 \text{ cfs}$$

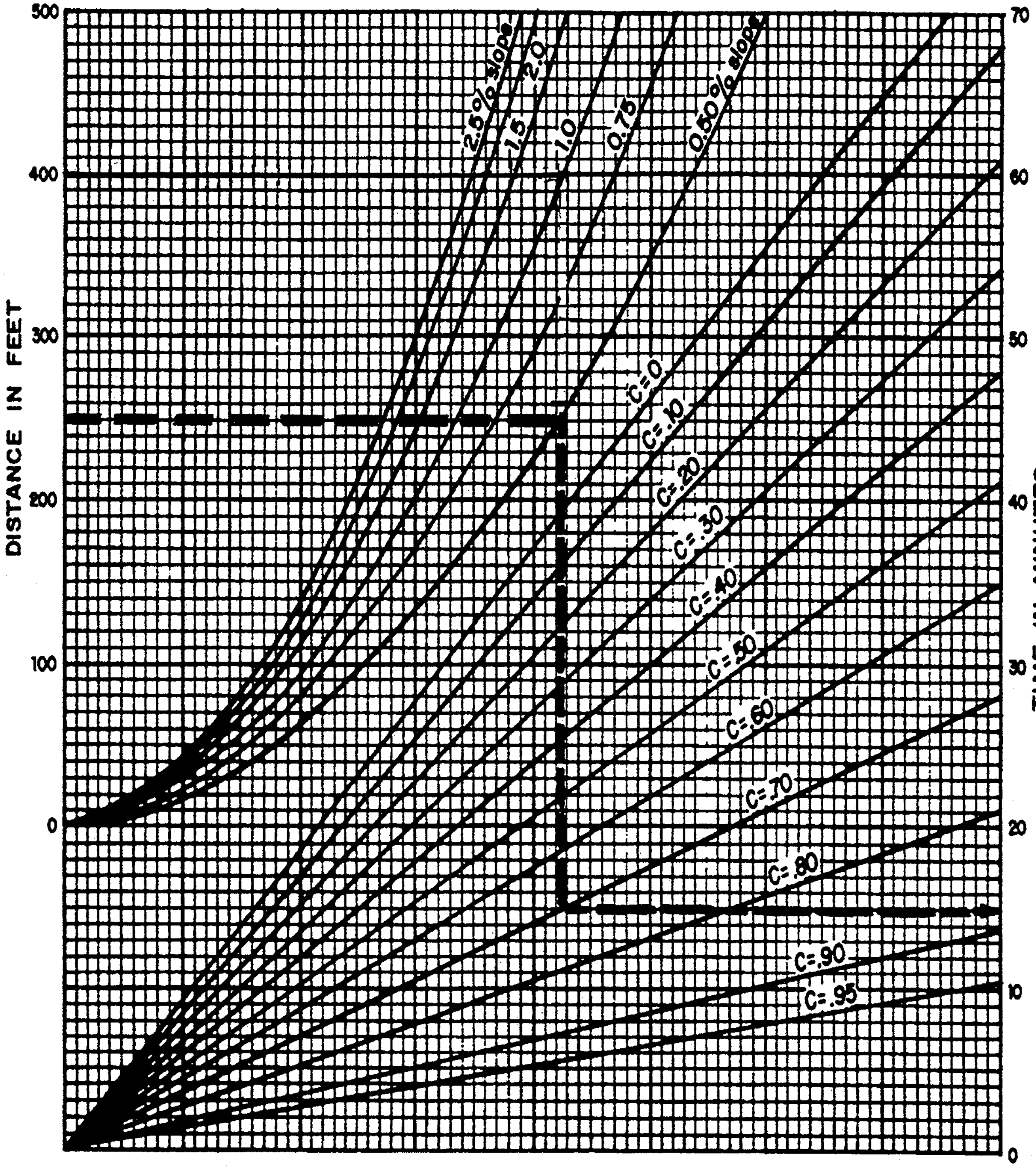
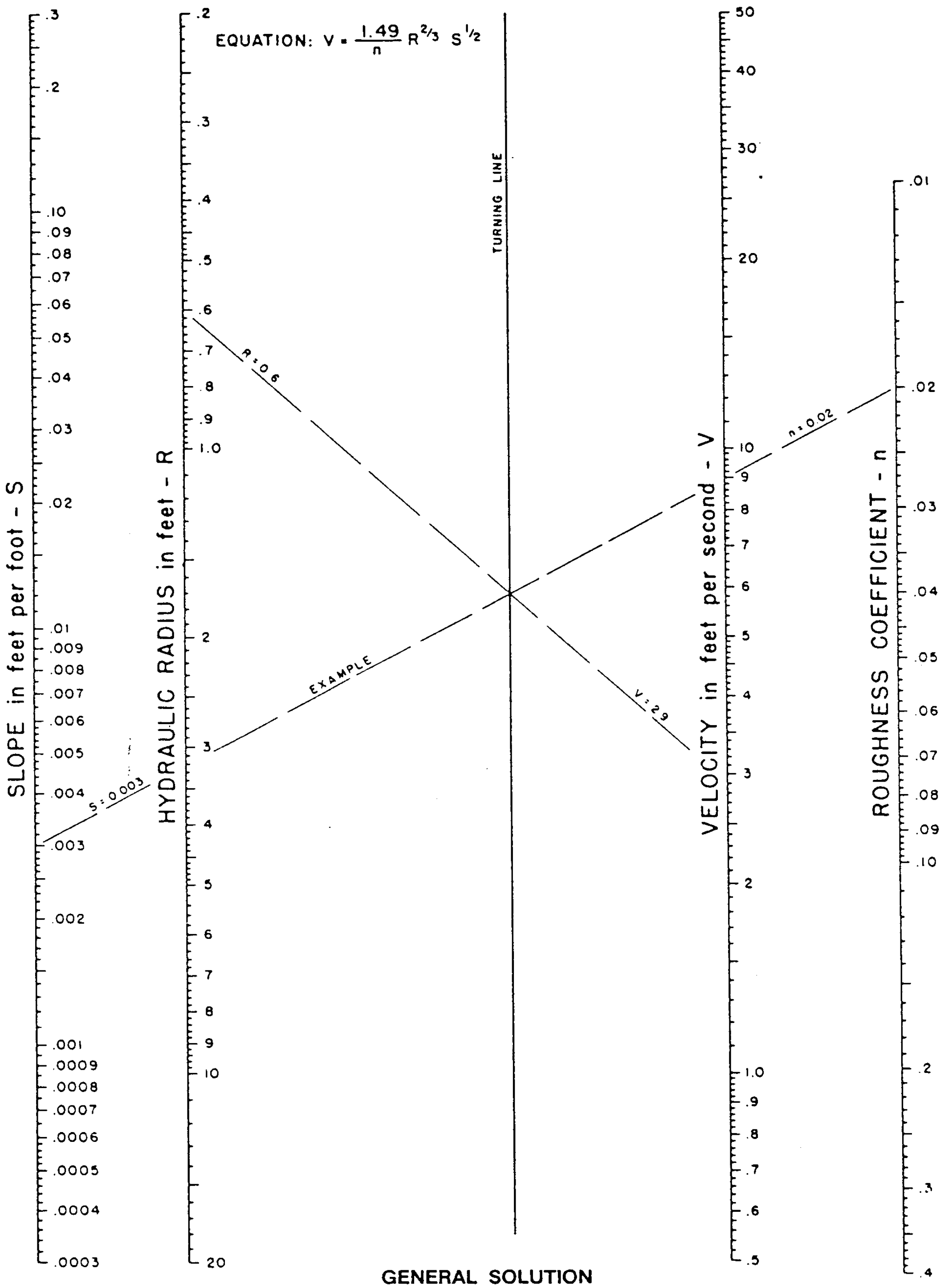


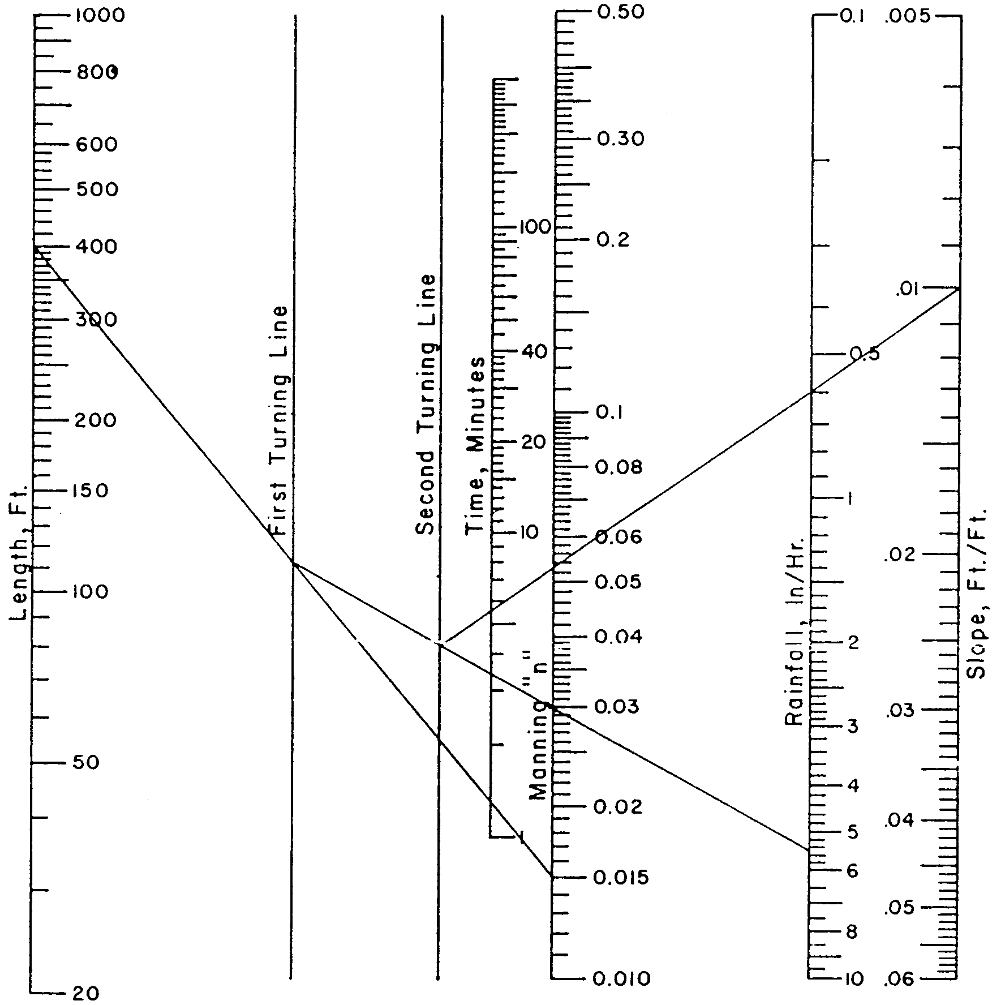
Figure 2.1.4-1 Rational Formula - Overland Time of Flow Nomograph
(Source: Airport Drainage, Federal Aviation Administration, 1965)



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

Figure 2.1.4-2 Manning's Equation Nomograph
(Source: USDOT, FHWA, HDS-3 (1961))

Equation solved by nomograph: $t_c\,(\mathrm{Sec}) = 56\,\dfrac{L_0^{.6}\,n^{.6}}{i^{.4}\,S_0^{.3}}$



Example:

L_0 = 400 ft.
n = 0.015
i = 5.5 in./hr.
S_0 = 0.01
t = 5.5 min.

ONE INCH is 25.4mm
ONE FOOT is 0.3048m

Figure 2.1.4-3 Kinematic Wave Nomograph
(Source: Manual For Erosion And Sediment Control In Georgia, 1996)

Table 2.1.4-2 Recommended Runoff Coefficient Values

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

2.1.5 SCS Hydrologic Method

2.1.5.1 Introduction

The Soil Conservation Service* (SCS) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, 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 SCS National Engineering Handbook, Section 4, Hydrology.

A typical application of the SCS method includes the following basic steps:

(1) Determination of curve numbers that represent different land uses within the drainage area.
(2) Calculation of time of concentration to the study point.
(3) Using the Type II or Type III rainfall distribution, total and excess rainfall amounts are determined. Note: See Figure 2.1.5-1 for the geographic boundaries for the different SCS rainfall distributions.
(4) Using the unit hydrograph approach, the hydrograph of direct runoff from the drainage basin can be developed.

2.1.5.2 Application

The SCS 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 2.1.5.7 can be used for drainage areas up to 2,000 acres. Thus, the SCS 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.

2.1.5.3 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 SCS 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 SCS 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 2.1.5-1).

* The Soil Conservation Service is now known as the Natural Resources Conservation Service (NRCS)

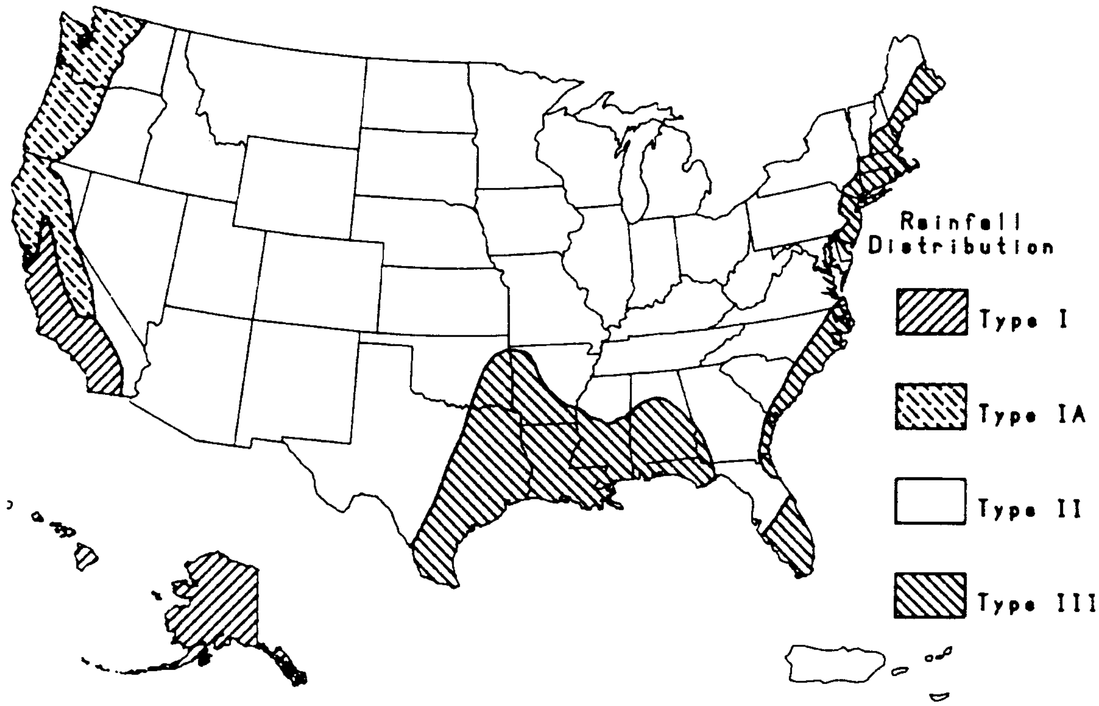


Figure 2.1.5-1 Approximate Geographic Boundaries for SCS Rainfall Distributions

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

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \quad \textbf{(2.1.4)}$$

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 SCS method for estimating I_a is:

$$I_a = 0.2S \quad \textbf{(2.1.5)}$$

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 2.1.4, the equation becomes:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \quad \textbf{(2.1.6)}$$

Where: S = 1000/CN - 10 and CN = SCS curve number

Figure 2.1.5-2 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.

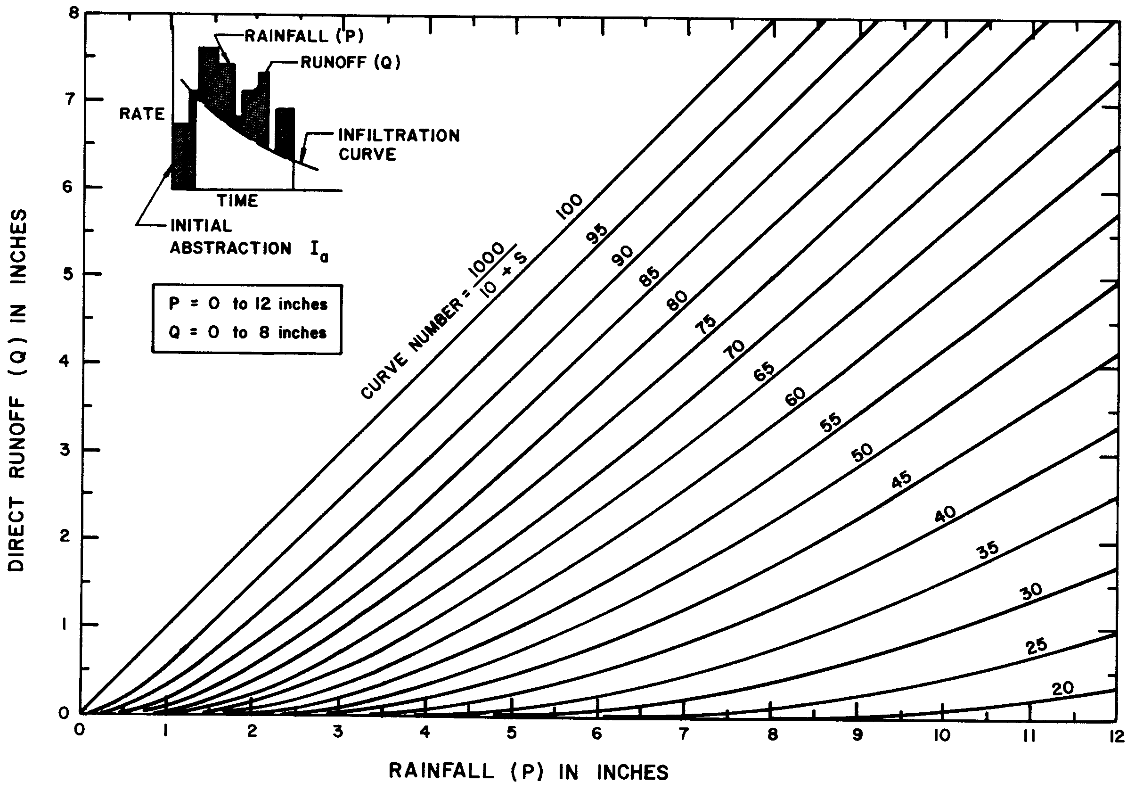


Figure 2.1.5-2
SCS Solution of the Runoff Equation
(Source: SCS, TR-55, Second Edition, June 1986)

Equation 2.1.6 can be rearranged so that the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

$$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}] \qquad (2.1.7)$$

2.1.5.4 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 SCS 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 SCS 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 moderately 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 downward 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 from local SCS offices for use in estimating soil type.

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 2.1.5-1 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 1/8 acre Soil group B	0.80	0.85	0.68
Meadow Good condition Soil group C	0.20	0.71	0.14

Total Weighted Curve Number = 0.68 + 0.14 = 0.82

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.

2.1.5.5 Urban Modifications of the SCS 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 2.1.5-1 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.

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 2.1.5-1 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 2.1.5-1 are not applicable, use Figure 2.1.5-3 to compute a composite CN. For example, Table 2.1.5-1 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 2.1.5-3 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Table 2.1.5-1 Runoff Curve Numbers[1]

Cover description			**Curve numbers for hydrologic 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 condition			30	58	71	78
Wood or forest land:	thin stand, poor cover		45	66	77	83
	good cover		25	55	70	77
Open space (lawns, 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 (town houses)		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 SCS 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.

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 2.1.5-4 if total impervious area is less than 30% or (2) use Figure 2.1.5-3 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.

When impervious area is less than 30%, obtain the composite CN by entering the right half of Figure 2.1.5-4 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 2.1.5-4 is 66. If all of the impervious area is connected, the resulting CN (from Figure 2.1.5-3) would be 68.

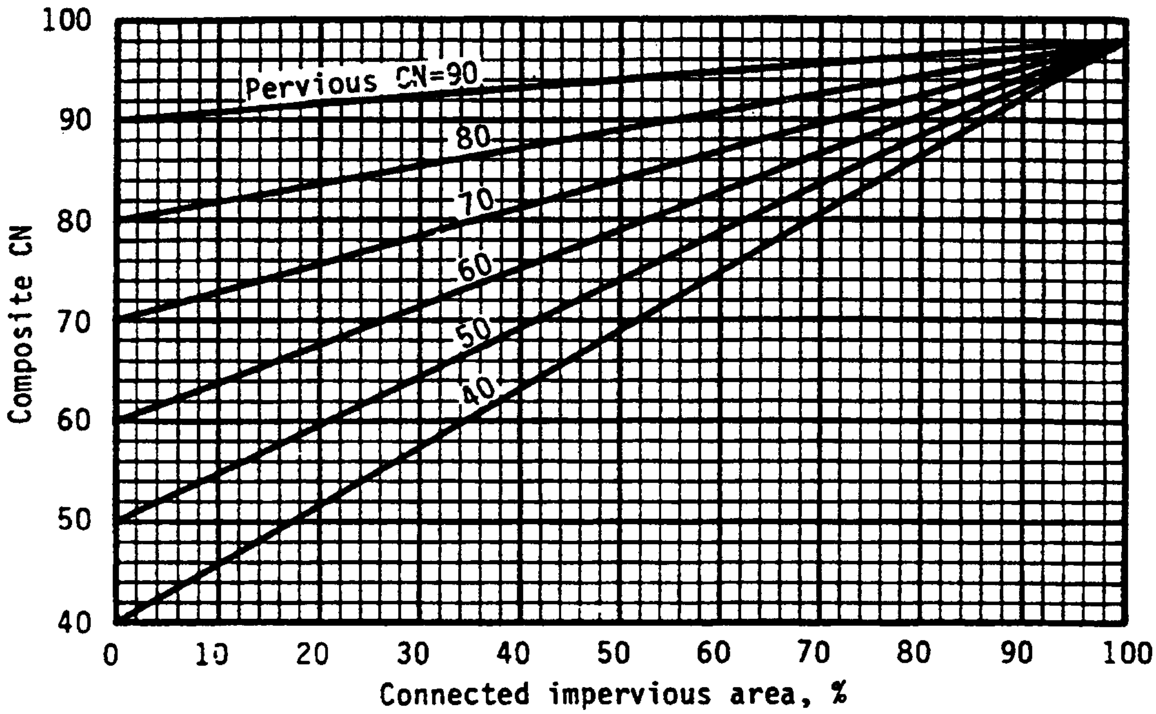


Figure 2.1.5-3 Composite CN with Connected Impervious Areas
(Source: SCS, TR-55, Second Edition, June 1986)

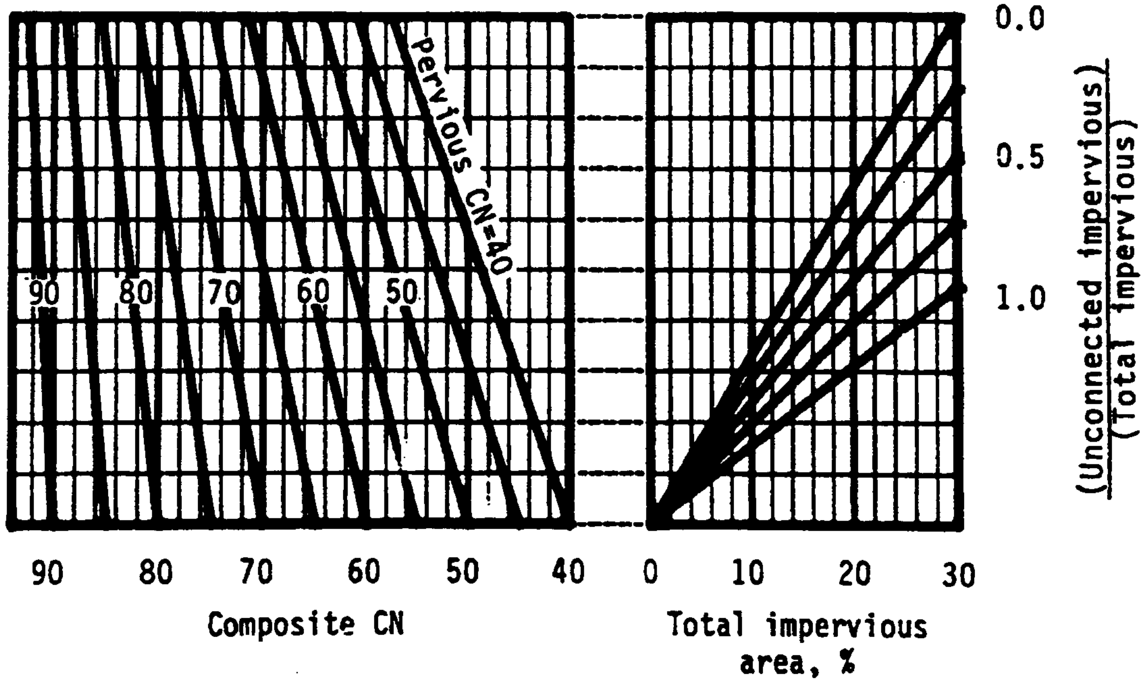


Figure 2.1.5-4 Composite CN with Unconnected Impervious Areas
(Total Impervious Area Less Than 30%)
(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.6 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:

$$T_t = \frac{L}{3600V} \quad \textbf{(2.1.8)}$$

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

Sheet Flow

Sheet flow can be calculated using the following formula:

$$T_t = \frac{0.42\,(nL)^{0.8}}{60\,(P_2)^{0.5}\,(S)^{0.4}} \qquad \textbf{(2.1.9)}$$

Where: T_t= travel time (hr)
n = Manning roughness coefficient (see Table 2.1.5-2)
L = flow length (ft),
P_2 = 2-year, 24-hour rainfall
S = land slope (ft/ft)

Table 2.1.5-2 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: SCS, 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 2.1.5-5, in which average velocity is a function of watercourse slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be computed from using Figure 2.1.5-5, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

Unpaved $\mathbf{V = 16.13(S)^{0.5}}$ **(2.1.10)**

Paved $\mathbf{V = 20.33(S)^{0.5}}$ **(2.1.11)**

Where: V = average velocity (ft/s)
S = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity using Figure 2.1.5-5 or equations 2.1.10 or 2.1.11, use equation 2.1.8 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 $$V = \frac{1.49\,(R)^{2/3}\,(S)^{1/2}}{n} \qquad \textbf{(2.1.12)}$$

Where: V = average velocity (ft/s)
R = hydraulic radius (ft) and is equal to A/P_w
A = cross sectional flow area (ft^2)
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 2.1.12, T_t for the channel segment can be estimated using equation 2.1.8.

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.

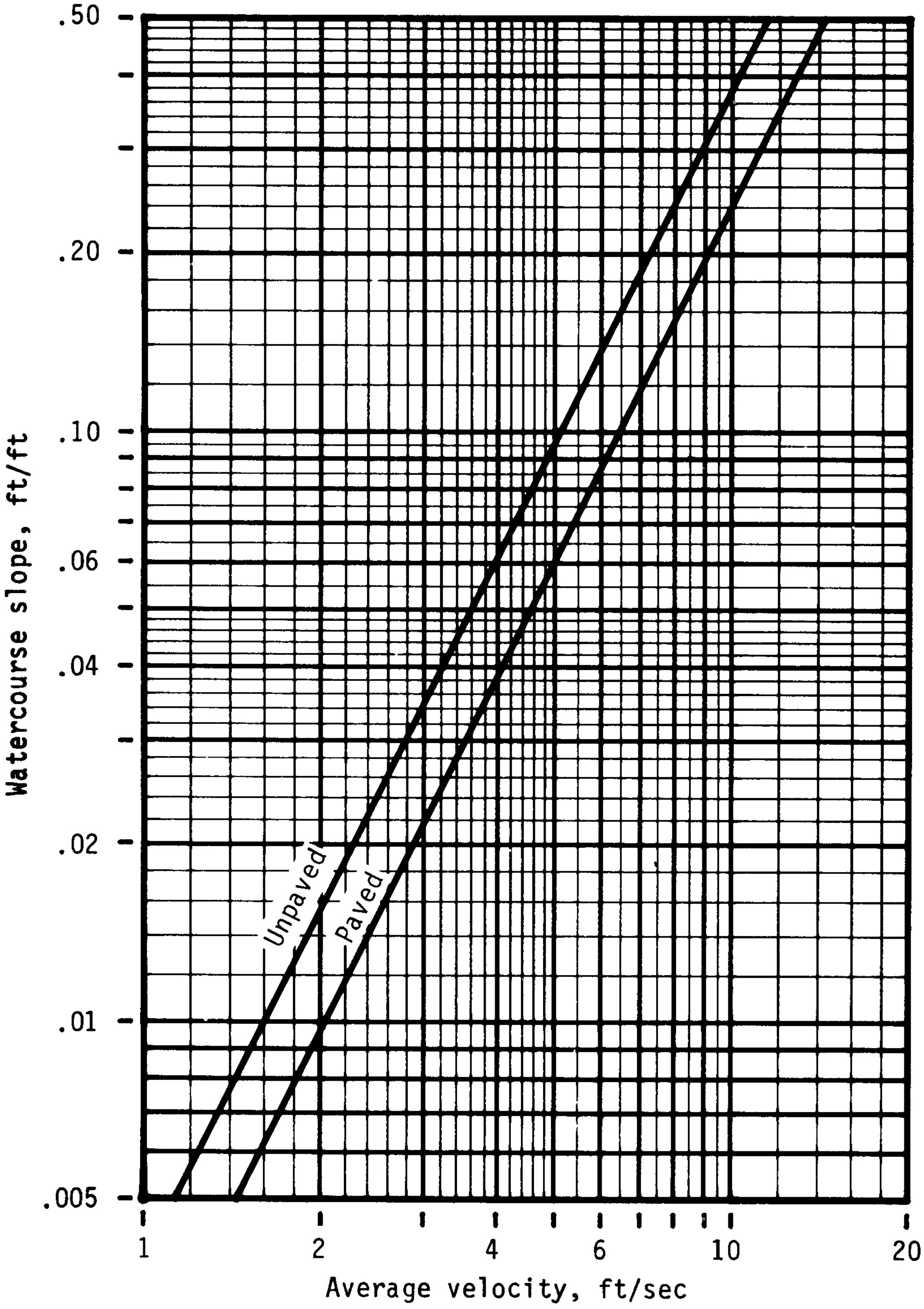


Figure 2.1.5-5 Average Velocities - Shallow Concentrated Flow
(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.7 Simplified SCS Peak Runoff Rate Estimation

The following SCS procedures were taken from the SCS 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:

$$\mathbf{Q_p = q_u A Q F_p} \qquad \textbf{(2.1.13)}$$

Where: Q_p = peak discharge (cfs)
q_u = unit peak discharge (cfs/mi^2/in)
A = drainage area (mi^2)
Q = runoff (in)
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:

(1) The 24-hour rainfall depth is determined from the rainfall tables in Appendix A for the selected location and return frequency.

(2) The runoff curve number, CN, is estimated from Table 2.1.5-1 and direct runoff, Q_p, is calculated using equation 2.1.13.

(3) The CN value is used to determine the initial abstraction, I_a, from Table 2.1.5-3, and the ratio I_a/P is then computed (P = accumulated 24-hour rainfall).

(4) The watershed time of concentration is computed using the procedures in subsection 2.1.5.6 and is used with the ratio I_a/P to obtain the unit peak discharge, q_{up}, from Figure 2.1.5-6 for the Type II rainfall distribution and Figure 2.1.5-7 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 2.1.5-6 and 2.1.5-7 are based on a peaking factor of 484. If a peaking factor of 300 is needed, these figures are not applicable and the simplified SCS method should not be used.

(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

(6) The peak runoff rate is computed using equation 2.1.13.

Table 2.1.5-3 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: SCS, TR-55, Second Edition, June 1986

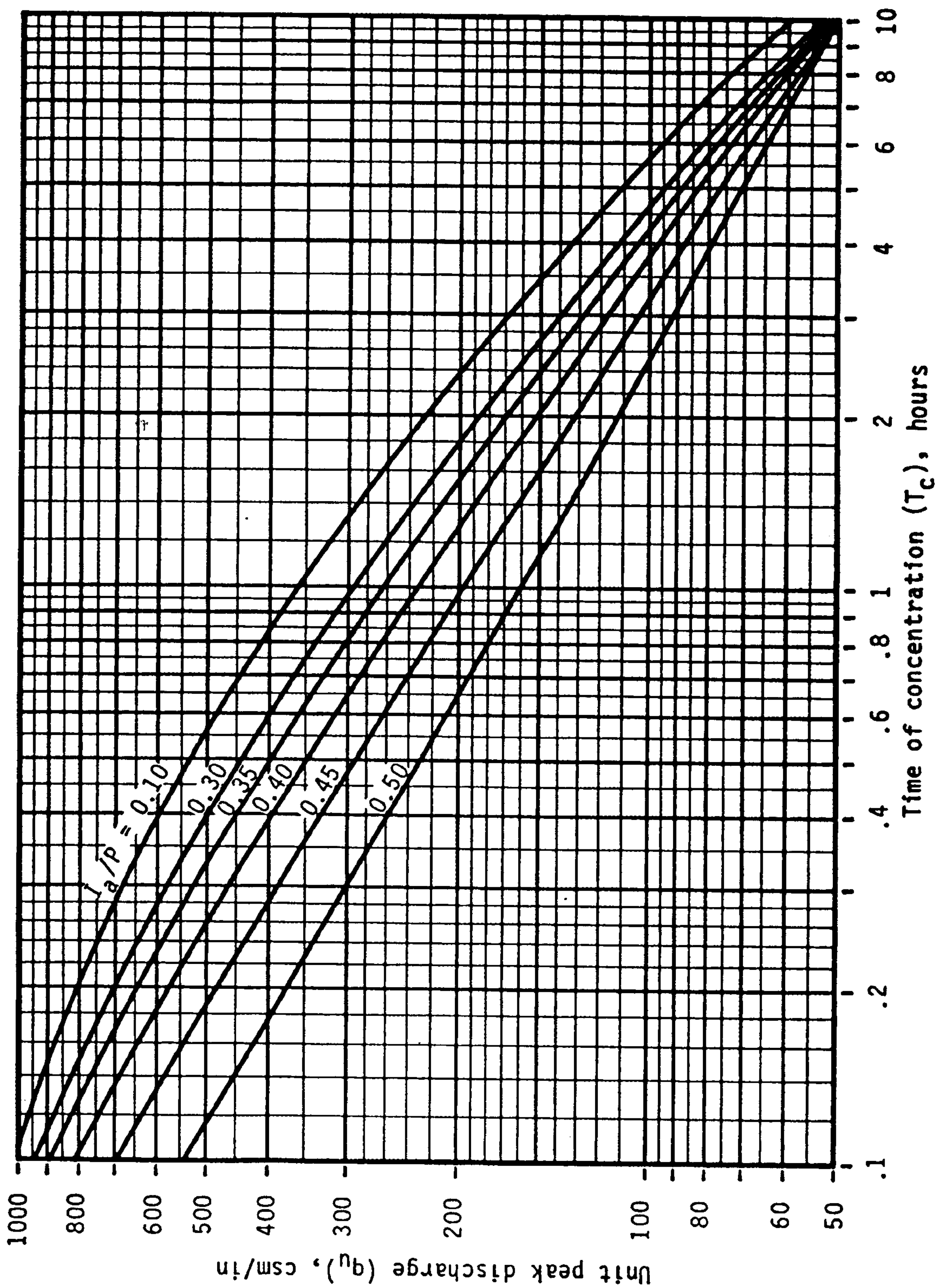


Figure 2.1.5-6
SCS Type II Unit Peak Discharge Graph
(Source: SCS, TR-55, Second Edition, June 1986)

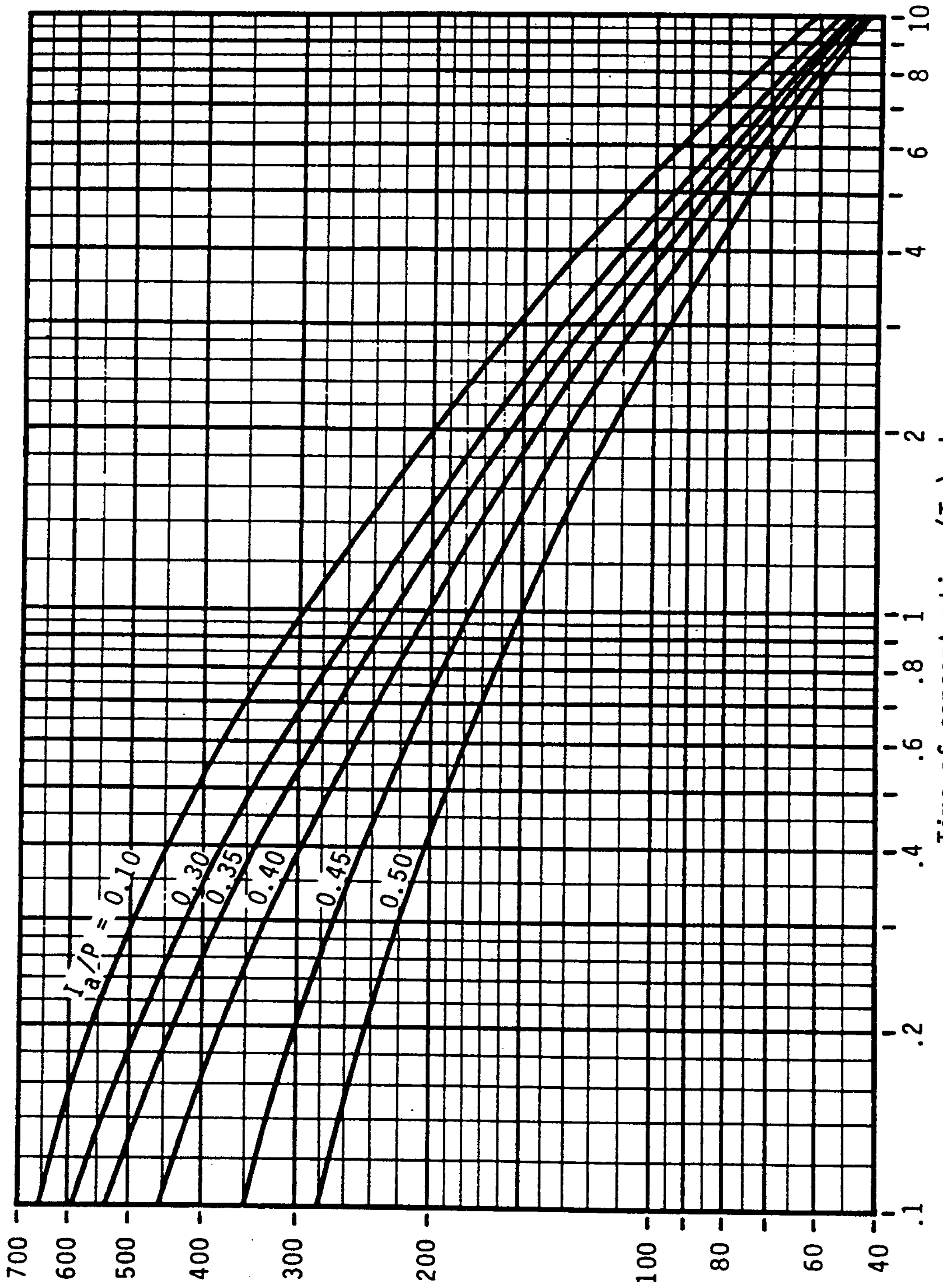


Figure 2.1.5-7
SCS Type III Unit Peak Discharge Graph
(Source: SCS, TR-55, Second Edition, June 1986)

2.1.5.8 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 7.92 inches (.33 in/hr x 24 hours – From Appendix A, Table A-10).
- The 1-year, 24-hour rainfall is 3.36 inches (.14 in/hr x 24 hours – From Appendix A, Table A-10).
- Composite weighted runoff coefficient is:

Dev. #	Area	% Total	CN	Composite CN
1	10 ac.	0.20	55	11.0
2	10 ac.	0.20	70	14.0
3	20 ac.	0.40	72	28.8
4	10 ac.	0.20	91	18.2
Total	50 ac.	1.00		72

* from Equation 2.1.6, Q (100-year) = 4.6 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	0.50

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

Segment 1 - Travel time from equation 2.1.9 with P_2 = 4.08 inches
(0.17 x 24 – Appendix A, Table A-10)

$$T_t = [0.42(0.24 \text{ X } 40)^{0.8}] / [(4.08)^{0.5} (.020)^{0.4}] = 6.07 \text{ minutes}$$

Segment 2 - Travel time from Figure 2.1.5-5 or equation 2.1.10
V = 2.1 ft/sec (from equation 2.1.10)
T_t = 750 / 60 (2.1) = 5.95 minutes

Segment 3 - Using equation 2.1.12
$V = (1.49/.06)(1.43)^{0.67}(.005)^{0.5} = 2.23$ ft/sec
T_t = 1100 / 60 (2.23) = 8.22 minutes

t_c = 6.07 + 5.95 + 8.22 = 20.24 minutes (.34 hours)

(3) Calculate I_a/P for C_n = 72 (Table 2.1.5-1), I_a = .778 (Table 2.1.5-3)

I_a/P = (.778 / 7.92) = .098 (Note: Use I_a/P = .10 to facilitate use of Figure 2.1.5-6. Straight line interpolation could also be used.)

(4) Unit discharge q_u (100-year) from Figure 2.1.5-6 = 650 csm/in, q_u (1-year) = 580 csm/in

(5) Calculate peak discharge with F_p = 1 using equation 2.1.13

Q_{100} = 650 (50/640)(4.6)(1) = 234 cfs

2.1.5.9 Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph from a drainage area. The SCS 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, SCS 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 homogeneous, where hydrographs need to be generated from sub-areas and then routed and combined at a point downstream, the engineer is referred to the procedures outlined by the SCS 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.

The unit hydrograph equations used in the SCS 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. SCS 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 2.1.6-1, 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 SCS method can be used without modification (peaking factor left at 484) in Regions 1, 2 and 4 generally when performing modeling analysis.

- The SCS method can be modified in that a peaking factor of 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 systems

The SCS 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:

(1) Development or selection of a design storm hyetograph. Often the SCS 24-hour storm described in subsection 2.1.5.3 is used. This storm is recommended for use in Georgia.

(2) Development of curve numbers and lag times for the watershed using the methods described in subsections 2.1.5.4, 2.1.5.5, and 2.1.5.6.

(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.

(4) Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the SCS rainfall-runoff equation (Equation 2.1.6).

(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”).

(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 SCS unit hydrograph approach with a peaking factor of 300, Figure 2.1.5-8 and Table 2.1.5-4 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 (Tp) and each value of q/q_u by q_u calculated as:

$$q_u = (PF\ A) / (T_p) \qquad \textbf{(2.1.14)}$$

Where: q_u= unit hydrograph peak rate of discharge (cfs)
PF = peaking factor (either 484 or 300)
A = area (mi^2)
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:

$$\frac{q}{q_u} = \left[\frac{t}{T_p} e^{\left(1-\frac{t}{T_p}\right)} \right]^X \qquad \textbf{(2.1.15)}$$

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

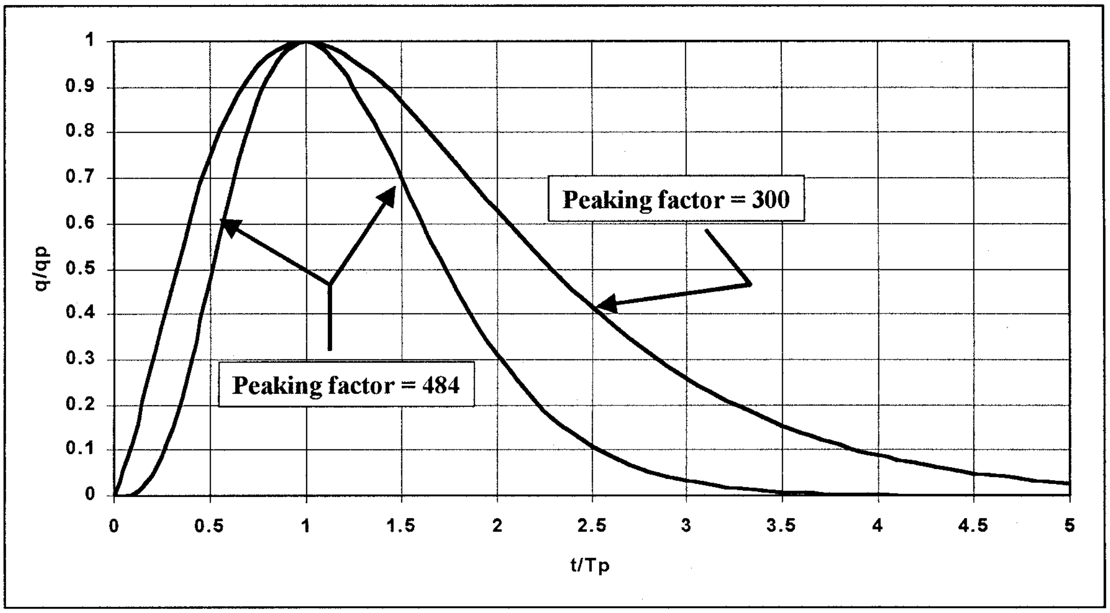


Figure 2.1.5-8 Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300

Table 2.1.5-4 Dimensionless Unit Hydrographs

	484		300	
t/Tt	**q/qu**	**Q/Qp**	**q/qu**	**Q/Qp**
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

Table 2.1.5-4 Dimensionless Unit Hydrographs (continued)

	484		300	
t/Tt	**q/qu**	**Q/Qp**	**q/qu**	**Q/Qp**
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
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

2.1.5.10 Example Problem 2

Compute the unit hydrograph for the 50-acre wooded watershed in example 2.1.5.8.

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

(2) Calculate q_{pu}

q_{pu} = PF A/T_p = (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 2.1.14 and 2.1.15, the table below has been derived.

Time		484		300	
t/Tp	time (min)	q/qpu	Q	q/qpu	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

Time		484		300	
t/Tp	time (min)	q/qpu	Q	q/qpu	q
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

2.1.6 U.S. Geological Survey Peak Flow and Hydrograph Method

2.1.6.1 Introduction

For the past 20 years the USGS has been collecting rain and streamflow data at various sites within the Atlanta metropolitan area and 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 discharge 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.

2.1.6.2 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.

2.1.6.3 Peak Discharge Equations

For a complete description of the USGS regression equations presented below, consult the USGS publication *Flood-Frequency Relations for Urban Streams in Georgia - 1994 Update, Water-Resources Investigation Report 95-4017*. Table 2.1.6-1 gives the USGS regression equations for urban streams in Georgia. Figure 2.1.6-1 shows the locations of the different regions throughout Georgia.

2.1.6.4 Peak Discharge Limitations for Urban Basins

Following are the limitations of the variables within the peak discharge equations. These equations should not be used on drainage areas which have physical characteristics outside the limits listed below:

Physical Characteristics	Minimum	Maximum	Units
A - Drainage Area	0.04	19.1	mi^2
TIA - Total Impervious Area	1.00	62	percent

Table 2.1.6-1 USGS Peak Flow Regression Equations

Frequency	Equations: Region 1	Equations: Region 2
2-year	$Q_2 = 167A^{0.73}TIA^{0.31}$	$Q_2 = 145A^{0.70}TIA^{0.31}$
5-year	$Q_5 = 301A^{0.71}TIA^{0.26}$	$Q_5 = 258A^{0.69}TIA^{0.26}$
10-year	$Q_{10} = 405A^{0.70}TIA^{0.21}$	$Q_{10} = 351A^{0.70}TIA^{0.21}$
25-year	$Q_{25} = 527A^{0.70}TIA^{0.20}$	$Q_{25} = 452A^{0.70}TIA^{0.20}$
50-year	$Q_{50} = 643A^{0.69}TIA^{0.18}$	$Q_{50} = 548A^{0.70}TIA^{0.18}$
100-year	$Q_{100} = 762A^{0.69}TIA^{0.17}$	$Q_{100} = 644A^{0.70}TIA^{0.17}$
200-year	$Q_{200} = 892A^{0.68}TIA^{0.16}$	$Q_{200} = 747A^{0.70}TIA^{0.16}$
500-year	$Q_{500} = 1063A^{0.68}TIA^{0.14}$	$Q_{500} = 888A^{0.70}TIA^{0.14}$
	Region 3	Region 4
2-year	$Q_2 = 54.6A^{0.69}TIA^{0.31}$	$Q_2 = 110A^{0.66}TIA^{0.31}$
5-year	$Q_5 = 99.7A^{0.69}TIA^{0.26}$	$Q_5 = 237A^{0.66}TIA^{0.26}$
10-year	$Q_{10} = 164A^{0.71}TIA^{0.21}$	$Q_{10} = 350A^{0.68}TIA^{0.21}$
25-year	$Q_{25} = 226A^{0.71}TIA^{0.20}$	$Q_{25} = 478A^{0.69}TIA^{0.20}$
50-year	$Q_{50} = 288A^{0.72}TIA^{0.18}$	$Q_{50} = 596A^{0.70}TIA^{0.18}$
100-year	$Q_{100} = 355A^{0.72}TIA^{0.17}$	$Q_{100} = 717A^{0.70}TIA^{0.17}$
200-year	$Q_{200} = 428A^{0.72}TIA^{0.16}$	$Q_{200} = 843A^{0.70}TIA^{0.16}$
500-year	$Q_{500} = 531A^{0.72}TIA^{0.14}$	$Q_{500} = 1017A^{0.71}TIA^{0.14}$
	Region - Rome	
2-year	$Q_2 = 107A^{0.73}TIA^{0.31}$	
5-year	$Q_5 = 183A^{0.71}TIA^{0.26}$	
10-year	$Q_{10} = 249A^{0.70}TIA^{0.21}$	
25-year	$Q_{25} = 316A^{0.70}TIA^{0.20}$	
50-year	$Q_{50} = 379A^{0.69}TIA^{0.18}$	
100-year	$Q_{100} = 440A^{0.69}TIA^{0.17}$	
200-year	$Q_{200} = 505A^{0.68}TIA^{0.16}$	
500-year	$Q_{500} = 589A^{0.68}TIA^{0.14}$	

For these equations:A = drainage area, mi^2
TIA = total impervious area, % (e.g., 30% would be input as 30 not .30)

Source: USGS, 1994

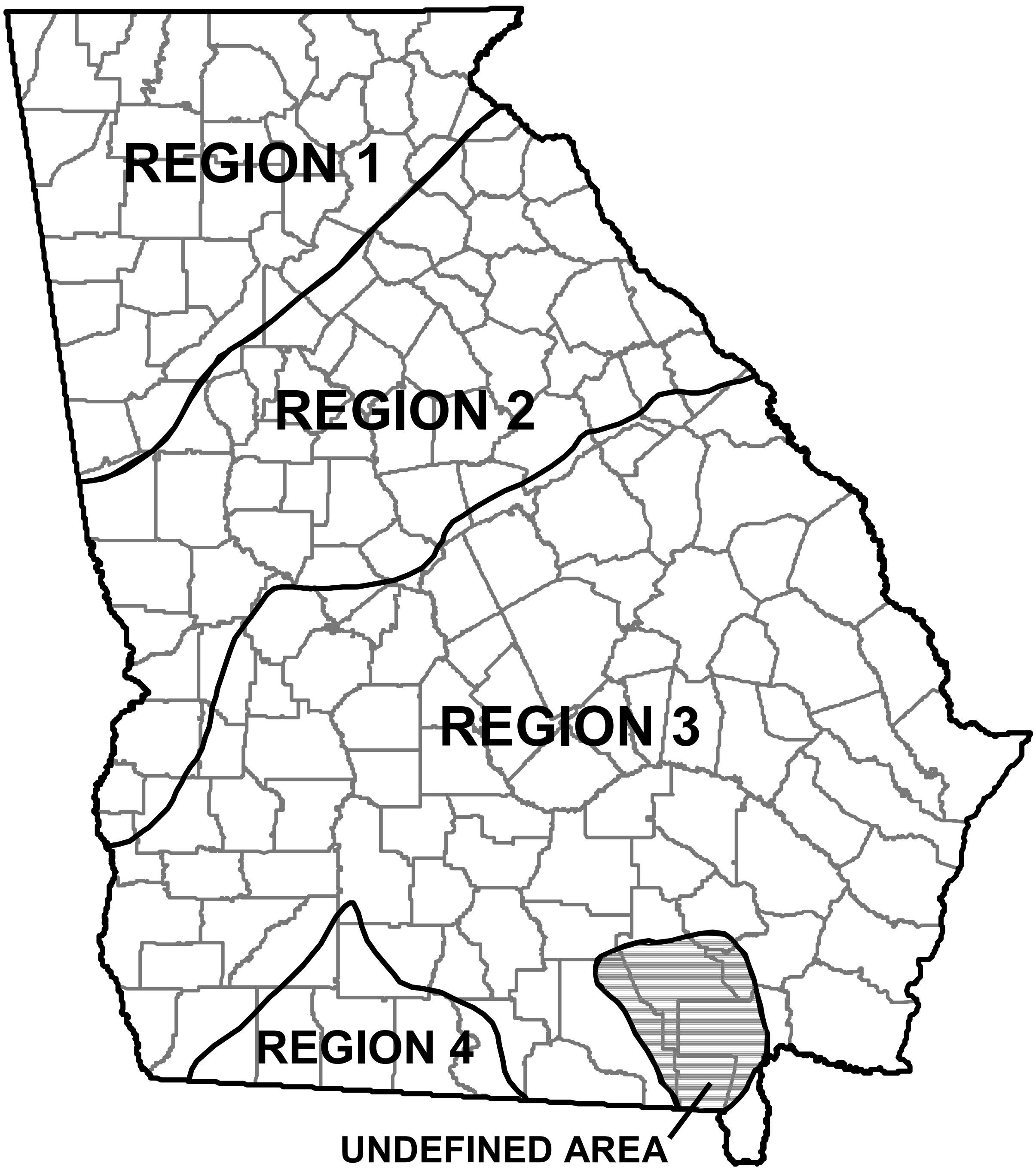


Figure 2.1.6-1 USGS Hydrologic Regions in Georgia
(Source: USGS, 1994)

2.1.6.5 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 dimensionless hydrograph, consult the USGS publication *Simulation of Flood Hydrographs for Georgia Streams, Water-Resources Investigation Report 86-4004*. Table 2.1.6-2 lists the time and discharge ratios for the dimensionless hydrograph.

Table 2.1.6-2 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

The lag time equations for calculating the dimensionless hydrograph are:

North of the Fall Line (rural):

$$\mathbf{T_L = 4.64A^{0.49}S^{-0.21}} \qquad \textbf{(2.1.16)}$$

South of the Fall Line (rural):

$$\mathbf{T_L = 13.6A^{0.43}S^{-0.31}} \qquad \textbf{(2.1.17)}$$

Regions 1, 2 and 3 (urban):

$$\mathbf{T_L = 7.86A^{0.35}TIA^{-0.22}S^{-0.31}} \qquad \textbf{(2.1.18)}$$

Region 4 (urban):

$$T_L = 6.10A^{0.35}TIA^{-0.22}S^{-0.31} \quad (2.1.19)$$

Where: T_L = lag time (hours)
A = drainage area (mi^2)
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.

2.1.6.6 Hydrograph Limitations

Following 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 below:

Physical Characteristics	Minimum	Maximum	Units
North of the Fall Line (rural)			
A - Drainage Area	0.3	500	mi^2
S - Main Channel Slope	5.0	200	feet per mile
South of the Fall Line (rural)			
A - Drainage Area	0.2	500	mi^2
S - Main Channel Slope	1.3	60	feet per mile
Regions 1, 2 & 3 (urban)			
A - Drainage Area	0.04	19.1	mi^2
S - Main Channel Slope	9.4	772.0	feet per mile
TIA - Total Impervious Area	1.0	61.6	percent
Region 4 (urban)			
A - Drainage Area	0.12	2.9	mi^2
S - Main Channel Slope	19.4	110.0	feet per mile
TIA - Total Impervious Area	6.1	42.4	percent

2.1.6.7 Rural (or Undeveloped) Basins

The USGS has recently revised the equation for estimating peak discharges for rural basins, as seen in Table 2.1.6-3. For a complete discussion of the development of these equations consult the USGS publication *Techniques for Estimating Magnitude and Frequency of Floods in Rural Basins Of Georgia, Water-Resources Investigations Report 93-4016*.

2.1.6.8 Rural (or Undeveloped) Basin Limitations

Following are the limitations associated with the rural basin equations given above:

Physical Characteristics	Minimum	Maximum	Units
Region 1 - A - Drainage Area	0.17	730	mi^2
Region 2 - A - Drainage Area	0.10	3,000	mi^2
Region 3 - A - Drainage Area	0.14	3,000	mi^2
Region 4 - A - Drainage Area	0.25	2,000	mi^2

Table 2.1.6-3 USGS Rural Peak Equations[1]

Frequency	Equations Region 1	Equations Region 2
Q_2	$207A^{0.654}$	$182A^{0.622}$
Q_5	$357A^{0.632}$	$311A^{0.616}$
Q_{10}	$482A^{0.619}$	$411A^{0.613}$
Q_{25}	$666A^{0.605}$	$552A^{0.610}$
Q_{50}	$827A^{0.595}$	$669A^{0.607}$
Q_{100}	$1010A^{0.584}$	$794A^{0.605}$
Q_{200}	$1220A^{0.575}$	$931A^{0.603}$
Q_{500}	$1530A^{0.563}$	$1130A^{0.601}$

Frequency	Equations Region 3	Equations Region 4
Q_2	$76A^{0.620}$	$142A^{0.591}$
Q_5	$133A^{0.620}$	$288A^{0.589}$
Q_{10}	$176A^{0.621}$	$410A^{0.591}$
Q_{25}	$237A^{0.623}$	$591A^{0.595}$
Q_{50}	$287A^{0.625}$	$748A^{0.599}$
Q_{100}	$340A^{0.627}$	$926A^{0.602}$
Q_{200}	$396A^{0.629}$	$1120A^{0.606}$
Q_{500}	$474A^{0.632}$	$1420A^{0.611}$

A - Drainage Area in mi^2

[1] For estimating discharges for a specific recurrence interval at sites where gaged data are available from the USGS, follow procedures outlined on pages 16 and 17 in the USGS publication *Techniques for Estimating Magnitude and Frequency of Floods in Rural Basins in Georgia, Water-Resources Investigation Report 93-4016, 1993*.

Source: USGS, 1993

2.1.6.9 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 mi^2
- Main Channel Slope = 117 ft/mi
- Total Impervious Area (TIA) = 32%

<u>Peak Discharge Calculations</u>

100-year Rural Peak Discharge:

$Q_{100} = 1010A^{0.584} = 1010(.273)^{0.584} = 473$ cfs

100-year Developed Peak Flow:

$Q_{100} = 762A^{0.69}(32)^{0.17}$

$Q_{100} = 762(.273)^{0.69}(32)^{0.17} = 561$ cfs

<u>Flood Hydrograph Calculations</u>

<u>Lag Time Calculations</u>

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

<u>Hydrograph Calculations</u>

Using the dimensionless USGS hydrograph given in Table 2.1.6-2, the following calculations are done to determine the ordinates of the flood hydrograph.

Time (t) = $t/T_L \times 0.53$ t/T_L from Table 2.1.6-2

Discharge (Q) = $Q/Q_p \times 561$ Q/Q_p from Table 2.1.6-2

Coordinates for the flood hydrograph are given in Table 2.1.6-4 on the next page.

Table 2.1.6-4 Flood Hydrograph

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

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

2.1.7 Water Quality Volume and Peak Flow

2.1.7.1 Water Quality Volume Calculation

The Water Quality Volume (WQ_v) is the 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 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:

$$\mathbf{R_v = 0.05 + 0.009(I)} \qquad \mathbf{(2.1.20)}$$

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:

$$\mathbf{WQ_v = \frac{1.2\ R_v\ A}{12}} \qquad \mathbf{(2.1.21)}$$

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 $1.2(R_v) = Q_{wv}$

2.1.7.2 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 SCS 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.

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

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

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

(Step 2) Once a CN is computed, the time of concentration (t_c) is computed (based on the methods described in this section).

(Step 3) Using the computed CN, t_c 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 SCS Peak Runoff Rate Estimation technique of subection 2.1.5.7. Use appropriate rainfall distribution type (either Type II or Type III in Georgia).

- Read initial abstraction (Ia), compute I_a/P
- Read the unit peak discharge (q_u) for appropriate tc
- Using WQ_v, compute the peak discharge (Q_{wq})

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

where Q_{wq} = the water quality peak discharge (cfs)
q_u = the unit peak discharge (cfs/mi²/inch)
A = drainage area (mi^2)
Q_{wv} = Water Quality Volume, in inches ($1.2R_v$)

2.1.7.3 Example Problem

Using the data and information from the example problem in subsection 2.1.5.8 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(R_V)(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 2.1.5-6 for I_a/P = 0.317 q_u = 535 cfs/mi^2/in

Compute water quality peak flow:

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

2.1.8 Water Balance Calculations

2.1.8.1 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 retention pond and wetland design.

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.

2.1.8.2 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):

$$\Delta V = \Sigma I - \Sigma O \qquad \textbf{(2.1.22)}$$

Where:
Δ = "change in"
V = pond volume (ac-ft)
Σ = "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 pond or wetland. Equation 2.1.22 can be changed to reflect these factors.

$$\Delta V = P + Ro + Bf - I - E - Et - Of \qquad \textbf{(2.1.23)}$$

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) – Monthly rainfall values can be obtained from State climatology data at:

http://climate.engr.uga.edu/info.html

Monthly values are commonly used for calculations of values over a season. Rainfall is then the direct amount that falls on the pond surface for the period in question. When multiplied by the pond surface area (in acres) it becomes acre-feet of volume. Table 2.1.8-1 shows monthly rainfall rates for Atlanta based on a 30-year period of record at Hartsfield-Atlanta International Airport.

Runoff (R_o) – 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 2.1.20 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 2.1.24 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

$$\mathbf{Q = 0.9\,PR_v} \qquad \textbf{(2.1.24)}$$

Where: P = precipitation (in)
Q = runoff volume (in)
R_v = volumetric runoff coefficient [see equation 2.1.20]

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:

$$\mathbf{Q = 0.235P/S^{0.64} - 0.161} \qquad \textbf{(2.1.25)}$$

Where: P = precipitation (in)
Q = runoff volume (in)
S = potential maximum retention (in) [see equation 2.1.6]

Table 2.1.8-1 Water Balance Values for Atlanta, Georgia

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Precipitation (ft)	0.40	0.40	0.48	0.36	0.36	0.30	0.42	0.31	0.29	0.25	0.32	0.36
Turf Evptr. (ft)	0.06	0.07	0.15	0.27	0.44	0.56	0.61	0.56	0.41	0.25	0.11	0.06

Annual Precipitation (ft)	4.25
Turf Evptr. (ft)	3.55

Source: Ferguson and Debo, 1990 and http://www.griffin.peachnet.edu/

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:

$$\mathbf{I = Ak_hG_h} \qquad \textbf{(2.1.26)}$$

Where: I = infiltration (ac-ft/day)
A = cross sectional area through which the water infiltrates (ac)
K_h = saturated hydraulic conductivity or infiltration rate (ft/day)
G_h = hydraulic gradient = pressure head/distance

G_h can be set equal to 1.0 for pond bottoms and 0.5 for pond sides steeper than about 4:1. Infiltration rate can be established through testing, though not always accurately. As a first cut estimate Table 2.1.8-2 can be used.

Table 2.1.8-2 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 lake 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 2.1.8-3 gives pan evaporation rate distributions for a typical 12-month period based on pan evaporation information from five stations in and around Georgia. Figure 2.1.8-1 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 of it 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 Table 2.1.8-3.

Table 2.1.8-3 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%

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. Values for turf are given in Table 2.1.8-1 based on the Blaney-Criddle method. 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.

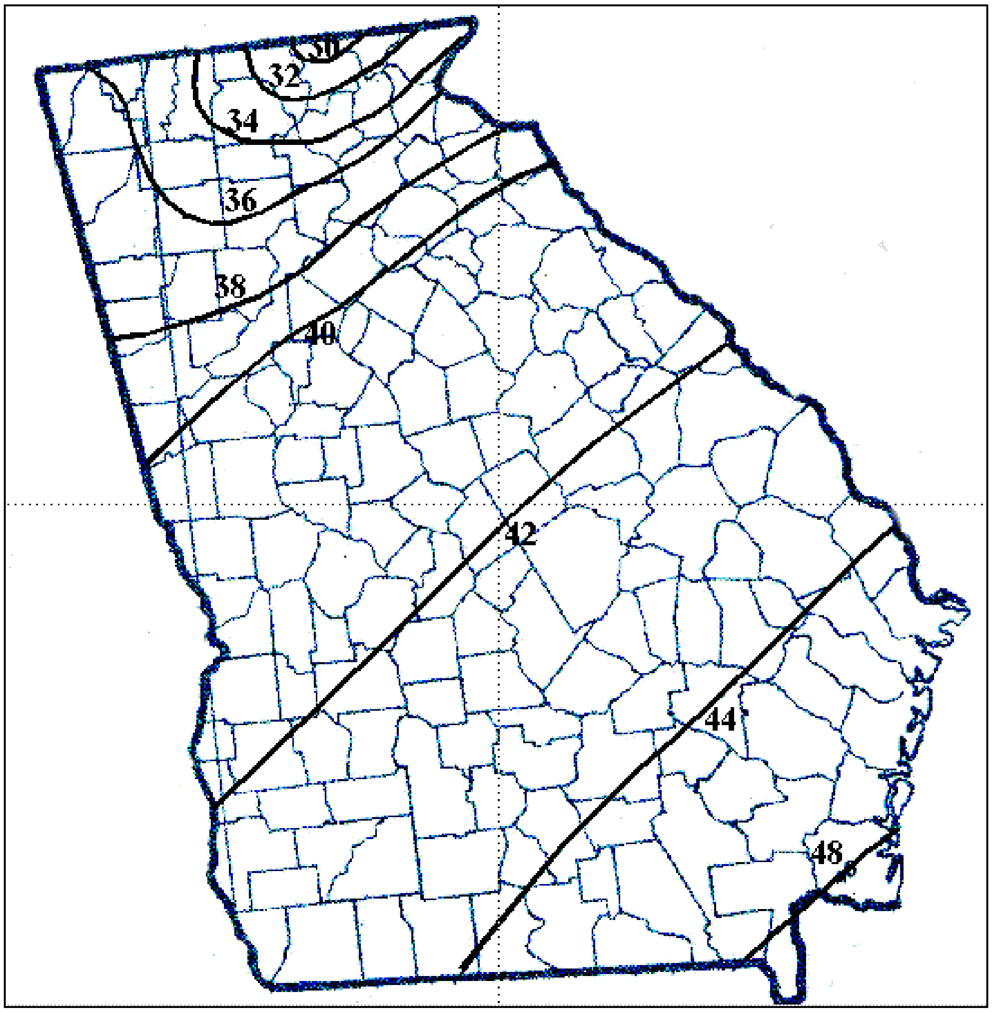


Figure 2.1.8-1 Average Annual Free Water Surface Evaporation (in inches)
(Source: NOAA, 1982)

2.1.8.3 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 2.1.20, $R_v = 0.05 + 0.009\ (75) = 0.73$. With the correction factor of 0.9 the watershed efficiency is 0.65.
- The annual lake evaporation from Figure 2.1.8-1 is about 42 inches.
- For a sandy clay loam the infiltration rate is I = 0.34 ft/day (Table 2.1.8-2).
- 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 2.1.8-4 shows summary calculations for this site for each month of the year.

Table 2.1.8-4 Summary Information for Austin Acres

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	**Ro (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	0.13	0.12	0.10	0.14
7	**E (ac-ft)**	0.06	0.08	0.13	0.18	0.22	0.23	0.23	0.21	0.16	0.12	0.08	0.06
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	1.37	0.36	0.00	0.00

<u>Explanation of Table:</u>

1. Months of year
2. Days per month
3. Monthly precipitation from web site is shown in Figure 2.1.8-2.
4. Distribution of evaporation by month from Table 2.1.8-3.
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 2.1.8-2 Augusta Precipitation Information

2.1.9 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.

2.1.9.1 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 2.1.9-1. 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.

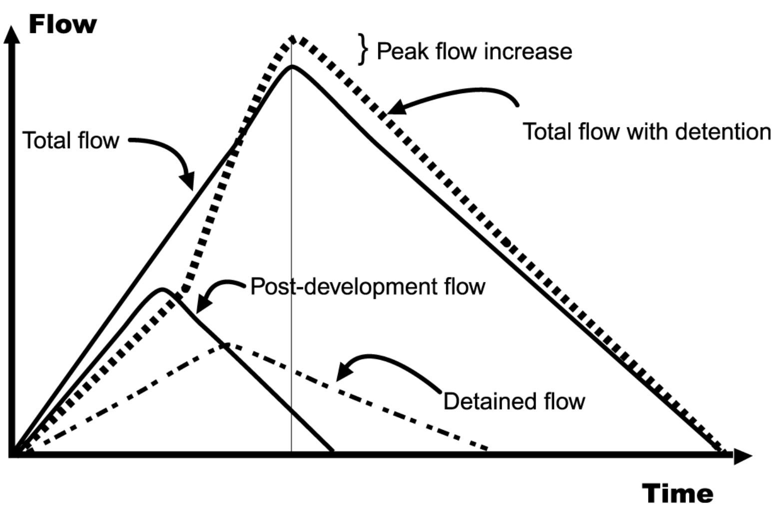


Figure 2.1.9-1 Detention Timing Example

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 2.1.9-2 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.

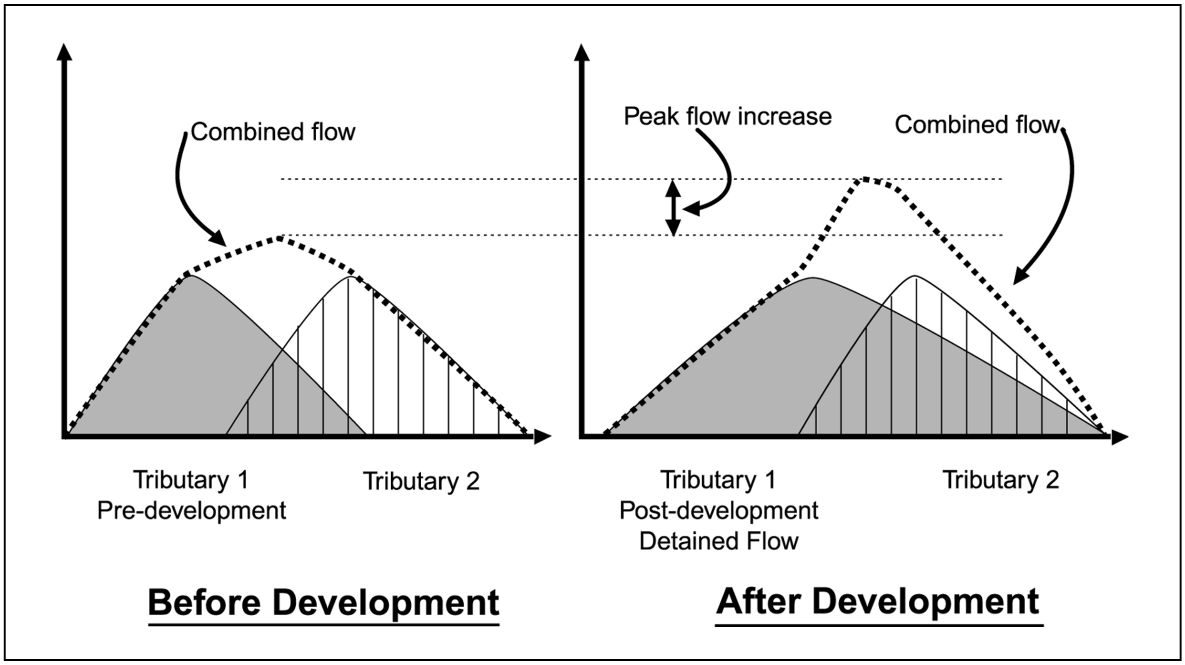


Figure 2.1.9-2 Effect of Increased Post-Development Runoff Volume with Detention on a Downstream Hydrograph

2.1.9.2 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 still need to be addressed.

2.1.9.3 Example Problem

Figure 2.1.9-3 illustrates the concept of the ten-percent rule for two sites in a watershed.

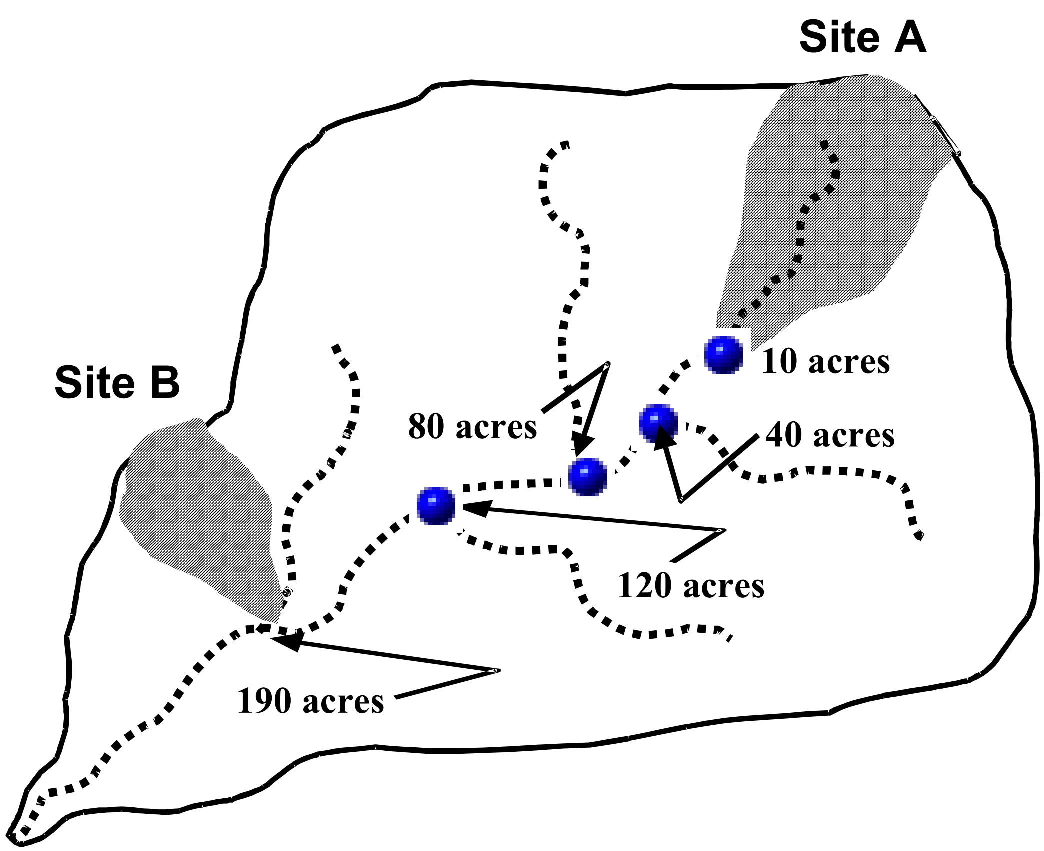


Figure 2.1.9-3 Example of the Ten-Percent Rule

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.

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SECTION

2.2

STORAGE DESIGN

2.2.1 General Storage Concepts

2.2.1.1 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 structural stormwater controls and/or nonstructural features and landscaped areas. Figure 2.2.1-1 illustrates various storage facilities that can be considered for a development site.

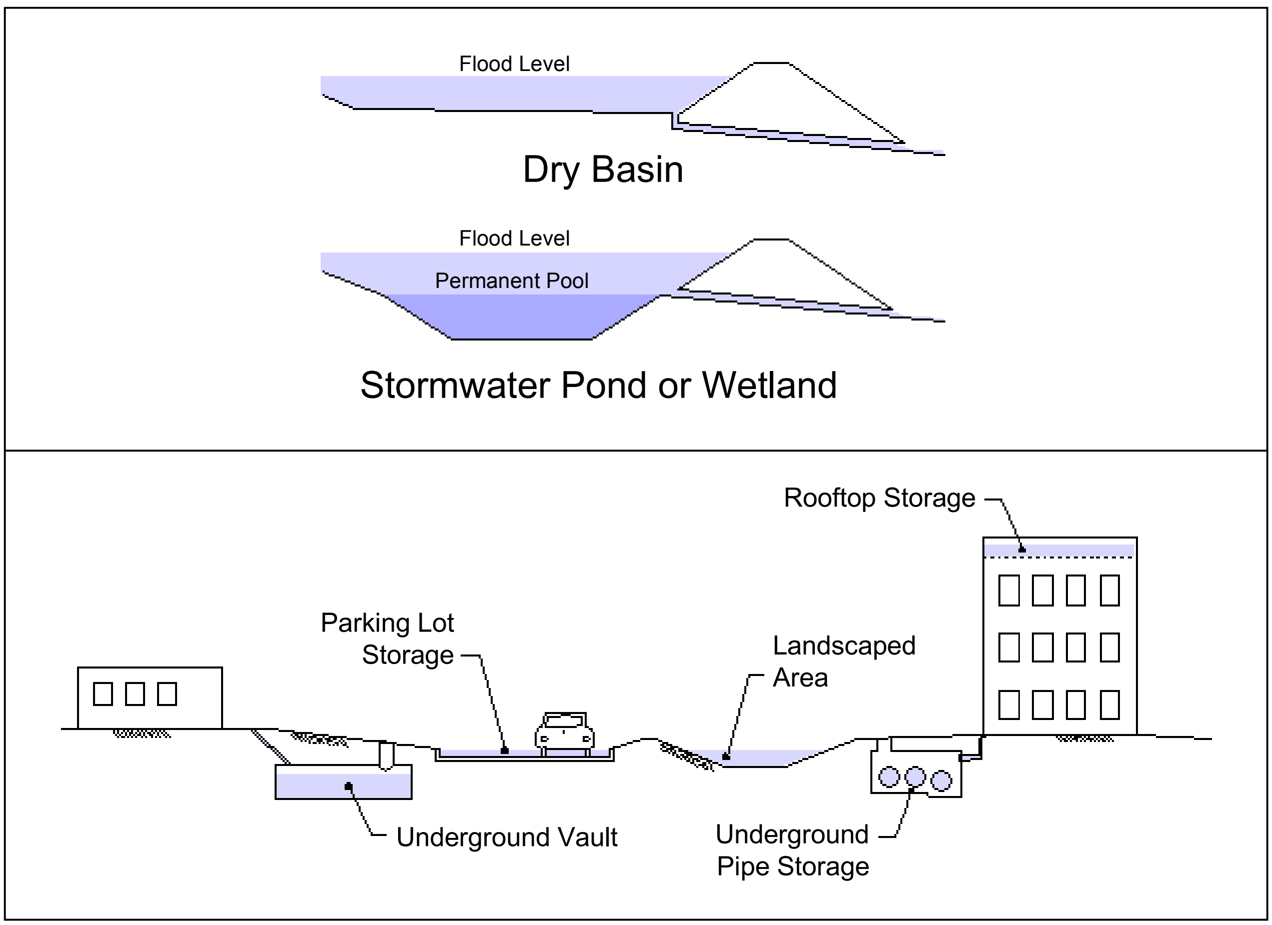

Figure 2.2.1-1 Examples of Typical Stormwater Storage Facilities

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 structural control designs (wet ED pond, micropool ED pond, and shallow ED marsh) 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, that 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 3.1.

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 2.2.1-2 illustrates on-line versus off-line storage.

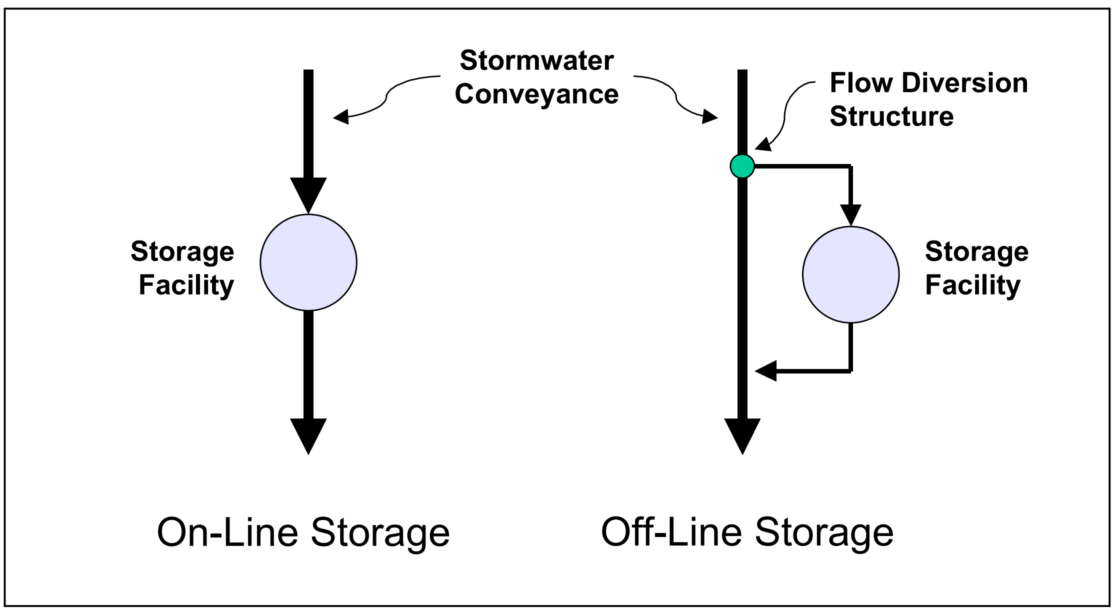

Figure 2.2.1-2 On-Line versus Off-Line Storage

2.2.1.2 Storage Classification

Stormwater storage(s) can be classified as either detention, extended detention or retention. Some facilities include one or more types of storage.

2.2.1.3 Stage-Storage Relationship

A stage-storage curve defines the relationship between the depth of water and storage volume in a storage facility (see Figure 2.2.1-3). The volume of storage can be calculated by using simple geometric formulas expressed as a function of depth.

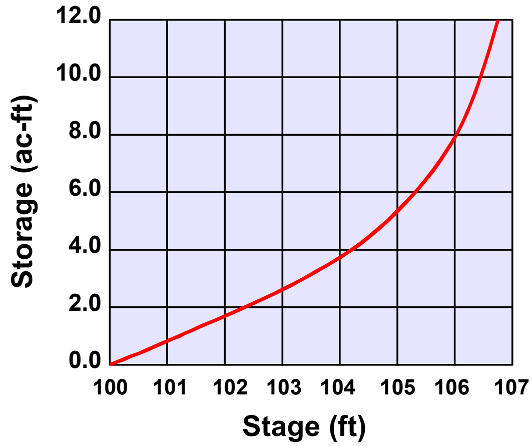

Figure 2.2.1-3 Stage-Storage Curve

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 2.2.1-4) is expressed as:

$$V_{1,2} = [(A_1 + A_2)/2]d \qquad (2.2.1)$$

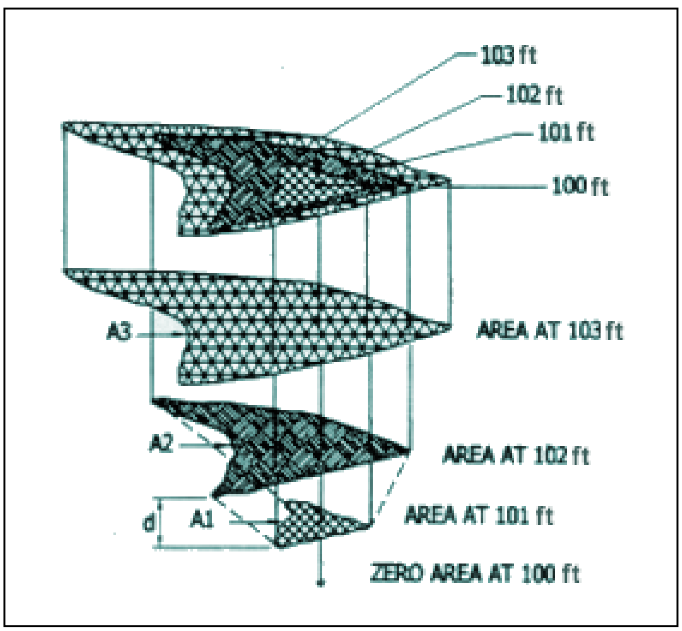

Figure 2.2.1-4 Double-End Area Method

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)

The frustum of a pyramid formula is expressed as:

$$V = d/3\ [A_1 + (A_1 \times A_2)^{0.5} + A_2]/3 \qquad (2.2.2)$$

Where: V = volume of frustum of a pyramid (ft^3)
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:

$$V = LWD + (L + W)\, ZD^2 + 4/3\, Z^2 D^3 \qquad (2.2.3)$$

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:

$$V = 1.047\, D\, (R_1^2 + R_2^2 + R_1 R_2) \qquad (2.2.4)$$

$$V = 1.047\, D\, (3\, R_1^2 + 3ZDR_1 + Z_2 D^2) \qquad (2.2.5)$$

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

2.2.1.4 Stage-Discharge Relationship

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility (see Figure 2.2.1-5). 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 2.3, *Outlet Structures*.

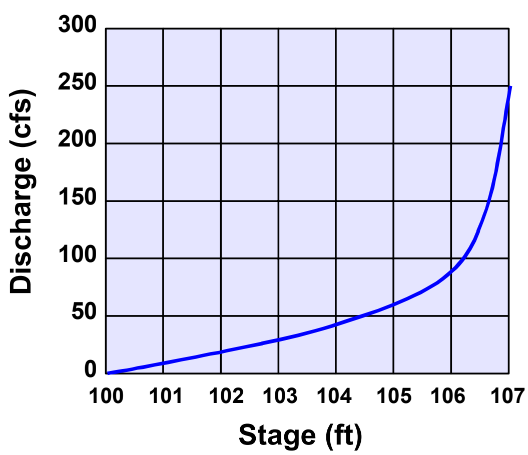

Figure 2.2.1-5 Stage-Discharge Curve

2.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.2.2-1 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 2.2.2-1 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	-
I	Inflow rate	cfs
L	Length	ft
Q,q	Peak inflow or outflow rate	cfs, in
R	Surface Radii	ft
S, V_S	Storage volume	ft^3
t_b	Time base on hydrograph	hrs
T_I	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	-

2.2.3 General Storage Design Procedures

2.2.3.1 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 downstream locations. Therefore, a downstream peak flow analysis should be performed as part of the storage facility design process (see subsection 2.1.9).

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 3 for further guidance and criteria for the design of structural stormwater controls.

2.2.3.2 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

2.2.3.3 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 runoff from the 25- (Q_{p25}), and 100-year (Q_f) design storms using the hydrologic methods outlined in Section 2.1. 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 2.2.4).

(Step 3) Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. 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 allowable 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 2.1.9).

(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 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.

2.2.4 Preliminary Detention Calculations

2.2.4.1 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.

2.2.4.2 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 2.2.4-1.

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

$$\mathbf{V_S = 0.5T_I\,(Q_i - Q_O)} \qquad \textbf{(2.2.6)}$$

Where: V_S = storage volume estimate (ft^3)
Q_i = peak inflow rate (cfs)
Q_O = peak outflow rate (cfs)
T_i = duration of basin inflow (s)

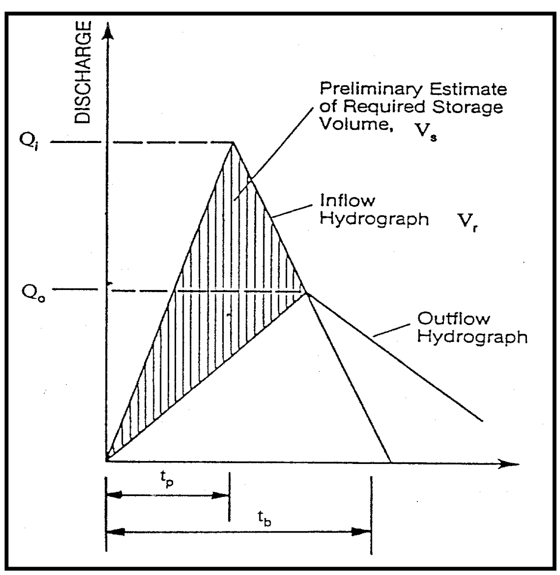

Figure 2.2.4-1 Triangular-Shaped Hydrographs
(For Preliminary Estimate of Required Storage Volume)

2.2.4.3 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 hydrograph, 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:

$$\frac{V_S}{V_r} = \frac{1.291\left(1 - \frac{Q_O}{Q_I}\right)^{0.753}}{\left(\frac{t_p}{t_b}\right)^{0.411}} \qquad \textbf{(2.2.7)}$$

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.

2.2.4.4 Peak Flow Reduction

A preliminary estimate 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 following equation (Wycoff and Singh, 1976):

$$Q_O/Q_i = 1 - 0.712(V_S/V_r)^{1.328}(t_b/t_p)^{0.546} \qquad \textbf{(2.2.8)}$$

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.

2.2.5 Channel Protection Volume Estimation

2.2.5.1 Introduction

The Simplified SCS Peak Runoff Rate Estimation approach (see subsection 2.1.5.7) 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 2.2.4 or the modified rational method in subsection 2.2.6 for preliminary detention calculations without formal routing.

2.2.5.2 Basic Approach

For CP_v estimation, using Figures 2.1.5-6 and 2.1.5-7 in Section 2.1, 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 2.2.5-1.

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 2.2.4-1 can also be used to estimate V_S/V_r.

$$V_S/V_r = 0.682 - 1.43\,(q_O/q_I) + 1.64\,(q_O/q_I)^2 - 0.804\,(q_O/q_I)^3 \qquad (2.2.9)$$

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:

$$V_S = \frac{(V_S/V_r)(Q_d)(A)}{12} \qquad (2.2.10)$$

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.

2.2.5.3 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

Other data include the following:

- Total impervious area = 18 acres
- % of pond and swamp area = 0

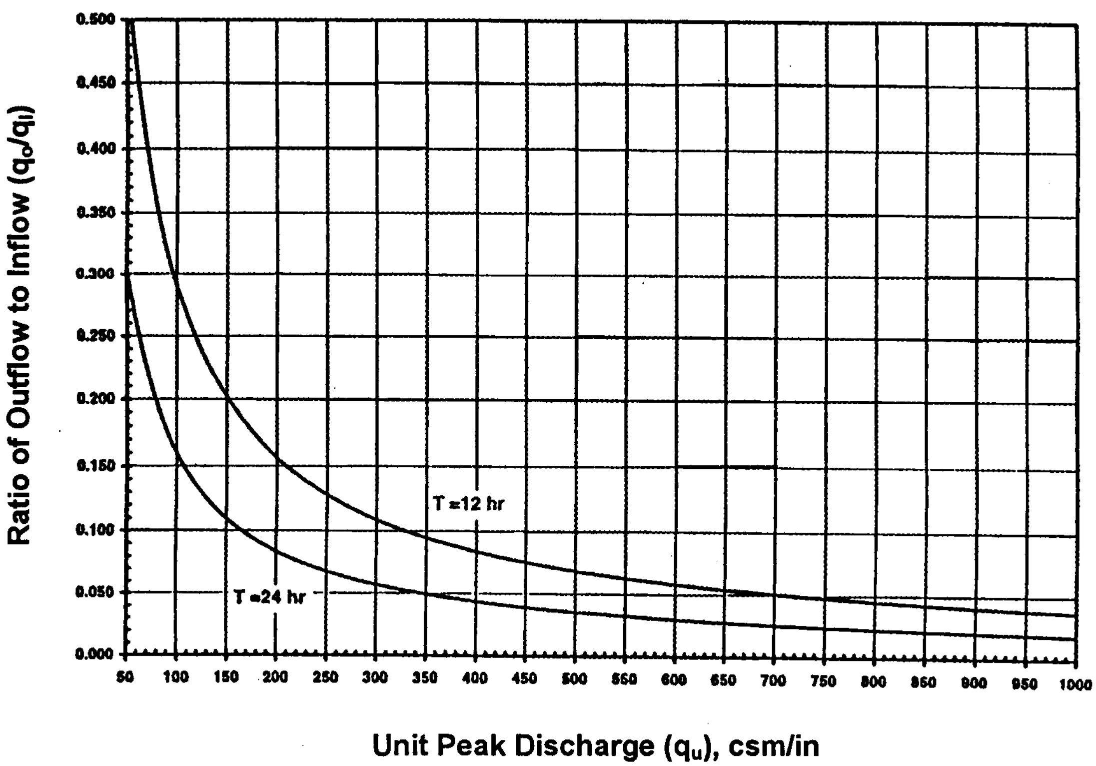

Figure 2.2.5-1 Detention Time vs. Discharge Ratios
(Source: MDE, 1998)

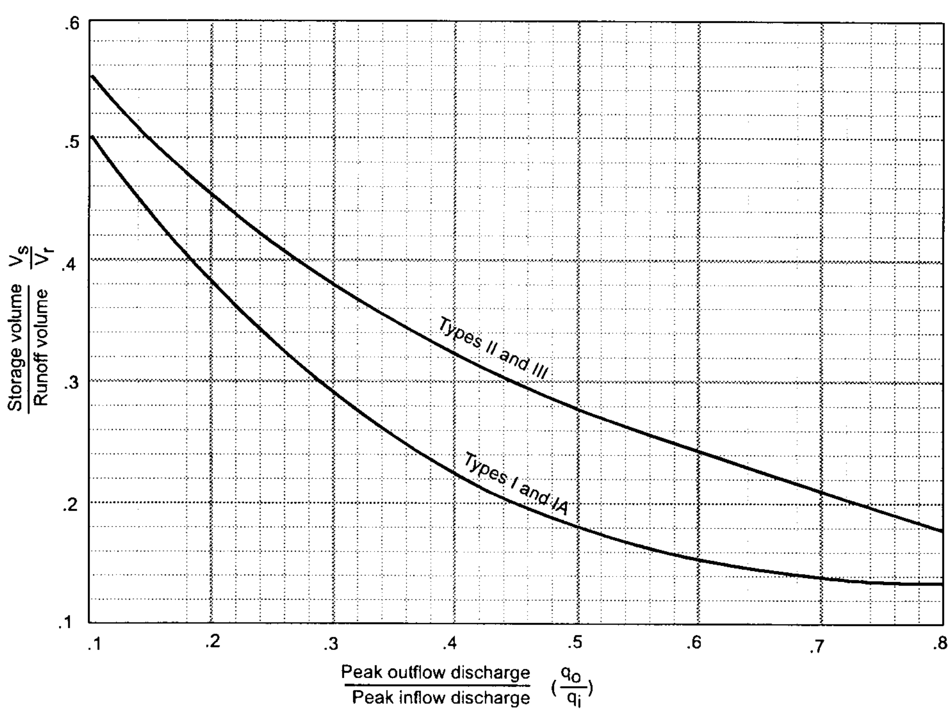

Figure 2.2.5-2
Approximate Detention Basin Routing for Rainfall Types I, IA, II, and III
(Source: TR-55, 1986)

Computations

(1) Calculate rainfall excess:

- The 100-year, 24-hour rainfall is 7.92 inches (.33 in/hr x 24 hours – From Appendix A, Table A-10).
- The 1-year, 24 hour rainfall is 3.36 inches (.14 in/hr x 24 hours – From Appendix A, Table A-10).
- Composite weighted runoff coefficient is:

Dev. #	Area (ac)	% Total	CN	Composite CN
1	10	0.20	55	11.0
2	10	0.20	70	14.0
3	20	0.40	72	28.8
4	10	0.20	91	18.2
Total	50	1.00		72

* From equation 2.1.6, Q (100-year) = 4.6 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	0.50 %

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

Segment 1 - Travel time from equation 2.1.9 with P_2 = 4.08 in
(0.17 x 24 – Appendix A, Table A-10)

$$T_t = [0.42(0.24 \text{ X } 40)^{0.8}] / [(4.08)^{0.5} (.020)^{0.4}] = 6.07 \text{ minutes}$$

Segment 2 - Travel time from Figure 2.1.5-5 or equation 2.1.10
V = 2.1 ft/sec (from equation 2.1.10)
T_t = 750 / 60 (2.1) = 5.95 minutes

Segment 3 - Using equation 2.1.12
$V = (1.49/.06)\ (1.43)^{0.67}\ (.005)^{0.5} = 2.23$ ft/sec
T_t = 1100 / 60 (2.23) = 8.22 minutes

t_c = 6.07 + 5.95 + 8.22 = 20.24 minutes (.34 hours)

(3) Calculate I_a/P for C_n = 72 (Table 2.1.5-1), I_a = .778 (Table 2.1.5-3)

I_a/P = (.778 / 7.92) = .098 (Note: Use I_a/P = .10 to facilitate use of Figure 2.1.5-6. Straight line interpolation could also be used.)

(4) Unit discharge q_u (100-year) from Figure 2.1.5-6 = 650 csm/in, q_u (1-year) = 580 csm/in

(5) Calculate peak discharge with F_p = 1 using equation 2.1.13

Q_{100} = 650 (50/640)(4.6)(1) = 234 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 2.2.5-1.
$q_O/q_I = 0.03$

For a Type II rainfall distribution,
$V_S/V_r = 0.682 – 1.43 (q_O/q_I) + 1.64 (q_O/q_I)^2 – 0.804 (q_O/q_I)^3$
$V_S/V_r = 0.682 – 1.43 (0.03) + 1.64 (0.03) – 0.804 (0.03) = 0.64$

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

2.2.6 The Modified Rational Method

2.2.6.1 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 2.2.6-1 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.

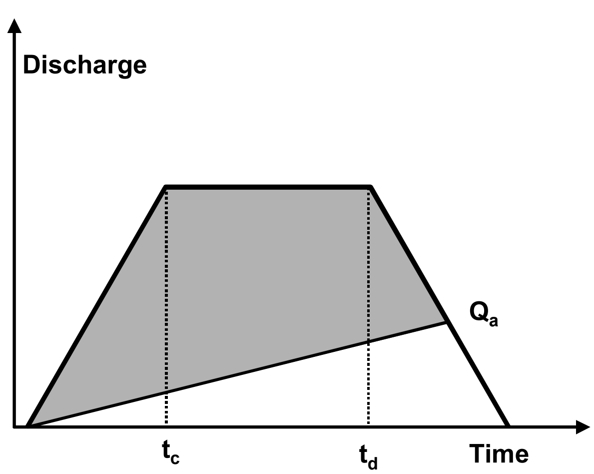

Figure 2.2.6-1 Modified Rational Definitions

2.2.6.2 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:

$$\mathbf{Q_a = C_a\ i\ A} \qquad \textbf{(2.2.11)}$$

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:

$$T_d = \sqrt{\frac{2CAab}{Q_a}} - b \qquad \textbf{(2.2.12)}$$

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 2.2.6-1

The required storage volume, in cubic feet can be obtained from equation 2.2.13.

$$\mathbf{V_{preliminary} = 60\ [CAa - (2CabAQ_a)^{1/2} + (Q_a/2)\ (b - t_c)]} \qquad \textbf{(2.2.13a)}$$

$$\mathbf{V_{max} = V_{preliminary} * P_{180}/P_{td}} \qquad \textbf{(2.2.13b)}$$

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 2.2.12.

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.

2.2.6.3 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 Appendix A. The following key information is obtained:

A = 5 acres Pre-development t_c = 21 minutes and C factor = 0.22
Slope is about 5% 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 2.2.11) (cfs)	3.67	4.96
a (from Table 2.2.6-1)	123.19	184.23
b (from Table 2.2.6-1)	15.91	19.96
V_{max} (equation 2.2.13) (ft^3)	16,017	23,199
P_{180} (from Table A-2) (in)	2.43	3.42
T_d (equation 2.2.12) (min)	49	57
P_{td} (from Table A-2) (in)	1.62	2.70
V_{max} (equation 2.2.13) (ft^3)	24,025	29,385

Table 2.2.6-1 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

Table 2.2.6-1 (continued)

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

References

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SECTION

2.3

OUTLET STRUCTURES

2.3.1 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.3.1-1 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 2.3.1-1 Symbols and Definitions

Symbol	Definition	Units
A,a	Cross sectional or surface area	ft^2
A_m	Drainage area	mi^2
B	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/s^2
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 rate	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
θ_g	Angle of the grate with respect to the horizontal	degrees

2.3.2 Primary Outlets

2.3.2.1 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.

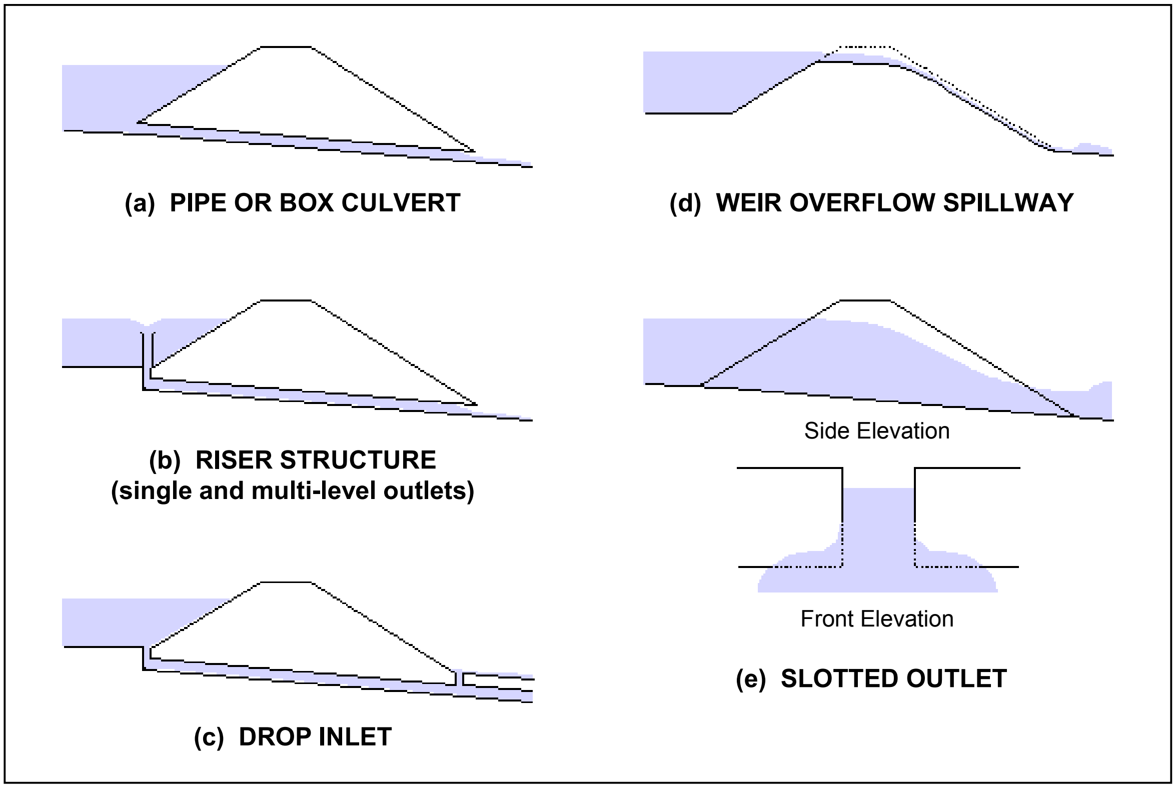

Figure 2.3.2-1 Typical Primary Outlets

2.3.2.2 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.

2.3.2.3 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 2.3.2-2(a), the orifice discharge can be determined using the standard orifice equation below.

$$Q = CA\,(2gH)^{0.5} \qquad (2.3.1)$$

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 the center of orifice 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 2.3.2-2(b).

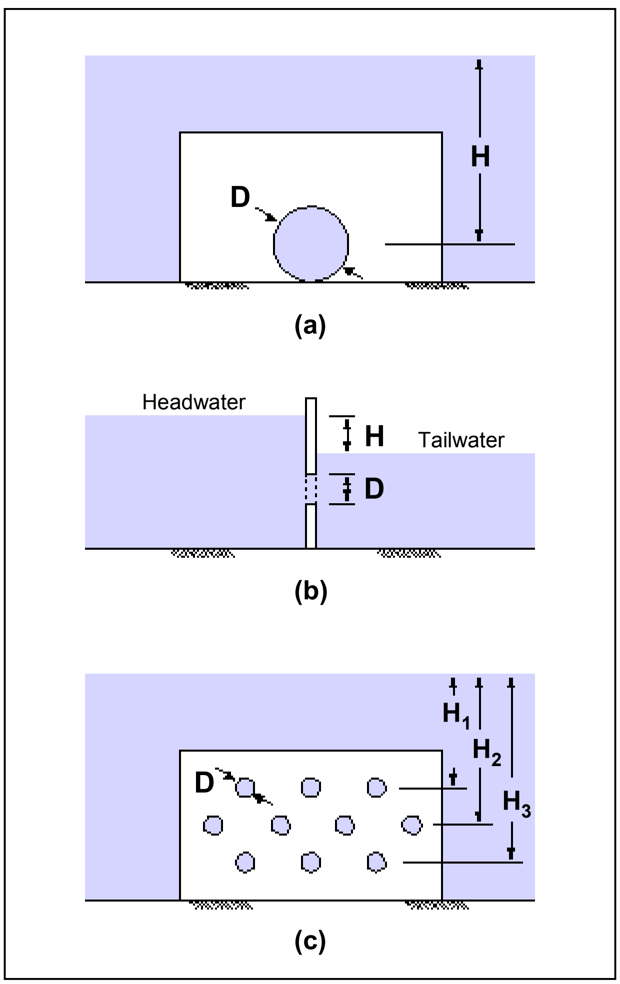

Figure 2.3.2-2 Orifice Definitions

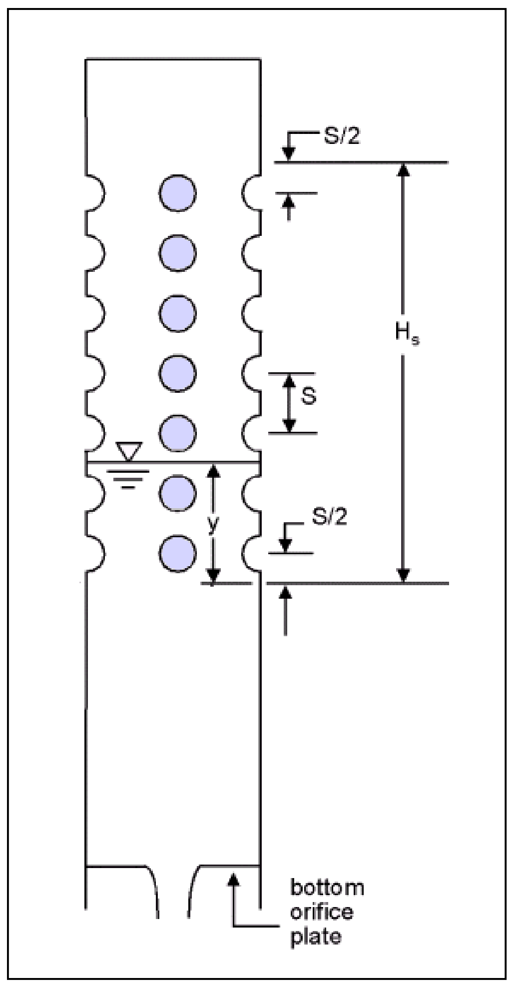

Figure 2.3.2-3 Perforated Riser

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:

$$\mathbf{Q = 0.6A\ (2gH)^{0.5} = 3.78D^2H^{0.5}} \qquad \mathbf{(2.3.2)}$$

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 2.3.2-2(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 2.3.2-1 gives appropriate dimensions. The vertical spacing between hole centerlines is always 4 inches.

Table 2.3.2-1 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

For rectangular slots the height is normally 2 inches with variable width. Only one column of rectangular slots is allowed.

Figure 2.3.2-4 provides a schematic of an orifice plate outlet structure for a wet ED pond showing the design pool elevations and the flow control mechanisms.

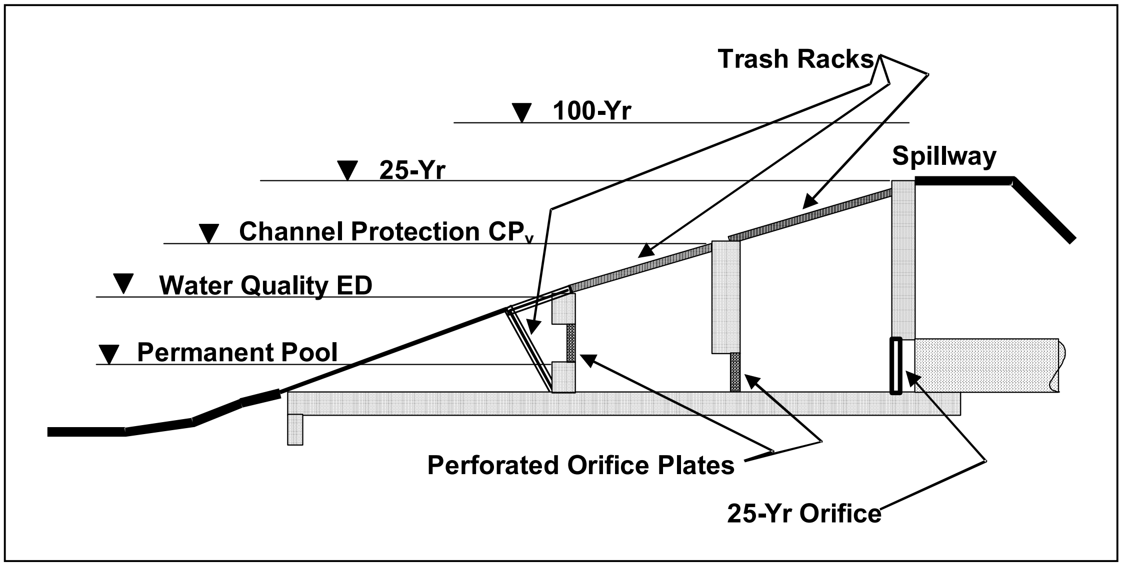

Figure 2.3.2-4 Schematic of Orifice Plate Outlet Structure

2.3.2.4 Perforated Risers

A special kind of orifice flow is a perforated riser as illustrated in Figure 2.3.2-3. 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 2.3.2-3, a shortcut formula has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988):

$$Q = C_p \frac{2A_p}{3H_s} \sqrt{2g} H^{3/2} \qquad \textbf{(2.3.3)}$$

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)

2.3.2.5 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 2.3.2.6). 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 4.3, *Culvert Design*, or by using equation 2.3.4 (NRCS, 1984).

The following equation is a general pipe flow equation that is derived through the use of the Bernoulli and continuity principles.

$$\mathbf{Q = a[(2gH) / (1 + k_m + k_pL)]0.5} \qquad \mathbf{(2.3.4)}$$

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)

2.3.2.6 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 2.3.2-5(a). The discharge equation for this configuration is (Chow, 1959):

$$\mathbf{Q = [(3.27 + 0.4(H/H_C)] LH^{1.5}} \qquad \mathbf{(2.3.5)}$$

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)

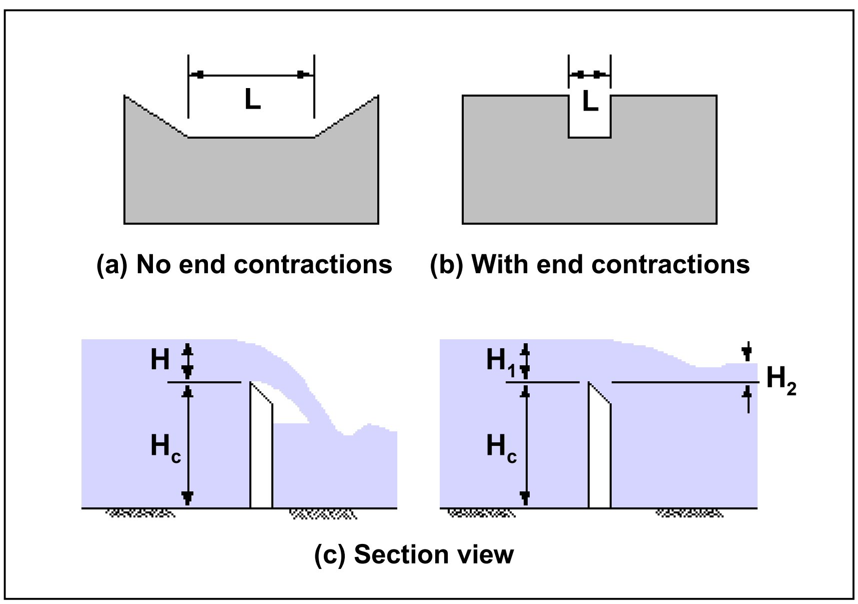

Figure 2.3.2-5 Sharp-Crested Weir

A sharp-crested weir with two end contractions is illustrated in Figure 2.3.2-5(b). The discharge equation for this configuration is (Chow, 1959):

$$Q = [(3.27 + 0.04(H/H_C)] (L - 0.2H) H^{1.5} \qquad (2.3.6)$$

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, 1976):

$$Q_S = Q_f (1 - (H_2/H_1)^{1.5})^{0.385} \qquad (2.3.7)$$

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)

2.3.2.7 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):

$$Q = CLH^{1.5} \qquad (2.3.8)$$

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 2.3.2-2.

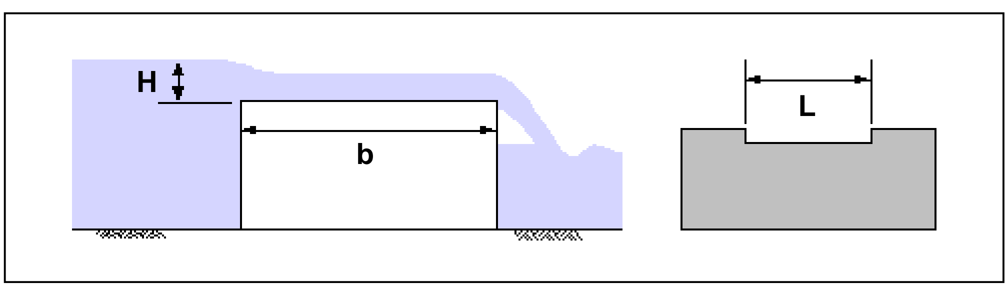

Figure 2.3.2-6 Broad-Crested Weir

Measured Head (H)* In feet	Weir Crest Breadth (b) 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

Table 2.3.2-2 Broad-Crested Weir Coefficient (C) Values

* Measured at least 2.5H upstream of the weir. Source: Brater and King (1976)

2.3.2.8 V-Notch Weirs

The discharge through a V-notch weir (Figure 2.3.2-7) can be calculated from the following equation (Brater and King, 1976).

$$\mathbf{Q = 2.5 \tan(\theta/2)\, H^{2.5}} \qquad \mathbf{(2.3.9)}$$

Where: Q = discharge (cfs)
θ = angle of V-notch (degrees)
H = head on apex of notch (ft)

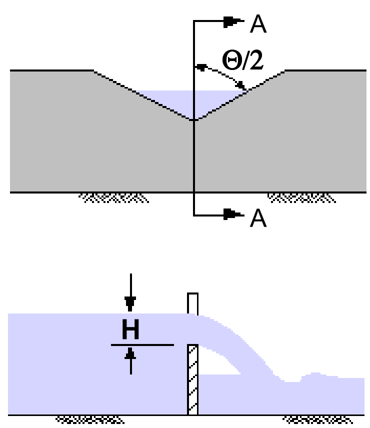

Figure 2.3.2-7 V-Notch Weir

2.3.2.9 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 2.3.2-8. Design equations for proportional weirs are (Sandvik, 1985):

$$Q = 4.97\ a^{0.5}\ b\ (H - a/3) \qquad (2.3.10)$$

$$x/b = 1 - (1/3.17)\ (\arctan\ (y/a)^{0.5}) \qquad (2.3.11)$$

Where: Q = discharge (cfs)
Dimensions a, b, H, x, and y are shown in Figure 2.3.2-8

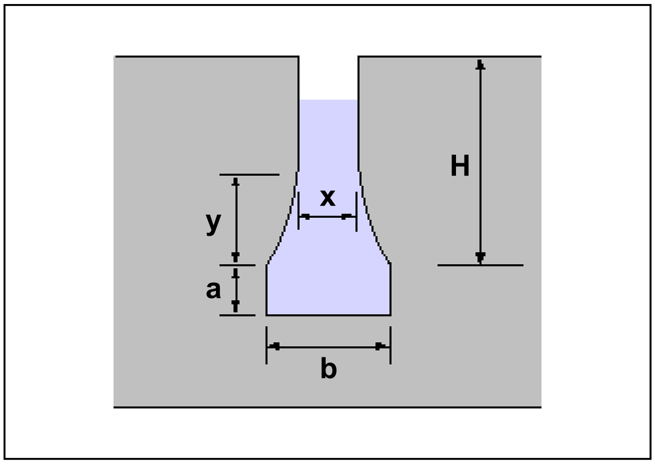

Figure 2.3.2-8 Proportional Weir Dimensions

2.3.2.10 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 2.3.2-9 shows an example of a riser designed for a wet ED pond. The orifice plate outlet structure in Figure 2.3.2-4 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 2.3.2-10) suitable for control of multiple storm flows. The design of multi-stage combination outlets is discussed later in this section.

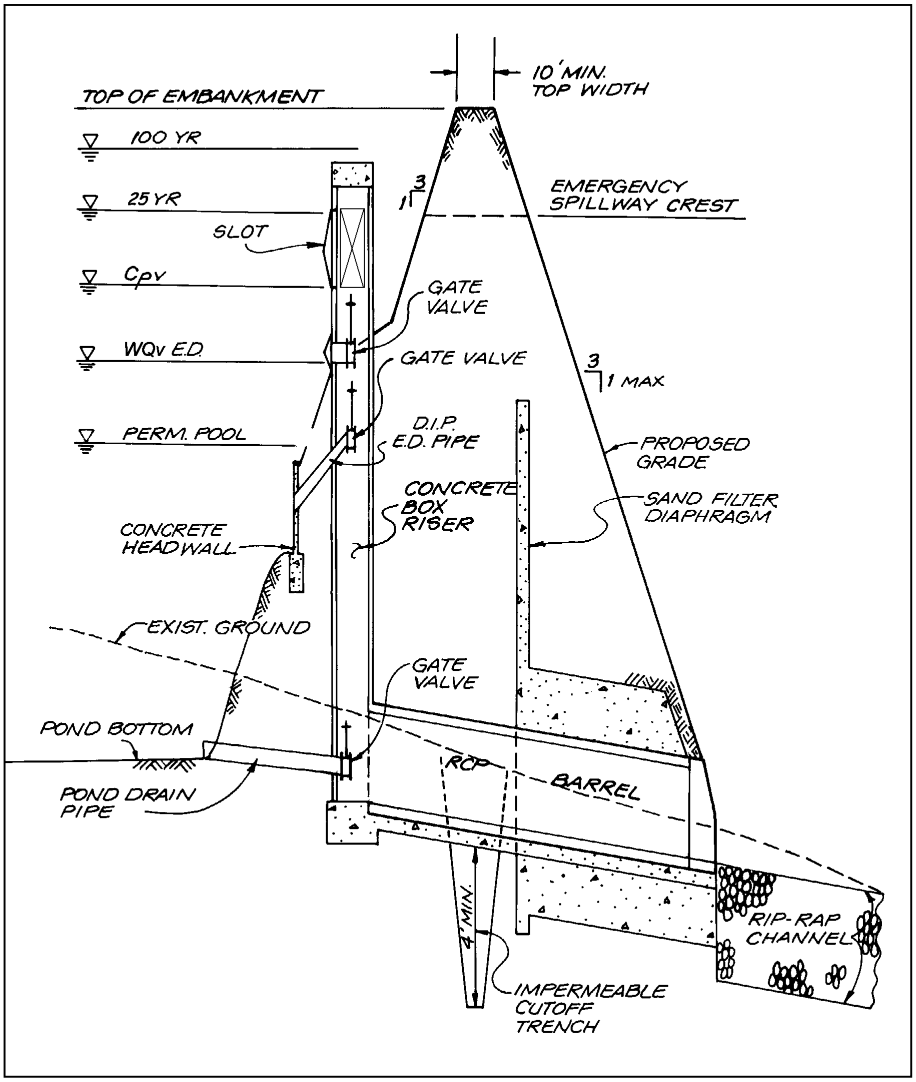

Figure 2.3.2-9 Schematic of Combination Outlet Structure

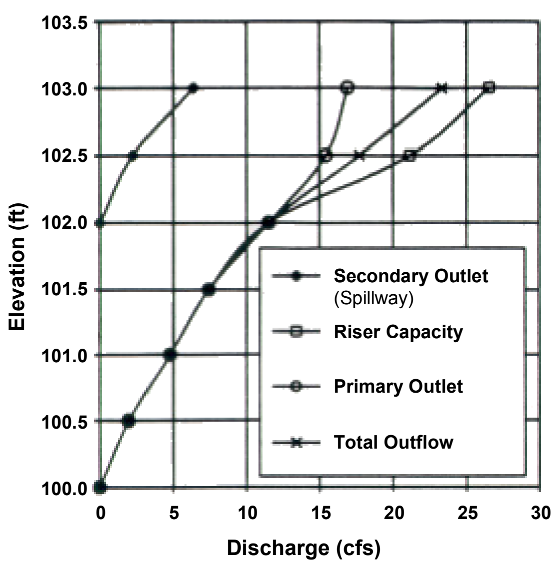

Figure 2.3.2-10 Composite Stage-Discharge Curve

2.3.3 Extended Detention (Water Quality and Channel Protection) Outlet Design

2.3.3.1 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 drawdown.

(This 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.

2.3.3.2 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 (2.3.8) 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.

2.3.3.3 Method 2: Average Hydraulic Head and Average Discharge

Using the data from the previous example (2.3.3.2) 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 (2.3.8) 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.

2.3.4 Multi-Stage Outlet Design

2.3.4.1 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. Figures 2.3.2-4 and 2.3.2-9 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 2.3.2-10)

2.3.4.2 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 2.1 and 2.2, 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 (WQ_v-ED) orifice using either Method 1 or Method 2 outlined in subsection 2.3.3. 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 2.3.5).

(Step 4) Design Channel Protection Outlet. Design the stream channel protection extended detention outlet (CP_v-ED) using either method from subsection 2.3.3. 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 Q_{p25} 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 2.3.4-1, 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 2.3.4-1 that as the elevation of the water increases further, the control can change from barrel inlet flow control to barrel pipe flow control. Figure 2.3.4-2 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 2.3.6). 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 4.5, *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.

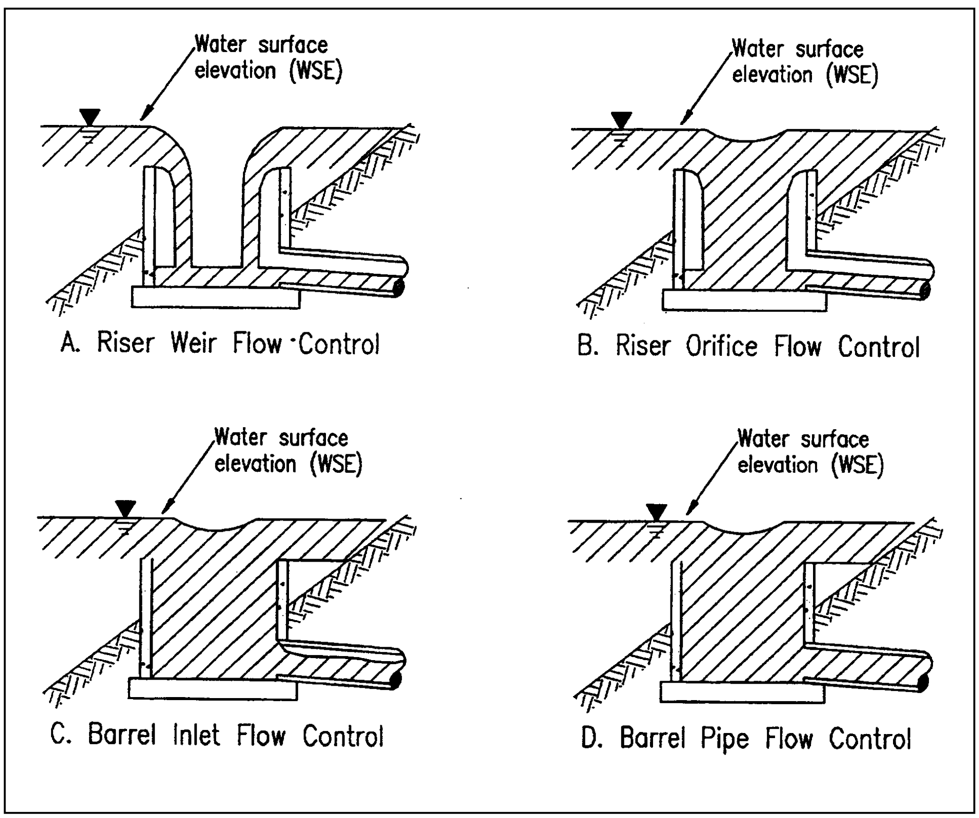

Figure 2.3.4-1 Riser Flow Diagrams
(Source: VDCR, 1999)

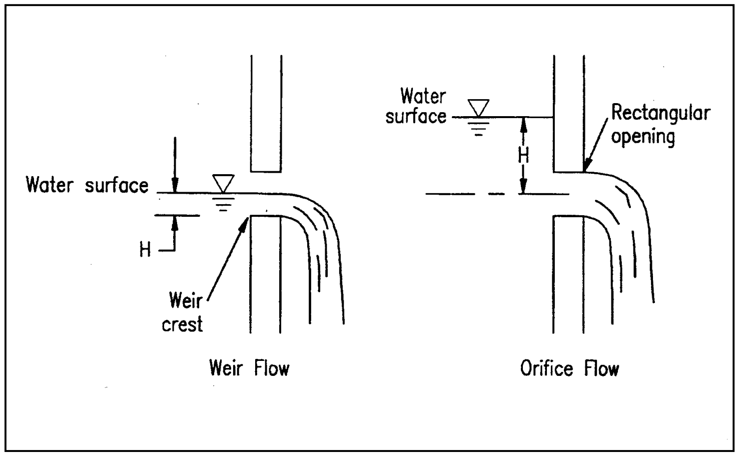

Figure 2.3.4-2 Weir and Orifice Flow
(Source: VDCR, 1999)

2.3.5 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 2.3.5-1). 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 2.3.5-2 and 2.3.5-3).
- 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 2.3.5-4).
- Internal orifice protection through the use of an adjustable gate valves can to achieve an equivalent orifice diameter.

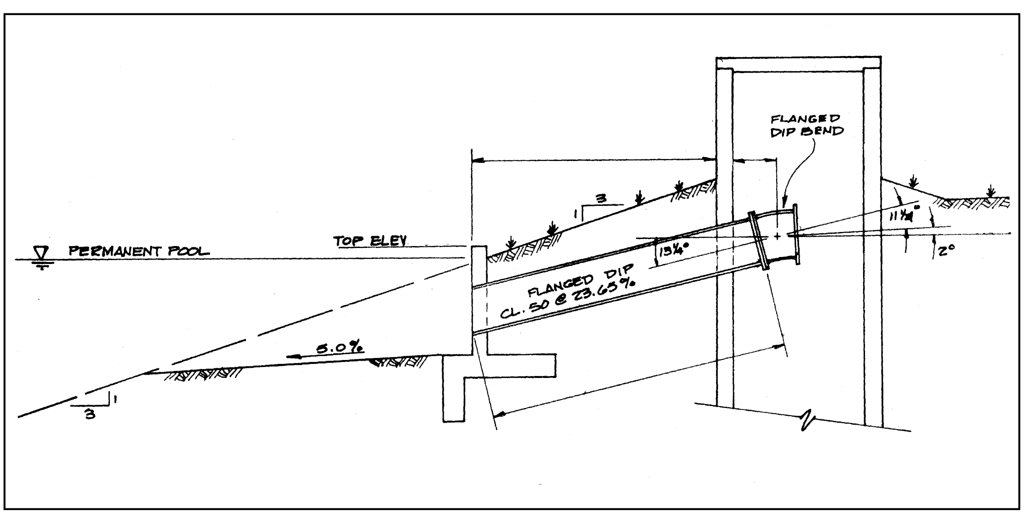

Figure 2.3.5-1 Reverse Slope Pipe Outlet

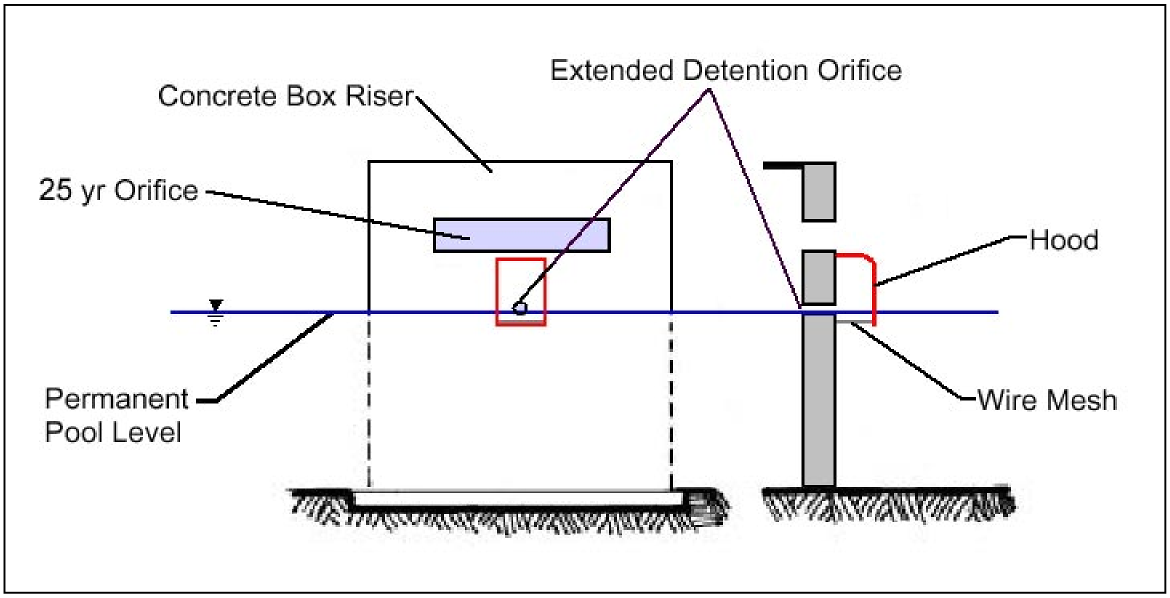

Figure 2.3.5-2 Hooded Outlet

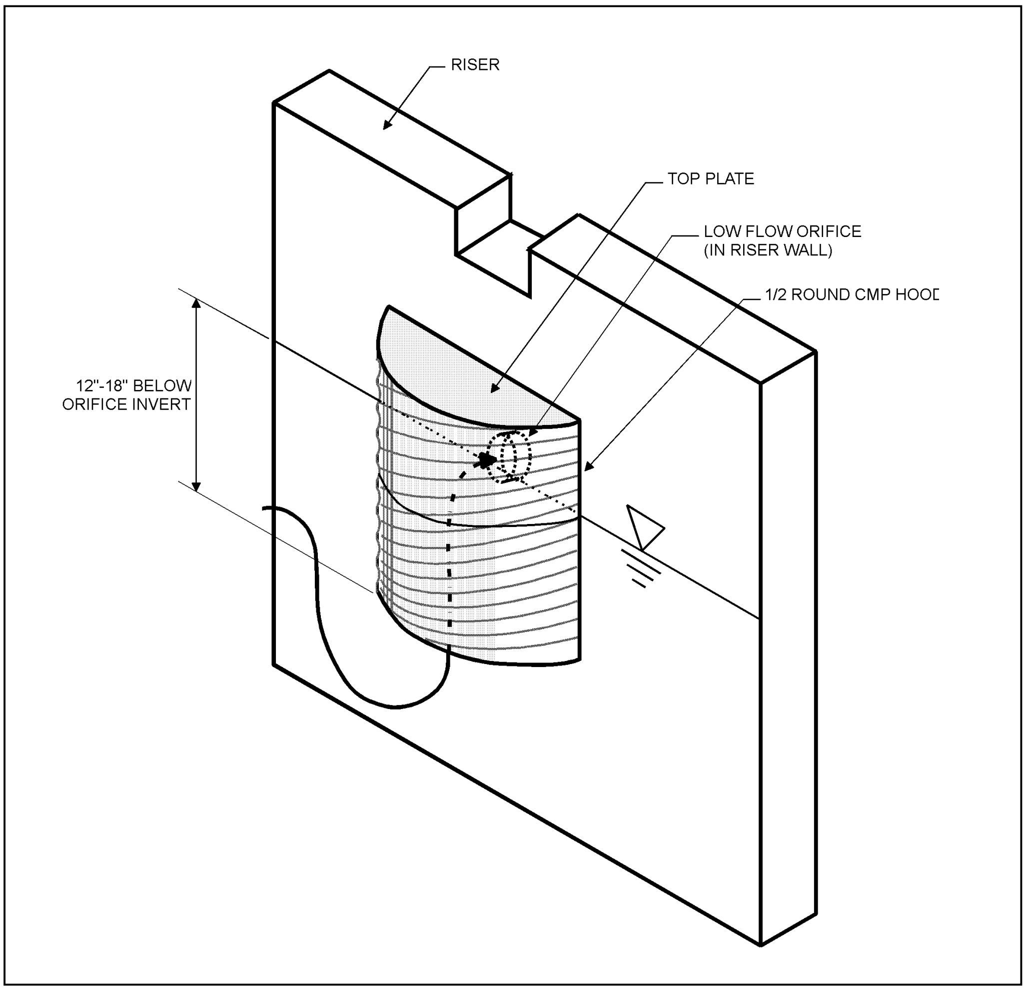

Figure 2.3.5-3 Half-Round CMP Orifice Hood

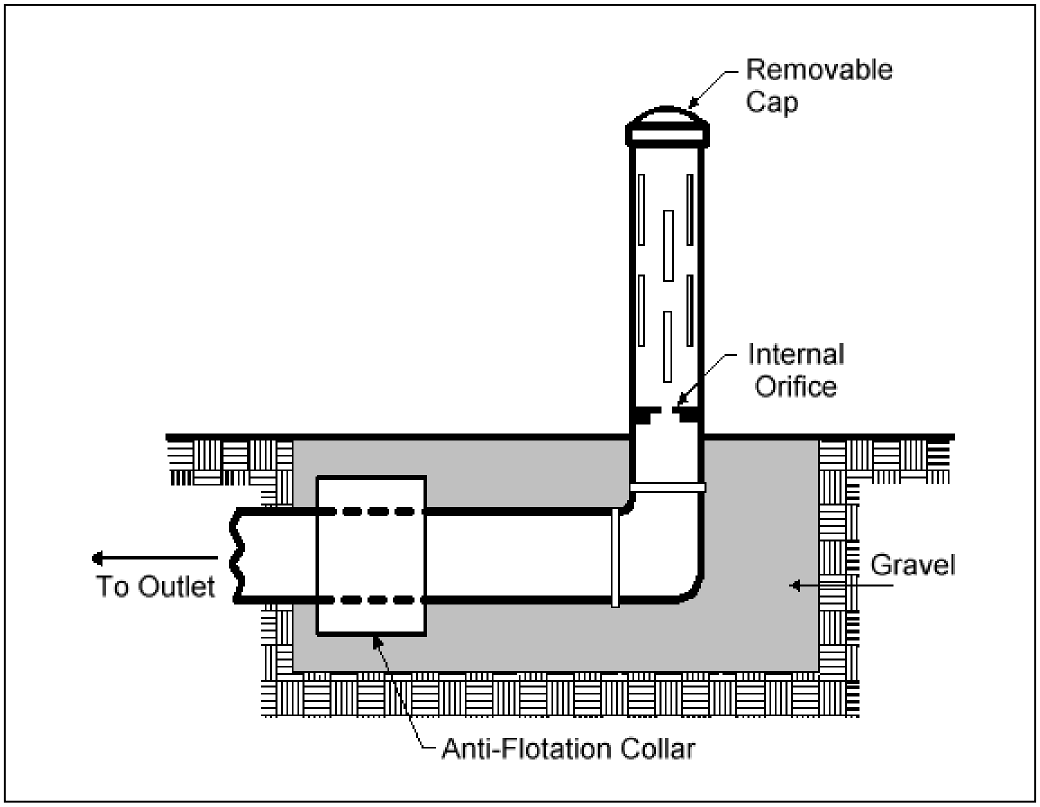

Figure 2.3.5-4 Internal Control for Orifice Protection

2.3.6 Trash Racks and Safety Grates

2.3.6.1 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 2.3.6-1. Additional track rack design can be found in Appendix C. 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.

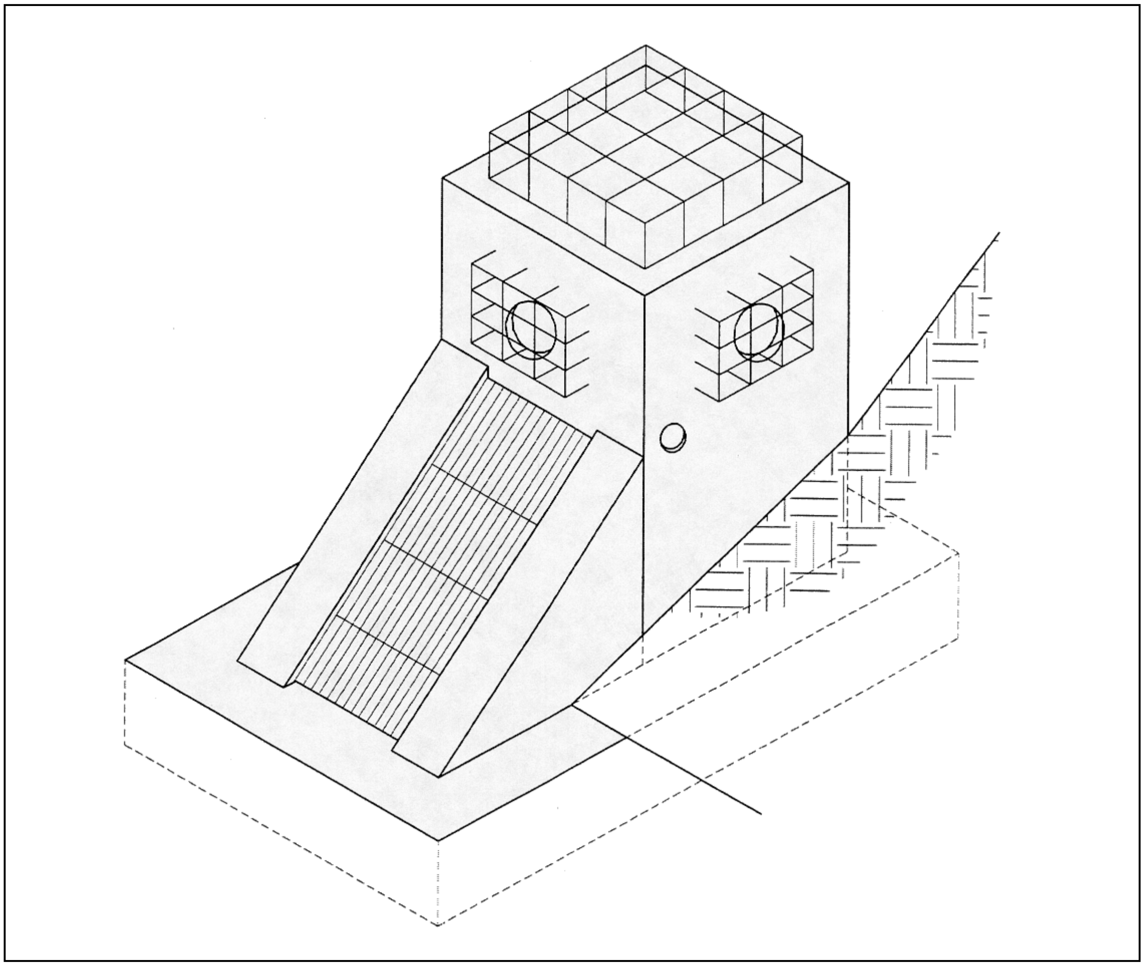

Figure 2.3.6-1 Example of Various Trash Racks Used on a Riser Outlet Structure
(Source: VDCR, 1999)

2.3.6.2 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 2.3.6-2 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.

$$\mathbf{H_g = K_{g1}\,(w/x)^{4/3}\,(V_u^2/2g)\,\sin\theta_g} \qquad \textbf{(2.3.12)}$$

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 faces
1.79 - circular bars
1.67 - rectangular bars with semicircular up- and downstream faces
w = maximum cross-sectional bar width facing the flow (in)
x = minimum clear spacing between bars (in)
V_u = approach velocity (ft/s)
θ_g = angle of the grate with respect to the horizontal (degrees)

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.

$$\mathbf{H_g = \frac{K_g^2 V_u^2}{2g}} \qquad \textbf{(2.3.13)}$$

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
  $K_{g2} = 0.00866 + 0.13589\,A_r + 6.0357\,A_r^2$

and A_r is the ratio of the area of the bars to the area of the grate section.

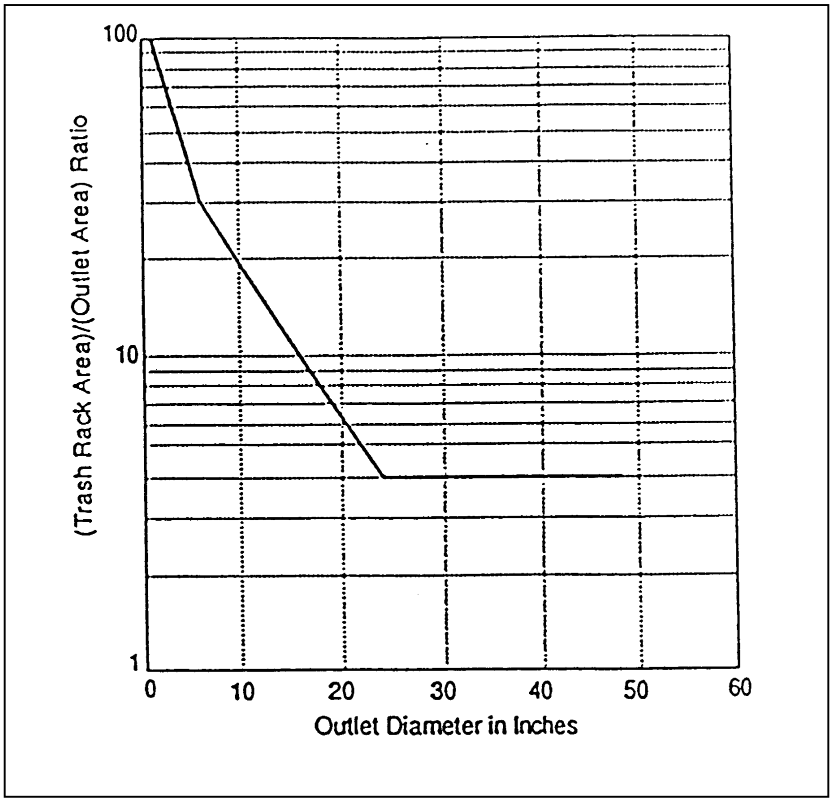

Figure 2.3.6-2 Minimum Rack Size vs. Outlet Diameter
(Source: UDCFD, 1992)

2.3.7 Secondary Outlets

2.3.7.1 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 2.3.7-1 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.

2.3.7.2 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 2.3.7-1). 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 4.4, *Open Channel Design*, for more information). Normally, it is assumed that critical depth occurs at the control section.

NRCS (SCS) 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 below.

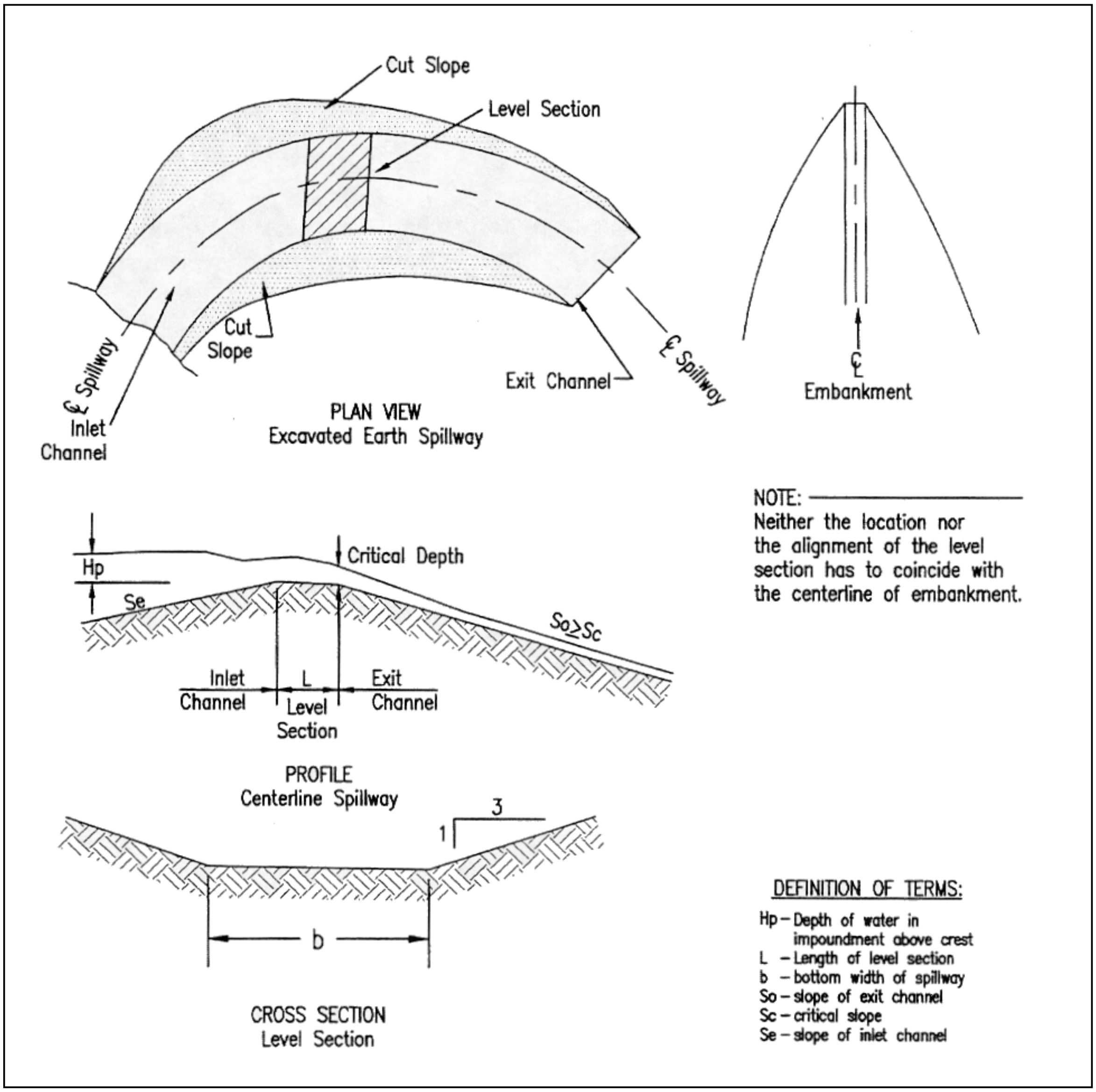

Figure 2.3.7-1 Emergency Spillway
(Source: VDCR, 1999)

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CHAPTER 3

STRUCTURAL STORMWATER CONTROLS

GEORGIA STORMWATER MANAGEMENT MANUAL
FIRST EDITION – AUGUST 2001

CHAPTER 3

STRUCTURAL STORMWATER CONTROLS

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STRUCTURAL STORMWATER CONTROLS OVERVIEW

3.1.1 Structural Stormwater Controls – Categories and Applicability

3.1.1.1 Introduction

Structural stormwater controls are engineered facilities intended to treat stormwater runoff and/or mitigate the effects of increased stormwater runoff peak rate, volume, and velocity due to urbanization. This section provides an overview of structural stormwater controls that can be used to address the minimum stormwater management standards outlined in Section 1.2.

In terms of the Unified Stormwater Sizing Criteria, a structural stormwater control, or set of structural controls, must:

- 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 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.

3.1.1.2 Structural Control Categories

The structural stormwater control practices recommended in this Manual have been placed into one of three categories based upon their applicability and ability to meet stormwater management goals:

General Application Structural Controls – General application structural controls are recommended for use with a wide variety of land uses and development types. These structural controls have a demonstrated ability to effectively treat the Water Quality Volume (WQ_v) and are presumed to be able to remove 80% of the annual average total suspended solids (TSS) load in typical post-development urban runoff when designed, constructed and maintained in accordance with recommended specifications. Several of the general application structural controls can also be designed to provide water quantity control, i.e. downstream channel protection (CP_v), overbank flood protection (Q_{p25}) and/or extreme flood protection (Q_f). General application controls are the recommended stormwater management facilities for a site wherever feasible and practical.

Limited Application Structural Controls – Limited application structural controls are those that are recommended only for limited use or for special site or design conditions. Generally, these practices either: (1) can not alone achieve the 80% TSS removal target, (2) are intended to address hotspot or specific land use constraints or conditions, and/or (3) may have high or special maintenance requirements that may preclude their use. Limited application controls are typically used for *water quality treatment only*. Some of these controls can be used as a pretreatment measure or in series with other structural controls to meet pollutant removal goals. Limited application structural controls should be considered primarily for commercial, industrial or institutional developments.

Detention Structural Controls – Detention structural controls are used only for providing water quantity control (CP_v, Q_{p25}, and/or Q_f), and are typically used downstream of a general application or limited application structural control.

In addition to the recommended *general application*, *limited application*, and *detention* structural controls, there are also a number of not recommended controls that are listed in subsection 3.1.6. These are identified structural stormwater control practices that fail to demonstrate an ability to meet the majority of the water quality goals and/or present difficulties in operation and maintenance, and are *not recommended* for use in Georgia.

3.1.1.3 General Application Structural Controls

Table 3.1.1-1 lists the general application structural stormwater control practices. These structural controls are recommended for use in a wide variety of applications. A detailed discussion of each of the general application controls, as well as design criteria and procedures can be found in Section 3.2.

Table 3.1.1-1 General Application Structural Controls

Structural Control	Description
Stormwater Ponds • Wet Pond • Wet Extended Detention Pond • Micropool Extended Detention Pond • Multiple Pond Systems	Stormwater ponds are constructed stormwater retention basins that have a permanent pool (or micropool) of water. Runoff from each rain event is detained and treated in the pool.
Stormwater Wetlands • Shallow Wetland • Extended Detention Shallow Wetland • Pond/Wetland Systems • 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.
Bioretention Areas	Bioretention areas are shallow stormwater basins or landscaped areas which utilize engineered soils and vegetation to capture and treat stormwater runoff. Runoff may be returned to the conveyance system, or allowed to partially exfiltrate into the soil.

Sand Filters • Surface Sand Filter • Perimeter 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, or allowed to partially exfiltrate into the soil.
Infiltration Trench	An infiltration trench is an excavated trench filled with stone aggregate used to capture and allow infiltration of stormwater runoff into the surrounding soils from the bottom and sides of the trench.
Enhanced Swales • Dry Swale • Wet Swale/Wetland Channel	Enhanced swales are 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.

3.1.1.4 Limited Application Structural Controls

Table 3.1.1-2 lists the limited application structural stormwater control practices, along with the rationale for limited use. These structural controls are recommended for use with particular land uses and densities, to meet certain water quality requirements, for limited usage on larger projects, or as part of a stormwater treatment train. A detailed discussion of each of the limited application controls, as well as design criteria and procedures can be found in Section 3.3.

Table 3.1.1-2 Limited Application Structural Controls

Structural Control	Description and Rationale for Limited Use
Biofilters • Filter Strip • Grass Channel	Both filter strips and grass channels provide "biofiltering" of stormwater runoff as it flows across the grass surface. However, by themselves these controls cannot meet the 80% TSS removal performance goal. Consequently, both filter strips and grass channels should only be used as pretreatment measure or as part of a treatment train approach. They are also acceptable for use as a site design credit (see Section 1.4).
Filtering Practices • Organic Filter • Underground Sand Filter	Organic filters are surface sand filters where organic materials such as a leaf compost or peat/sand mixture as the filter media. These media may be able to provide enhanced removal of some contaminants, such as heavy metals. Given their potentially high maintenance requirements, they should only be used in environments that warrant their use. Underground sand filters are sand filter systems located in an underground vault. These systems should only be considered for extremely high density or space-limited sites.

Wetland Systems • Submerged Gravel Wetlands	Submerged gravel wetlands systems use wetland plants in a submerged gravel or crushed rock media to remove stormwater pollutants. These systems should only be used in mid- to high-density environments where the use of other structural controls may be precluded. The long-term maintenance burden of these systems is uncertain.
Hydrodynamic Devices • Gravity (Oil-Grit) Separator	Hydrodynamic controls 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. These controls typically do not meet the 80% TSS removal performance goal and therefore should only be used as a pretreatment measure and as part of a treatment train approach.
Porous Surfaces • Porous Concrete • Modular Porous Paver Systems	Porous surfaces are permeable pavement surfaces with an underlying stone reservoir to temporarily store surface runoff before it infiltrates into the subsoil. Porous concrete is the term for a mixture of course aggregate, portland cement and water that allows for rapid infiltration of water. Modular porous paver systems consist of open void paver units laid on a gravel subgrade. Both porous concrete and porous paver systems provide water quality and quantity benefits, but have high workmanship and maintenance requirements, as well as high failure rates.
Chemical Treatment • Alum Treatment	Alum treatment provides for the removal of suspended solids from stormwater runoff entering a wet pond by injecting liquid alum into storm sewer lines on a flow-weighted basis during rain events. Alum treatment should only be considered for large-scale projects where high water quality is desired.
Proprietary Systems • Commercial Stormwater Controls	Proprietary controls are manufactured structural control systems available from commercial vendors 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, as well as in pretreatment applications. However, proprietary systems are often more costly than other alternatives, may have high maintenance requirements, and often lack adequate independent performance data, particularly for use in Georgia conditions.

3.1.1.5 Detention Structural Controls

Table 3.1.1-3 lists the detention structural stormwater control practices. These structural controls are recommended only for providing water quantity control, i.e. channel protection, overbank flood protection and/or extreme flood protection in a stormwater treatment train. A detailed discussion of each of the detention controls, as well as design criteria and procedures can be found in Section 3.4.

Due to the potential for pollutant resuspension and outlet clogging, detention structural controls are not intended to treat stormwater runoff and should be used downstream of other water quality structural control in a treatment train.

Table 3.1.1-3 Detention Structural Controls	
Structural Control	**Description**
Dry Detention / Dry Extended Detention Basins	Dry detention basins and dry extended detention (ED) basins are surface facilities intended to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts.
Multi-Purpose Detention Areas	Multi-purpose detention areas are site areas used for one or more specific activities, such as parking lots and rooftops, which are also designed for the temporary storage of runoff.
Underground Detention	Underground detention tanks and vaults are an alternative to surface dry detention for space-limited areas where there is not adequate land for a dry detention basin or multi-purpose detention area.

3.1.1.6 Not Recommended Structural Controls

The following structural controls in Table 3.1.1-4 are *not recommended* for use in Georgia to meet stormwater management objectives, as they fail to demonstrate an ability to meet the majority of the water quality treatment goals and/or present difficulties in operation and maintenance. Check with the local review authority for additional structural stormwater controls that may not be allowed in a particular community.

Table 3.1.1-4 Not Recommended Structural Controls	
Structural Control	**Rationale for Lack of Recommendation**
• Infiltration Basin	While in theory, infiltration basins provide excellent pollutant removal capabilities, the reality is that infiltration basins have historically experienced high rates of failure due to clogging associated with poor design, construction and maintenance. In addition, because many areas in Georgia having soils with high clay content, the infiltration basin has limited applicability. They would typically have an unacceptably high maintenance burden.
• Porous Asphalt	Porous asphalt surfaces are easily clogged by clays, silts and oils resulting in a potentially high maintenance burden to maintain the effectiveness of this structural control. Further, summer heat in Georgia can cause the asphalt to melt, destroying the porous properties of the surface.
• Media Filter Inserts	Media filter inserts such as catch basin inserts and filter systems are easily clogged and require a high degree of regular maintenance and replacement to achieve the intended water quality treatment performance and should not be used for areas of new development or redevelopment. These structural control may serve a potential use in stormwater retrofitting.

3.1.1.7 Using Other or New Structural Stormwater Controls

Innovative technologies should be allowed and encouraged providing there is sufficient documentation as to their effectiveness and reliability. Communities can allow 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 structural stormwater control 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 structural control conforms with local and/or State criteria for treatment, maintenance, and environmental impact.

3.1.2 Structural Stormwater Control Pollutant Removal Capabilities

General and limited application structural stormwater controls are intended to provide water quality treatment for stormwater runoff. Though each of these structural controls provides pollutant removal capabilities, the relative capabilities vary between structural control practices and for different pollutant types.

Pollutant removal capabilities for a given structural stormwater control practice are based on a number of factors including the physical, chemical and/or biological processes that take place in the structural control and the design and sizing of the facility. In addition, pollutant removal efficiencies for the same structural control type and facility design can vary widely depending on the tributary land use and area, incoming pollutant concentration, rainfall pattern, time of year, maintenance frequency and numerous other factors.

To assist the designer in evaluating the relative pollutant removal performance of the various structural control options, Table 3.1.2-1 provides design removal efficiencies for each of the general and limited application control practices. 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 structural control 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 structural control is sized, designed, constructed and maintained in accordance with recommended specifications in this Manual.

Where the pollutant removal capabilities of an individual structural stormwater control are not deemed sufficient for a given site application, additional controls may be used in series in a "treatment train" approach. More detail on using structural stormwater controls in series are provided in subsection 3.1.6.

For additional information and data on the range of pollutant removal capabilities for various structural stormwater controls, the reader is referred to 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

Table 3.1.2-1 Design Pollutant Removal Efficiencies for Structural Stormwater Controls					
Structural Control	**Total Suspended Solids**	**Total Phosphorus**	**Total Nitrogen**	**Fecal Coliform**	**Metals**
General Application Structural Controls					
Stormwater Ponds	80	50	30	70*	50
Stormwater Wetlands	80	40	30	70*	50
Bioretention Areas	80	60	50	---	80
Sand Filters	80	50	25	40	50
Infiltration Trench	80	60	60	90	90
Enhanced Dry Swale	80	50	50	---	40
Enhanced Wet Swale	80	25	40	---	20
Limited Application Structural Controls					
Filter Strip	50	20	20	---	40
Grass Channel	50	25	20	---	30
Organic Filter	80	60	40	50	75
Underground Sand Filter	80	50	25	40	50
Submerged Gravel Wetland	80	50	20	70	50
Gravity (Oil-Grit) Separator	40	5	5	---	---
Porous Concrete	**	50	65	---	60
Modular Porous Paver Systems	**	80	80	---	90
Alum Treatment	90	80	60	90	75
Proprietary Systems	***	***	***	***	***

* If no resident waterfowl population present

** Due to the potential for clogging, porous concrete and modular block paver systems should not be used for the removal of sediment or other coarse particulate pollutants

*** The performance of specific proprietary commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data

--- Insufficient data to provide design removal efficiency

3.1.3 Structural Stormwater Control Selection

3.1.3.1 General Application Control Screening Process

Outlined below is a screening process for General Application structural stormwater controls. This process is intended to assist the site designer and design engineer in the selection of the most appropriate structural controls for a development site, and provides guidance on factors to consider in their location.

In general the following four criteria should be evaluated in order to select the appropriate structural control(s) or group of controls for a development:

- Stormwater Treatment Suitability
- Water Quality Performance
- Site Applicability
- Implementation Considerations

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 structural control on site, or may require a permit, need to be considered.

The following pages provide a selection process for comparing and evaluating various general application structural stormwater controls using two screening matrices and a list of location and permitting factors. These tools are provided to assist the design engineer in selecting the subset of structural controls that will meet the stormwater management and design objectives for a development site or project.

Step 1 Overall Applicability

Through the use of the first matrix (Table 3.1.3-1) the site designer evaluates and screens the overall applicability of the full set of general application structural controls as well as the constraints of the site in question. The following are the details of the various screening categories and individual characteristics used to evaluate the structural controls.

Stormwater Management Suitability

The first columns of Matrix 1 examine the capability of each structural control option to provide water quality treatment, downstream channel protection, overbank flood protection, and extreme flood protection. A blank entry means that the structural control 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 structural control may be needed at a site (e.g., a bioretention area used in conjunction with dry detention storage).

> *Ability to treat the Water Quality Volume (WQ_v).* This indicates whether a structural control provides treatment of the water quality volume (WQ_v).
>
> *Ability to provide Channel Protection (CP_v).* This indicates whether the structural control can be used to provide the extended detention of the channel protection volume (CP_v). The presence of a check mark indicates that the structural control can be used to meet CP_v requirements. A star indicates that the structural control may be sized to provide channel protection in certain situations, for instance on small sites.

Ability to provide Overbank Flood Protection (Q_{p25}). This indicates whether a structural control can be used to meet the overbank flood protection criteria. The presence of a check mark indicates that the structural control can be used to provide peak reduction of the 25-year storm event.

Ability to provide Extreme Flood Protection (Q_f). This indicates whether a structural control can be used to meet the extreme flood protection criteria. The presence of a check mark indicates that the structural control can be used to provide peak reduction of the 100-year storm event.

Relative Water Quality Performance

The second group of columns in Matrix 1 provide an overview of the pollutant removal performance of each structural control option, when designed, constructed and maintained according to the criteria and specifications in this Manual.

Ability to provide TSS and Sediment Removal. This column indicates the capability of a structural control to remove sediment in runoff. All of the general application structural controls are presumed to remove 80% of the average annual total suspended solids (TSS) load in typical urban post-development runoff (and a proportional removal of other pollutants).

Ability to provide Nutrient Treatment. This column indicates the capability of a structural control to remove the nutrients nitrogen and phosphorus in runoff, which may be of particular concern with certain downstream receiving waters.

Ability to provide Bacteria Removal. This column indicates the capability of a structural control to remove bacteria in runoff. This capability may be of particular focus in areas with public beaches, shellfish beds, or to meet water regulatory quality criteria under the Total Maximum Daily Load (TMDL) program.

Ability to accept Hotspot Runoff. This last column indicates the capability of a structural control to treat runoff from designated hotspots. Hotspots are land uses or activities with higher potential pollutant loadings. Examples of hotspots might include: gas stations, convenience stores, marinas, public works storage areas, vehicle service and maintenance areas, commercial nurseries, and auto recycling facilities. A check mark indicates that the structural control may be used on hotspot site, however it may have specific design restrictions. Please see the specific design criteria of the structural control for more details.

Site Applicability

The third group of columns in Matrix 1 provide an overview of the specific site conditions or criteria that must be met for a particular structural control 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 the structural control for more details.

Drainage Area. This column indicates the approximate minimum or maximum drainage area that is considered suitable for the structural control practice. 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 can be 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, or design variations used to maintain a permanent pool (e.g., liners).

Space Required (Space Consumed). This comparative index expresses how much space a structural control typically consumes at a site in terms of the approximate area required as a percentage of the area draining to the control.

Slope. This column evaluates the effect of slope on the structural control 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. This column provides an estimate of the minimum elevation difference needed at a site (from the inflow to the outflow) to allow for gravity operation within the structural control.

Water Table. This column indicates the minimum depth to the seasonally high water table from the bottom or floor of a structural control.

Implementation Considerations

The last group of columns of Matrix 1 provide additional considerations for the applicability of each structural control option.

Residential Subdivision Use. This column identifies whether or not a structural control is suitable for typical residential subdivision development (not including high-density or ultra-urban areas).

Ultra-Urban. This column identifies those structural controls that are appropriate for use in very high-density (ultra-urban) areas, or areas where space is a premium.

Construction Cost. The structural controls are ranked according to their relative construction cost per impervious acre treated as determined from cost surveys.

Maintenance. This column assesses the relative maintenance effort needed for a structural stormwater control, in terms of three criteria: frequency of scheduled maintenance, chronic maintenance problems (such as clogging) and reported failure rates. It should be noted that **all structural controls** require routine inspection and maintenance.

Table 3.1.2-1 Structural Control Screening Matrix 1 – Overall Applicability

General Application Controls

STRUCTURAL CONTROL CATEGORY	STRUCTURAL CONTROL	STORMWATER TREATMENT SUITABILITY				WATER QUALITY PERFORMANCE*				SITE APPLICABILITY					IMPLEMENTATION CONSIDERATIONS			
		Water Quality	Channel Protection	Overbank Flood Protection	Extreme Flood Protection	TSS / Sediment Removal Rate	Nutrient Removal Rate (TP/TN)	Bacteria Removal Rate	Hotspot Application	Drainage Area (acres)	Space Req'd (% of tributary imp. Area)	Site Slope	Minimum Head Required	Depth to Water Table	Residential Subdivision Use	High Density / Ultra-Urban	Capital Cost	Maintenance Burden
Stormwater Ponds	Wet Pond	✓	✓	✓	✓	80%	50%/30%	70%	✓	25 min**	2-3%	15% max	6 to 8 ft	2 feet, if hotspot or aquifer	✓		Low	Low
	Wet ED Pond	✓	✓	✓	✓				✓						✓		Low	Low
	Micropool ED Pond	✓	✓	✓	✓				✓	10 min**					✓		Low	Moderate
	Multiple Ponds	✓	✓	✓	✓				✓	25 min**					✓		Low	Low
Stormwater Wetlands	Shallow Wetland	✓	✓	✓	✓	80%	40%/30%	70%	✓	25 min	3-5%	8% max	3 to 5 ft	2 feet, if hotspot or aquifer	✓		Moderate	Moderate
	Shallow ED Wetland	✓	✓	✓	✓				✓						✓		Moderate	Moderate
	Pond/Wetland	✓	✓	✓	✓				✓				6 to 8 ft		✓		Moderate	Moderate
	Pocket Wetland	✓	✓						✓	5 min			2 to 3 ft	below WT	✓	✓	Moderate	High
Bioretention	Bioretention Areas	✓	✪			80%	60%/50%	*Insuff. data*	✓	5 max***	5%	6% max	5 ft	2 feet	✓	✓	Moderate	Moderate
Sand Filters	Surface Sand Filter	✓	✪			80%	50%/25%	40%	✓	10 max***	2-3%	6% max	5 ft	2 feet		✓	High	High
	Perimeter Sand Filter	✓	✪						✓	2 max***			2 to 3 ft			✓	High	High
Infiltration	Infiltration Trench	✓	✪			80%	60%/60%	90%		5 max	2-3%	6% max	1 ft	4 feet	✓	✓	High	High
Enhanced Swales	Dry Swale	✓	✪			80%	50%/50%	*Insuff. data*	✓	5 max	10-20%	4% max	3 to 5 ft	2 feet	✓		Moderate	Low
	Wet Swale	✓	✪			80%	25%/40%	*Insuff. data*	✓	5 max			1 ft	below WT	✓		High	Low

✓ -- Meets suitability criteria

✪ -- Can be incorporated into the structural control in certain situations

* Pollutant removal rates are average removal efficiencies for design purposes

** Smaller area acceptable with adequate water balance and anti-clogging device

*** Drainage area can be larger in some instances

Step 2 Specific Criteria

The second matrix (Table 3.1.3-2) provides an overview of various specific design criteria and specifications, or exclusions for a structural control that may be present due to a site's general physiographic character, soils, or location in a watershed with special water resources considerations.

<u>Physiographic Factors</u>

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 northeastern and northwestern parts of the State. Karst and major carbonaceous rock areas are generally found in the northwest and southwest 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 structural controls require a hydraulic head to move stormwater runoff through the facility.
- High Relief may limit some the use of some structural controls that need flat or gently sloping areas to settle out sediment or to reduce velocities. In other cases high relief may impact dam heights to the point that a structural control becomes infeasible.
- Karst terrain can limit the use of some structural controls 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.

<u>Soils</u>

The 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.

<u>Special Watershed or Stream Considerations</u>

The design of structural stormwater controls is fundamentally influenced by the nature of the downstream water body that will be receiving the stormwater discharge. 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, the designer should consult with the appropriate review authority to determine if their development project is subject to additional structural control 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 structural control or the exclusion of one or more controls may need to be considered within these watersheds or areas. Examples of important watershed factors to consider include:

Primary 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 structural controls can have adverse downstream impacts on cold-water streams, and their design may need to be modified or use restricted.

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 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 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.

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 level of pollutant removal for the pollutants of concern, such as bacteria pathogens, nutrients, sediment 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, structural controls should be explicitly designed to maximize bacteria removal.

Table 3.1.2-2 Structural Control Screening Matrix 2 – Specific Criteria

General Application Controls

STRUCTURAL CONTROL CATEGORY	PHYSIOGRAPHIC FACTORS			SOILS	SPECIAL WATERSHED CONSIDERATIONS				
	Low Relief	High Relief	Karst		Trout Stream	High Quality Stream	Aquifer Protection	Reservoir Protection	Shellfish / Beach
Stormwater Ponds	Limit maximum normal pool depth to about 4 feet (dugout) Providing pond drain can be problematic	Embankment heights restricted	Require poly or clay liner Max ponding depth Geotechnical tests	"A" soils may require pond liner "B" soils may require infiltration testing	Limit use of due to thermal impacts Limit ED to 12 hrs Offline design and provide shading	Evaluate for stream warming	May require liner if "A" soils are present Pretreat hotspots 2 to 4 ft separation distance from water table		Moderate bacteria removal Design for waterfowl prevention Provide 48 hr ED for max coliform dieoff
Stormwater Wetlands		Embankment Heights restricted	Require poly-liner Geotechnical tests	"A" soils may require pond liner	Limit use of due to thermal impacts Offline design and provide shading	Evaluate for stream warming	May require liner if "A" soils are present Pretreat hotspots 2 to 4 ft separation distance from water table		Provide 48 hr ED for max coliform dieoff
Bioretention & Sand Filters	Several design variations will likely be limited by low head		Use poly-liner or impermeable membrane to seal bottom	Clay or silty soils may require pretreatment	Evaluate for stream warming	Evaluate for stream warming	Needs to be designed with no exfiltration (i.e. outflow to groundwater)		Moderate to high coliform removal
Infiltration	Minimum distance to water table of 2 feet	Maximum slope of 6% Trenches must have flat bottom	GENERALLY NOT ALLOWED	Infiltration rate > 0.5 inch/hr			Maintain safe distance from wells and water table No hotspot runoff	Maintain safe distance from bedrock and water table Pretreat runoff	Maintain safe distance from water table
Enhanced Swales	Generally feasible however slope <1% may lead to standing water in dry swales	Often infeasible if slopes are 4% or greater					Hotspot runoff must be adequately treated	Hotspot runoff must be adequately treated	Poor coliform removal

Step 3 Location and Permitting Considerations

In the last step, a site designer assesses the physical and environmental features at the site to determine the optimal location for the selected structural control or group of controls. The checklist below (Table 3.1.3-3) 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 structural control within an area that is expressly prohibited by law.
- Locating a structural control 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 control within the regulated area.
- Structural stormwater controls 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 stormwater structural controls. Consultation with the appropriate regulatory agency is the best strategy.

Table 3.1.3-3 Location and Permitting Checklist

Site Feature	Location and Permitting Guidance
Jurisdictional Wetland (Waters of the U.S) U.S. Army Corps of Engineers Section 404 Permit	• Jurisdictional wetlands should be delineated prior to siting structural control. • Use of natural wetlands for stormwater quality treatment is contrary to the goals of the Clean Water Act and should be avoided. • Stormwater should be treated prior to discharge into a natural wetland. • Structural controls may also be *restricted* in local buffer zones, although they may be utilized as a non-structural filter strip (i.e., accept sheet flow). • Should justify that no practical upland treatment alternatives exist. • Where practical, excess stormwater flows should be conveyed away from jurisdictional wetlands.
Stream Channel (Waters of the U.S) 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. • 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. • Stormwater should be treated prior to discharge into Waters of the U.S. • In-stream ponds for stormwater quality treatment are highly discouraged. • Must justify that no practical upland treatment alternatives exist. • Temporary runoff storage preferred over permanent pools. • Implement measures that reduce downstream warming.
Georgia Planning Act Groundwater Recharge Areas	• Prevention of groundwater contamination • Covers about 23% of State. Detailed mapping available at Regional Development Centers. • Permanent stormwater infiltration devices are prohibited in areas having high pollution susceptibility.
Georgia Planning Act Water Supply watersheds	• Specific stream and reservoir buffer requirements. • May be imperviousness limitations • May be specific structural control requirements.

Table 3.1.3-3 Location and Permitting Checklist (continued)	
Site Feature	**Location and Permitting Guidance**
100 Year Floodplain Local Stormwater review Authority	• Grading and fill for structural control 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. • Floodplain fill cannot raise the floodplain water surface elevation by more than a tenth of a foot.
Stream Buffer Check with appropriate review authority whether stream buffers are required	• Consult local authority for stormwater policy. • Structural controls are discouraged in the streamside zone (within 25 feet or more of streambank, depending on the specific regulations). • There are specific additional requirements related to River Corridor Protection, the Metropolitan River Protection Act, and the Georgia Scenic Rivers Act (which include wider and more stringent buffers).
Utilities Local Review Authority	• Call appropriate agency to locate existing utilities prior to design. • Note the location of proposed utilities to serve development. • Structural controls are discouraged within utility easements or rights of way for public or private utilities.
Roads Local DOT, DPW, or State DOT	• Consult local DOT or DPW for any setback requirement from local roads. • Consult DOT for setbacks from State maintained roads. • Approval must also be obtained for any stormwater discharges to a local or state-owned conveyance channel.
Structures Local Review Authority	• Consult local review authority for structural control setbacks from structures. • Recommended setbacks for each structural control group are provided in the performance criteria in this manual.
Septic Drain fields Local Health Authority	• Consult local health authority. • Recommended setback is a minimum of 50 feet from drain field edge.
Water Wells Local Health Authority	• 100-foot setback for stormwater infiltration. • 50-foot setback for all other structural controls.

3.1.3.2 Limited Application Control Screening Process

Outlined below a screening process for Limited Application structural controls designed to assist the site designer and design engineer in the evaluation of the performance and applicability of the various limited application controls. Through the use of the Screening Matrix 3 (Table 3.1.3-4) the site designer can evaluate and screen the list of Limited Application structural controls to determine if a particular control or set of control(s) is appropriate.

As with the general application controls, the site designer assesses the physical and environmental features at the site to determine the optimal location for the selected structural control or group of controls using Table 3.1.3-3 (Location and Permitting Checklist).

Evaluation Criteria

The following are the details of the various screening categories and individual characteristics used to evaluate the structural controls.

Water Quality Treatment

> *Ability to Meet 80% TSS Reduction Goal*. This column indicates whether or not a limited/ special application control can meet or be used towards meeting the goal of reducing the post-development TSS loading by 80%. '*Yes'* means that the structural control can meet the 80% TSS removal performance goal when designed, constructed and maintained according to the criteria and specifications in this Manual. *'No'* means that the structural control has a TSS removal efficiency that does not meet the 80% goal, however the control can contribute toward meeting the goal either individually or as part of set of controls used in series (see 3.1.4 for more details). Specific design pollutant removal rates for TSS and other pollutants can be found in Table 3.1.2-1.

Site Applicability

The next two columns in Matrix 3 provide an overview of the specific site conditions or criteria that must be met for a particular limited application structural control to be suitable. Please see the specific criteria section of the structural control for more details.

> *Drainage Area*. This column indicates the approximate minimum or maximum drainage area that is considered suitable for the structural control practice.
>
> *Space Required (Space Consumed)*. This comparative index expresses how much space a structural control typically consumes at a site in terms of the approximate area required as a percentage of the impervious area draining to the control.

Implementation Considerations

The last group of columns in Matrix 3 provide additional considerations for the applicability of each structural control options.

> *Pretreatment Control*. This column indicates that the structural control is ideally used for the pretreatment of runoff in a stormwater treatment train (see Section 3.1.3).
>
> *Residential Subdivision*. This column identifies whether or not a structural control is suitable for typical residential subdivision development (not including high-density or ultra-urban areas).
>
> *Ultra-Urban*. This column identifies those structural controls that are appropriate for use in very high-density (ultra-urban) areas, or areas where space is a premium.
>
> *Construction Cost*. The structural controls are ranked according to their relative construction cost per impervious acre treated as determined from cost surveys.

Maintenance. This column assesses the relative maintenance effort needed for a structural stormwater control, in terms of three criteria: frequency of scheduled maintenance, chronic maintenance problems (such as clogging) and reported failure rates. It should be noted that **all structural controls** require routine inspection and maintenance.

Commercially Manufactured Systems Available. This column indicates if a structural control is available as a pre-manufactured commercial product from a vendor.

Table 3.1.2-4 Structural Control Screening Matrix 3

Limited Application Controls

STRUCTURAL CONTROL CATEGORY	STRUCTURAL CONTROL	WATER QUALITY	SITE APPLICABILITY		IMPLEMENTATION CONSIDERATIONS					
		Able to Meet 80% TSS Reduction Goal	Drainage Area (acres)	Space Req'd (% of tributary imp. Area)	Pretreatment Control	Residential Subdivision Use	High Density / Ultra-Urban	Capital Cost	Maintenance Burden	Commercially Manufactured Systems Available?
Biofilters	Filter Strip	No*	2 max	20-25%	✓	✓		Low	Moderate	
	Grass Channel	No*	5 max	10-20%	✓	✓		Low	Low	
Filtering Practices	Organic Filter	Yes	10 max**	2-3%			✓	High	High	
	Underground Sand Filter	Yes	5 max	None			✓	High	High	Yes
Wetland Systems	Submerged Gravel Wetland	Yes	5 max**	2-3%			✓	High	High	
Hydro-dynamic Devices	Gravity (Oil-Grit) Separator	No*	1 max**	None	✓		✓	High	High	Yes
Porous Surfaces	Porous Concrete[1]	No[2]	5 max	Varies			✓	Medium	High	
	Modular Porous Paver Systems[1]	No[2]	5 max	Varies		✓	✓	High	High	Yes
Chemical Treatment	Alum Treatment System	Yes	25 min	None		✓	✓	High	High	
Proprietary Systems	Commercial Stormwater Controls	***	***	***	***	***	***	***	***	Yes

✓ -- Meets suitability criteria

* Provides less than 80% TSS removal efficiency. May be used in pretreatment and as part of a "treatment train"

** Drainage area can be larger in some instances

*** The application, performance and maintenance requirements of specific commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data

[1] Porous surfaces provide water quantity benefits by reducing the effective impervious area

[2] Due to the potential for clogging, porous surfaces should not be used for the removal of sediment or other coarse particulate pollutants

3.1.3.3 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 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 tributary to a phosphorus limited drinking water reservoir. Low permeability soils limit infiltration practices.

Objective: Avoid additional disruptions to receiving channel 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 the following:

- Sediment: 80%
- Phosphorus: 40%

Activity/Runoff Characteristics: The proposed site is to 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).

Table 3.1.3-5 lists the results of the selection analysis using Matrices 1 and 2 described previously.

While there is a downstream reservoir to consider, there are no special watershed factors nor physiographic factors that preclude the use of any of the practices from the General Application structural control 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 removes an infiltration trench from being considered. Due to the need to provide overbank flood control as well as channel protection storage, a micropool ED pond will likely be needed, unless some downstream regional storage is available to control the overbank flood.

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. The 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 additionally high oil and grease trapping ability.

The site drainage system can be designed so that the bioretention and/or sand filters drain to the micropool ED pond for redundant treatment. Vegetated dry swales could also be used to convey runoff to the pond, which would provide pretreatment. Pocket wetlands and wet swales were eliminated from consideration due to potential for nuisance conditions. Underground sand filters could also be used at the site; however, cost and aesthetic considerations were significant enough to eliminate from consideration.

Table 3.1.3-5 Sample Structural Control Selection Matrix

General Application Structural Control Alternative	Stormwater Treatment Suitability	Site Applicability	Implementation Considerations	Physiographic Factors/Soils	Special Watershed Considerations	Other Issues
Wet Pond	✓	X				
Wet ED Pond	✓	X				
Micropool ED Pond	✓	✓	✓	✓	none	
Multiple Ponds	✓	X				
Shallow Wetland	✓	X				
ED Shallow Wetland	✓	X				
Pocket Wetland	✓	✓	✓	✓	none	Odor / mosquitoes
Infiltration Trench	✓[1]	✓	✓	X		
Surface Sand Filter	✓[1]	✓[2]	✓	✓	none	Aesthetics
Perimeter SF	✓[1]	✓[2]	✓	✓	none	Higher cost
Bioretention	✓[1]	✓[2]	✓	✓	none	
Dry Swale	✓[1]	✓[2]	✓	✓	none	
Wet Swale	✓[1]	✓[2]	✓	✓	none	Odor / mosquitoes

Notes:
1. Only when used with another structural control that provides water quantity control
2. Can treat a portion of the site

3.1.4 On-Line Versus Off-Line Structural Controls

3.1.4.1 Introduction

Structural stormwater control are designed to be either "on-line" or "off-line." On-line facilities are designed to receive, but not necessarily control or treat, the entire runoff volume up to the Q_{p25} or Q_f event. On-line structural controls must be able to handle the entire range of storm flows.

Off-line facilities on the other hand are designed to receive only a specified flow rate through the use of a flow regulator (i.e. diversion structure, flow splitter, etc). Flow regulators are typically used to divert the water quality volume (WQ_v) to an off-line structural control 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 3.1.4.1 shows an example of an off-line sand filter and a off-line enhanced dry swale.

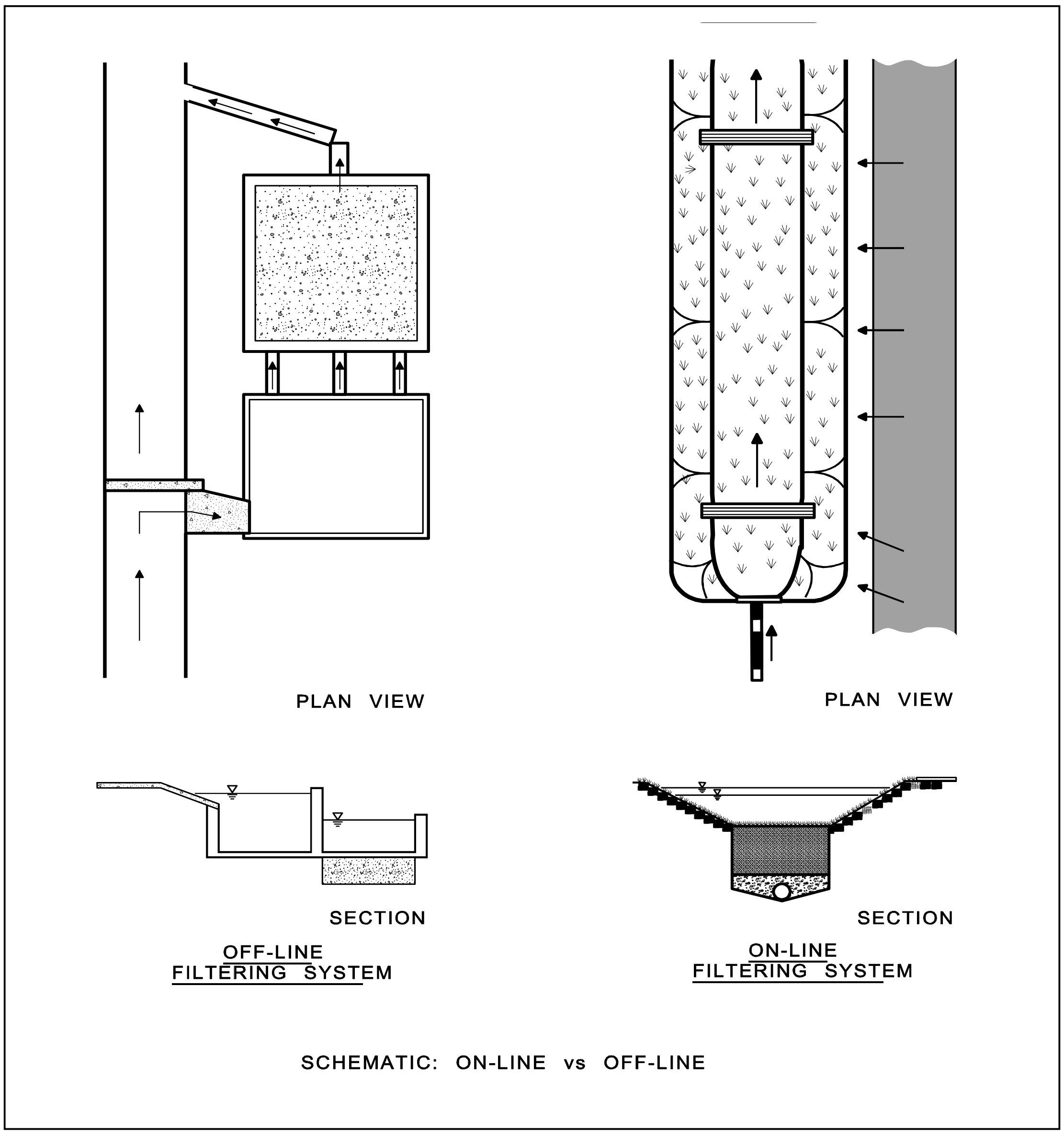

Figure 3.1.4-1 Example of On-Line versus Off-Line Structural Controls
(Source: CWP, 1996)

3.1.4.2 Flow Regulators

Flow regulation to off-line structural stormwater controls can be achieved by either:

- Diverting the water quality volume or other specific maximum flow rate to an off-line structural stormwater control, 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 2.1.7.2 in Section 2.1.

Flow regulators can be flow splitter devices, diversion structures, or overflow structures. An number of examples are shown below and in Appendix C.

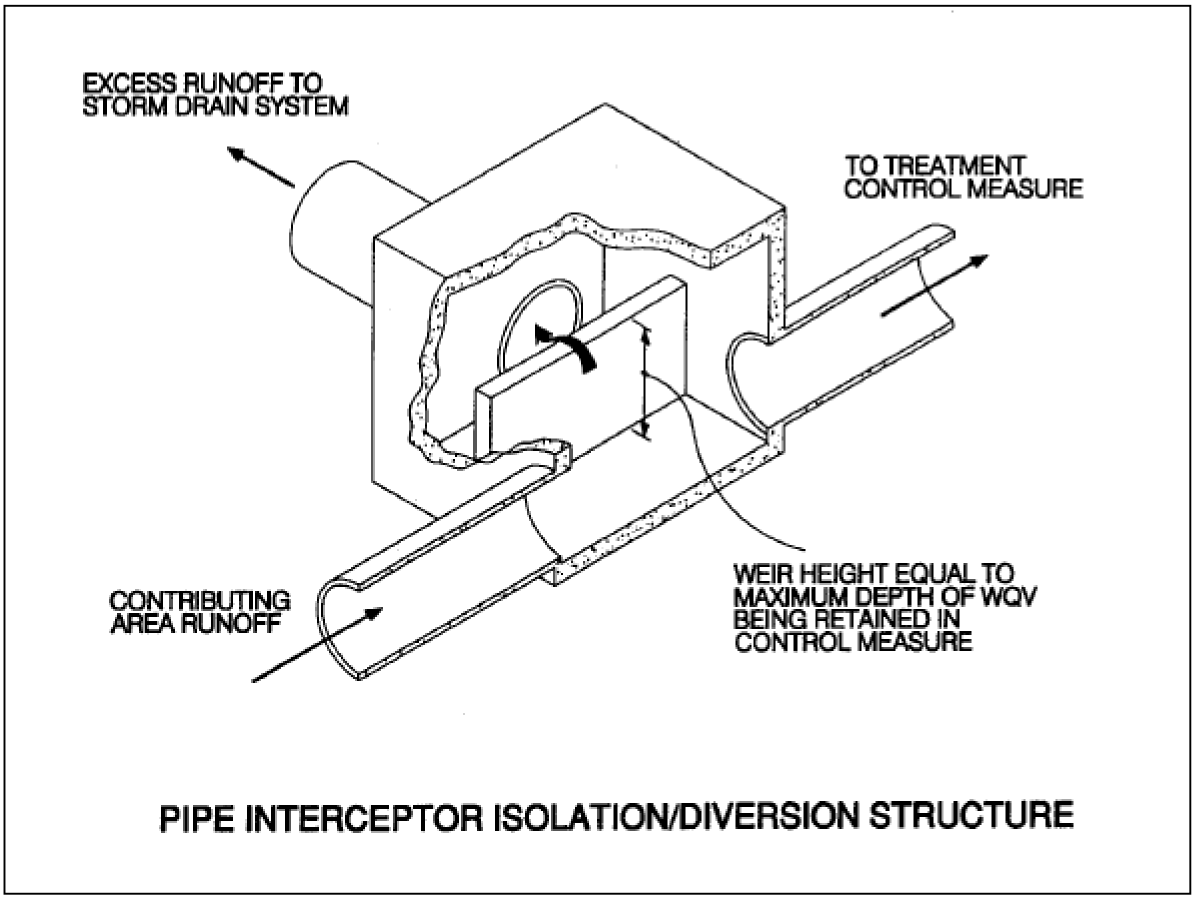

Figure 3.1.4-2 Pipe Interceptor Diversion Structure
(Source: City of Sacramento, 2000)

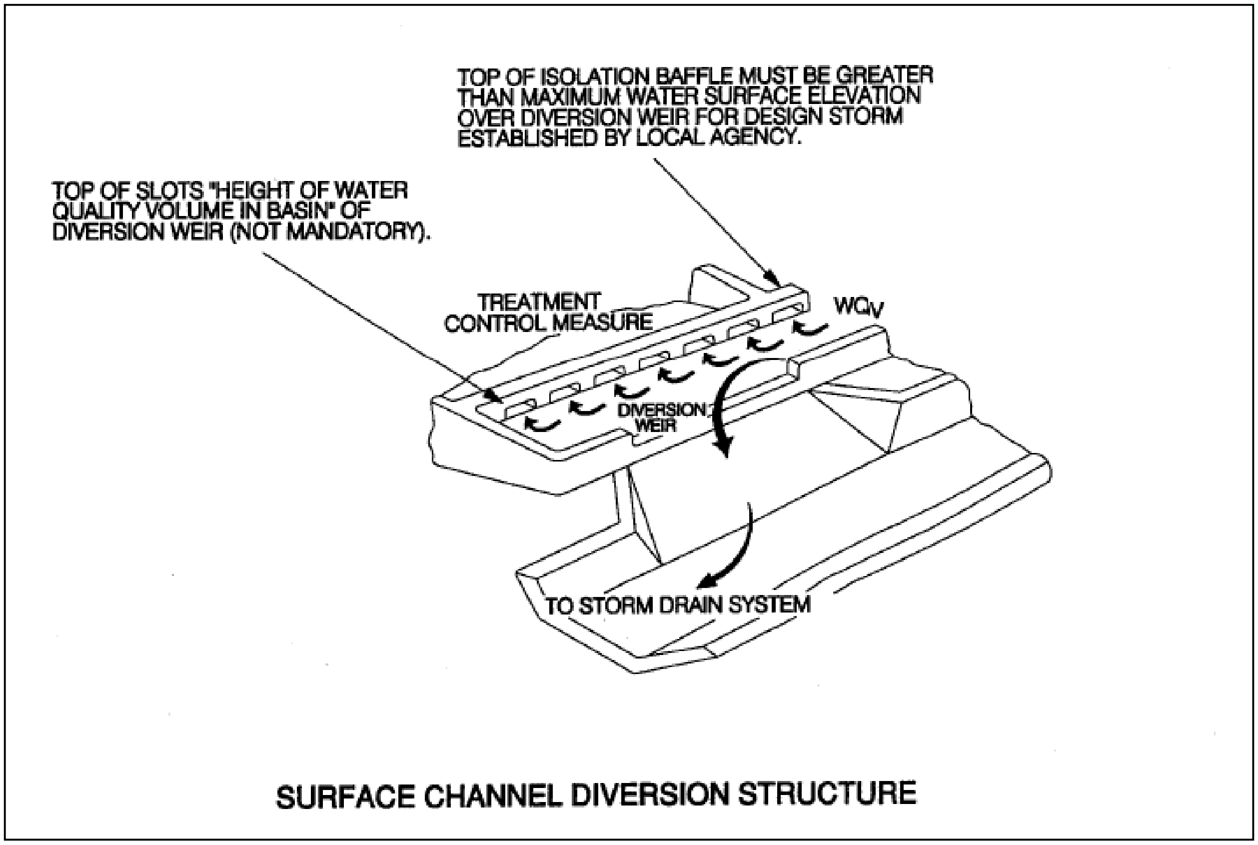

Figure 3.1.4-3 Surface Channel Diversion Structure
(Source: City of Sacramento, 2000)

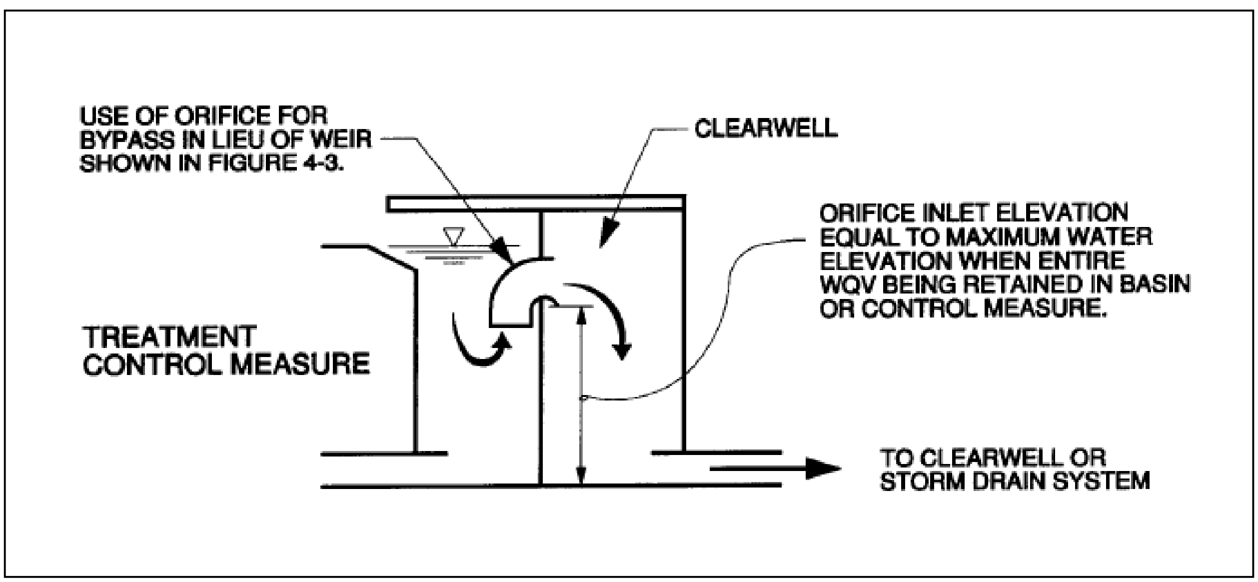

Figure 3.1.4-4 Outlet Flow Regulator
(Source: City of Sacramento, 2000)

3.1.5 Regional vs. On-site Stormwater Management

3.1.5.1 Introduction

Using individual, on-site structural stormwater controls 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 controls in a subwatershed rather than require on-site controls (see Figure 3.1.4-1). For this Manual, regional stormwater controls 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 controls is either eliminated or reduced.

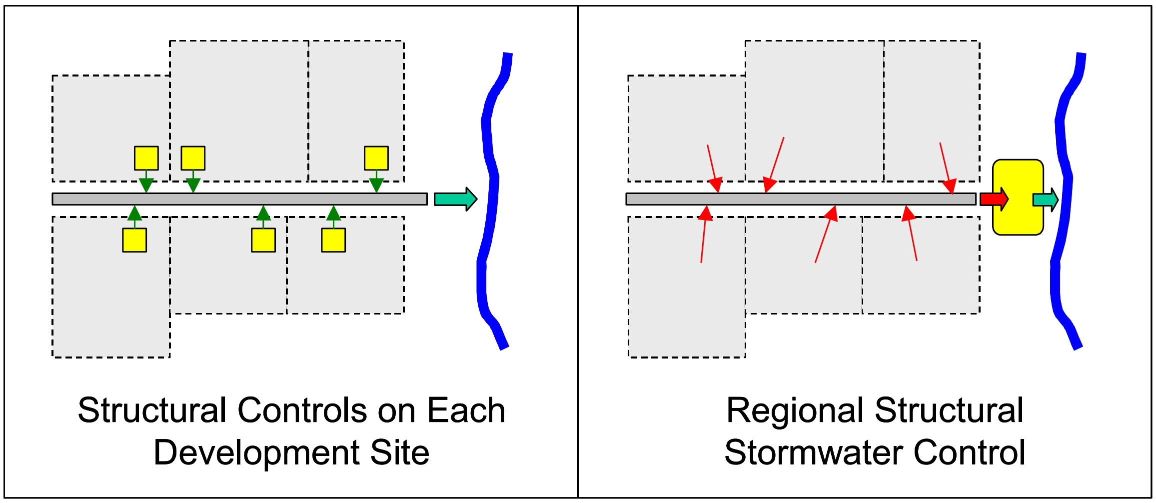

Figure 3.1.5-1 On-site versus Regional Stormwater Management

3.1.5.2 Advantages and Disadvantages of Regional Stormwater Controls

Regional stormwater facilities are significantly more cost-effective because it is easier and less expensive to build, operate, and maintain one large facility than several small ones. Regional stormwater controls are generally better maintained than individual site controls because they are large, highly visible and typically the responsibility of the local government. In addition, a larger facility poses less of a safety hazard than numerous small ones because it is more visible and is easier to secure.

There are also several disadvantages to regional stormwater controls. 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 control. 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.

Below are summarized some of the “pros” and “cons” of regional stormwater controls.

Advantages of Regional Stormwater Controls

- **Reduced Construction Costs** – Design and construction of a single regional stormwater control facility can be far more cost-effective than numerous individual on-site structural controls.
- **Reduced Operation and Maintenance Costs** – Rather than multiple owners and associations being responsible for the maintenance of several storm water 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 the local government.
- **Maximum Utilization of Developable Land** – Developers would be able to maximize the utilization of the proposed development for the purpose intended by minimizing the land normally set aside for the construction of stormwater structural controls.
- **Retrofit Potential** – Regional facilities can be used by a community to mitigate existing developed areas that have insufficient or no structural controls 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 controls 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 and streambank erosion.
- **Ponding Impacts** – Upstream inundation from a regional facility impoundment can eliminate floodplains, wetlands, and other habitat.

3.1.5.3 Important Considerations for the Use of Regional Stormwater Controls

If a community decides to implement a regional stormwater control, 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 man-made conveyances (i.e. storm drains, pipes, concrete channels, etc) then on-site structural controls for water quality and downstream channel protection will need to 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 controls 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 a context of a stormwater master planning or watershed planning to be effective.

3.1.6 Using Structural Stormwater Controls in Series

3.1.6.1 Stormwater Treatment Trains

The minimum stormwater management standards are an integrated planning and design approach whose components work together to limit the 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 and nonstructural and structural controls that work to attain water quality and quantity goals. This is illustrated in Figure 3.1.6-1.

Figure 3.1.6-1 Generalized Stormwater Treatment Train

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 80% TSS reduction goal, but do provide calculable water quality benefits that may be applied towards meeting the WQv 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)
- Limited application structural controls that provide pretreatment
- Pretreatment facilities such as sediment forebays on general application structural controls

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:

- General application structural controls
- Limited application structural controls
- Detention structural controls

3.1.6.2 Use of Multiple Structural Controls in Series

Many combinations of structural controls in series may exist for a site. Figure 3.1.5-2 provides a number of hypothetical examples of how the unified stormwater sizing criteria may be addressed by using structural stormwater controls.

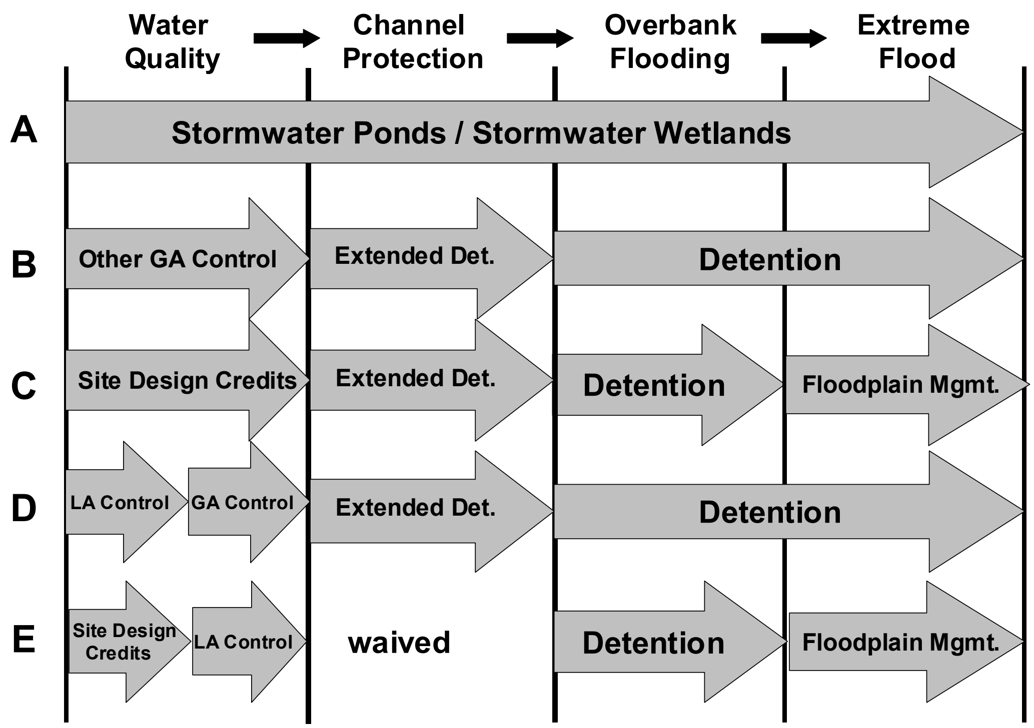

Figure 3.1.6-2 Examples of Structural Controls Used in Series

Referring to Figure 3.1.6-2 by line letter:

A. Two general application (GA) structural controls, *stormwater ponds* and *stormwater wetlands*, can be used to meet all of the unified stormwater sizing criteria in a single facility.

B. The other general application structural controls (*bioretention, sand filters, infiltration trench and enhanced swale*) are typically used in combination with detention controls to meet the unified stormwater sizing criteria. The detention facilities are located downstream from the water quality controls either on-site or combined into a regional or neighborhood facility.

C. Line C indicates the condition where an environmentally sensitive large lot subdivision has been developed that can be designed so as to waive the water quality treatment requirement altogether. However, detention controls may still be required for downstream channel protection, overbank flood protection and extreme flood protection.

D. Where a limited application (LA) structural control does not meet the 80% TSS removal criteria, another downstream structural control must be added. For example, urban hotspot land may be fit or retrofit with devices adjacent to parking or service areas designed to remove petroleum hydrocarbons. These devices may also serve as pre-treatment devices removing the coarser fraction of sediment. One or more downstream structural controls is then used to meet the full 80% TSS removal goal, and well as water quantity control.

E. In line E site design credits have been employed to partially reduce the water quality volume requirement. In this case, for a smaller site, a well designed and tested Limited Application structural control provides adequate TSS removal while a dry detention pond handles the overbank flooding criteria. For this location, direct discharge to a large stream and local downstream floodplain management practices have eliminated the need for channel protection volume and extreme flood protection structural controls on site.

The combinations of structural stormwater controls are limited only by the need to employ measures of proven effectiveness and meet local regulatory and physical site requirements. Figures 3.1.6-3 through 3.1.6-5 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 3.1.6-3 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.

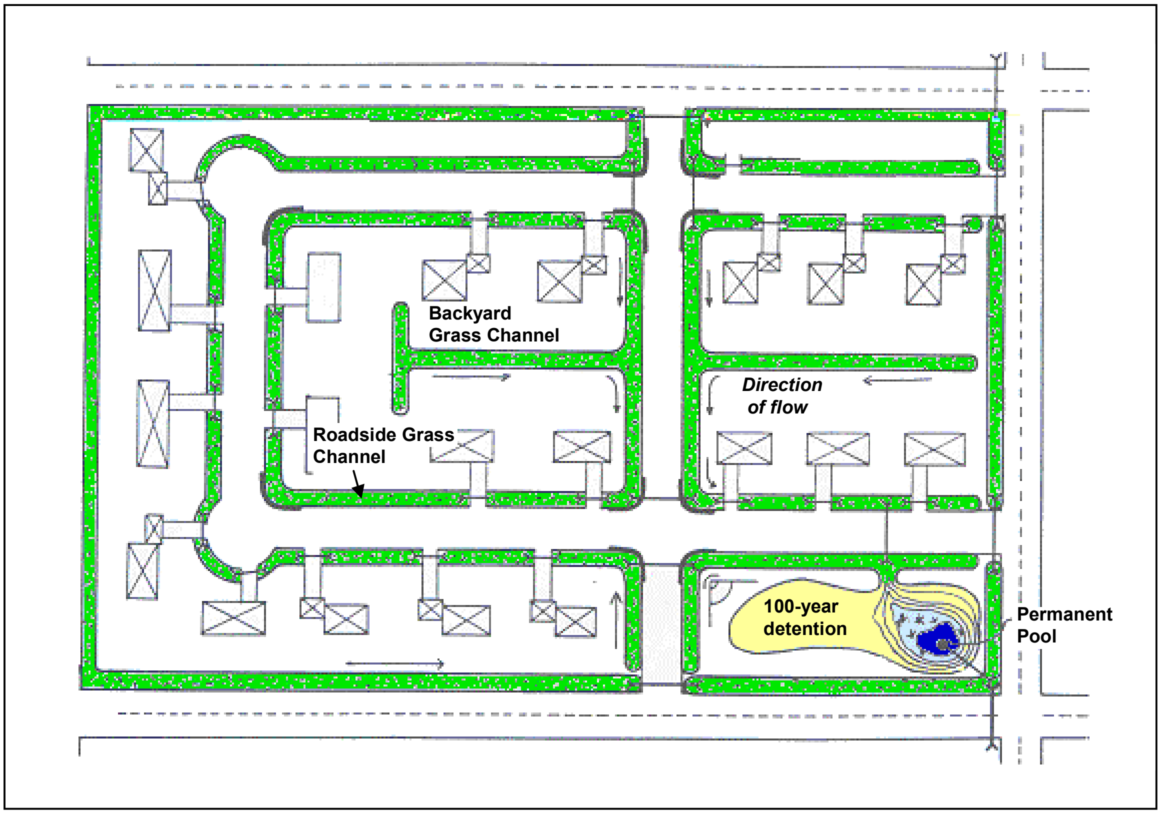

Figure 3.1.6-3 Example Treatment Train – Residential Subdivision
(Adapted from: NIPC, 2000)

A gas station and convenience store is depicted in Figure 3.1.6-4. 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 perimeter sand filter for removal of finer particles and TSS.
No stormwater control 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 3.1.5-5 shows an example treatment train for a commercial shopping center. In this case, runoff from rooftops and parking lots drains to 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 to 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.

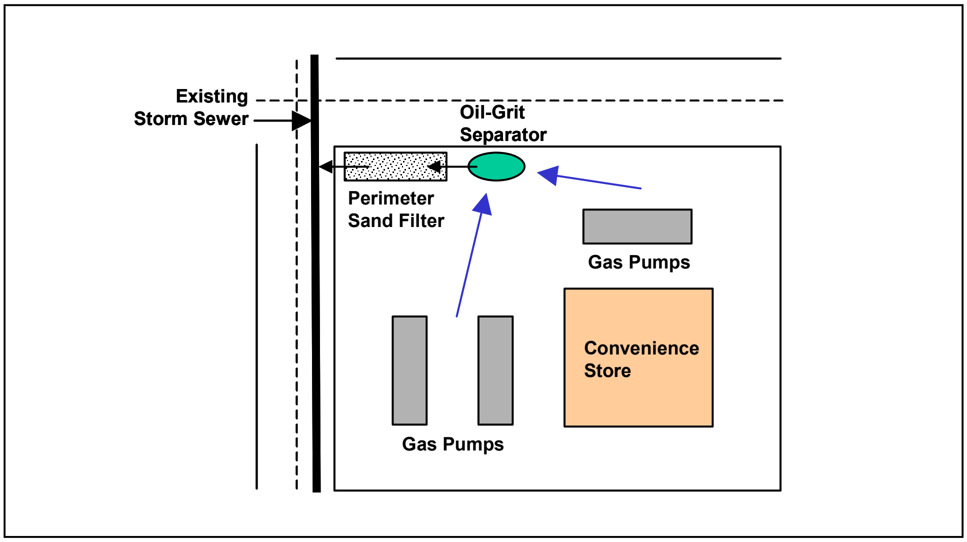

Figure 3.1.6-4 Example Treatment Train – Commercial Development

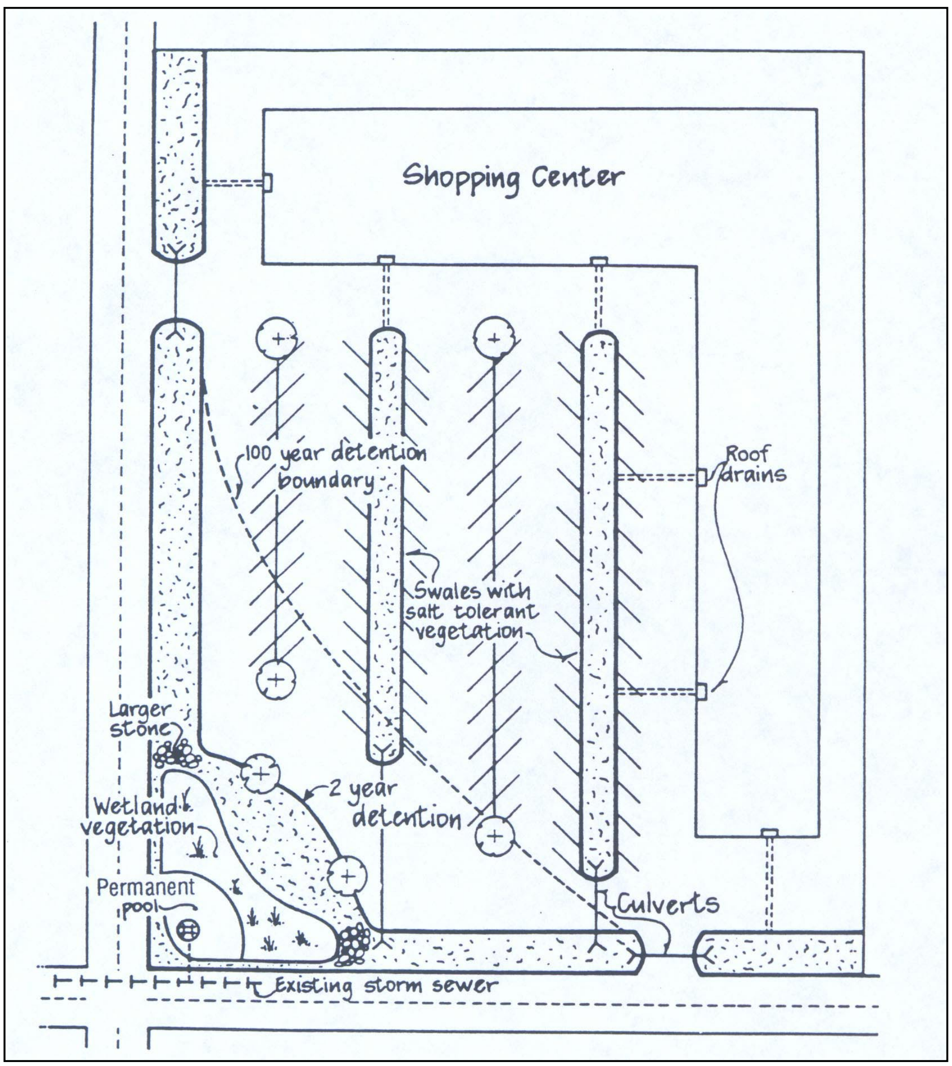

Figure 3.1.6-5 Example Treatment Train – Commercial Development
(Source: NIPC, 2000)

3.1.6.3 Calculation of Pollutant Removal for Structural Controls in Series

For two or more structural stormwater controls used in combination, it is often important to have an estimate of the pollutant removal efficiency of the treatment train. Pollutant removal rates for structural controls in series are not additive. For pollutants in particulate form, the actual removal rate (expressed in terms of percentage of pollution removed) varies directly with the pollution concentration and sediment size distribution of runoff entering a facility.

For example, a stormwater pond facility will have a much higher pollutant removal percentage for very turbid runoff than for clearer water. When two stormwater ponds are placed in series, the second pond will treat an incoming particulate pollutant load very different from the first pond. The upstream pond captures the easily removed larger sediment sizes, passing on an outflow with a lower concentration of TSS but with a higher proportion of finer particle sizes. Hence, the removal capability of the second pond for TSS is considerably less than the first pond. Recent findings suggest that the second pond in series can provide as little as half the removal efficiency of the upstream pond.

To estimate the pollutant removal rate of structural controls in series, a method is used in which the removal efficiency of a downstream structural control is reduced to account for the pollutant removal of the upstream control(s). The following steps are used to determine the pollutant removal:

- For each drainage area list the structural controls in order, upstream to downstream, along with their expected average pollutant removal rates from Table 3.1.2-1 for the pollutants of concern.

- For any general application structural control located downstream from another general application control or a limited application structural control that has TSS removal rates equivalent to 80%, the designer should use <u>50%</u> of the normal pollutant removal rate for the second control in series. For a general application structural control located downstream from a limited application structural control that cannot achieve the 80% TSS reduction goal the designer should use <u>75%</u> of the normal pollutant removal rate for the second control in series.

  For example, if a general application structural control has an 80% TSS removal rate, then a 40% TSS removal rate would be assumed for this control if it were placed downstream from another general application control in the treatment train (0.5 x 80%). If it were placed downstream from a limited application structural control that cannot achieve the 80% TSS reduction goal a 60% TSS removal rate would be assumed (0.75 x 80%). Use this rule with caution depending on the actual pollutant of concern and make allowance for differences among structural control pollutant removal rates for different pollutants. Actual data from similar situations should be used to temper or override this rule of thumb where available.

- For cases where a limited application control is sited upstream from a general application control in the treatment train, the downstream general application structural control is given full credit for removal of pollutants.

- Apply the following equation for calculation of approximate total accumulated pollution removal for controls in series:

  Final Pollutant Removal = (Total load * Control1 removal rate) + (Remaining load * Control2 removal rate) + … for other Controls in series.

Example

TSS is the pollutant of concern and a commercial device is inserted that has a 20% sediment removal rate. A stormwater pond is designed at the site outlet. A second stormwater pond is located downstream from the first one in series. What is the total TSS removal rate? The following information is given:

Control 1 (Commercial Device) = 20% TSS removal
Control 2 (Stormwater Pond 1) = 80% TSS removal (use 1.0 x design removal rate)
Control 3 (Stormwater Pond 2) = 40% TSS removal (use 0.5 x design removal rate)

Then applying the controls in order and working in terms of “units” of TSS starting at 100 units:

For Control 1: 100 units of TSS * 20% removal rate = 20 units removed
100 units - 20 units removed = 80 units of TSS remaining

For Control 2: 80 units of TSS * 80% removal rate = 64 units removed
80 units - 64 units removed = 16 units of TSS remaining

For Control 3: 16 units of TSS * 40% removal rate = 6 units removed
16 units - 6 units removed = 10 units TSS remaining

For the treatment train in total = 100 units TSS – 10 units TSS remaining = **90% removal**

3.1.6.4 Routing with WQ_v Removed

When off-line structural controls such as bioretention areas, sand filters and infiltration trenches capture and remove the water quality volume (WQ_v), downstream structural controls do not have to account for this volume during design. That is, the WQ_v may be subtracted from the total volume that would otherwise need to be routed through the downstream structural controls.

From a calculation standpoint this would amount to removing the initial WQ_v from the beginning of the runoff hydrograph – thus creating a “notch” in the runoff hydrograph. Since most commercially available hydrologic modeling packages cannot handle this type of action, the following method has been created to facilitate removal from the runoff hydrograph of approximately the WQ_v:

- Enter the horizontal axis on Figure 3.1.4-6 with the impervious percentage of the watershed and read upward to the predominant soil type (interpolation between curves is permitted)
- Read left to the factor
- Multiply the curve number for the sub-watershed that includes the water quality basin by this factor – this provides a smaller curve number

The difference in curve number will generate a runoff hydrograph that has a volume less than the original volume by an amount approximately equal to the WQ_v. This method should be used only for bioretention areas, filter facilities and infiltration trenches where the drawdown time is ≥ 24 hours.

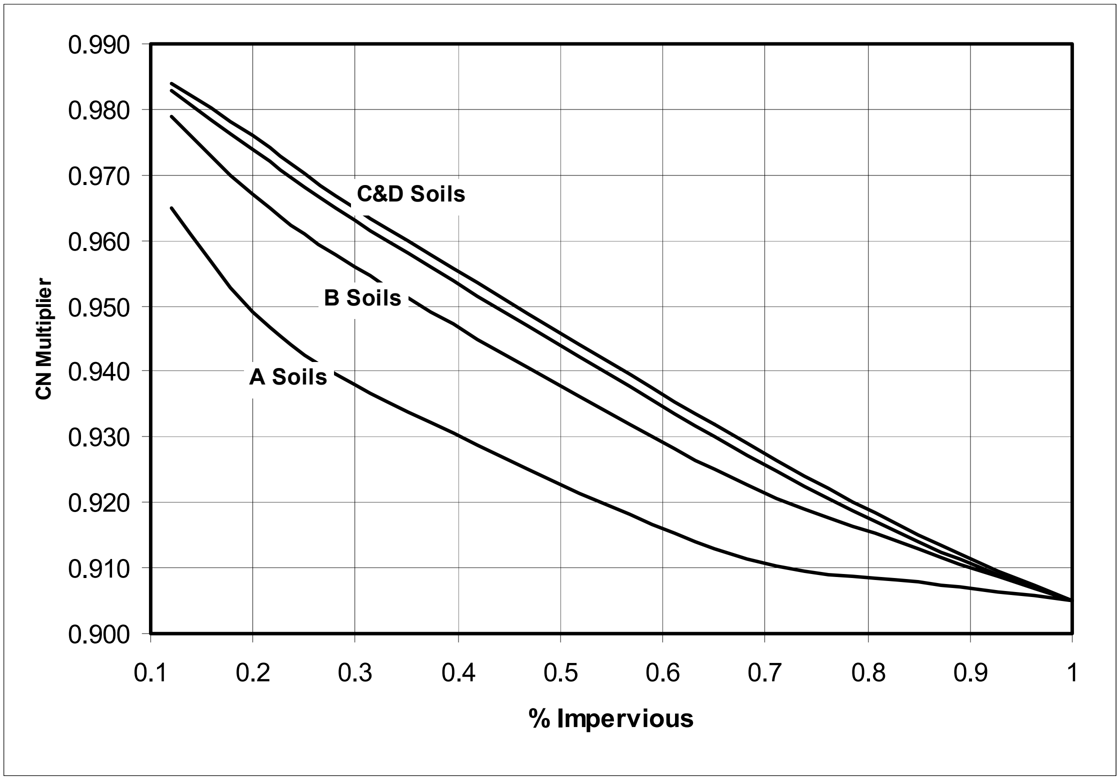

Figure 3.1.6-6 Curve Number Adjustment Factor

<u>Example</u>

A site design employs an infiltration trench for the WQ_v and has a curve number of 72, is B type soil, and has an impervious percentage of 60%, the factor from Figure 3.1.4-1 is 0.92. The curve number to be used in calculation of a runoff hydrograph for the quantity controls would be: (72*0.92) = 66.

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SECTION

3.2

GENERAL APPLICATION STRUCTRUAL STORMWATER CONTROLS

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3.2.1 Stormwater Ponds

General Application
Structural Stormwater Control

Description: Constructed stormwater retention basin that has a permanent pool (or micropool). Runoff from each rain event is detained and treated in the pool primarily through settling and biological uptake mechanisms.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Minimum contributing drainage area of 25 acres; 10 acres for micropool ED pond
- A sediment forebay or equivalent upstream pretreatment must be provided
- Minimum length to width ratio for the pond is 1.5:1
- Maximum depth of the permanent pool should not exceed 8 feet
- Side slopes to the pond should not exceed 3:1 (h:v)

ADVANTAGES / BENEFITS:

- Moderate to high removal rate of urban pollutants
- High community acceptance
- Opportunity 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

MAINTENANCE REQUIREMENTS:

- Remove debris from inlet and outlet structures
- Maintain side slopes / remove invasive vegetation
- Monitor sediment accumulation and remove periodically

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
50/30%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
50%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
70%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORMWATER MANAGEMENT SUITABILITY

- ✓ **Water Quality**
- ✓ **Channel Protection**
- ✓ **Overbank Flood Protection**
- ✓ **Extreme Flood Protection**

Accepts Hotspot Runoff: *Yes*
(2 feet of separation distance required to water table)

FEASIBILITY CONSIDERATIONS

M-H	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *No*

Drainage Area: *10-25 acres min.*

Soils: *Hydrologic group 'A' and 'B' soils may require pond liner*

Other Considerations:

- *Outlet clogging*
- *Safety bench*
- *Landscaping*

L=Low **M**=Moderate **H**=High

3.2.1.1 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 already 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 storm for downstream channel protection, as well as normal detention of larger storm events (25-year and, optionally, the 100-year 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 and located properly.

There are several different 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 3.2.1-1 shows a number of examples of stormwater pond variants. Section 3.2.1.8 provides plan view and profile schematics for the design of a wet pond, wet extended detention pond, micropool extended detention pond, and multiple pond system.

> *Conventional dry detention basins* do not provide a permanent pool and are **not recommended** for general application use to meet water quality criteria, as they fail to demonstrate an ability to meet the majority of the water quality goals. In addition, dry detention basins are prone to clogging and resuspension of previously settled solids and require a higher frequency of maintenance than wet ponds if used for untreated stormwater flows. These facilities can be used in combination with appropriate water quality controls to provide channel protection, and overbank and extreme flood storage. Please see a further discussion in subsection 3.4.1 (*Dry Detention Basins*).

Wet Pond

Wet ED Pond

Micropool ED Pond

Wet ED Pond

Figure 3.2.1-1 Stormwater Pond Examples

3.2.1.2 Stormwater Management Suitability

Stormwater ponds are designed to control both stormwater quantity and quality. Thus, a stormwater pond can be used to address all of the *unified stormwater sizing criteria* for a given drainage area.

<u>Water Quality</u>

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.

Section 3.2.1.3 provides median pollutant removal efficiencies that can be used for planning and design purposes.

<u>Channel Protection</u>

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 storm (Q_p) 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 storm peak flow (Q_f). Where this is not required, the pond structure is designed to safely pass extreme storm flows.

3.2.1.3 Pollutant Removal Capabilities

All of the stormwater pond design variants 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 ponds 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 – 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 (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

3.2.1.4 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 – Land requirements may preclude use
- Regional Stormwater Control – YES

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – A minimum of 25 acres is needed for wet pond and wet ED pond to maintain a permanent pool, 10 acres minimum for micropool ED pond. A smaller drainage area may be acceptable with an adequate water balance and anti-clogging device.
- Space Required – Approximately 2 to 3% of the tributary drainage area
- Site Slope – There should be more than 15% slope across the pond site.
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 6 to 8 feet

- 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 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

- Trout Streams – Consideration should be given to the thermal influence of stormwater pond outflows on downstream trout waters.

3.2.1.5 Planning and Design Criteria

*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 followed.*

A. LOCATION AND SITING

- Stormwater ponds should have a minimum contributing drainage area of 25 acres or more for 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 areas with a high water table). In these cases a water balance may be performed (see subsection 2.1.8 for details). Ensure that an appropriate anti-clogging device is provided for the pond outlet.

- A stormwater pond should be sited such that the 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.

- Minimum setback requirements for stormwater pond facilities (when not specified by local ordinance or criteria):
  - From a property line – 10 feet
  - From a private well – 100 feet; if well is downgradient from a hotspot land use then the minimum setback is 250 feet
  - From a septic system tank/leach field – 50 feet

- All utilities should be located outside of the pond/basin site.

B. GENERAL DESIGN

- A well-designed stormwater pond consists of:

  (1) Permanent pool of water,
  (2) Overlying zone in which runoff control volumes are stored, and
  (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 need to include a sediment forebay at the inflow** to the basin to allow heavier sediments to drop out of suspension before the runoff enters the permanent pool. (A sediment forebay schematic can be found in Appendix C)

- Additional pond design features include an **emergency spillway, maintenance access, safety bench, pond buffer, and appropriate native landscaping**.

Figures 3.2.1-4 thru 3.2.1-7 in subsection 3.2.1.8 provide plan view and profile schematics for the design of a wet pond, wet ED pond, micropool ED pond and multiple pond system.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

In general, pond designs are unique for each site and application. However, there are number of geometric ratios and limiting depths for pond design 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: Approximately 0.1 inch per impervious acre

- Proper geometric design is essential to prevent hydraulic short-circuiting (unequal distribution of inflow), which results in the 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 or islands can be added within the permanent pool to increase the flow path.

- Maximum depth of the permanent pool should generally not exceed 8 feet to avoid stratification and anoxic conditions. Minimum depth for the pond bottom should be 3 to 4 feet. Deeper depths near the outlet will yield cooler bottom water discharges that may mitigate downstream thermal effects.

- Side slopes to the pond should not usually exceed 3:1 (h:v) without safety precautions or if mowing is anticipated and should terminate on a safety bench (see Figure 3.2.1-2). The safety bench requirement may be waived if slopes are 4:1 or gentler.

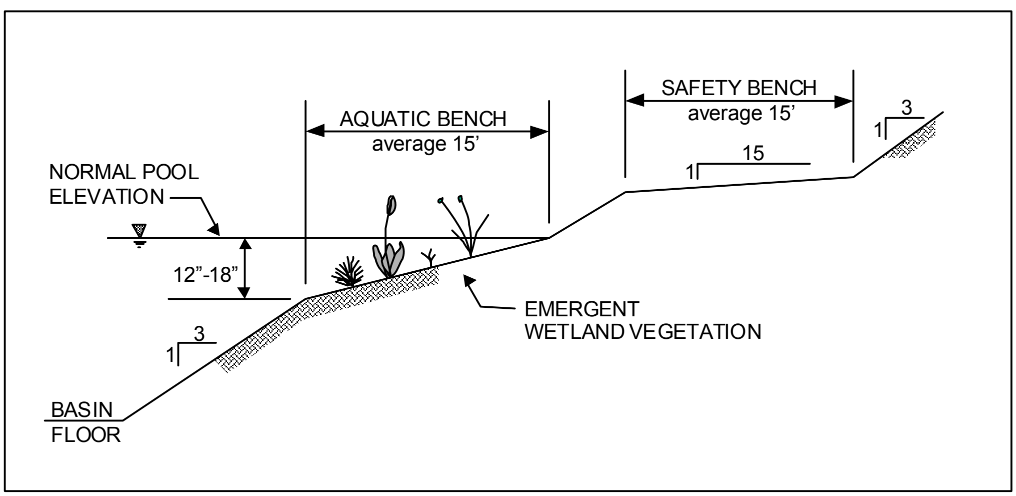

Figure 3.2.1-2 Typical Stormwater Pond Geometry Criteria

- The perimeter of all deep pool areas (4 feet or greater in depth) 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 3.2.1-2).

- The contours and shape of the permanent pool should be irregular to provide a more natural landscaping effect.

D. 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 to 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.

- 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 nonerosive.

E. 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 3.2.1-3). The riser should be located within the embankment for maintenance access, safety and aesthetics.

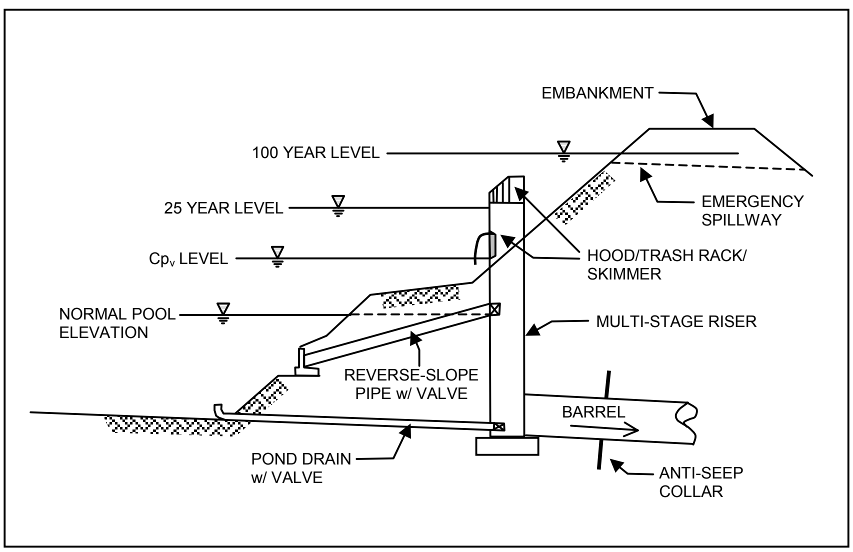

Figure 3.2.1-3 Typical Pond Outlet Structure

- 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 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 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 wet pond providing *only* water quality treatment can use a simple overflow weir as the outlet structure.

  In the case of a wet ED pond or micropool ED pond, 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 (if design is for a wet ED or micropool ED pond) 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 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. See Section 4.5 (*Energy Dissipation Design*) for more guidance.

- 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 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 3.2.1.6 as well as Section 2.2 (*Storage Design*) and Section 2.3 (*Outlet Structures*) for additional information and specifications on pond routing and outlet works.

F. EMERGENCY SPILLWAY

- An emergency spillway is to 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.

G. 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 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.

H. SAFETY FEATURES

- All embankments and spillways must be designed to State of Georgia guidelines for dam safety (see Appendix H).

- 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 dropoffs 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, and 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.

I. LANDSCAPING

- Aquatic vegetation can play an important role in pollutant removal in a stormwater pond. In addition, vegetation can enhance the appearance of the pond, stabilize side slopes, serve as wildlife habitat, and can temporarily conceal unsightly trash and debris. Therefore, wetland plants should be encouraged in a pond design, along the aquatic bench (fringe wetlands), the 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 F (*Landscaping and Aesthetics Guidance)*.

- 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 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 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, it is advisable to excavate large and 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 pond locations is strongly encouraged.

J. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

Physiographic Factors - Local terrain design constraints

- Low Relief – Maximum normal pool depth is limited; providing pond 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 generally require pond liner; group “B” soils may require infiltration testing

Special Downstream Watershed Considerations

- Trout Stream – Micropool ED pond best alternative; design wet ponds and wet ED ponds offline and provide shading to minimize thermal impact; limit WQ_v-ED to 12 hours
- Aquifer Protection – Reduce potential groundwater contamination by preventing infiltration of hotspot runoff. May require liner for type “A” and “B” soils; Pretreat hotspots; 2 to 4 foot separation distance from water table
- Swimming Area/Shellfish – Design for geese prevention (see Appendix F); provide 48-hour ED for maximum coliform dieoff.

3.2.1.6 Design Procedures

Step 1. Compute runoff control volumes from the Unified Stormwater Sizing Criteria

Calculate the Water Quality Volume (WQ_v), Channel Protection Volume (Cp_v), Overbank Flood Protection Volume (Q_p), and the Extreme Flood Volume (Q_f).

Details on the Unified Stormwater Sizing Criteria are found in Section 1.4.

Step 2. Determine if the development site and conditions are appropriate for the use of a stormwater pond

Consider the Application and Site Feasibility Criteria in subsections 3.2.1.4 and 3.2.1.5-A (Location and Siting).

Step 3. Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 3.2.1.5-J. (Additional Site-Specific Design Criteria and Issues).

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.

Step 4. 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 pond. The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 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. Size extended detention volume to 0.5 WQ_v.

Micropool ED Pond: Size permanent pool volume to 25 to 30% of WQ_v. Size extended detention volume to remainder of WQ_v.

Step 6. Determine pond location and preliminary geometry. Conduct pond grading and determine storage available for permanent pool (and water quality extended detention if wet ED pond or micropool ED pond)

This step involves initially grading the pond (establishing contours) and determining the elevation-storage relationship for the pond.

- Include safety and aquatic benches.
- Set WQ_v permanent pool elevation (and WQ_v-ED elevation for wet ED and micropool ED pond) based on volumes calculated earlier.

See subsection 3.2.1.5-C (Physical Specifications / Geometry) for more details.

Step 7. Compute extended detention orifice release rate(s) and size(s), and establish Cp_v elevation

Wet Pond: The Cp_v elevation is determined from the stage-storage relationship and the orifice is then sized to release the channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold water streams). 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 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. 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 Q_{p25} (25-year storm) 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 storm.

Step 9. Design embankment(s) and spillway(s)

Size emergency spillway, calculate 100-year water surface elevation, set top of embankment elevation, and analyze safe passage of the Extreme Flood Volume (Q_f).

At final design, provide safe passage for the 100-year 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 (see Appendix H).

Step 11. Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

See subsection 3.2.1.5-D through H for more details.

Step 12. Prepare 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 3.2.1.5-I (Landscaping) and Appendix F for more details.

See Appendix D-1 for a Stormwater Pond Design Example

3.2.1.7 Inspection and Maintenance Requirements

Table 3.2.1-1 Typical Maintenance Activities for Ponds
(Source: WMI, 1997)

Activity	Schedule
• Clean and remove debris from inlet and outlet structures. • Mow side slopes.	Monthly
• If wetland components are included, inspect for invasive vegetation.	Semiannual Inspection
• Inspect for damage, paying particular attention to the control structure. • Check for signs of eutrophic conditions. • Note signs of hydrocarbon build-up, and remove appropriately. • Monitor for sediment accumulation in the facility and forebay. • Examine to ensure that inlet and outlet devices are free of debris and operational. • 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
• 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

Additional Maintenance Considerations and Requirements

- A sediment marker should be located in the forebay to determine when sediment removal is required.
- Sediments excavated from stormwater ponds that do not receive runoff from designated hotspots are not considered toxic or hazardous material and can be safely disposed of by either land application or landfilling. Sediment testing may be required prior to sediment disposal when a hotspot land use is present.
- Periodic mowing of the pond buffer is only required along maintenance rights-of-way and the embankment. The remaining buffer can be managed as a meadow (mowing every other year) or forest.
- Care should be exercised during pond drawdowns to prevent downstream discharge of sediments, anoxic water, or high flows with erosive velocities. The approving jurisdiction should be notified before draining a stormwater pond.

Regular inspection and maintenance is critical to the effective operation of stormwater ponds as designed. Maintenance responsibility for a pond and its buffer 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.

3.2.1.8 Example Schematics

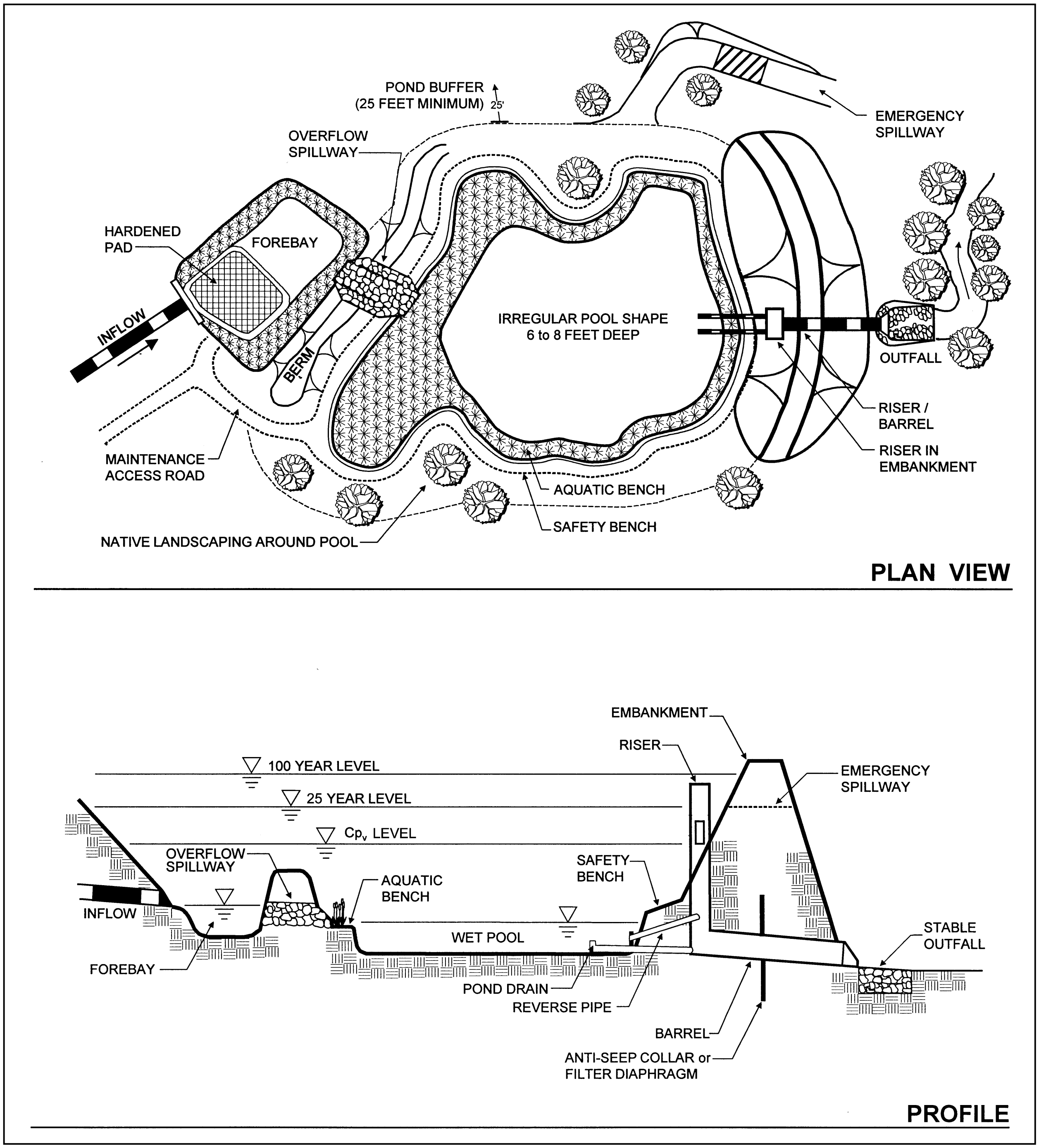

Figure 3.2.1-4 Schematic of Wet Pond

(Source: Center for Watershed Protection)

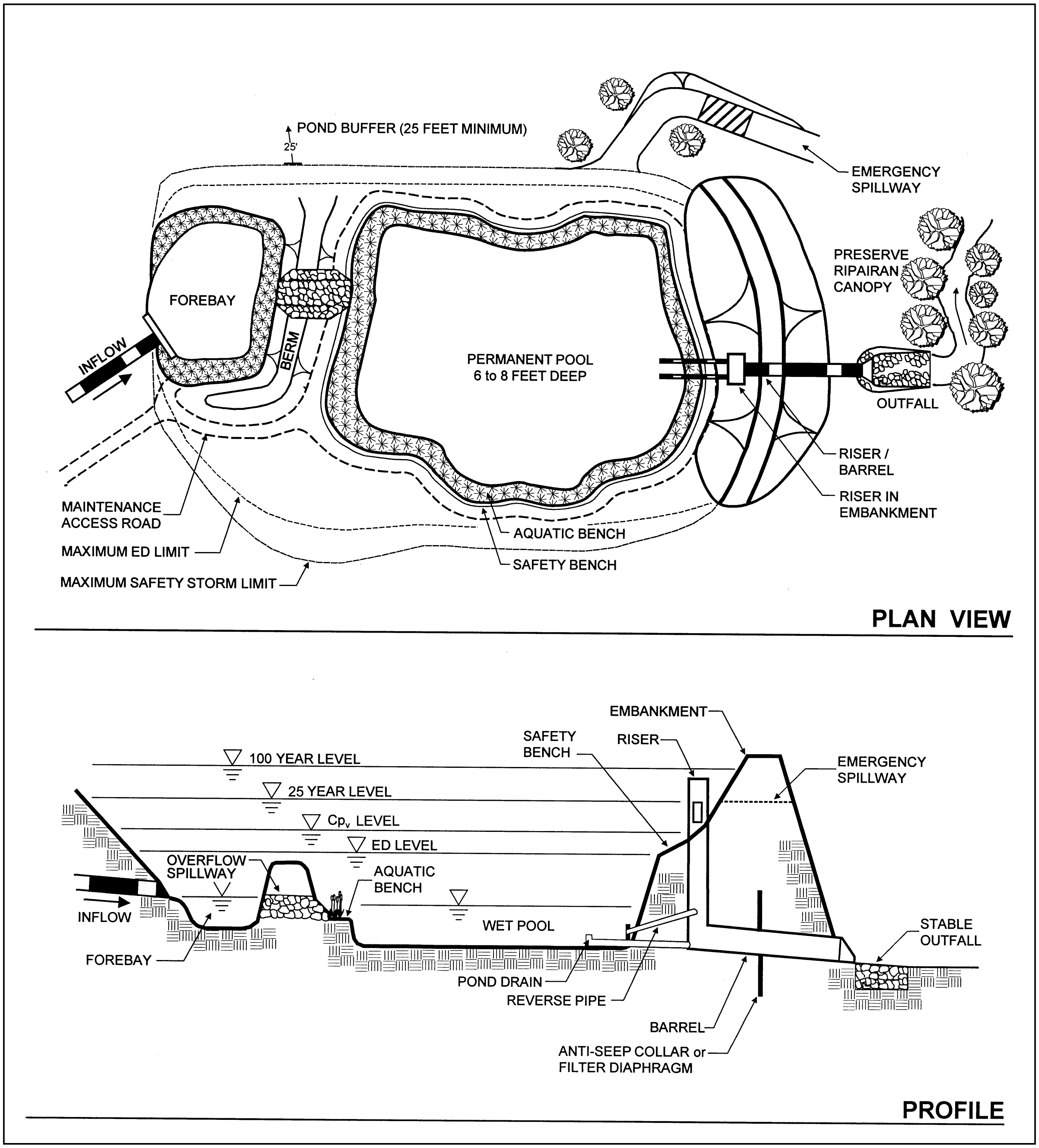

Figure 3.2.1-5 Schematic of Wet Extended Detention Pond

(Source: Center for Watershed Protection)

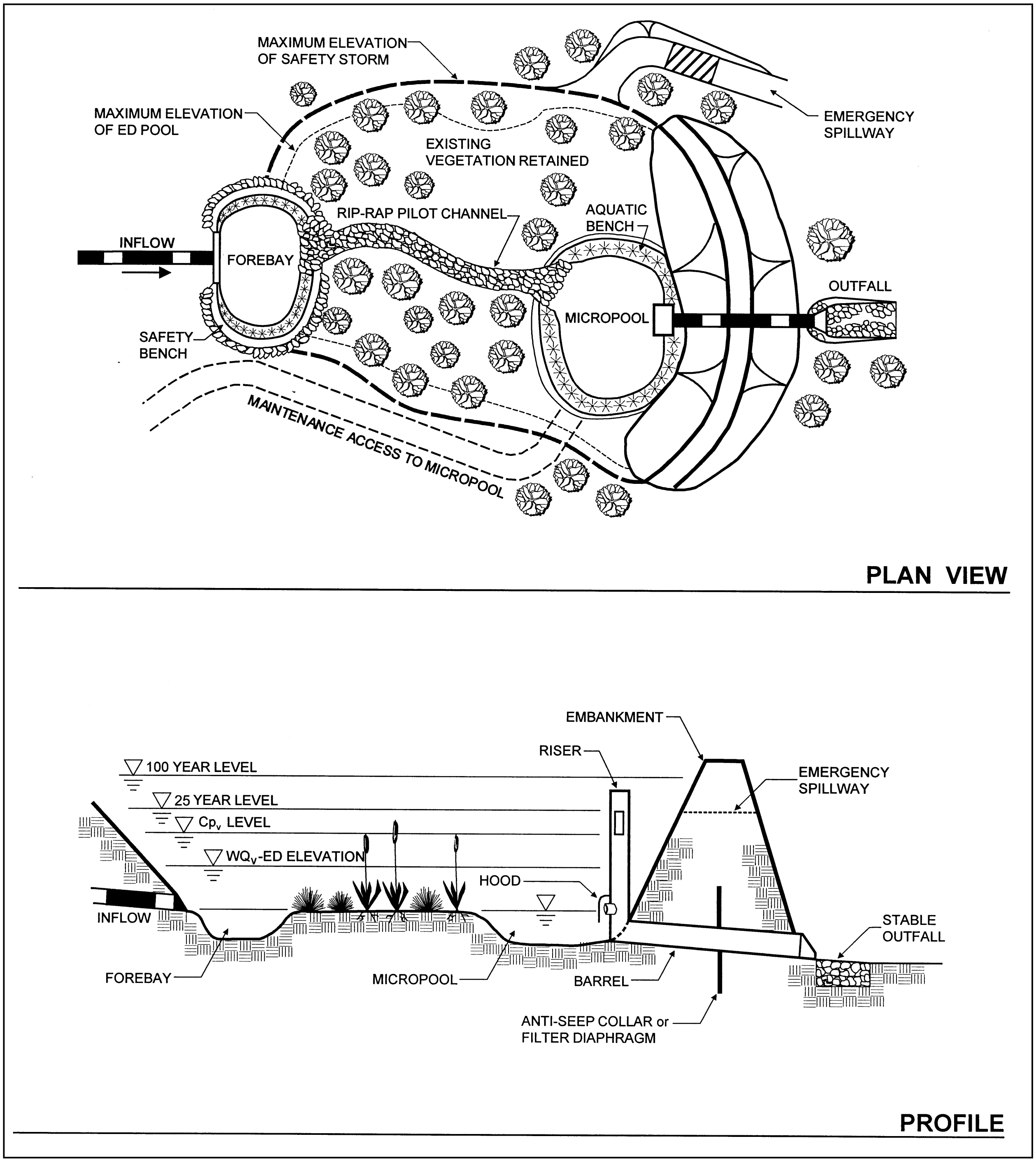

Figure 3.2.1-6 Schematic of Micropool Extended Detention Pond

(Source: Center for Watershed Protection)

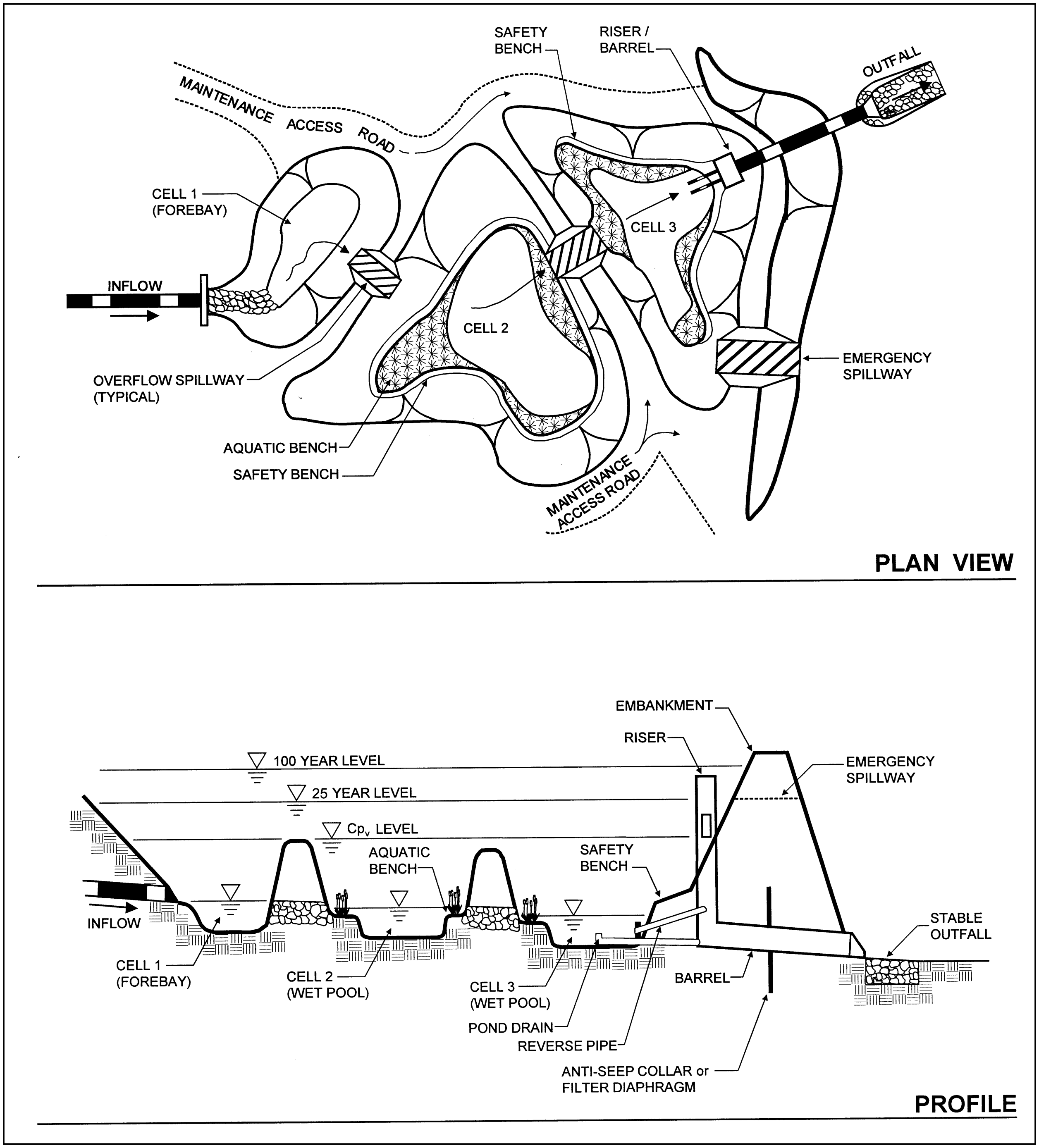

Figure 3.2.1-7 Schematic of Multiple Pond System

(Source: Center for Watershed Protection)

3.2.1.9 Design Forms

Design Procedure Form: Stormwater Ponds

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQ_v volume requirements
Compute Runoff Coefficient, R_v — R_v = ________
Compute WQv — WQ_v = ________ acre-ft

1b. Compute Cp_v — Cp_v = ________ acre-ft
Compute average release rate — release rate = ________ cfs
Compute Q_{p-25} — Q_{p-25} = ________ acre-ft
Add 15% to the required Q_{p-25} volume — Q_{p-25} * 15% = ________ acre-ft
Compute (as necessary) Q_f — Q_f = ________ acre-ft

STORMWATER POND DESIGN

2. Is the use of a stormwater pond appropriate? — ***See subsections 3.2.1.4 and 3.2.1.5 - A***

3. Confirm local design criteria and applicability

4. Pretreatment volume — Vol_{pre} = ________ acre-ft
Vol_{pre} = I (0.1")(1'/12")

5. Allocation of Permanent Pool Volume and ED Volume

Wet Pond: $Vol_{pool} = WQ_v$ — Vol_{pool} = ________ acre-ft

Wet ED Pond: $Vol_{pool} = 0.5\,(WQ_v)$ — Vol_{pool} = ________ acre-ft
$Vol_{ED} = 0.5\,(WQ_v)$ — Vol_{ED} = ________ acre-ft

Micropool ED Pond: $Vol_{pool} = 0.25\,(WQ_v)$ — Vol_{pool} = ________ acre-ft
$Vol_{ED} = 0.75\,(WQ_v)$ — Vol_{ED} = ________ acre-ft

6. Conduct grading and determine storage available for permanent pool (and WQv-ED volume if applicable) — *Prepare an elevation-storage table and curve using the average area method for computing volumes.*

Elevation MSL	Area ft^2	Average Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3	Cumulative Volume ac-ft	Volume above Permanent Pool ac-ft

7. WQ_v Orifice Computations
Average ED release rate (if applicable) — release rate = ________ cfs
Average head, h = (ED elev. - Permanent pool elev.) / 2 — h = ________ ft
Area of orifice from orifice equation
$Q = CA(2gh)^{0.5}$ — A = ________ ft^2
diameter = ________ in
Discharge equation Q = (h)^0.5 — factor = ________ (h)^0.5

Compute release rate for Cp_v-ED control and establish Cp_v elevation — WSEL = ________ ft-NGVD
Release rate = — release rate = ________ cfs
Average head, h = (Cp_v elev. - Permanent pool elev.) / 2 — h = ________ ft
Area of orifice from orifice equation — A = ________ ft^2
$Q = CA(2gh)^{0.5}$ — diameter = ________ in
Discharge equation Q = (h)^0.5 — factor = ________ (h)^0.5

8. Calculate Q_{p-25} release rate and WSEL

Set up a stage-storage-discharge relationship

Elevation	Storage	Low Flow	Riser			Barrel		Emergency	Total
		WQv-ED	Cpv-ED	High Storage		Inlet	Pipe	Spillway	Storage
				Orif.	Weir				
MSL	ac-ft	H(ft) Q(cfs)	H(ft) Q(cfs)	H Q	H Q	H(ft) Q(cfs)	H(ft) Q(cfs)	H(ft) Q(cfs)	Q(cfs)

Q_{p-25} = pre-dev. peak discharge - (WQ_v-ED release + Cp_v-ED release)

Q_{p-25} = ____________ cfs

Maximum head =
Use weir equation for slot length ($Q = CLH^{3/2}$)

H = ____________ ft
L = ____________ ft

Check inlet condition
Check outlet condition

Use culvert charts (Section 4.3)

9. Size emergency spillway, calculate 100-year WSEL and set top of embankment elevation

$WSEL_{25}$ = ____________ ft
$WSEL_{100}$ = ____________ ft
Q_{ES} = ____________ cfs
Q_{PS} = ____________ cfs

10. Investigate potential pond hazard classification

See Appendix H

11. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features.

See subsection 3.2.1.5 - D through H

12. Attach landscaping plan

See Appendix F

Notes: ____

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3.2.2 Stormwater Wetlands

Description: Constructed wetland systems used for stormwater management. Runoff volume is both stored and treated in the wetland facility.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Minimum contributing drainage area of 25 acres; 5 acres for pocket wetland
- Minimum dry weather flow path of 2:1 (length:width) should be provided from inflow to outflow
- Minimum of 35% of total surface area should have a depth of 6 inches or less; 10 to 20% of surface area should be deep pool (1.5- to 6-foot depth)

ADVANTAGES / BENEFITS:

- Good nutrient removal
- Provides natural wildlife habitat
- Relatively low maintenance costs

DISADVANTAGES / LIMITATIONS:

- Requires large land area
- Needs continuous baseflow for viable wetland
- Sediment regulation is critical to sustain wetlands

MAINTENANCE REQUIREMENTS:

- Replace wetland vegetation to maintain at least 50% surface area coverage
- Remove invasive vegetation
- Monitor sediment accumulation and remove periodically

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
40/30%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
50%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
70%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORMWATER MANAGEMENT SUITABILITY

- ✓ **Water Quality**
- ✓ **Channel Protection**
- ✓ **Overbank Flood Protection**
- ✓ **Extreme Flood Protection**

Accepts Hotspot Runoff: *Yes*

(2 feet of separation distance required to water table)

IMPLEMENTATION CONSIDERATIONS

M-H	**Land Requirement**
M	**Capital Cost**
	Maintenance Burden:
M	Shallow Wetland
M	ED Shallow Wetland
H	Pocket Wetland
M	Pond/Wetland

Residential Subdivision Use: *Yes*

High-Density/Ultra-Urban: *No*

Drainage Area: *25 acres min.*

Soils: *Hydrologic group 'A' and 'B' soils may require liner*

L=Low **M**=Moderate **H**=High

3.2.2.1 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. 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.

There are several design variations of the stormwater wetland, each design differing in the relative amounts of shallow and deep water, and dry storage above the wetland. These include the shallow wetland, the extended detention shallow wetland, pond/wetland system and pocket wetland. Below are descriptions of each design variant:

- **Shallow Wetland** – 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** – 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 specified in the ED zone.
- **Pond/Wetland Systems** – The pond/wetland system has two separate cells: a wet pond and a shallow marsh. The wet pond traps sediments 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** – A pocket wetland is intended for smaller drainage areas of 5 to 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 application 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 3.3.5.

3.2.2.2 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.

<u>Water Quality</u>

Pollutants are removed from stormwater runoff in a wetland through uptake by wetland vegetation and algae, vegetative filtering, and through gravitational settling in the slow moving marsh flow. Other pollutant removal mechanisms are also at work in a stormwater wetland, including chemical

Shallow Wetland

Shallow ED Wetland

Newly Constructed Shallow Wetland

Pocket Wetland

Figure 3.2.2-1 Stormwater Wetland Examples

and biological decomposition, and volatilization. Section 3.2.2.3 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). This is accomplished 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_p) 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 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.

3.2.2.3 Pollutant Removal Capabilities

All of the stormwater wetland design variants 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 wetland facilities 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 – 40%**
- **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 wetlands, 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

3.2.2.4 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 is needed to maintain wetland conditions; 5 acres for pocket wetland
- 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
- 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 wetland is typically below 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 an actual subsurface analysis and permeability tests*.

Other Constraints / Considerations

- Trout Streams – Consideration should be given to the thermal influence of stormwater wetland outflows on downstream trout waters.

3.2.2.5 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 followed.*

A. LOCATION AND SITING

- Stormwater wetlands should normally have a minimum contributing drainage area of 25 acres or more. For a pocket wetland, the minimum drainage area is 5 acres.

- 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 (see subsection 2.1.8 for details).

- 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.

- Minimum setback requirements for stormwater wetland facilities (when not specified by local ordinance or criteria):
  - From a property line – 10 feet
  - From a private well – 100 feet; if well is downgradient from a hotspot land use then the minimum setback is 250 feet
  - From a septic system tank/leach field – 50 feet

- All utilities should be located outside of the wetland site.

B. GENERAL DESIGN

- A well-designed stormwater wetland consists of:

  (1) Shallow marsh areas of varying depths with wetland vegetation,
  (2) Permanent micropool, and
  (3) Overlying zone in which runoff control volumes are stored.

  Pond/wetland systems also include a stormwater pond facility (see Section 3.2.1, *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. (Design information for sediment forebays can be found in Appendix B)

- Additional pond design features include an **emergency spillway, maintenance access, safety bench, wetland buffer, and appropriate wetland vegetation and native landscaping**.

Figures 3.2.2-3 through 3.2.2-6 in subsection 3.2.2.8 provide plan view and profile schematics for the design of a shallow wetland, ED shallow wetland, pond/wetland system, and pocket wetland.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

In general, wetland designs are unique for each site and application. However, there are 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. Table 3.2.2-1 provides the recommended physical specifications and geometry for the various stormwater wetland design variants.

Table 3.2.2-1 Recommended Design Criteria for Stormwater Wetlands
Modified from Massachusetts DEP, 1997; Schueler, 1992

Design Criteria	Shallow Wetland	ED Shallow Wetland	Pond/ Wetland	Pocket Wetland
Length to Width Ratio (minimum)	2:1	2:1	2:1	2:1
Extended Detention (ED)	No	Yes	Optional	Optional
Allocation of WQ_v Volume (pool/marsh/ED) in %	25/75/0	25/25/50	70/30/0 (includes pond volume)	25/75/0
Allocation of Surface Area (deepwater/low marsh/high marsh/semi-wet) in %	20/35/40/5	10/35/45/10	45/25/25/5 (includes pond surface area)	10/45/40/5
Forebay	Required	Required	Required	Optional
Micropool	Required	Required	Required	Required
Outlet Configuration	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Hooded broad-crested weir

Depth:
Deepwater: 1.5 to 6 feet below normal pool elevation
Low marsh: 6 to 18 inches below normal pool elevation
High marsh: 6 inches or less below normal pool elevation
Semi-wet zone: Above normal pool elevation

- The stormwater wetland should be designed with the recommended proportion of “depth zones.” Each of the four wetland design variants has depth zone allocations which are given as a percentage of the stormwater wetland surface area. Target allocations are found in Table 3.2.2-1. The four basic depth zones are:

  Deepwater zone
  From 1.5 to 6 feet deep. 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
  From 6 to 18 inches below the normal permanent pool or water surface elevation. This zone is suitable for the growth of several emergent wetland plant species.

  High marsh zone
  From 6 inches below the pool to the normal pool elevation. 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.

  Semi-wet zone
  Those areas above the permanent pool that are inundated during larger storm events. This zone supports a number of species that can survive flooding.

- A minimum dry weather flow path of 2:1 (length to width) is required from inflow to outlet across the stormwater wetland and should ideally be greater than 3:1. This path may be achieved by constructing internal dikes or berms, using marsh plantings, and by using multiple cells. Finger dikes are commonly used in surface flow systems to create serpentine configurations and prevent short-circuiting. Microtopography (contours along the bottom of a wetland or marsh that provide a variety of conditions for different species needs and increases the surface area to volume ratio) is encouraged to enhance wetland diversity.

- A 4- to 6-foot deep micropool must be included in the design at the outlet to prevent the outlet from clogging and resuspension of sediments, and to mitigate thermal effects.

- Maximum depth of any permanent pool areas should generally not exceed 6 feet.

- The volume of the extended detention must not comprise more than 50% of the total WQ_v, and its maximum water surface elevation must not extend more than 3 feet above the normal pool. Q_p and/or Cp_v storage can be provided above the maximum WQ_v elevation within the wetland.

- The perimeter of all deep pool areas (4 feet or greater in depth) should be surrounded by safety and aquatic benches similar to those for stormwater ponds (see subsection 3.2.1).

- The contours of the wetland should be irregular to provide a more natural landscaping effect.

D. 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 WQ_v requirement and may be subtracted from WQ_v 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 nonerosive.

E. 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 3.2.2-2) 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 a 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.

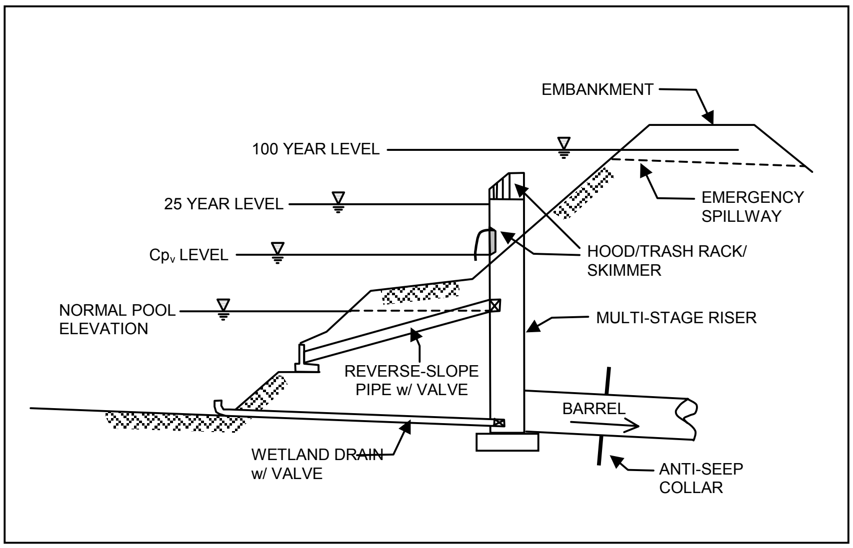


Figure 3.2.2-2 Typical Wetland Facility Outlet Structure

- The water quality outlet (if design is for an ED shallow wetland) 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 4.5 (*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 3.2.2.6 as well as Section 2.2 (*Storage Facility Design*) and Section 2.3 (*Outlet Structures*) for additional information and specifications on pond routing and outlet works.

F. EMERGENCY SPILLWAY

- An emergency spillway is to 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.

G. 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.

H. SAFETY FEATURES

- All embankments and spillways must be designed to State of Georgia guidelines for dam safety (see Appendix H).

- 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.

I. 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 and waterfowl 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, it is advisable to excavate large and deep holes around the proposed planting sites and backfill these with uncompacted topsoil.

Guidance on establishing wetland vegetation can be found in Appendix F (*Landscaping and Aesthetics Guidance)*.

J. 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-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 F); provide 48-hour ED for maximum coliform dieoff.

3.2.2.6 Design Procedures

Step 1. Compute runoff control volumes from the Unified Stormwater Sizing Criteria

> Calculate the Water Quality Volume (WQ_v), Channel Protection Volume (Cp_v), Overbank Flood Protection Volume (Q_p), and the Extreme Flood Volume (Q_f).
>
> Details on the Unified Stormwater Sizing Criteria are found in Section 1.4.

Step 2. Determine if the development site and conditions are appropriate for the use of a stormwater wetland

> Consider the Application and Site Feasibility Criteria in subsections 3.2.2.4 and 3.2.2.5-A (Location and Siting).

Step 3. Confirm local design criteria and applicability

> Consider any special site-specific design conditions/criteria from subsection 3.2.2.5-J (Additional Site-Specific Design Criteria and Issues).
>
> 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.

Step 4. 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 pond. The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 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. Allocate the WQ_v volume among marsh, micropool, and ED volumes

> Use recommended criteria from Table 3.2.2-1.

Step 6. Determine wetland location and preliminary geometry, including distribution of wetland depth zones

> This step involves initially laying out the wetland design and determining the distribution of wetland surface area among the various depth zones (high marsh, low marsh, and deepwater). Set WQ_v permanent pool elevation (and WQ_v-ED elevation for ED shallow wetland) based on volumes calculated earlier.
>
> See subsection 3.2.2.5-C (Physical Specification / Geometry) for more details.

Step 7. Compute extended detention orifice release rate(s) and size(s), and establish Cp_v elevation

> *Shallow Wetland and Pocket Wetland:* 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 streams). 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.

ED Shallow Wetland: 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 should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged one 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 Q_{p25} (25-year storm) 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 storm.

Step 9. Design embankment(s) and spillway(s)

Size emergency spillway, calculate 100-year water surface elevation, set top of embankment elevation, and analyze safe passage of the Extreme Flood Volume (Q_f).

At final design, provide safe passage for the 100-year event. Attenuation may not be required.

Step 10. Investigate potential pond/wetland hazard classification

The design and construction of stormwater management ponds and wetlands are required to follow the latest version of the State of Georgia dam safety rules (see Appendix H).

Step 11. Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

See subsection 3.2.2.5-D through H for more details.

Step 12. Prepare Vegetation and Landscaping Plan

A landscaping plan for the wetland facility and its buffer should be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation.

See subsection 3.2.2.5-I (Landscaping) and Appendix F for more details.

3.2.2.7 Inspection and Maintenance Requirements

Table 3.2.2-2 Typical Maintenance Activities for Wetlands
(Adapted from WMI, 1997 and CWP, 1998)

Activity	Schedule
• Replace wetland vegetation to maintain at least 50% surface area coverage in wetland plants after the second growing season.	One-Time Activity
• Clean and remove debris from inlet and outlet structures. • Mow side slopes.	Frequently (3 to 4 times/year)
• Monitor wetland vegetation and perform replacement planting as necessary.	Semi-annual Inspection (first 3 years)
• Examine stability of the original depth zones and microtopographical features. • Inspect for invasive vegetation, and remove where possible. • Inspect for damage to the embankment and inlet/outlet structures. Repair as necessary. • Note signs of hydrocarbon build-up, and remove appropriately. • Monitor for sediment accumulation in the facility and forebay. • Examine to ensure that inlet and outlet devices are free of debris and operational.	Annual Inspection
• Repair undercut or eroded areas.	As Needed
• Harvest wetland plants that have been “choked out” by sediment build-up.	Annually
• Removal of sediment from the forebay.	5 to 7 years or after 50% of the total forebay capacity has been lost
• Monitor sediment accumulations, and remove sediment when the pool volume has become reduced significantly, plants are “choked” with sediment, or the wetland becomes eutrophic.	10 to 20 years or after 25% of the wetland volume has been lost

Additional Maintenance Considerations and Requirements

- Maintenance requirements for constructed wetlands are particularly high while vegetation is being established. Monitoring during these first years is crucial to the future success of the wetland as a stormwater structural control. Wetland facilities should be inspected after major storms (greater than 2 inches of rainfall) during the first year of establishment to assess bank stability, erosion damage, flow channelization, and sediment accumulation within the wetland. For the first 3 years, inspections should be conducted at least twice a year.

- A sediment marker should be located in the forebay to determine when sediment removal is required.

- Accumulated sediments will gradually decrease wetland storage and performance. The effects of sediment deposition can be mitigated by the removal of the sediments.

- Sediments excavated from stormwater wetlands that do not receive runoff from designated hotspots are not considered toxic or hazardous material and can be safely disposed of by either land application or landfilling. Sediment testing may be required prior to sediment disposal when a hotspot land use is present. Sediment removed from stormwater wetlands should be disposed of according to an approved erosion and sediment control plan.

- Periodic mowing of the wetland buffer is only required along maintenance rights-of-way and the embankment. The remaining buffer can be managed as a meadow (mowing every other year) or forest.

Regular inspection and maintenance is critical to the effective operation of stormwater wetlands as designed. Maintenance responsibility for a wetland facility and its buffer 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.

3.2.2.8 Example Schematics

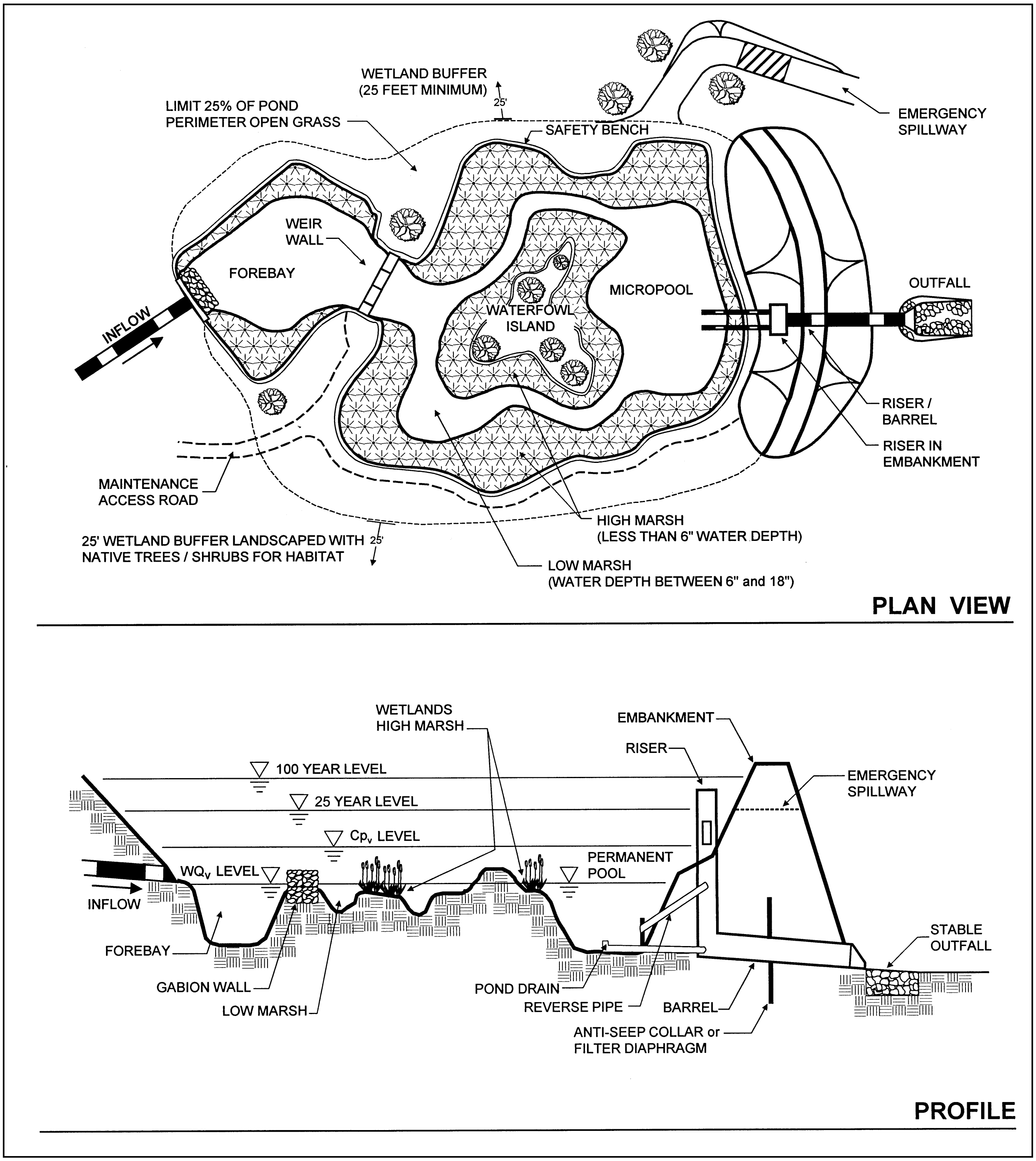


Figure 3.2.2-3 Schematic of Shallow Wetland

(Source: Center for Watershed Protection)

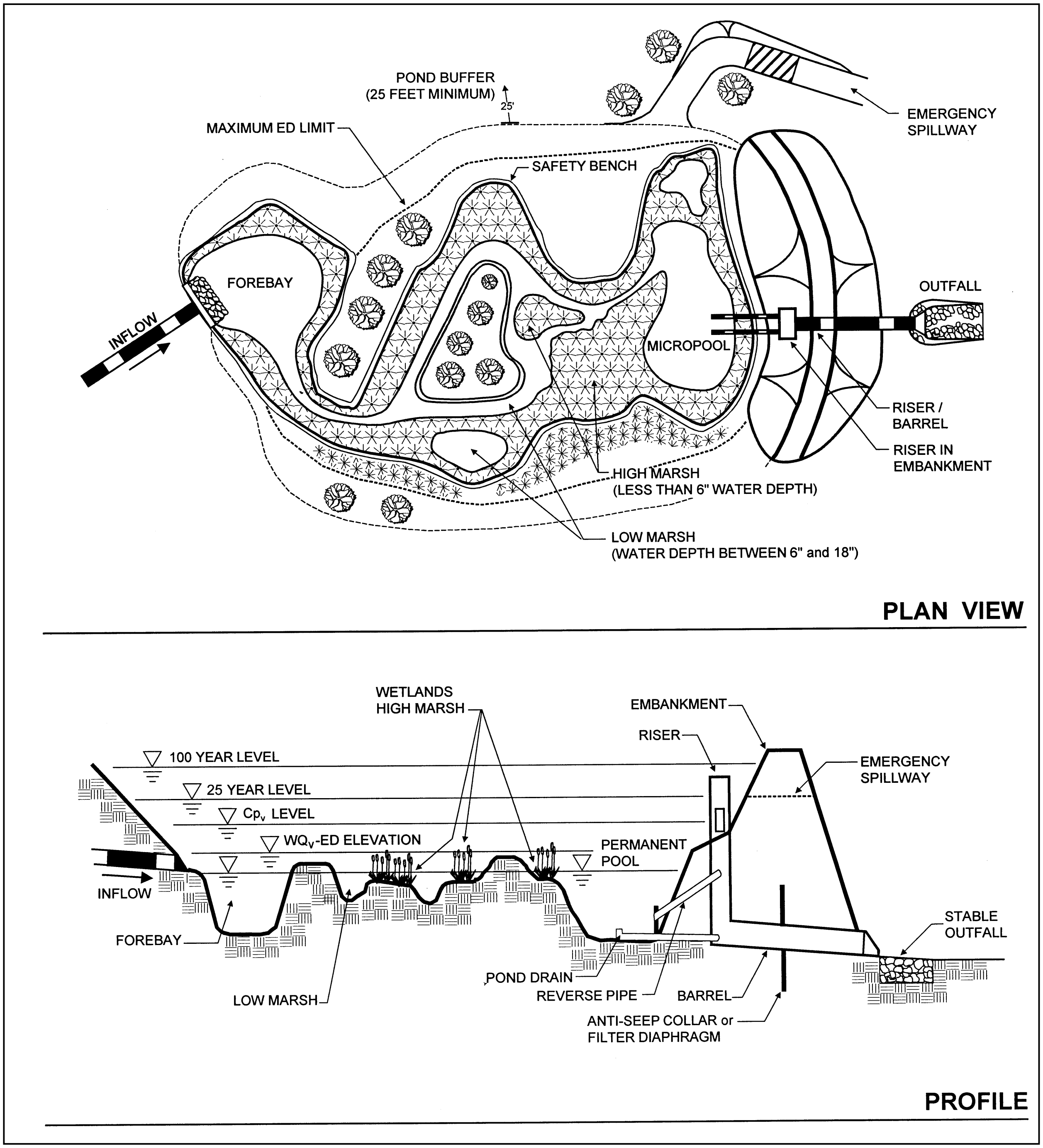


Figure 3.2.2-4 Schematic of Extended Detention Shallow Wetland

(Source: Center for Watershed Protection)

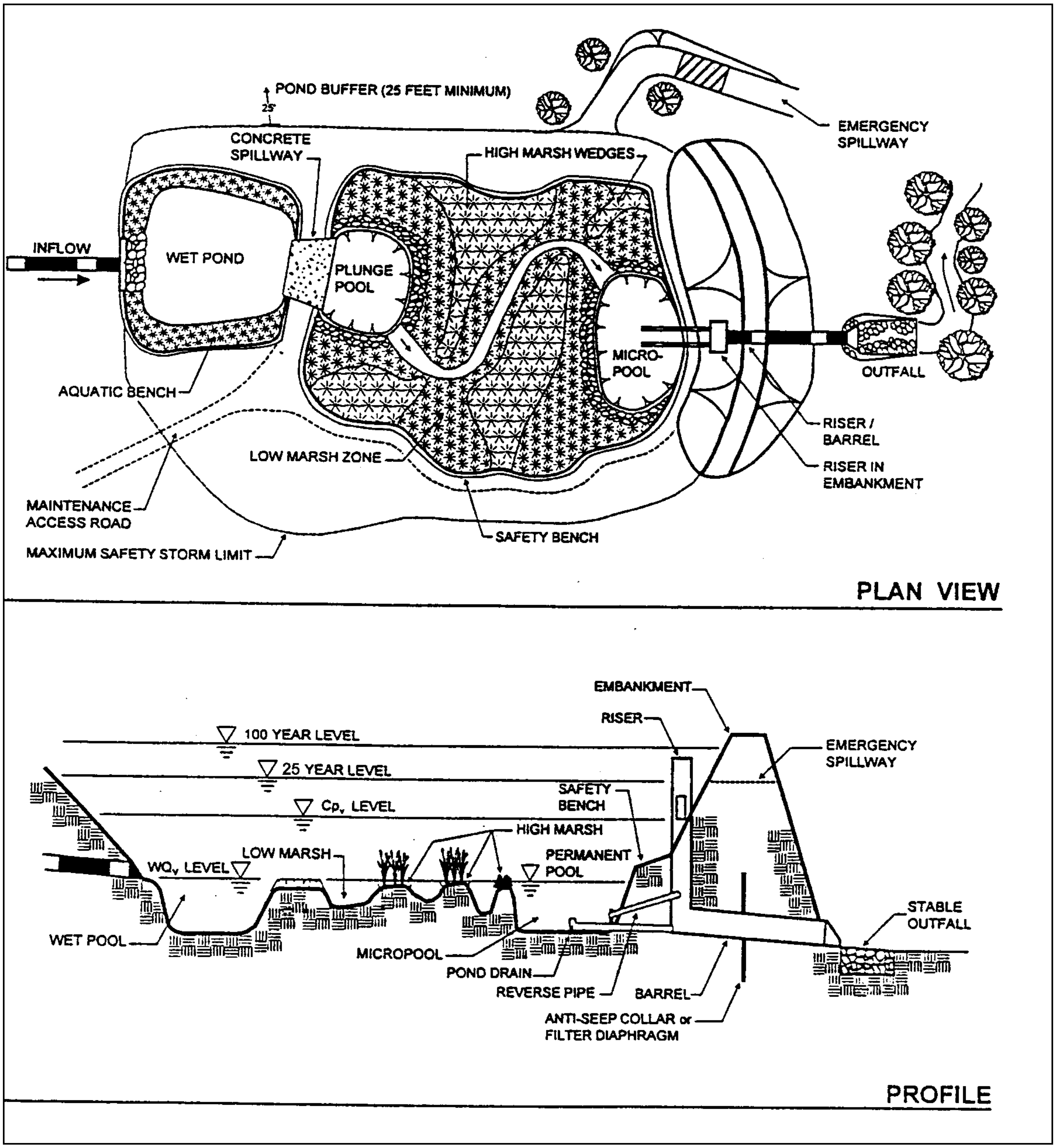


Figure 3.2.2-5 Schematic of Pond/Wetland System

(Source: Center for Watershed Protection)

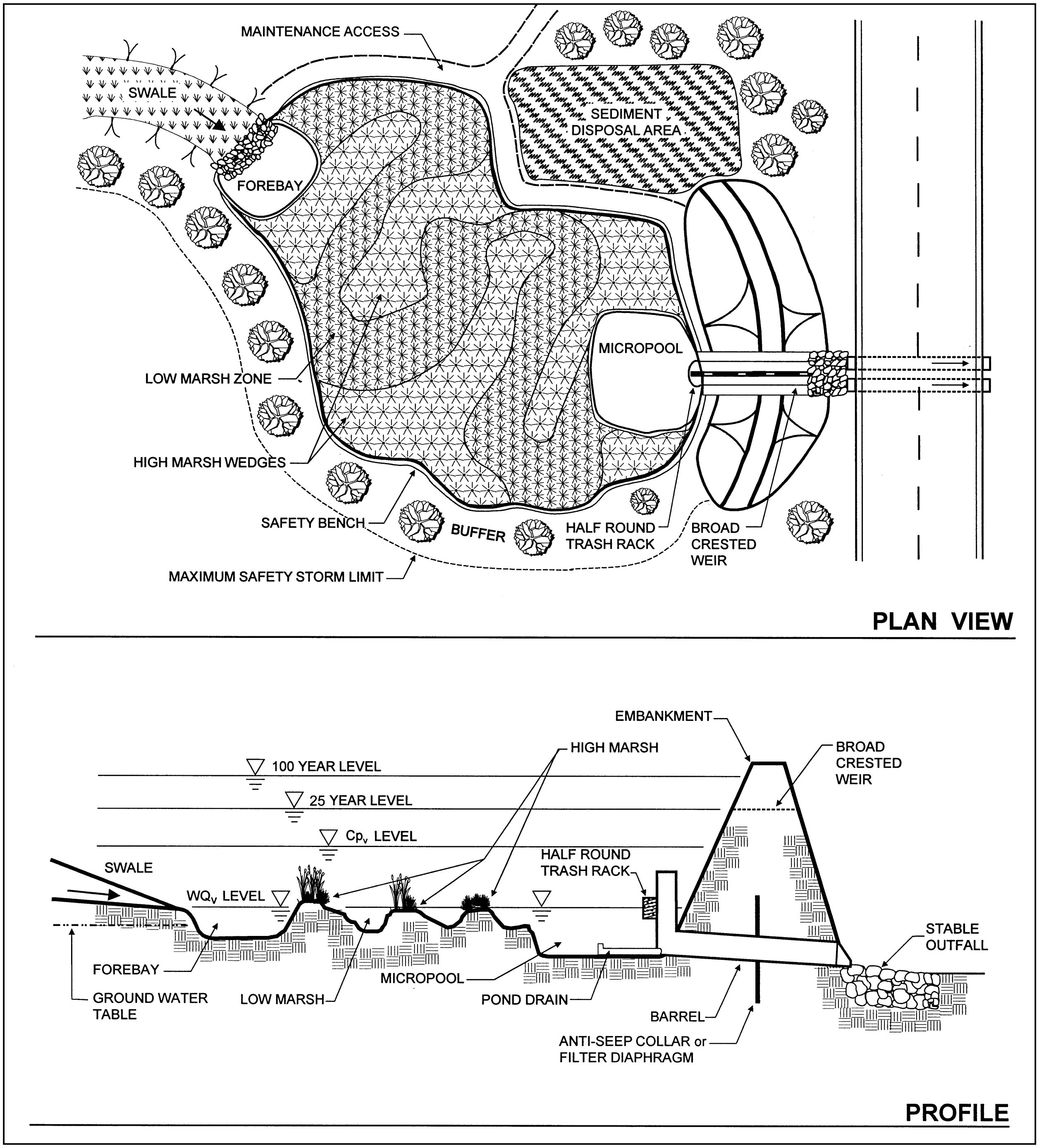


Figure 3.2.2-6 Schematic of Pocket Wetland

(Source: Center for Watershed Protection)

3.2.2.9 Design Forms

Design Procedure Form: Stormwater Wetlands

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQ_v volume requirements
Compute Runoff Coefficient, R_v — R_v = ________
Compute WQv — WQ_v = ________ acre-ft

1b. Compute Cp_v — Cp_v = ________ acre-ft
Compute average release rate — release rate = ________ cfs
Compute Q_{p-25} — Q_{p-25} = ________ acre-ft
Add 15% to the required Q_{p-25} volume — Q_{p-25} * 15% = ________ acre-ft
Compute (as necessary) Q_f — Q_f = ________ acre-ft

STORMWATER WETLAND DESIGN

2. Is the use of a stormwater wetland appropriate? — ***See subsections 3.2.2.4 and 3.2.2.5 - A***

3. Confirm local design criteria and applicability

4. Pretreatment volume — Vol_{pre} = ________ acre-ft
Vol_{pre} = I (0.1")(1'/12")

5. Allocation of Pool, Marsh, and ED Volumes

Shallow Wetland: Vol_{pool} = 0.2 (WQv) — Vol_{pool} = ________ acre-ft
Vol_{marsh} = 0.7 (WQv) — Vol_{marsh} = ________ acre-ft

Shallow ED Wetland: Vol_{pool} = 0.1 (WQv) — Vol_{pool} = ________ acre-ft
Vol_{marsh} = 0.3 (WQv) — Vol_{marsh} = ________ acre-ft
Vol_{ED} = 0.5 (WQv) — Vol_{ED} = ________ acre-ft

Pocket Wetland: Vol_{pool} = 0.1 (WQv) — Vol_{pool} = ________ acre-ft
Vol_{marsh} = 0.8 (WQv) — Vol_{marsh} = ________ acre-ft

6. Allocation of Surface Area

Pool/Deepwater Wetland Zone (1.5 - 6 feet deep) — $Area_{water}$ = ________ acres, % = ________
Low Marsh Wetland Zone (6-18 inches deep) — $Area_{low}$ = ________ acres, % = ________
High Marsh Wetland Zone (0-6 inches deep) — $Area_{high}$ = ________ acres, % = ________
Semi-Wet Wetland Zone (above pool depth) — $Area_{semi}$ = ________ acres, % = ________
100.00%

Conduct grading and determine storage available for marsh zones (and ED if applicable), and compute orifice size — *Prepare an elevation-storage table and curve using the average area method for computing volumes.*

Elevation MSL	Area ft^2	Average Area ft^2	Depth ft	Volume ft^3	Cummulative Volume ft^3	Cummulative Volume ac-ft	Volume above Permanent Pool ac-ft

7. WQ_v Orifice Computations
   Average ED release rate (if applicable) — release rate = ________ cfs
   Average head, h = (ED elev. - Permanent pool elev.) / 2 — h = ________ ft
   Area of orifice from orifice equation
   $Q = CA(2gh)^{0.5}$ — A = ________ ft^2
   diameter = ________ in
   Discharge equation Q = (h)^0.5 — factor = ________ (h)^0.5

   Compute release rate for Cp_v-ED control and establish Cp_v elevation — WSEL = ________ ft-NGVD
   Release rate = — release rate = ________ cfs
   Average head, h = (Cp_v elev. - Permanent pool elev.) / 2 — h = ________ ft
   Area of orifice from orifice equation — A = ________ ft^2
   $Q = CA(2gh)^{0.5}$ — diameter = ________ in
   Discharge equation Q = (h)^0.5 — factor = ________ (h)^0.5

8. Calculate Q_{p-25} release rate and WSEL — *Set up a stage-storage-discharge relationship*

Elevation	Storage	Low Flow	Riser			Barrel		Emergency	Total
		WQv-ED	Cpv-ED	High Storage		Inlet	Pipe	Spillway	Storage
				Orif.	Weir				
MSL	ac-ft	H(ft) Q(cfs)	H(ft) Q(cfs)	H Q	H Q	H(ft) Q(cfs)	H(ft) Q(cfs)	H(ft) Q(cfs)	Q(cfs)

   Q_{p-25} = pre-dev. peak discharge - (WQ_v-ED release + Cp_v-ED release) — Q_{p-25} = ________ cfs

   Maximum head = — H = ________ ft
   Use weir equation for slot length ($Q = CLH^{3/2}$) — L = ________ ft

   Check inlet condition — ***Use culvert charts***
   Check outlet condition — ***(Section 4.3)***

9. Size emergency spillway, calculate 100-year WSEL and set top of embankment elevation
   $WSEL_{25}$ = ________ ft
   $WSEL_{100}$ = ________ ft
   Q_{ES} = ________ cfs
   Q_{PS} = ________ cfs

10. Investigate potential pond hazard classification — ***See Appendix H***

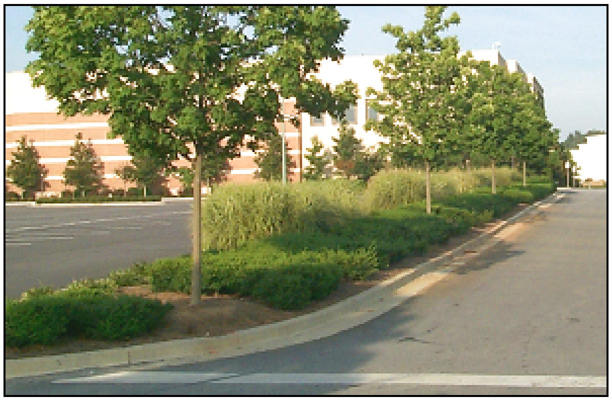
11. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features. — ***See subsection 3.2.2.5 - D through H***

12. Attach landscaping plan (including wetland vegetation) — ***See Appendix F***

Notes: ____

3.2.3 Bioretention Areas

General Application
Structural Stormwater Control



Description: Shallow stormwater basin or landscaped area that utilizes engineered soils and vegetation to capture and treat runoff.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Maximum contributing drainage area of 5 acres
- Often located in "landscaping islands"
- Treatment area consists of grass filter, sand bed, ponding area, organic/mulch layer, planting soil, and vegetation
- Typically requires 5 feet of head

ADVANTAGES / BENEFITS:

- Applicable to small drainage areas
- Good for highly impervious areas, particularly parking lots
- Good retrofit capability
- Relatively low maintenance requirements
- Can be planned as an aesthetic feature

DISADVANTAGES / LIMITATIONS:

- Requires extensive landscaping
- Not recommended for areas with steep slopes

MAINTENANCE REQUIREMENTS:

- Inspect and repair/replace treatment area components

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
60/50%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
M	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORMWATER MANAGEMENT SUITABILITY

- ☑ **Water Quality**
- ✪ **Channel Protection**
- ☐ **Overbank Flood Protection**
- ☐ **Extreme Flood Protection**

Accepts Hotspot Runoff: *Yes (requires impermeable liner)*

✪ in certain situations

IMPLEMENTATION CONSIDERATIONS

M	**Land Requirement**
M	**Capital Cost**
L	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *5 acres max.*

Soils: *Planting soils must meet specified criteria; No restrictions on surrounding soils*

Other Considerations:

- *Use of native plants is recommended*

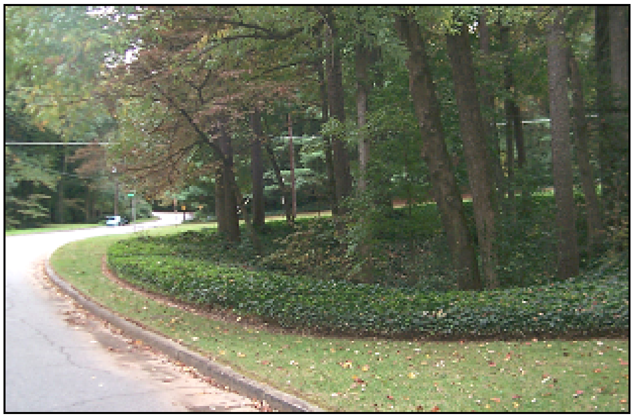
L=Low **M**=Moderate **H**=High

3.2.3.1 General Description

Bioretention areas (also referred to as *bioretention filters* or *rain gardens*) are structural stormwater controls that capture and temporarily store the water quality volume (WQ_v) using soils and vegetation in shallow basins or landscaped areas to remove pollutants from stormwater runoff.

Bioretention areas are engineered facilities in which runoff is conveyed as sheet flow to the "treatment area," which consists of a grass buffer strip, ponding area, organic or mulch layer, planting soil, and vegetation. An optional sand bed can also be included in the design to provide aeration and drainage of the planting soil. The filtered runoff is typically collected and returned to the conveyance system, though it can also be exfiltrated into the surrounding soil in areas with porous soils.

There are numerous design applications, both on- and off-line, for bioretention areas. These include use on single-family residential lots (*rain gardens*), as off-line facilities adjacent to parking lots, along highway and road drainage swales, within larger landscaped pervious areas, and as landscaped islands in impervious or high-density environments. Figures 3.2.3-1 and 3.2.3-2 illustrate a number of examples of bioretention facilities in both photographs and drawings.



Single-Family Residential "Rain Garden"

Landscaped Island

Newly Constructed Bioretention Area

Newly Planted Bioretention Area After Storm

Figure 3.2.3-1 Bioretention Area Examples

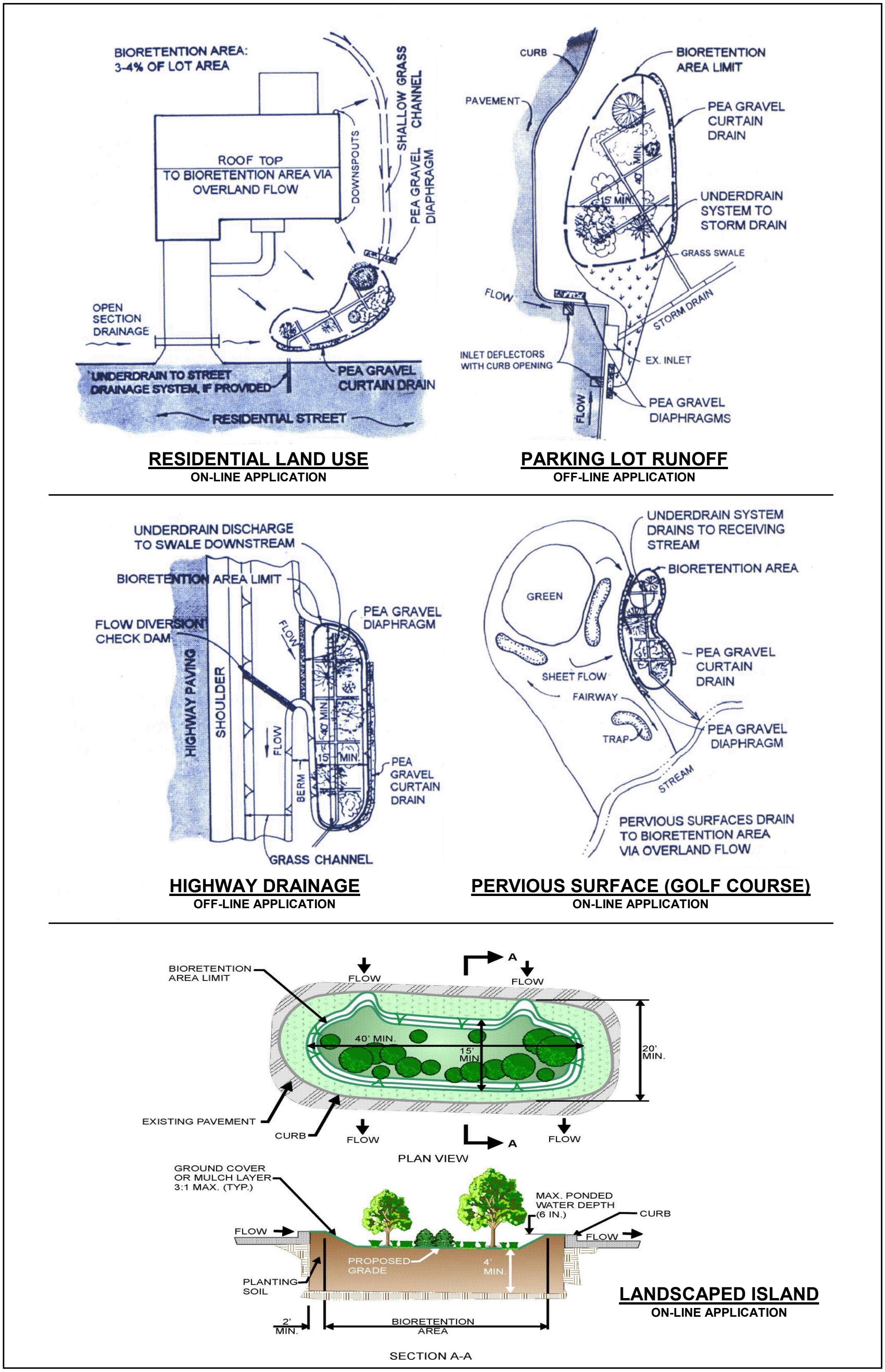


Figure 3.2.3-2 Bioretention Area Applications
(Source: Claytor and Schueler, 1996)

3.2.3.2 Stormwater Management Suitability

Bioretention areas are designed primarily for stormwater quality, i.e. the removal of stormwater pollutants. Bioretention can provide limited runoff quantity control, particularly for smaller storm events. 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 structural control to provide channel protection as well as overbank flood protection. It is important to ensure that a bioretention area safely bypasses higher flows.

Water Quality

Bioretention is an excellent stormwater treatment practice due to the variety of pollutant removal mechanisms. Each of the components of the bioretention area is designed to perform a specific function (see Figure 3.2.3-3). The *grass filter strip (or grass channel)* 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 facility 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 the surrounding soils. Finally, a *sand bed* provides for positive drainage and aerobic conditions in the planting soil and provides a final polishing treatment media.

Section 3.2.3.3 provides median pollutant removal efficiencies that can be used for planning and design purposes.

Channel Protection

For smaller sites, a bioretention area may be designed to capture the entire channel protection volume Cp_v in either an off- or on-line configuration. Given that a bioretention facility is typically designed to completely drain over 48 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 bioretention facility, another structural control must be used to provide Cp_v extended detention.

Overbank Flood Protection

Another structural control must be used in conjunction with a bioretention area 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 removed and treated in the bioretention area may be taken in the overbank flood protection and extreme flood protection calculations (see Section 3.1).

3.2.3.3 Pollutant Removal Capabilities

Bioretention areas 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 bioretention areas 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 – 60%**
- **Total Nitrogen – 50%**
- **Fecal Coliform – insufficient data**
- **Heavy Metals – 80%**

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

3.2.3.4 Application and Site Feasibility Criteria

Bioretention areas are suitable for many types of development, from single-family residential to high-density commercial projects. Bioretention is also well suited for small lots, including those of 1 acre or less. Because of its ability to be incorporated in landscaped areas, the use of bioretention is extremely flexible. Bioretention areas are an ideal structural 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 can also be used to retrofit existing development with stormwater quality treatment capacity.

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 maximum; 0.5 to 2 acres are preferred.
- Space Required – Approximately 5% of the tributary impervious area is required; minimum 200 ft^2 area for small sites (10 feet x 20 feet)
- Site Slope – No more than 6% slope
- Minimum Head – Elevation difference needed at a site from the inflow to the outflow: 5 feet
- Minimum Depth to Water Table – A separation distance of 2 feet recommended between the bottom of the bioretention facility and the elevation of the seasonally high water table.
- Soils – No restrictions; engineered media required

Other Constraints / Considerations

- Aquifer Protection – Do not allow exfiltration of filtered hotspot runoff into groundwater

3.2.3.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of a bioretention facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A. LOCATION AND SITING

- Bioretention areas should have a maximum contributing drainage area of 5 acres or less; 0.5 to 2 acres are preferred. Multiple bioretention areas can be used for larger areas.

- Bioretention areas can either be used to capture sheet flow from a drainage area or function as an off-line device. On-line designs should be limited to a maximum drainage area of 0.5 acres.

- When used in an off-line configuration, the water quality volume (WQ_v) is diverted to the bioretention area through the use of a flow splitter. Stormwater flows greater than the WQ_v are diverted to other controls or downstream (see Section 3.1 for more discussion of off-line systems and design guidance for diversion structures and flow splitters).

- Bioretention 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 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.

B. GENERAL DESIGN

- A well-designed bioretention area consists of:

  (1) Grass filter strip (or grass channel) between the contributing drainage area and the ponding area,
  (2) Ponding area containing vegetation with a planting soil bed,
  (3) Organic/mulch layer,
  (4) Gravel and perforated pipe underdrain system to collect runoff that has filtered through the soil layers (bioretention areas can optionally be designed to infiltrate into the soil – see description of infiltration trenches for infiltration criteria).

- A bioretention area design will also include some of the following:

  - Optional **sand filter layer** to spread flow, filter runoff, and aid in aeration and drainage of the planting soil.
  - **Stone diaphragm** at the beginning of the grass filter strip to reduce runoff velocities and spread flow into the grass filter.
  - **Inflow diversion** or an **overflow structure** consisting of one of five main methods:
    - Use a flow diversion structure
    - For curbed pavements use an inlet deflector (see Figure 3.2.3-6).
    - Use a slotted curb and design the parking lot grades to divert the WQ_v into the facility. Bypass additional runoff to a downstream catch basin inlet. Requires temporary ponding in the parking lot (see Figure 3.2.3-5).
    - Figure 3.2.3-2c illustrates the use of a short deflector weir (maximum height 6 inches) designed to divert the maximum water quality peak flow into the bioretention area.
    - An in-system overflow consisting of an overflow catch basin inlet and/or a pea gravel curtain drain overflow.

See Figure 3.2.3-3 for an overview of the various components of a bioretention area. Figure 3.2.3-4 provides a plan view and profile schematic of an on-line bioretention area. An example of an off-line facility is shown in Figure 3.2.3-5.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

- Recommended minimum dimensions of a bioretention area are 10 feet wide by 20 feet long. All designs except small residential applications should maintain a length to width ratio of at least 2:1.

- The planting soil filter bed is sized using a Darcy's Law equation with a filter bed drain time of 48 hours and a coefficient of permeability (k) of 0.5 ft/day.

- The maximum recommended ponding depth of the bioretention areas is 6 inches.

- The planting soil bed must be at least 4 feet in depth. Planting soils should be sandy loam, loamy sand, or loam texture with a clay content ranging from 10 to 25%. The soil must have an infiltration rate of at least 0.5 inches per hour and a pH between 5.5 and 6.5. In addition, the planting soil should have a 1.5 to 3% organic content and a maximum 500 ppm concentration of soluble salts.

- For on-line configurations, a grass filter strip with a pea gravel diaphragm is typically utilized (see Figure 3.2.3-3) as the pretreatment measure. The required length of the filter strip depends on the drainage area, imperviousness, and the filter strip slope. Design guidance on filter strips for pretreatment can be found in subsection 3.3.1 (*Filter Strip)*.

- For off-line applications, a grass channel with a pea gravel diaphragm flow spreader is used for pretreatment. The length of the grass channel depends on the drainage area, land use, and channel slope. The minimum grassed channel length should be 20 feet. Design guidance on grass channels for pretreatment can be found in subection 3.3.2 (*Grass Channel)*.

- The mulch layer should consist of 2 to 4 inches of commercially available fine shredded hardwood mulch or shredded hardwood chips.

- The sand bed should be 12 to 18 inches thick. Sand should be clean and have less than 15% silt or clay content.

- Pea gravel for the diaphragm and curtain, where used, should be ASTM D 448 size No. 6 ($^1/_8$" to ¼").

- The underdrain collection system is equipped with a 6-inch perforated PVC pipe (AASHTO M 252) in an 8-inch gravel layer. The pipe should have 3/8-inch perforations, spaced at 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. A permeable filter fabric is placed between the gravel layer and the planting soil bed.

D. PRETREATMENT / INLETS

- Adequate pretreatment and inlet protection for bioretention systems is provided when all of the following are provided: (a) grass filter strip below a level spreader, or grass channel, (b) pea gravel diaphragm and (c) an organic or mulch layer.

E. 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.

F. EMERGENCY SPILLWAY

- An overflow structure and nonerosive overflow channel must be provided to safely pass flows from the bioretention area that exceed the storage capacity to a stabilized downstream area or watercourse. If the system is located off-line, the overflow should be set above the shallow ponding limit.

- The high flow overflow system <u>within</u> the structure consists of a yard drain catchbasin (Figure 3.2.3-3), though any number of conventional systems could be used. The throat of the catch basin inlet is normally placed 6 inches above the mulch layer at the elevation of the shallow ponding area.

G. MAINTENANCE ACCESS

- Adequate access must be provided for all bioretention facilities for inspection, maintenance, and landscaping upkeep, including appropriate equipment and vehicles.

H. SAFETY FEATURES

- Bioretention areas generally do not require any special safety features. Fencing of bioretention facilities is not generally desirable.

I. LANDSCAPING

- Landscaping is critical to the performance and function of bioretention areas.

- A dense and vigorous vegetative cover should be established over the contributing pervious drainage areas before runoff can be accepted into the facility.

- The bioretention area should be vegetated to resemble a terrestrial forest ecosystem, with a mature tree canopy, subcanopy of understory trees, scrub layer, and herbaceous ground cover. Three species each of both trees and scrubs are recommended to be planted.

- The tree-to-shrub ratio should be 2:1 to 3:1. On average, the trees should be spaced 8 feet apart. Plants should be placed at regular intervals to replicate a natural forest. Woody vegetation should not be specified at inflow locations.

- After the trees and shrubs are established, the ground cover and mulch should be established.

- Choose plants based on factors such as whether native or not, resistance to drought and inundation, cost aesthetics, maintenance, etc. Planting recommendations for bioretention facilities 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 bioretention in Georgia, and their planting and establishment, can be found in Appendix F, *Landscaping and Aesthetics Guidance*.

J. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

Physiographic Factors - Local terrain design constraints

- Low Relief – Use of bioretention areas may be limited by low head
- High Relief – Ponding area surface must be relatively level
- Karst – Use poly-liner or impermeable membrane to seal bottom

Soils

- No restrictions

Special Downstream Watershed Considerations

- Trout Stream – Evaluate for stream warming
- Aquifer Protection – No restrictions, if designed with no exfiltration (i.e. outflow to groundwater)

3.2.3.6 Design Procedures

Step 1. Compute runoff control volumes from the Unified Stormwater Sizing Criteria

Calculate the Water Quality Volume (WQ_v), Channel Protection Volume (Cp_v), Overbank Flood Protection Volume (Q_p), and the Extreme Flood Volume (Q_f).

Details on the Unified Stormwater Sizing Criteria are found in Section 1.4.

Step 2. Determine if the development site and conditions are appropriate for the use of a bioretention area

Consider the Application and Site Feasibility Criteria in subsections 3.2.3.4 and 3.2.3.5-A (Location and Siting).

Step 3. Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 3.2.3.5-J (Additional Site-Specific Design Criteria and Issues).

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.

Step 4. Compute WQ_v peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion structures (see subsection 2.1.7).
.
(a) Using WQ_v (or total volume to be captured), 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 bioretention area.

Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 6. Determine size of bioretention ponding/filter area

The required planting soil filter bed area is computed using the following equation (based on Darcy's Law):

$$\mathbf{A_f = (WQv)\,(d_f) \,/\, [\,(k)\,(h_f + d_f)\,(t_f)]}$$

where:
A_f = surface area of ponding area (ft^2)
WQ_v = water quality volume (or total volume to be captured)
d_f = filter bed depth
(4 feet minimum)
k = coefficient of permeability of filter media (ft/day)
(use 0.5 ft/day for silt-loam)
h_f = average height of water above filter bed (ft)
(typically 3 inches, which is half of the 6-inch ponding depth)
t_f = design filter bed drain time (days)
(2.0 days or 48 hours is recommended maximum)

Step 7. Set design elevations and dimensions of facility

See subsection 3.2.3.5-C (Physical Specifications/Geometry).

Step 8. Design conveyances to facility (off-line systems)

See the example figures to determine the type of conveyances needed for the site.

Step 9. Design pretreatment

Pretreat with a grass filter strip (on-line configuration) or grass channel (off-line), and stone diaphragm.

Step 10. Size underdrain system

See subsection 3.2.3.5-C (Physical Specifications/Geometry)

Step 11. Design emergency overflow

An overflow must be provided to bypass and/or convey larger flows to the downstream drainage system or stabilized watercourse. Nonerosive velocities need to be ensured at the outlet point.

Step 12. 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 3.2.3.5-I (Landscaping) and Appendix F for more details.

See Appendix D-2 for a Bioretention Area Design Example

3.2.3.7 Inspection and Maintenance Requirements

Table 3.2.3-1 Typical Maintenance Activities for Bioretention Areas
(Source: EPA, 1999)

Activity	Schedule
• Pruning and weeding to maintain appearance. • Mulch replacement when erosion is evident. • Remove trash and debris.	As needed
• Inspect inflow points for clogging (off-line systems). Remove any sediment. • Inspect filter strip/grass channel for erosion or gullying. Re-seed or sod as necessary. • Trees and shrubs should be inspected to evaluate their health and remove any dead or severely diseased vegetation.	Semi-annually
• The planting soils should be tested for pH to establish acidic levels. If the pH is below 5.2, limestone should be applied. If the pH is above 7.0 to 8.0, then iron sulfate plus sulfur can be added to reduce the pH.	Annually
• Replace mulch over the entire area. • Replace pea gravel diaphragm if warranted.	2 to 3 years

Additional Maintenance Considerations and Requirements

- The surface of the ponding area may become clogged with fine sediment over time. Core aeration or cultivating of unvegetated areas may be required to ensure adequate filtration.

Regular inspection and maintenance is critical to the effective operation of bioretention facilities as designed. Maintenance responsibility for a bioretention area 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.

3.2.3.8 Example Schematics

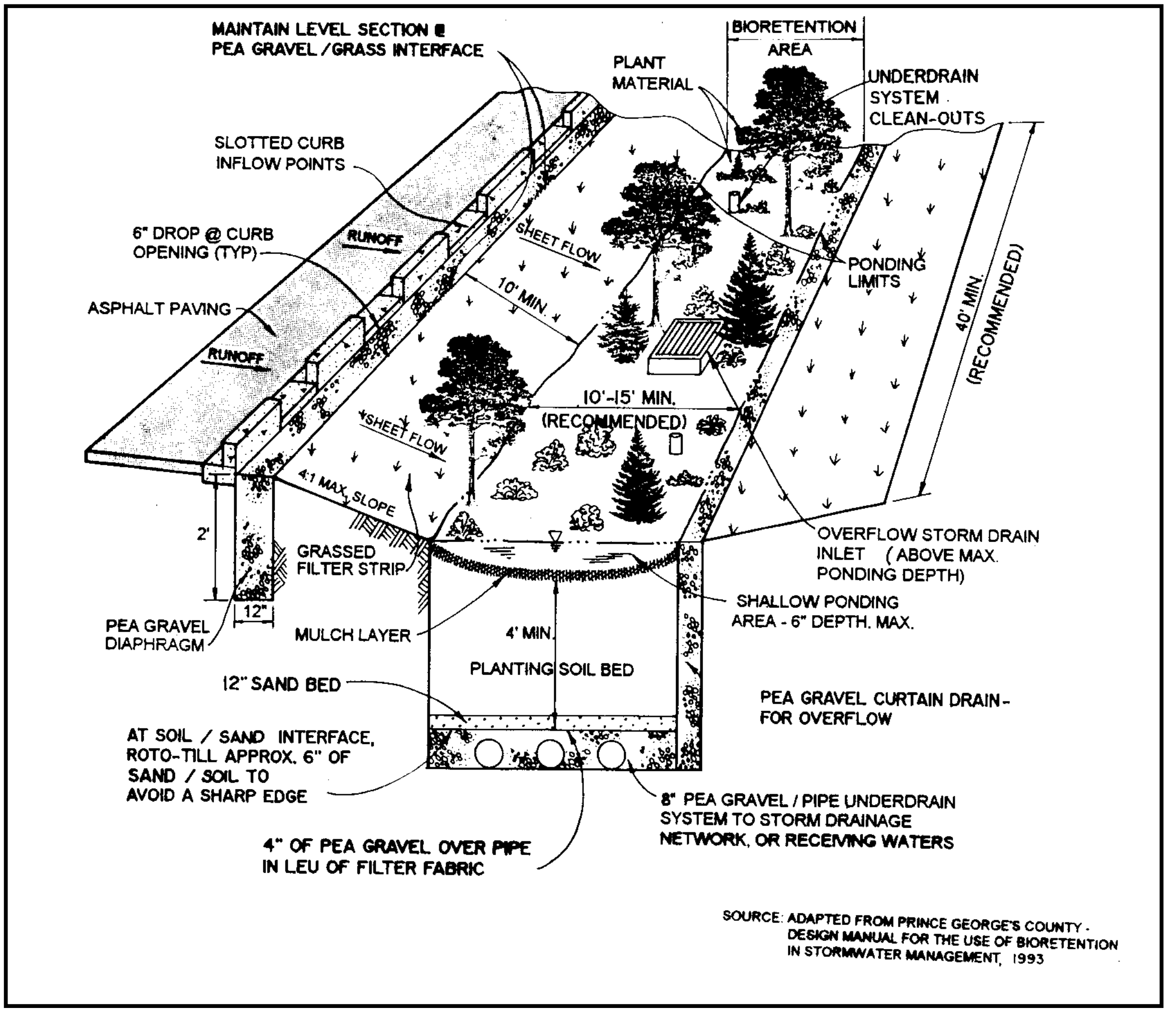


Figure 3.2.3-3 Schematic of a Typical Bioretention Area

(Source: Claytor and Schueler, 1996)

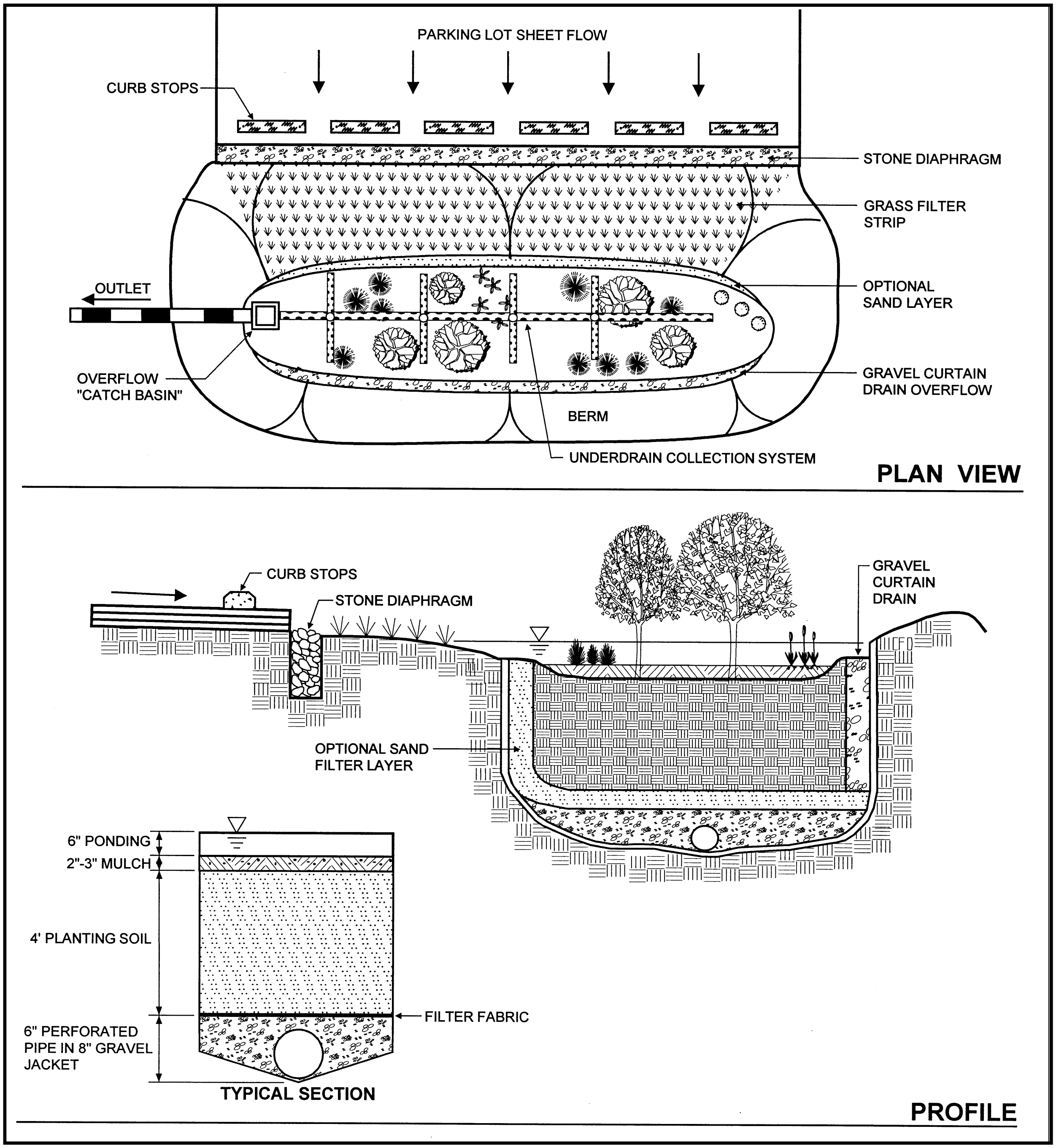


Figure 3.2.3-4 Schematic of a Typical On-line Bioretention Area

(Source: Claytor and Schueler, 1996)

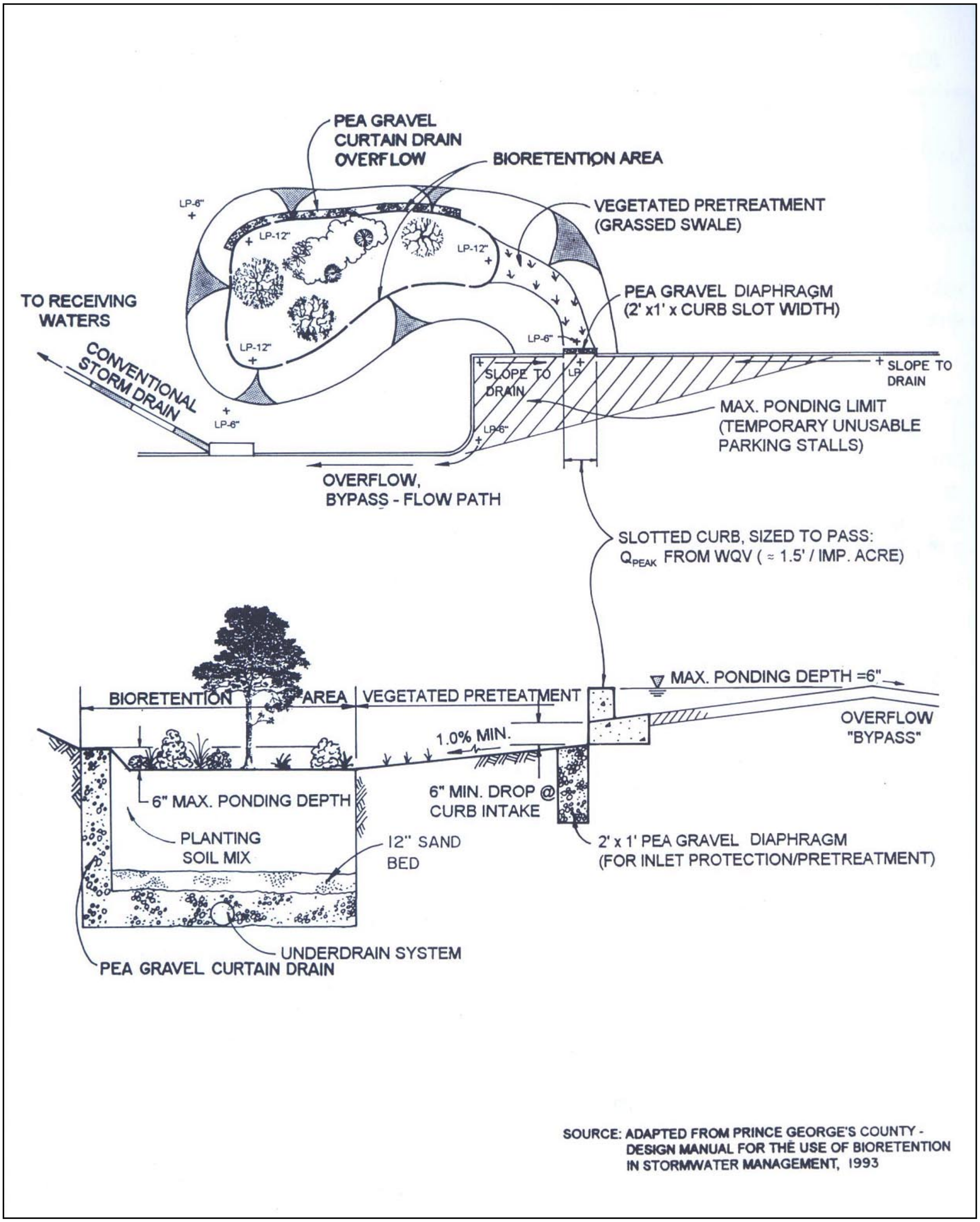


Figure 3.2.3-5 Schematic of a Typical Off-line Bioretention Area

(Source: Claytor and Schueler, 1996)

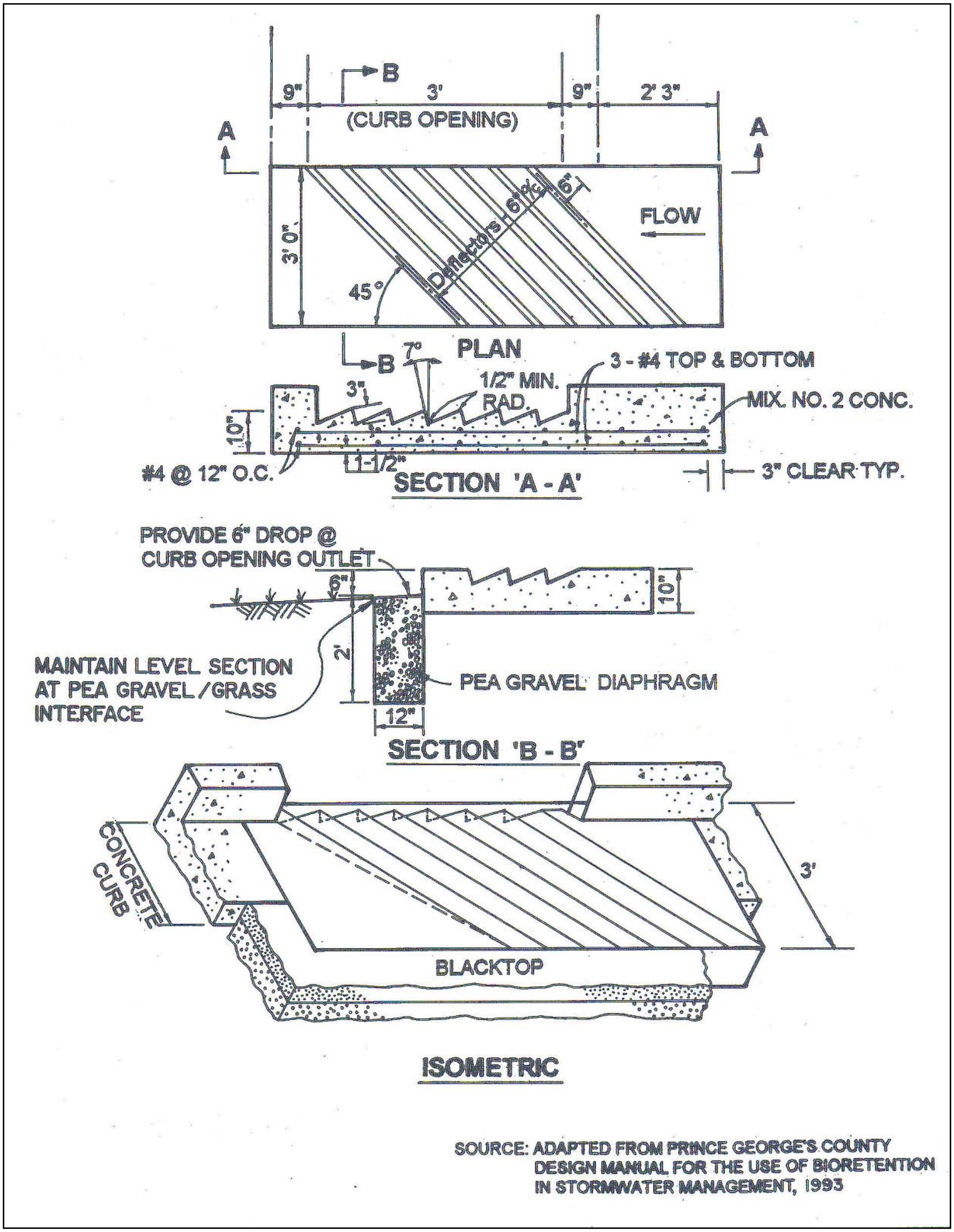


Figure 3.2.3-6 Schematic of a Typical Inlet Deflector

(Source: Claytor and Schueler, 1996)

3.2.3.9 Design Forms

Design Procedure Form: Bioretention Areas

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQ_v volume requirements	
Compute Runoff Coefficient, R_v	R_v = ________
Compute WQv	WQ_v = ________ acre-ft
1b. Compute Cp_v	Cp_v = ________ acre-ft
Compute average release rate	release rate = ________ cfs
Compute Q_{p-25}	Q_{p-25} = ________ acre-ft
Compute (as necessary) Q_f	Q_f = ________ acre-ft
BIORETENTION AREA DESIGN	
2. Is the use of a stormwater pond appropriate?	***See subsections 3.2.3.4 and 3.2.3.5 - A***
3. Confirm local design criteria and applicability	
4. Determine size of bioretention filter area	A_f = ________ ft^2
5. Set design elevations and dimensions	Length = ________ ft Width = ________ ft ________ elevation top of facility ________ other elev: ________ ________ other elev: ________ ________ other elev: ________
6. Conveyance to bioretention facililty	_____Online or _____ Offline?
7. Pretreatment	Type: ________________
8. Size underdrain area Based on guidance: Approx. 10% A_f	Length = ________ ft
9. Overdrain design	Type: ________________ Size: ________
10. Emergency storm weir design Overflow weir - Weir equation	Length = ________ ft
11. Choose plants for planting area	*Select native plants based on resistance to drought and inundation, cost, aesthetics, maintenance, etc.* ***See Appendix F***

Notes: ____

____

3.2.4 Sand Filters

General Application
Structural Stormwater Control

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.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Typically requires 2 to 6 feet of head
- 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/chamber
- Replace sand filter media as needed

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
50/25%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
50%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
40%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORMWATER MANAGEMENT SUITABILITY

- ✓ **Water Quality**
- ✪ **Channel Protection**
- ☐ **Overbank Flood Protection**
- ☐ **Extreme Flood Protection**

Accepts Hotspot Runoff: *Yes (requires impermeable liner)*

✪ in certain situations

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

3.2.4.1 General Description

Sand filters (also referred to as *filtration basins*) are structural stormwater controls 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 install.

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 typically 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 high density areas and is thus considered a limited application structural control. See subsection 3.3.4 for more details.

Surface Sand Filter

Perimeter Sand Filter

Figure 3.2.4-1 Sand Filter Examples

3.2.4.2 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 structural control 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.

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. Section 3.2.4.3 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 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 structural control must be used in conjunction with a sand filter system to reduce the post-development peak flow of the 25-year 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.

Credit for the volume of runoff removed and treated by the sand filter may be taken in the overbank flood protection and extreme flood protection calculations (see Section 3.1).

3.2.4.3 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 (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

3.2.4.4 Application and Site Feasibility Criteria

Sand filter systems are well suited for highly impervious areas where land available for structural controls 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.

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

3.2.4.5 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.*

A. LOCATION AND SITING

- 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.

B. 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, runoff is collected by a perforated pipe and gravel underdrain system. Figure 3.2.4-6 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 3.2.4-7 provides plan view and profile schematics of a perimeter sand filter.

C. 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 3.2.4-2 illustrates the distribution of the treatment volume (0.75 WQ_v) 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.

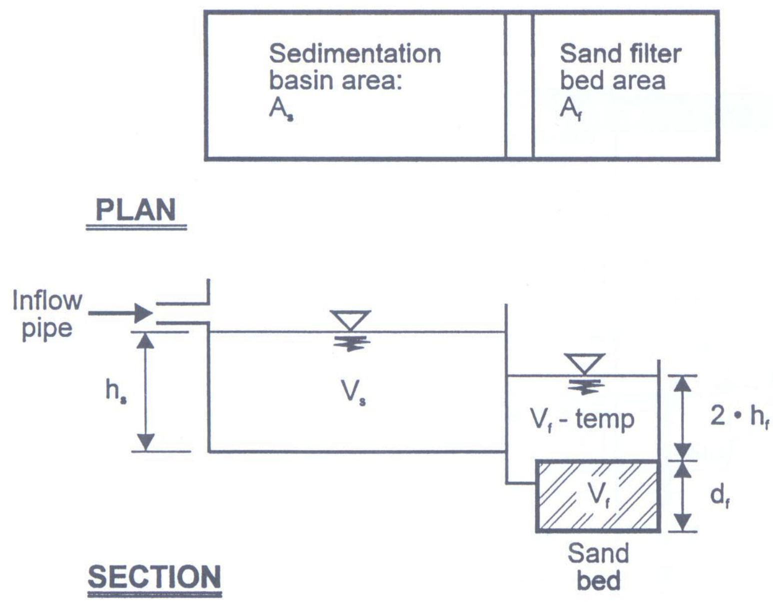


Figure 3.2.4-2 Surface Sand Filter Volumes
Source: Claytor and Schueler, 1996

- The filter media consists of an 18-inch layer of clean washed medium sand (meeting ASTM C-33 concrete sand or GADOT 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 3.2.4-4 illustrates a typical media cross section.

- The filter bed is equipped with a 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% (GADOT 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 WQ_v prior to filtration. Figure 3.2.4-3 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 GADOT Fine Aggregate Size No. 10) on top of the underdrain system. Figure 3.2.4-4 illustrates a typical media cross section.

- The perimeter sand filter is equipped with a 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% (GADOT No.3 Stone). Aggregate contaminated with soil shall not be used.

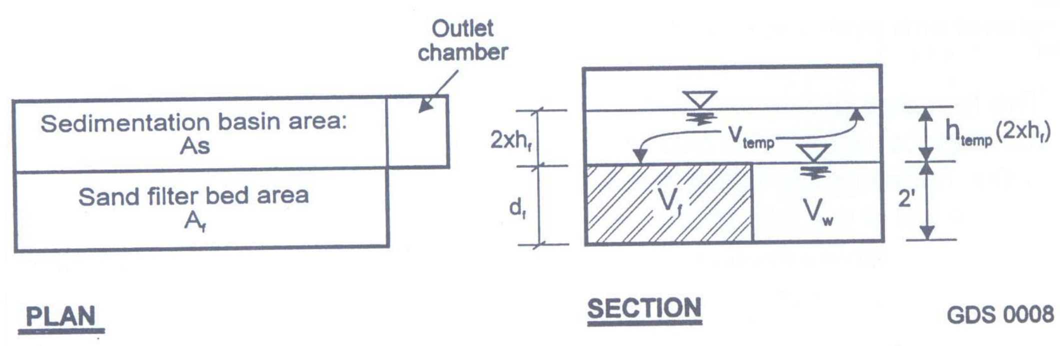


Figure 3.2.4-3 Perimeter Sand Filter Volumes
(Source: Claytor and Schueler, 1996)

D. 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 3.2.4-5 shows a typical inlet pipe from the sedimentation basin to the filter media basin for the surface sand filter.

E. 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).

F. 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.

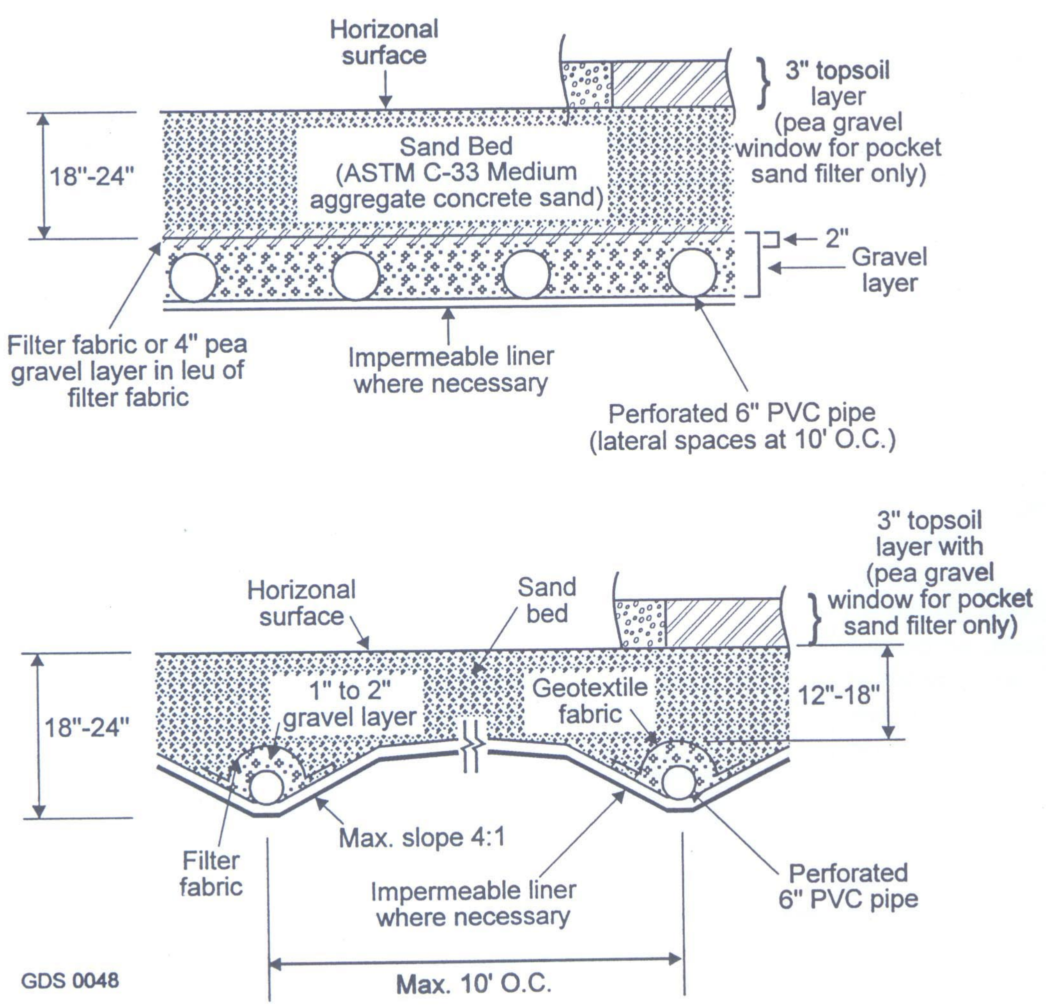


Figure 3.2.4-4 Typical Sand Filter Media Cross Sections
(Source: Claytor and Schueler, 1996)

G. 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.

H. SAFETY FEATURES

- Surface sand filter facilities can be fenced to prevent access. Inlet and access grates to perimeter sand filters may be locked.

I. LANDSCAPING

- Surface 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.

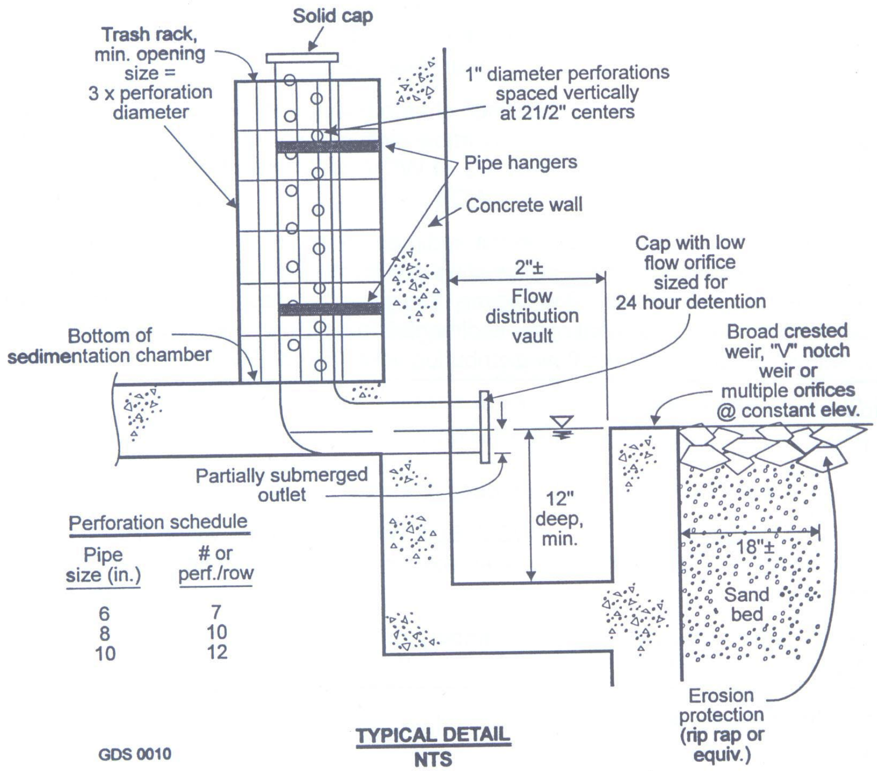


Figure 3.2.4-5 Surface Sand Filter Perforated Stand-Pipe
(Source: Claytor and Schueler, 1996)

J. 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)
- 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

3.2.4.6 Design Procedures

Step 1. Compute runoff control volumes from the Unified Stormwater Sizing Criteria

Calculate the Water Quality Volume (WQ_v), Channel Protection Volume (Cp_v), Overbank Flood Protection Volume (Q_p), and the Extreme Flood Volume (Q_f).

Details on the Unified Stormwater Sizing Criteria are found in Section 1.4.

Step 2. Determine if the development site and conditions are appropriate for the use of a surface or perimeter sand filter.

Consider the Application and Site Feasibility Criteria in subsections 3.2.4.4 and 3.2.4.5-A (Location and Siting).

Step 3. Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 3.2.4.5-J (Additional Site-Specific Design Criteria and Issues).

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.

Step 4. Compute WQ_v peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion structures (see subsection 2.1.7).

(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):

$$\mathbf{A_f = (WQ_v)\,(d_f) / [(k)\,(h_f + d_f)\,(t_f)]}$$

where:
A_f = surface area of filter bed (ft^2)
d_f = filter bed depth
(typically 18 inches, no more than 24 inches)
k = coefficient of permeability of filter media (ft/day)
(use 3.5 ft/day for sand)
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)
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 3.2.4.5-C (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:

$$\mathbf{A_s = -(Q_o/w) * Ln\ (1\text{-}E)}$$

Where:

A_s = sedimentation basin surface area (ft^2)
Q_o = rate of outflow = the WQ_v over a 24-hour period
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 < 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:

$$\mathbf{V_{min} = 0.75\ WQ_v = V_s + V_f + V_{f\text{-}temp}}$$

(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 (see example)

(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 3.2.4-5-D through H 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 h_f 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 D-3 for a Sand Filter Design Example

3.2.4.7 Inspection and Maintenance Requirements

Table 3.2.4-1. Typical Maintenance Activities for Sand Filters
(Source: WMI, 1997; Pitt, 1997)

Activity	Schedule
• Ensure that contributing area, facility, inlets and outlets are clear of debris. • Ensure that the contributing area is stabilized and mowed, with clippings removed. • Remove trash and debris. • Check to ensure that the filter surface is not clogging (also check after moderate and major storms). • Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system. • If permanent water level is present (perimeter sand filter), ensure that the chamber does not leak, and normal pool level is retained.	Monthly
• 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. • Make sure that there is no evidence of deterioration, spalling or cracking of concrete. • Inspect grates (perimeter sand filter). • Inspect inlets, outlets and overflow spillway to ensure good condition and no evidence of erosion. • Repair or replace any damaged structural parts. • Stabilize any eroded areas. • Ensure that flow is not bypassing the facility. • Ensure that no noticeable odors are detected outside the facility.	Annually
• 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. • Replace any filter fabric that has become clogged.	As needed

Additional Maintenance Considerations and Requirements

- A record should be kept of the dewatering time for a sand filter to determine if maintenance is necessary.

- When the filtering capacity of the sand filter facility diminishes substantially (i.e., when water ponds on the surface of the filter bed for more than 48 hours), then the top layers of the filter media (topsoil and 2 to 3 inches of sand) will need to be removed and replaced. This will typically need to be done every 3 to 5 years for low sediment applications, more often for areas of high sediment yield or high oil and grease.

- Removed sediment and media may usually be disposed of in a landfill.

Regular inspection and maintenance is critical to the effective operation of sand filter facilities as designed. Maintenance responsibility for a sand filter system 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.

3.2.4.8 Example Schematics

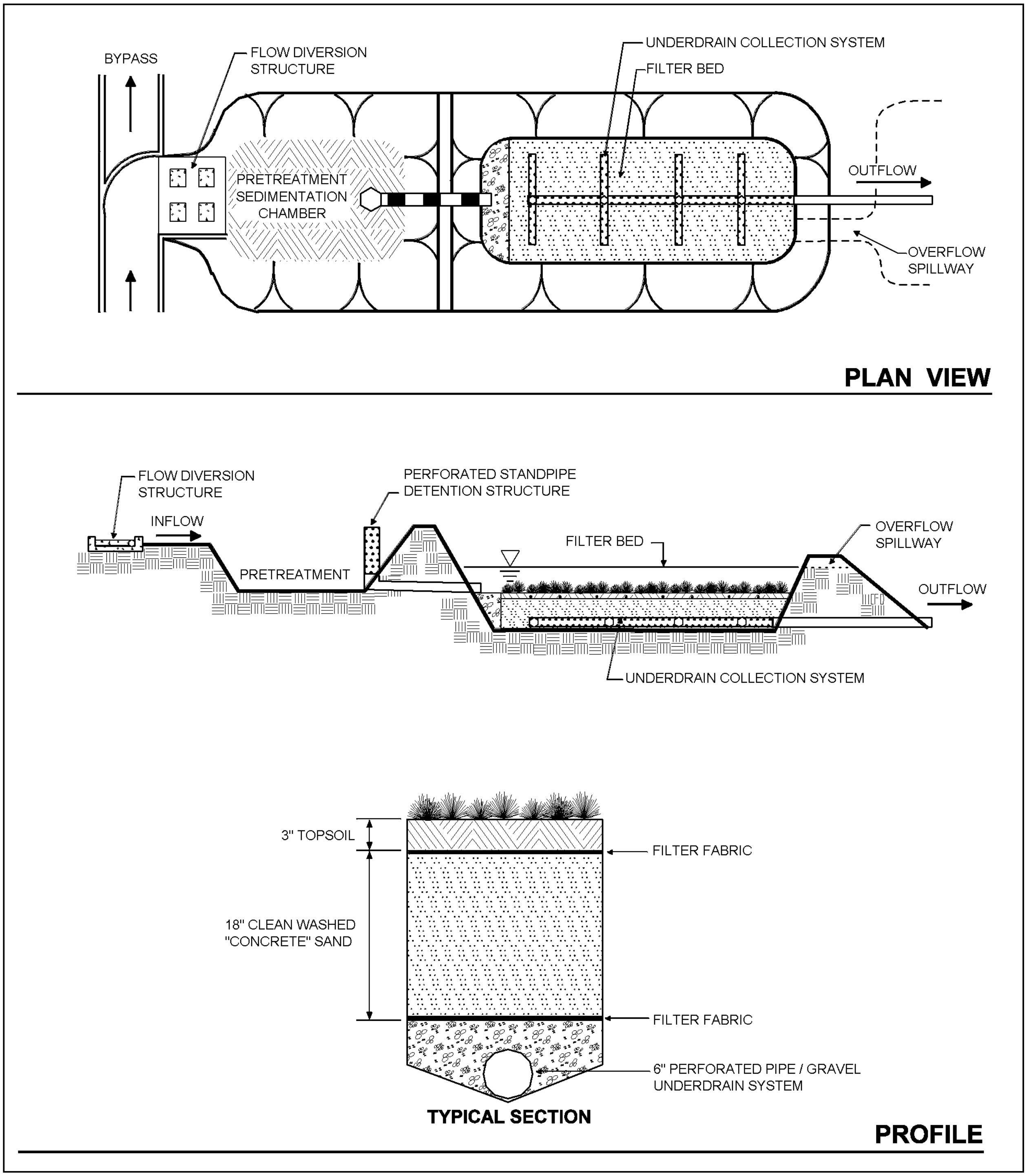


Figure 3.2.4-6 Schematic of Surface Sand Filter

(Source: Center for Watershed Protection)

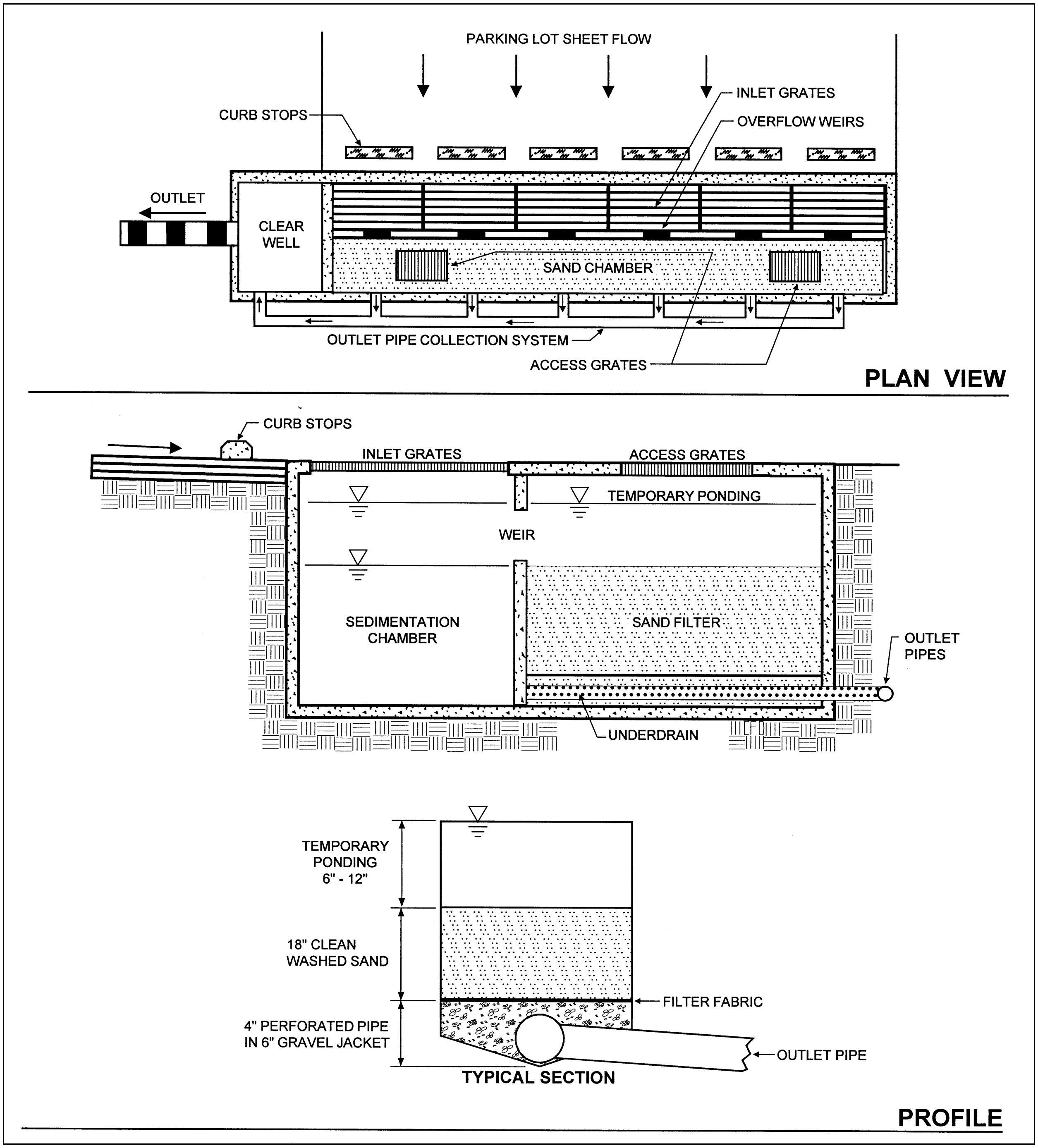


Figure 3.2.4-7 Schematic of Perimeter Sand Filter

(Source: Center for Watershed Protection)

3.2.4.9 Design Forms

Design Procedure Form: Sand Filters

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQ_v volume requirements	
Compute Runoff Coefficient, R_v	R_v = ________
Compute WQv	WQ_v = ________ acre-ft
1b. Compute Cp_v	Cp_v = ________ acre-ft
Compute average release rate	release rate = ________ cfs
Compute $Q_{p\text{-}25}$	$Q_{p\text{-}25}$ = ________ acre-ft
Compute (as necessary) Q_f	Q_f = ________ acre-ft
SAND FILTER DESIGN	
2. Is the use of a sand filter appropriate?	Low Point in development area = ________
	Low Point at stream invert = ________
	Total available head = ________
	Average depth, h_f = ________
	See subsections 3.2.2.4 and 3.2.2.5 - A
3. Confirm local design criteria and applicability.	***See subsection 3.2.2.5 - J***
4. Compute WQ_v peak discharge (Q_{wq})	
Compute Curve Number	CN = ________
Compute Time of Concentration t_c	t_c = ________ hour
Compute Q_{wq}	Q_{wq} = ________ cfs
5. Size flow diversion structure	
Low flow orifice - Orifice equation	A = ________ ft^2
	diam. = ________ inch
Overflow weir - Weir equation	Length = ________ ft
6. Size filtration bed chamber	
Compute area from Darcy's Law	A_f = ________ ft^2
Using length to width (2:1) ratio	L = ________ ft
	W = ________ ft
7. Size sedimentation chamber	
Compute area from Camp-Hazen equation	A_s = ________ ft^2
Given W from step 5, compute Length	L = ________ ft
8. Compute V_{min}	V_{min} = ________ ft^3
9. Compute volume within practice	
Surface sand filter	
Volume within filter bed	V_f = ________ ft^3
Temporary storage above filter bed	$V_{f\text{-}temp}$ = ________ ft^3
Sedimentation chamber (remaining volume)	V_s = ________ ft^3
Height in sedimentation chamber	h_s = ________ ft
Perforated stand pipe - Orifice equation	A = ________ ft^2
	diameter = ________ in
Perimeter sand filter	
Compute volume in filter bed	V_f = ________ ft^3
Compute wet pool storage	V_w = ________ ft^3
Compute temporary storage	V_{temp} = ________ ft^3
	h_{temp} = ________ ft
10. Compute overflow weir sizes	
Compute overflow - Orifice equation	Q = ________ cfs
Weir from sedimenation chamber- Weir equation	Length = ________ ft
Weir from filtration chamber- Weir equation	Length = ________ ft

Notes: ____

____

3.2.5 Infiltration Trench

General Application
Structural Stormwater Control

Description: Excavated trench filled with stone aggregate used to capture and allow infiltration of stormwater runoff into the surrounding soils from the bottom and sides of the trench.

KEY CONSIDERATIONS

DESIGN CRITERIA:

- Soil infiltration rate of 0.5 in/hr or greater required
- Excavated trench (3 to 8 foot depth) filled with stone media (1.5- to 2.5-inch diameter)**;** pea gravel and sand filter layers
- A sediment forebay and grass channel, or equivalent upstream pretreatment, must be provided
- Observation well to monitor percolation

ADVANTAGES / BENEFITS:

- Provides for groundwater recharge
- Good for small sites with porous soils

DISADVANTAGES / LIMITATIONS:

- Potential for groundwater contamination
- High clogging potential; should not be used on sites with fine-particled soils (clays or silts) in drainage area
- Significant setback requirements
- Restrictions in karst areas
- Geotechnical testing required, two borings per facility

MAINTENANCE REQUIREMENTS:

- Inspect for clogging
- Remove sediment from forebay
- Replace pea gravel layer as needed

POLLUTANT REMOVAL

80%	**Total Suspended Solids**
60/60%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
90%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
90%	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORMWATER MANAGEMENT SUITABILITY

- ✓ **Water Quality**
- ✪ **Channel Protection**
- ☐ **Overbank Flood Protection**
- ☐ **Extreme Flood Protection**

Accepts Hotspot Runoff: *No*

✪ in certain situations

IMPLEMENTATION CONSIDERATIONS

M	**Land Requirement**
H	**Capital Cost**
H	**Maintenance Burden**

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *5 acres max.*

Soils: *Pervious soils required (0.5 in/hr or greater)*

Other Considerations:

- *Must not be placed under pavement or concrete*

L=Low **M**=Moderate **H**=High

3.2.5.1 General Description

Infiltration trenches are excavations typically filled with stone to create an underground reservoir for stormwater runoff (see Figure 3.2.5-1). This runoff volume gradually exfiltrates through the bottom and sides of the trench into the subsoil over a 2-day period and eventually reaches the water table. By diverting runoff into the soil, an infiltration trench not only treats the water quality volume, but also helps to preserve the natural water balance on a site and can recharge groundwater and 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 trenches must be carefully sited to avoid the potential of groundwater contamination.

Infiltration trenches are not intended to trap sediment and must always be designed with a sediment forebay and grass channel or filter strip, 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.

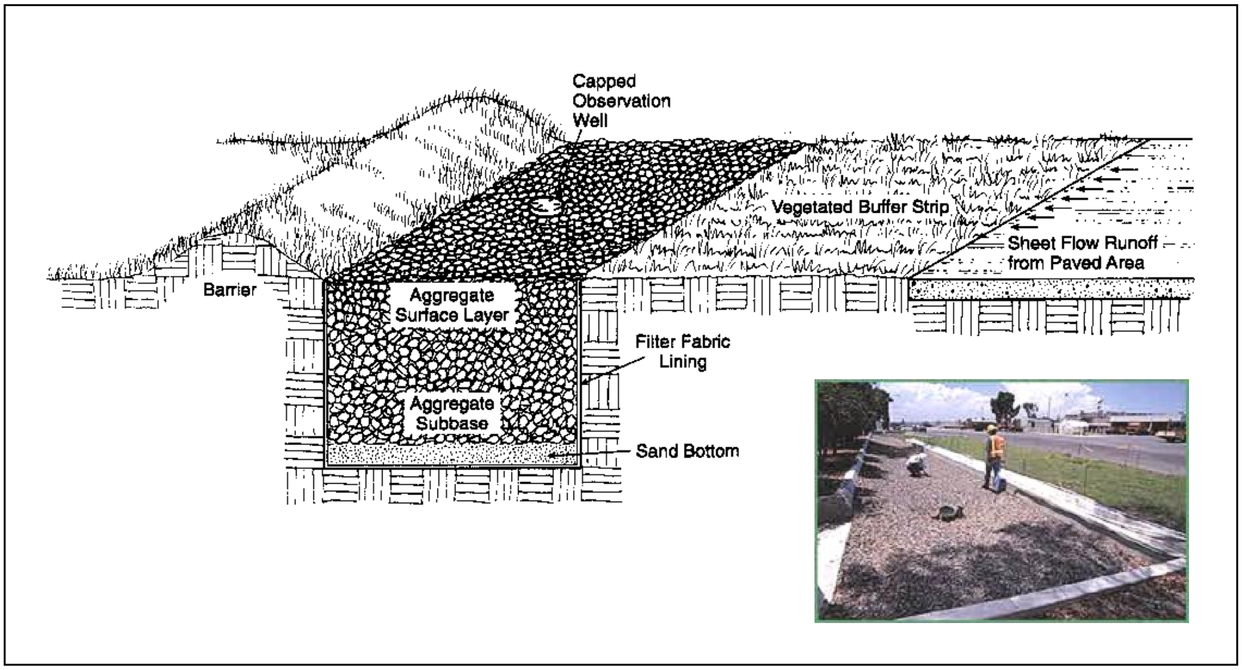


Figure 3.2.5-1 Infiltration Trench Example

3.2.5.2 Stormwater Management Suitability

Infiltration trenches are designed primarily for stormwater quality, i.e. the removal of stormwater pollutants. However, they can provide limited runoff quantity control, particularly for smaller storm events. For some smaller sites, trenches can be designed to capture and infiltrate the channel protection volume (Cp_v) in addition to WQ_v. An infiltration trench will need to be used in conjunction with another structural control to provide overbank and extreme flood protection, if required.

<u>Water Quality</u>

Using the natural filtering properties of soil, infiltration trenches can remove a wide variety of pollutants from stormwater through sorption, precipitation, filtering, and bacterial and chemical degradation. Sediment load and other suspended solids are removed from runoff by pretreatment measures in the facility that treat flows before they reach the trench surface.

Section 3.2.5.3 provides median pollutant removal efficiencies that can be used for planning and design purposes.

Channel Protection

For smaller sites, an infiltration trench may be designed to capture and infiltrate the entire channel protection volume Cp_v in either an off- or on-line configuration. For larger sites, or where only the WQ_v is diverted to the trench, another structural control must be used to provide Cp_v extended detention.

Overbank Flood Protection

Another structural control must be used in conjunction with an infiltration trench system to reduce the post-development peak flow of the 25-year storm (Q_p) to pre-development levels (detention).

Extreme Flood Protection

Infiltration trench facilities must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the filter bed and facility.

Credit for the volume of runoff removed and treated by the infiltration trench may be taken in the overbank flood protection and extreme flood protection calculations (see Section 3.1).

3.2.5.3 Pollutant Removal Capabilities

An infiltration trench is 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 infiltration trenches 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 – 60%**
- **Total Nitrogen – 60%**
- **Fecal Coliform – 90%**
- **Heavy Metals – 90%**

For additional information and data on pollutant removal capabilities for infiltration trenches, 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

3.2.5.4 Application and Site Feasibility Criteria

Infiltration trenches are generally suited for medium-to-high density residential, commercial and institutional developments where the subsoil is sufficiently permeable to provide a reasonable infiltration rate and the water table is low enough to prevent groundwater contamination. They are applicable primarily for impervious areas where there are not high levels of fine particulates (clay/silt soils) in the runoff and should only be considered for sites where the sediment load is relatively low.

Infiltration trenches can either be used to capture sheet flow from a drainage area or function as an off-line device. Due to the relatively narrow shape, infiltration trenches can be adapted to many different types of sites and can be utilized in retrofit situations. Unlike some other structural stormwater controls, they 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 trenches should not be used for manufacturing and 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 trenches are also not suitable in areas with karst geology without adequate geotechnical testing by qualified individuals and in accordance with local requirements.

The following criteria should be evaluated to ensure the suitability of an infiltration trench 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

- <u>Drainage Area</u> – 5 acres maximum
- <u>Space Required</u> – Will vary depending on the depth of the facility
- <u>Site Slope</u> – No more than 6% slope (for pre-construction facility footprint)
- <u>Minimum Head</u> – Elevation difference needed at a site from the inflow to the outflow: 1 foot
- <u>Minimum Depth to Water Table</u> – 4 feet recommended between the bottom of the infiltration trench and the elevation of the seasonally high water table, may be reduced to 2 feet in coastal areas
- <u>Soils</u> – Infiltration rate greater than 0.5 inches per hour required (typically hydrologic group "A", some group "B" soils)

Other Constraints / Considerations

- <u>Aquifer Protection</u> – No hotspot runoff allowed; Meet setback requirements in design criteria

3.2.5.5 Planning and Design Criteria

*The following criteria are to be considered **minimum** standards for the design of an infiltration trench facility. Consult with the local review authority to determine if there are any variations to these criteria or additional standards that must be followed.*

A. LOCATION AND SITING

- To be suitable for infiltration, underlying soils should have an infiltration rate (f_c) of <u>0.5 inches per hour or greater</u>, as initially determined from NRCS soil textural classification, and subsequently confirmed by field geotechnical tests. The minimum geotechnical testing is one test hole per 5,000 square feet, with a minimum of two borings per facility (taken within the proposed limits of the facility). Infiltration trenches cannot be used in fill soils.

- Infiltration trenches should have a contributing drainage area of 5 acres or less.

- Soils on the drainage area tributary to an infiltration trench should have a clay content of less than 20% and a silt/clay content of less than 40% to prevent clogging and failure.

- There should be at least 4 feet between the bottom of the infiltration trench and the elevation of the seasonally high water table. *This value can be reduced to 2 feet for coastal areas of Georgia*.

- Clay lenses, bedrock or other restrictive layers below the bottom of the trench will reduce infiltration rates unless excavated.

- Minimum setback requirements for infiltration trench 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)

- When used in an off-line configuration, the water quality volume (WQ_v) is diverted to the infiltration trench through the use of a flow splitter. Stormwater flows greater than the WQ_v are diverted to other controls or downstream using a diversion structure or flow splitter.

- To reduce the potential for costly maintenance and/or system reconstruction, it is strongly recommended that the trench be located in an open or lawn area, with the top of the structure as close to the ground surface as possible. Infiltration trenches shall not be located beneath paved surfaces, such as parking lots.

- Infiltration trenches are designed for intermittent flow and must be allowed to drain and allow reaeration of the surrounding soil between rainfall events. They must not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

B. GENERAL DESIGN

- A well-designed infiltration trench consists of:

  (1) Excavated shallow trench backfilled with sand, coarse stone, and pea gravel, and lined with a filter fabric;
  (2) Appropriate pretreatment measures; and
  (3) One or more observation wells to show how quickly the trench dewaters or to determine if the device is clogged.

Figure 3.2.5-2 provides a plan view and profile schematic for the design of an off-line infiltration trench facility. An example of an on-line infiltration trench is shown in Figure 3.2.5-1.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

- The required trench storage volume is equal to the water quality volume (WQ_v). For smaller sites, an infiltration trench can be designed with a larger storage volume to include the channel protection volume (Cp_v).

- A trench must be designed to fully dewater the entire WQ_v within 24 to 48 hours after a rainfall event. The slowest infiltration rate obtained from tests performed at the site should be used in the design calculations.

- Trench depths should be between 3 and 8 feet, to provide for easier maintenance. The width of a trench must be less than 25 feet.

- Broader, shallow trenches reduce the risk of clogging by spreading the flow over a larger area for infiltration.

- The surface area required is calculated based on the trench depth, soil infiltration rate, aggregate void space, and fill time (assume a fill time of 2 hours for most designs).

- The bottom slope of a trench should be flat across its length and width to evenly distribute flows, encourage uniform infiltration through the bottom, and reduce the risk of clogging.

- The stone aggregate used in the trench 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 shall not be used. A porosity value (void space/total volume) of 0.32 should be used in calculations, unless aggregate specific data exist.

- A 6-inch layer of clean, washed sand is placed on the bottom of the trench to encourage drainage and prevent compaction of the native soil while the stone aggregate is added.

- The infiltration trench is lined on the sides and top by an appropriate geotextile filter fabric that prevents soil piping but has greater permeability than the parent soil. The top layer of filter fabric is located 2 to 6 inches from the top of the trench and serves to prevent sediment from passing into the stone aggregate. Since this top layer serves as a sediment barrier, it will need to be replaced more frequently and must be readily separated from the side sections.

- The top surface of the infiltration trench above the filter fabric is typically covered with pea gravel. The pea gravel layer improves sediment filtering and maximizes the pollutant removal in the top of the trench. In addition, it can easily be removed and replaced should the device begin to clog. Alternatively, the trench can be covered with permeable topsoil and planted with grass in a landscaped area.

- An observation well must be installed in every infiltration trench and should consist of a perforated PVC pipe, 4 to 6 inches in diameter, extending to the bottom of the trench (see Appendix B for a schematic of an observation well). The observation well will show the rate of dewatering after a storm, as well as provide a means of determining sediment levels at the bottom and when the filter fabric at the top is clogged and maintenance is needed. It should be installed along the centerline of the structure, flush with the ground elevation of the trench. A visible floating marker should be provided to indicate the water level. The top of the well should be capped and locked to discourage vandalism and tampering.

- The trench excavation should be limited to the width and depth specified in the design. Excavated material should be placed away from the open trench so as not to jeopardize the stability of the trench sidewalls. The bottom of the excavated trench shall not be loaded in a way that causes soil compaction, and should be scarified prior to placement of sand. The sides of the trench shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling. All infiltration trench facilities should be protected during site construction and should be constructed after upstream areas have been stabilized.

D. PRETREATMENT / INLETS

- Pretreatment facilities **must always** be used in conjunction with an infiltration trench to prevent clogging and failure.

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

- For an off-line configuration, pretreatment should consist of a sediment forebay, vault, plunge pool, or similar sedimentation chamber (with energy dissipaters) sized to 25% of the water quality volume (WQ_v). Exit velocities from the pretreatment chamber must be nonerosive for the 2-year design storm.

E. OUTLET STRUCTURES

- Outlet structures are not required for infiltration trenches.

F. EMERGENCY SPILLWAY

- Typically for off-line designs, there is no need for an emergency spillway. However, a nonerosive overflow channel should be provided to safely pass flows that exceed the storage capacity of the trench to a stabilized downstream area or watercourse.

G. MAINTENANCE ACCESS

- Adequate access should be provided to an infiltration trench facility for inspection and maintenance.

H. SAFETY FEATURES

- In general, infiltration trenches are not likely to pose a physical threat to the public and do not need to be fenced.

I. LANDSCAPING

- Vegetated filter strips and buffers should fit into and blend with surrounding area. Native grasses are preferable, if compatible. The trench may be covered with permeable topsoil and planted with grass in a landscaped area

J. 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

3.2.5.6 Design Procedures

Step 1. Compute runoff control volumes from the Unified Stormwater Sizing Criteria

Calculate the Water Quality Volume (WQ_v), Channel Protection Volume (Cp_v), Overbank Flood Protection Volume (Q_p), and the Extreme Flood Volume (Q_f).

Details on the Unified Stormwater Sizing Criteria are found in Section 1.4.

Step 2. Determine if the development site and conditions are appropriate for the use of an infiltration trench.

Consider the Application and Site Feasibility Criteria in subsections 3.2.5.4 and 3.2.5.5-A (Location and Siting).

Step 3. Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 3.2.5.5-J (Additional Site-Specific Design Criteria and Issues).

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.

Step 4. Compute WQ_v peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion (see subsection 2.1.7).

(a) Using WQ_v (or total volume to be infiltrated), 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 infiltration trench.

Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 6. Size infiltration trench

The area of the trench can be determined from the following equation:

$$A = \frac{WQ_v}{(nd + kT / 12)}$$

Where:
A = Surface Area
WQ_v = Water Quality Volume (or total volume to be infiltrated)
n = porosity
d = trench depth (feet)
k = percolation (inches/hour)
T= Fill Time (time for the practice to fill with water), in hours

A porosity value *n* = 0.32 should be used.

All infiltration systems should be designed to fully dewater the entire WQ_v within 24 to 48 hours after the rainfall event.

A fill time T=2 hours can be used for most designs

See subsection 3.2.5.5-C (Physical Specifications/Geometry) for more specifications.

Step 7. Determine pretreatment volume and design pretreatment measures

Size pretreatment facility to treat 25% of the water quality volume (WQ_v) for off-line configurations.

See subsection 3.2.5.5-D (Pretreatment / Inlets) for more details.

Step 8. Design spillway(s)

Adequate stormwater outfalls should be provided for the overflow exceeding the capacity of the trench, ensuring nonerosive velocities on the down-slope.

See Appendix D-4 for an Infiltration Trench Design Example

3.2.5.7 Inspection and Maintenance Requirements

Table 3.2.4-2 Typical Maintenance Activities for Infiltration Trenches
(Source: EPA, 1999)

Activity	Schedule
• Ensure that contributing area, facility and inlets are clear of debris. • Ensure that the contributing area is stabilized. • Remove sediment and oil/grease from pretreatment devices, as well as overflow structures. • Mow grass filter strips should be mowed as necessary. Remove grass clippings.	Monthly
• Check observation wells following 3 days of dry weather. Failure to percolate within this time period indicates clogging. • Inspect pretreatment devices and diversion structures for sediment build-up and structural damage. • Remove trees that start to grow in the vicinity of the trench.	Semi-annual Inspection
• Replace pea gravel/topsoil and top surface filter fabric (when clogged).	As needed
• Perform total rehabilitation of the trench to maintain design storage capacity. • Excavate trench walls to expose clean soil.	Upon Failure

Additional Maintenance Considerations and Requirements

- A record should be kept of the dewatering time of an infiltration trench to determine if maintenance is necessary.
- Removed sediment and media may usually be disposed of in a landfill.

Regular inspection and maintenance is critical to the effective operation of infiltration trench facilities as designed. Maintenance responsibility for a infiltration trench 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.

3.2.5.8 Example Schematics

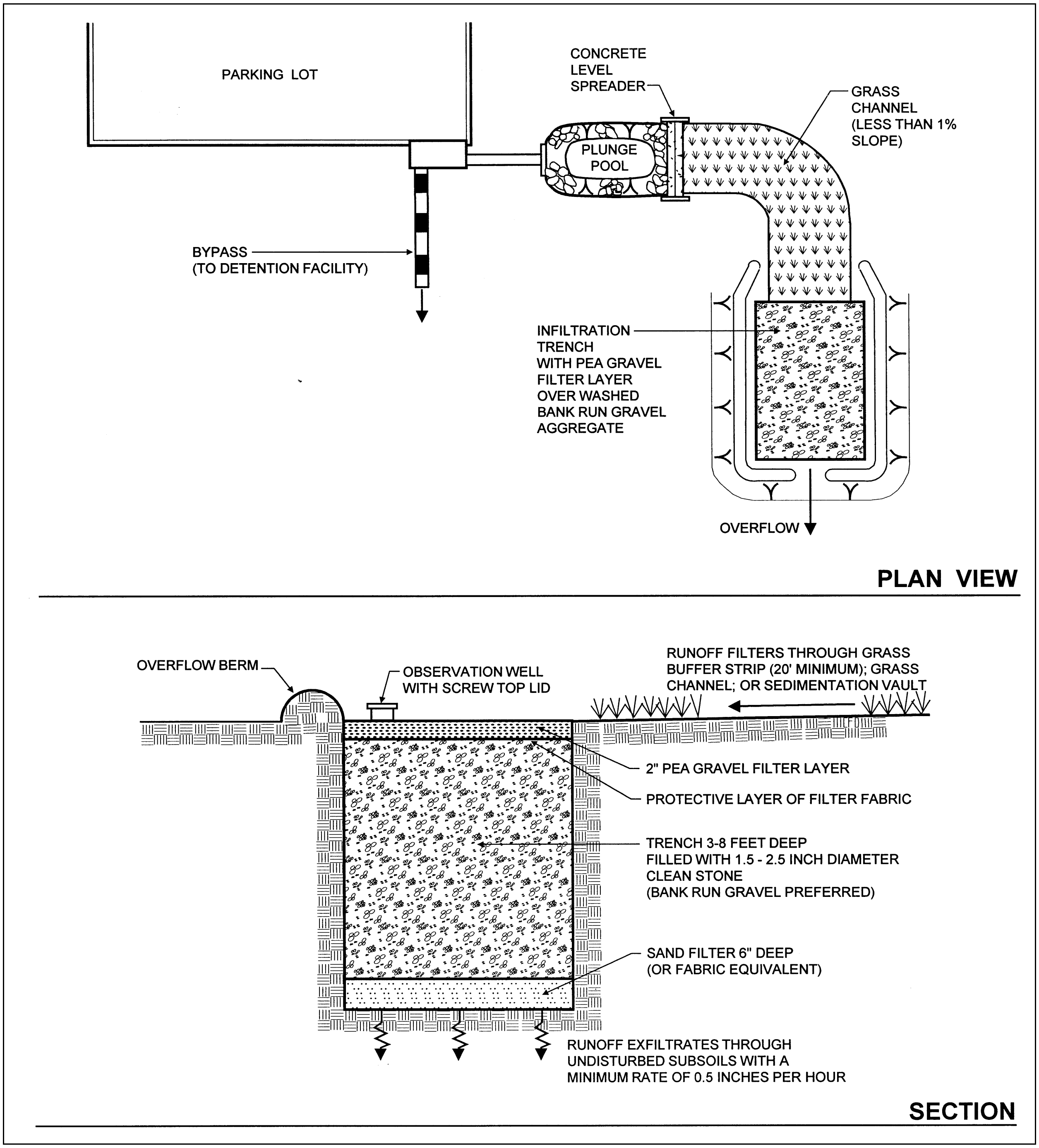
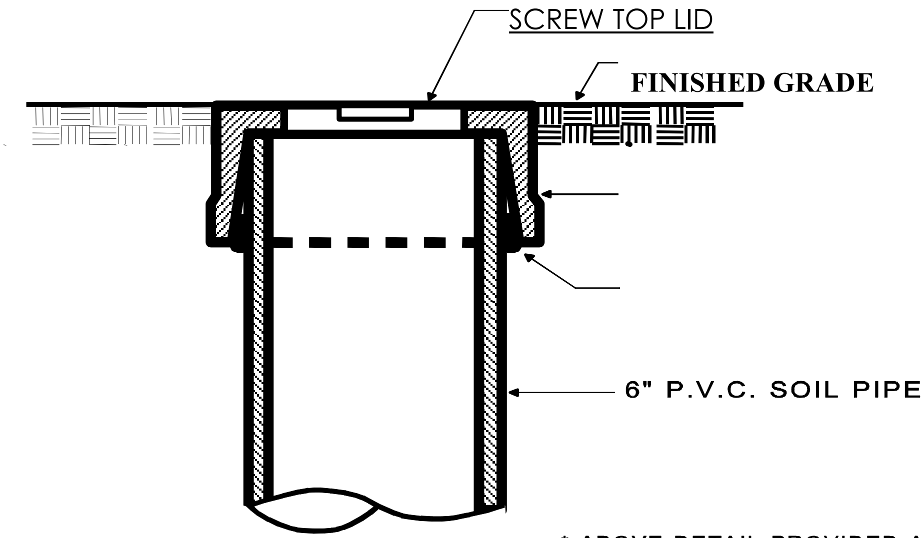


Figure 3.2.5-2 Schematic of Infiltration Trench

(Source: Center for Watershed Protection)



* ABOVE DETAIL PROVIDED AS SCHEMATIC
SCREW TOP P.V.C. WELL CAP ONLY

EACH OBSERVATION WELL / CLEANOUT SHALL INCLUDE THE FOLLOWING:

1. FOR AN UNDERGROUND FLUSH MOUNTED OBSERVATION WELL / CLEANOUT, PROVIDE A TUBE MADE OF NON-CORROSIVE MATERIAL, SCHEDULE 40 OR EQUAL, AT LEAST THREE FEET LONG WITH AN INSIDE DIAMETER OF AT LEAST 6 INCHES.

2. THE TUBE SHALL HAVE A FACTORY ATTACHED CAST IRON OR HIGH IMPACT PLASTIC COLLAR WITH RIBS TO PREVENT ROTATION WHEN REMOVING SCREW TOP LID. THE SCREW TOP LID SHALL BE CAST IRON OR HIGH OMPACT PLASTIC THAT WILL WITHSTAND ULTRA-VIOLET RAYS.

OBSERVATION WELL DETAIL

Figure 3.2.5-3. Observation Well Detail

3.2.5.9 Design Forms

Design Procedure Form: Infiltration Trench

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQ_v volume requirements	
Compute Runoff Coefficient, R_v	R_v = ________
Compute WQv	WQ_v = ________ acre-ft
1b. Compute Cp_v	Cp_v = ________ acre-ft
Compute average release rate	release rate = ________ cfs
Compute $Q_{p\text{-}25}$	$Q_{p\text{-}25}$ = ________ acre-ft
Compute (as necessary) Q_f	Q_f = ________ acre-ft
INFILTRATION TRENCH DESIGN	
2. Is the use of a infiltration trench appropriate?	***See subsections 3.2.5.4 and 3.2.5.5 - A***
3. Confirm local design criteria and applicability.	***See subsection 3.2.5.5 - J***
4. Compute WQ_v peak discharge (Q_{wq})	
Compute Curve Number	CN = ________
Compute Time of Concentration t_c	t_c = ________ hour
Compute Q_{wq}	Q_{wq} = ________ cfs
5. Size infiltration trench	Area = ________ ft^2
Width must be less than 25 ft	Width = ________ ft
	Length = ________ ft
6. Size the flow diversion structures	
Low flow orifice from orifice equation	
$Q = CA(2gh)^{0.5}$	A = ________ ft^2
	diam. = ________ inch
Overflow weir from weir equation	
$Q = CLH^{3/2}$	Length = ________ ft
7. Pretreatment volume (for offine designs)	
$Vol_{pre} = 0.25(WQ_v)$	Vol_{pre} = ________ ft^3
8. Design spillway(s)	

Notes: ____

____

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3.2.6 Enhanced Swales

General Application
Structural Stormwater Control

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.

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**
50/50%	**Nutrients** - Total Phosphorus / Total Nitrogen removal
40%	**Metals** - Cadmium, Copper, Lead, and Zinc removal
No data	**Pathogens** - Coliform, Streptococci, E.Coli removal

STORMWATER MANAGEMENT SUITABILITY

- ✓ **Water Quality**
- ✪ **Channel Protection**
- ☐ **Overbank Flood Protection**
- ☐ **Extreme Flood Protection**

Accepts Hotspot Runoff: *Yes (requires impermeable liner)*

✪ in certain situations

IMPLEMENTATION CONSIDERATIONS

H	**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

3.2.6.1 General Description

Enhanced swales (also referred to as *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 overlays an underdrain system. Dry swales are sized to allow the entire WQ_v 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 (Wetland Channel)** – 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.

Enhanced Dry Swale

Enhanced Wet Swale

Figure 3.2.6-1 Enhanced Swale Examples

Dry and wet swales are not to be confused with a *filter strip* or *grass channel*, which are Limited Application structural controls and 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 limited application structural controls in subsections 3.3.1 and 3.3.2, respectively.

3.2.6.2 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.

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 accumulation and biological removal.

Section 3.2.6.3 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 structural control 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 structural control 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 structural control 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.

3.2.6.3 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 (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

3.2.6.4 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 nonerosive velocities. 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 – 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 for hotspots

3.2.6.5 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.*

A. LOCATION AND SITING

- 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 nonerosive 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.

B. GENERAL DESIGN

- Both types of enhanced swales are designed to treat the WQ_v 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 the use of a pea gravel flow spreader 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 overlays 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 is collected and conveyed by a perforated pipe and gravel underdrain system to the outlet. Figure 3.2.6-2 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 3.2.6-3 provides a plan view and profile schematic for the design of a wet swale system.

C. 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 WQ_v 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 nonerosive 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 bed of the dry swale consists of a permeable soil layer of at least 30 inches in depth, above a 4-inch diameter perforated PVC pipe (AASHTO M 252) longitudinal underdrain in a 6-inch gravel layer. 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.

- 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 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.

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.

D. 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.

- A pea gravel diaphragm and gentle side slopes should be provided along the top of channels to provide pretreatment for lateral sheet flows.

E. OUTLET STRUCTURES

Dry Swale

- 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.

F. EMERGENCY SPILLWAY

- Enhanced swales must be adequately designed to safely pass flows that exceed the design storm flows.

G. MAINTENANCE ACCESS

- Adequate access should be provided for all dry and wet swale systems for inspection and maintenance.

H. SAFETY FEATURES

- Ponding depths should be limited to a maximum of 18 inches.

I. LANDSCAPING

Landscape design should specify proper grass species and wetland plants based on specific site, soils and hydric conditions present along the channel. Below is some specific guidance for dry and wet swales:

Dry Swale

- Information on appropriate turf grass species for Georgia can be found in Appendix F (*Landscaping and Aesthetics Guidance)*.

Wet Swale

- Emergent vegetation should be planted, or wetland soils may be spread on the swale bottom for seed stock.

- Information on establishing wetland vegetation and appropriate wetland species for Georgia can be found in Appendix F (*Landscaping and Aesthetics Guidance)*.

- 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 2.1.8 for guidance on water balance calculations.

J. 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

3.2.6.6 Design Procedures

Step 1. Compute runoff control volumes from the Unified Stormwater Sizing Criteria

Calculate the Water Quality Volume (WQ_v), Channel Protection Volume (Cp_v), Overbank Flood Protection Volume (Q_p), and the Extreme Flood Volume (Q_f).

Details on the Unified Stormwater Sizing Criteria are found in Section 1.4.

Step 2. Determine if the development site and conditions are appropriate for the use of an enhanced swale system (dry or wet swale).

Consider the Application and Site Feasibility Criteria in subsections 3.2.6.4 and 3.2.6.5-A (Location and Siting).

Step 3. Confirm local design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 3.2.6.5-J (Additional Site-Specific Design Criteria and Issues).

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.

Step 4. Determine pretreatment volume

The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage. The forebay storage volume counts toward the total WQ_v requirement, and should be subtracted from the WQ_v for subsequent calculations.

Step 5. Determine swale dimensions

Size bottom width, depth, length, and slope necessary to store WQ_v with less than 18 inches of ponding at the downstream 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 3.2.6.5-C (Physical Specifications / Geometry) for more details

Step 6. Compute number of check dams (or similar structures) required to detain WQ_v

Step 7. Calculate draw-down time

Dry swale: Planting soil should pass a maximum rate of 1.5 feet in 24 hours and must completely filter WQ_v within 48 hours.

Wet swale: Must hold the WQ_v.

Step 8. 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 9. Design low flow orifice at downstream headwalls and checkdams

Design orifice to pass WQ_v in 6 hours. Use Orifice equation.

Step 10. Design inlets, sediment forebay(s), and underdrain system (dry swale)

See subsection 3.2.6.5-D through H for more details.

Step 11. 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 3.2.6.5-I (Landscaping) and Appendix F for more details.

See Appendix D-5 for an Enhanced Swale Design Example

3.2.6.7 Inspection and Maintenance Requirements

Table 3.2.6-1 Typical Maintenance Activities for Enhanced Swales
(Source: WMI, 1997; Pitt, 1997)

Activity	Schedule
• For dry swales, mow grass to maintain a height of 4 to 6 inches. Remove grass clippings.	As needed (frequent/seasonally)
• Inspect grass along side slopes for erosion and formation of rills or gullies and correct. • Remove trash and debris accumulated in the inflow forebay. • Inspect and correct erosion problems in the sand/soil bed of dry swales. • Based on inspection, plant an alternative grass species if the original grass cover has not been successfully established. • Replant wetland species (for wet swale) if not sufficiently established. • Inspect pea gravel diaphragm for clogging and correct the problem.	Annually (Semi-annually the first year)
• Roto-till or cultivate the surface of the sand/soil bed of dry swales if the swale does not draw down within 48 hours. • Remove sediment build-up within the bottom of the swale once it has accumulated to 25% of the original design volume.	As needed

⚠ Regular inspection and maintenance is critical to the effective operation of an enhanced swale system as designed. Maintenance responsibility for a dry or wet 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.

3.2.6.8 Example Schematics

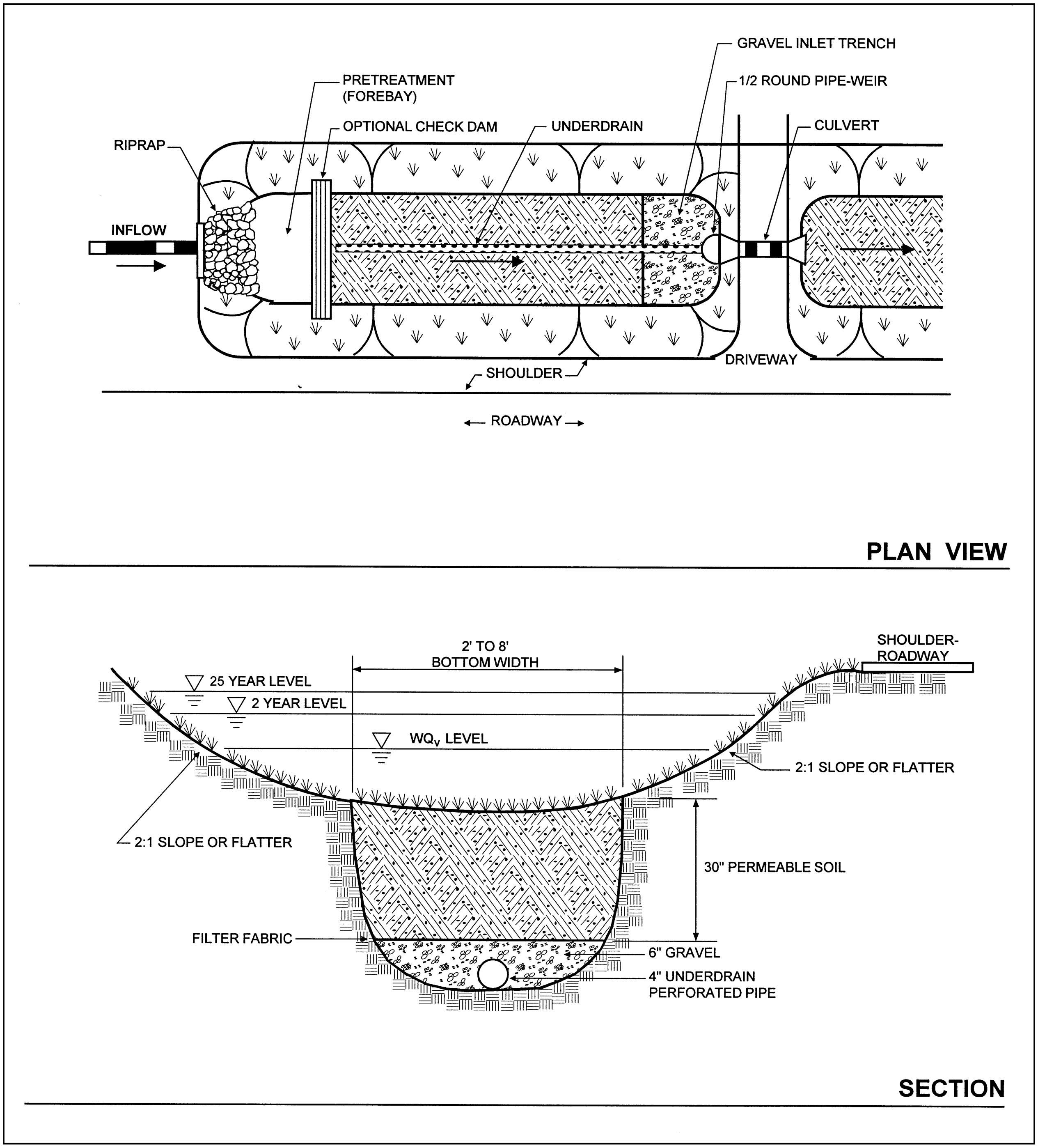

Figure 3.2.6-2 Schematic of Dry Swale

(Source: Center for Watershed Protection)

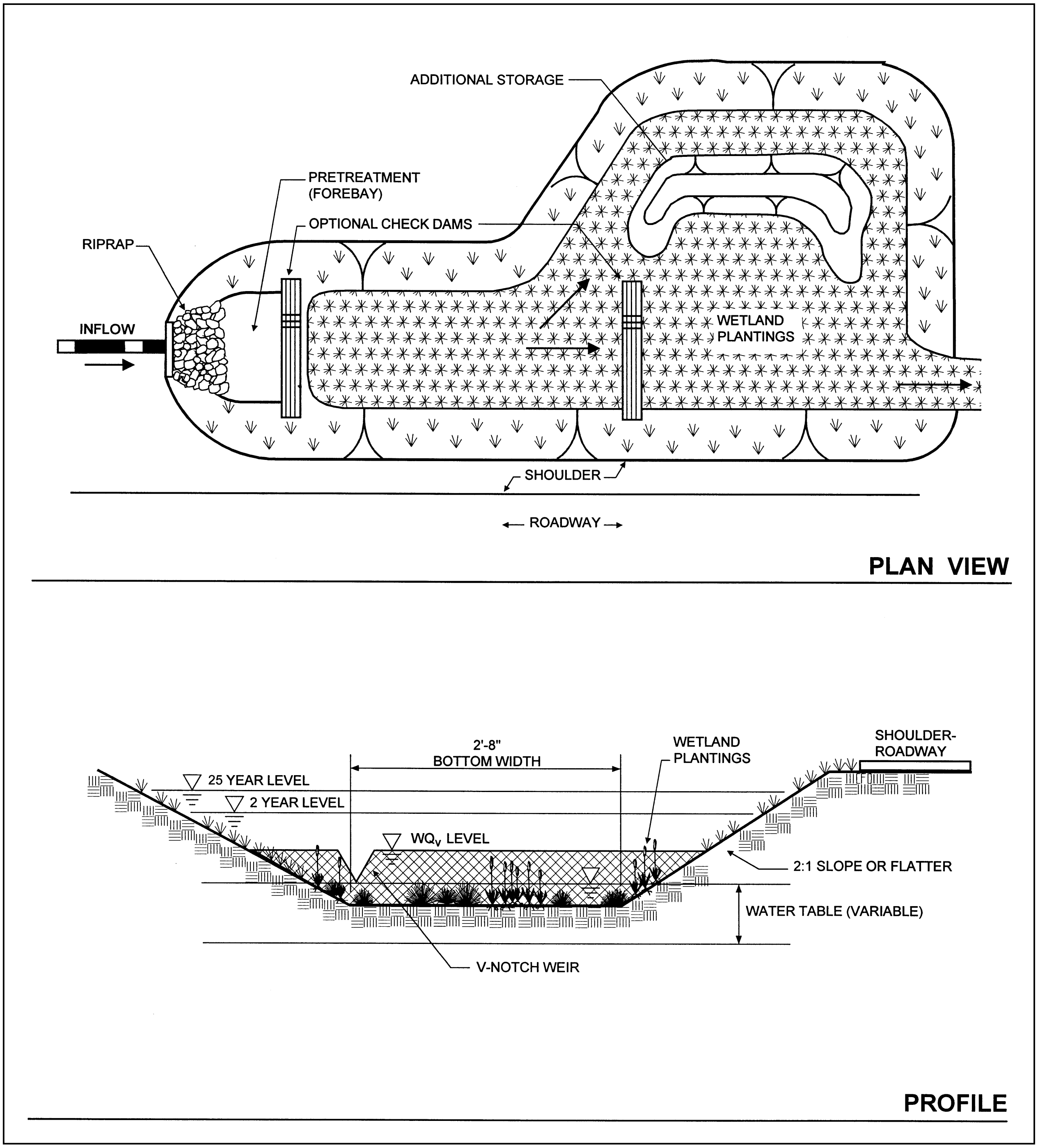

Figure 3.2.6-3 Schematic of Wet Swale

(Source: Center for Watershed Protection)

3.2.6.9 Design Forms

Design Procedure Form: Enhanced Swales

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQ_v volume requirements	
Compute Runoff Coefficient, R_v	R_v = ________
Compute WQv	WQ_v = ________ acre-ft
1b. Compute Cp_v	Cp_v = ________ acre-ft
Compute average release rate	release rate = ________ cfs
Compute Q_{p-25}	Q_{p-25} = ________ acre-ft
Compute (as necessary) Q_f	Q_f = ________ acre-ft
ENHANCED SWALE DESIGN	
2. Is the use of an enhanced swale appropriate?	***See subsections 3.2.6.4 and 3.2.6.5 - A***
3. Confirm local design criteria and applicability.	***See subsection 3.2.6.5 - J***
4. Pretreatment Volume Vol_{pre} = I (0.1")(1'/12")	Vol_{pre} = ________ acre-ft
5. Determine swale dimensions Assume trapezoidal channel with max depth of 18 inches	Length = ________ ft Width = ________ ft Side Slopes = ________ Area = ________ ft^2
6. Compute number of check dams (or similar structures) required to detain WQ_v	Slope = ________ ft/ft Depth = ________ ft Distance = ________ ft Number = ________ each
7. Calculate draw-down time Require k = 1.5 ft per day for dry swales	t = ________ hr
8. Check 25-year velocity erosion potential and freeboard Requires separate computer analysis for velocity	V_{min} = ________ fps
Overflow wier (use weir equation) Use weir equation for slot length ($Q = CLH^{3/2}$)	Weir Length = ________ ft
9. Design low flow orifice at headwall Area of orifice from orifice equation $Q = CA(2gh)^{0.5}$	Area = ________ ft^2 diam = ________ inch
10. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features.	***See subsection 3.2.6.5 - D through H***
11. Attach landscaping plan (including wetland vegetation)	***See Appendix F***

Notes: ____

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SECTION

3.3

LIMITED APPLICATION STRUCTRUAL STORMWATER CONTROLS

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3.3.1 Filter Strip

Limited Application
Structural Stormwater Control

Description: 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.

REASONS FOR LIMITED USE	STORMWATER MANAGEMENT SUITABILITY
• Cannot alone achieve the 80% TSS removal target	☑ **Water Quality** ☐ **Channel/Flood Protection**
KEY CONSIDERATIONS	**SPECIAL APPLICATIONS**
• 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 • Can provide groundwater recharge • Reasonably low construction cost • Large land requirement • Requires periodic repair, regrading, and sediment removal to prevent channelization	☑ **Pretreatment** ☐ **High Density/Ultra-Urban** ☑ **Other:** Use in buffer system; treating runoff from pervious areas **Residential Subdivision Use**: *Yes*

3.3.1.1 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 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. Filter strips are often used as a stormwater site design credit (see Section 1.4 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 while contained within the filter strip. To be effective, however, 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 the contact time with the runoff, thus reducing the overall width of the filter strip required to treat stormwater runoff. Filter strips are typically an on-line practice, so they must be designed to withstand the full range of storm events without eroding.

3.3.1.2 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.

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 – 50%**
- **Total Phosphorus – 20%**
- **Total Nitrogen – 20%**
- **Fecal Coliform – insufficient data**
- **Heavy Metals – 40%**

3.3.1.3 Design Criteria and Specifications

General Criteria

- Filter strips should be used to treat small 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.

- Filter strips should be integrated within site designs.

- 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 be designed for slopes between 2% and 6%. Greater slopes than this would encourage the formation of concentrated flow. Flatter slopes would encourage standing water.

- 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 F for a list of appropriate grasses for use in Georgia.

- The filter strip should be at least 15 feet long to provide filtration and contact time for water quality treatment. 25 feet is preferred (where available), though length will normally be dictated by design method.

- Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion.

- An effective flow spreader is to 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 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.

- Pedestrian traffic across the filter strip should be limited through channeling onto sidewalks.

- Maximum discharge loading per foot of filter strip width (perpendicular to flow path) is found using the Manning's equation:

$$q = \frac{0.00236}{n} Y^{\frac{5}{3}} S^{\frac{1}{2}} \qquad \textbf{(3.3.1)}$$

Where: q = discharge per foot of width of filter strip (cfs/ft)
Y = allowable depth of flow (inches)
S = slope of filter strip (percent)
n = Manning's "n" roughness coefficient

(use 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:

$$W_{fMIN} = \frac{Q}{q} \qquad \textbf{(3.3.2)}$$

Where: W_{fMIN} = minimum filter strip width perpendicular to flow (feet)

<u>Filter without Berm</u>

- Size filter strip (parallel to flow path) for a contact time of 5 minutes minimum

- Equation for filter length is based on the SCS TR55 travel time equation (SCS, 1986):

$$L_f = \frac{(T_t)^{1.25} (P_{2-24})^{0.625} (S)^{0.5}}{3.34n} \qquad \textbf{(3.3.3)}$$

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

(use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)

<u>Filter Strips with Berm</u>

- Size outlet pipes to ensure that the bermed area drains within 24 hours.

- Specify grasses resistant to frequent inundation within the shallow ponding limit.

- Berm material should be 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 WQ_v 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 bioretention areas 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 3.3.3-1 provides sizing guidance for bioretention filter strips for pretreatment.

Table 3.3.1-1 Bioretention 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 slope (max = 6%)	< 2%	> 2%	< 2%	> 2%	< 2%	> 2%	< 2%	> 2%
Filter strip minimum length (feet)	10	15	20	25	10	12	15	18

3.3.1.4 Inspection and Maintenance Requirements

Table 3.3.1-2. Typical Maintenance Activities for Filter Strips
(Source: CWP, 1996)

Activity	Schedule
• Mow grass to maintain a 2 to 4 inch height.	Regularly (frequently)
• Inspect pea gravel diaphragm for clogging and remove built-up sediment. • Inspect vegetation for rills and gullies and correct. Seed or sod bare areas. • Inspect to ensure that grass has established. If not, replace with an alternative species.	Annual Inspection (Semi-annual first year)

Additional Maintenance Considerations and Requirements

- Filter strips require similar maintenance to other vegetative practices. Maintenance is very important for filter strips, particularly in terms of ensuring that flow does not short circuit the practice.

3.3.1.5 Example Schematic

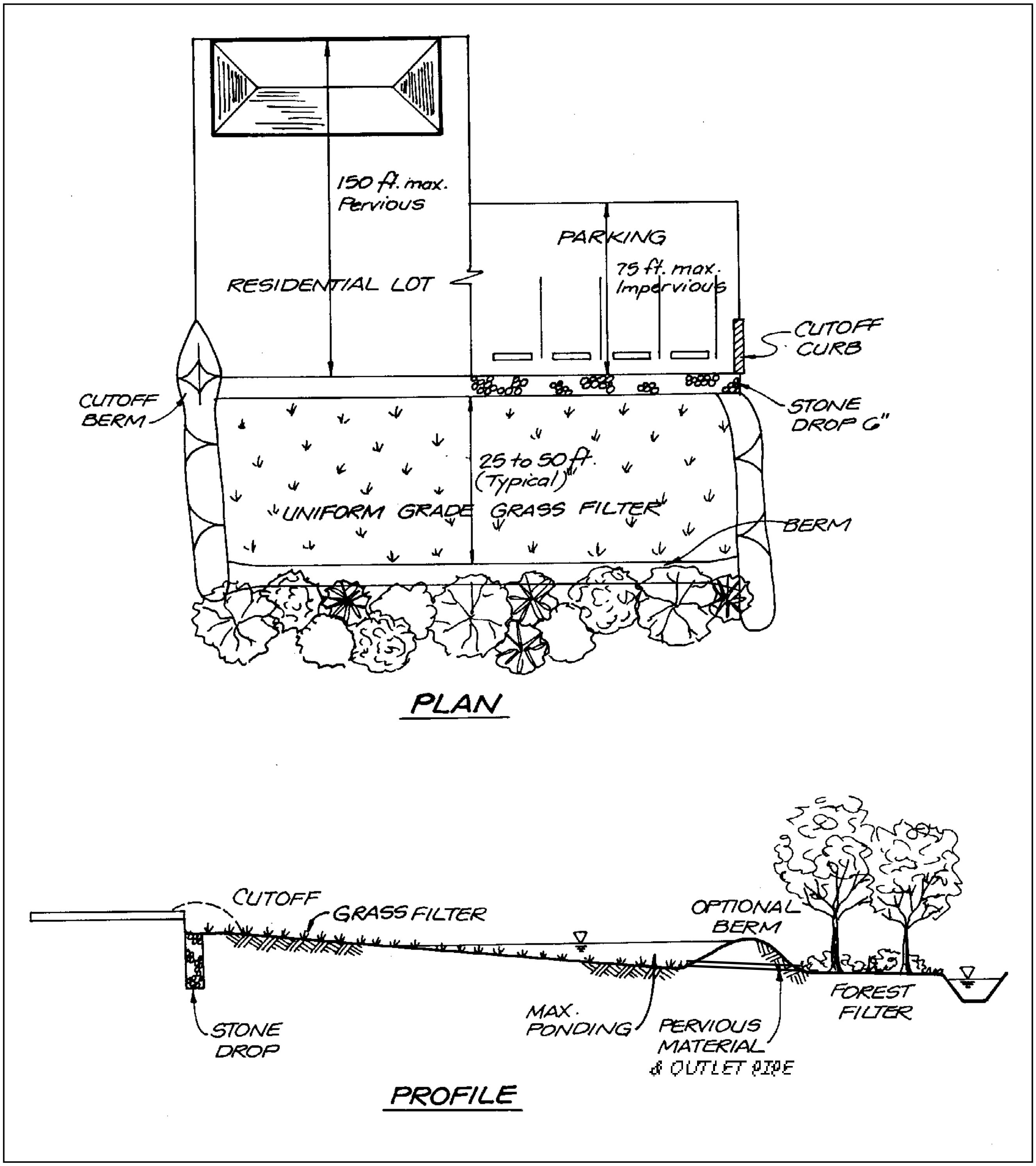

Figure 3.3.1-1 Schematic of Filter Strip (with Berm)

3.3.1.6 Design Example

Basic Data

Small commercial lot 150 feet deep x 100 feet wide located in Smyrna

- Drainage area (A) = 0.34 acres
- Impervious percentage (I) = 70%
- Slope equals 4%, Manning's n = 0.25

Calculate Maximum Discharge Loading Per Foot of Filter Strip Width

Using equation 3.3.1:

$q = 0.00236/0.25 * (1.0)^{5/3} * (4)^{1/2} = 0.019$ cfs/ft

Water Quality Peak Flow

See subsection 2.1.7 for details

Compute the Water Quality Volume in inches:

$WQ_v = 1.2\ (0.05 + 0.009 * 70) = 0.82$ inches

Compute modified CN for 1.2-inch rainfall (P=1.2):

$$\begin{aligned} CN &= 1000/[10+5P+10Q-10(Q^2+1.25*Q*P)^{½}] \\ &= 1000/[10+5*1.2+10*0.82-10(0.82^2+1.25*0.82*1.2)^{½}] \\ &= 96.09 \text{ (Use CN = 96)} \end{aligned}$$

For CN = 96 and an estimated time of concentration (T_c) of 8 minutes (0.13 hours), compute the Q_{wq} for a 1.2 inch storm.

From Section 2.1, $I_a = 0.083$, therefore $I_a/P = 0.083/1.2 = 0.069$.

For a Type II storm (using the limiting values) $q_u = 950$ csm/in, and therefore:

Q_{wq} = (950 csm/in) (0.34ac/640ac/mi^2) (0.82") = 0.41 cfs

Minimum Filter Width

Using equation 3.3.2:

$W_{fMIN} = Q/q = 0.41/0.019 = 22$ feet

Since the width of the lot is 100 feet, the actual width of the filter strip will depend on site grading and the ability to deliver the drainage to the filter strip in sheet flow through a pea gravel filled trench.

Filter without Berm

- 2-year, 24-hour storm (see Appendix A) = 0.17 in/hr or 0.17*24= 4.08 inches
- Use 5 minute travel (contact) time

Using equation 3.3.3:

$L_f = (5)^{1.25} * (4.08)^{0.625} * (4)^{0.5} / (3.34 * 0.25) = 43$ feet

Note: Reducing the filter strip slope to 2% and planting a denser grass (raising the Manning n to 0.35) would reduce the filter strip length to 22 feet. Sensitivity to slope and Manning's n changes are illustrated for this example in Figure 3.3.1-2.

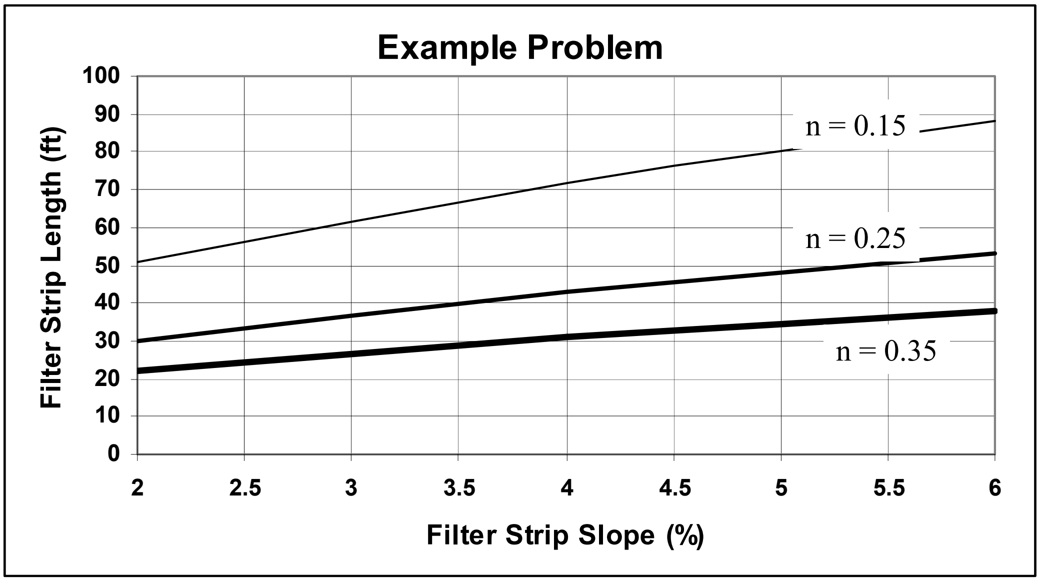

Figure 3.3.1-2 Example Problem Sensitivity of Filter Strip Length to Slope and Manning's n Values

Filter With Berm

- Pervious berm height is 6 inches

Compute the Water Quality Volume in cubic feet:

$WQ_v = R_v * 1.2/12 * A = (0.05 + 0.009 * 70) * 1.2/12 * 0.34 = 0.023$ Ac-ft or 1,007 ft^3

For a berm height of 6 inches the "wedge" of volume captured by the filter strip is:

Volume $= W_f * ½ * L_f * 0.5 = 0.25W_fL_f = 1{,}007\ ft^3$

For a maximum width of the filter of 100 feet, the length of the filter would then be 40 feet.

For a 1-foot berm height, the length of the filter would be 20 feet.

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3.3.2 Grass Channel

Limited Application
Structural Stormwater Control

Description: Vegetated open channels designed to filter stormwater runoff and meet velocity targets for the water quality design storm and the 2-year storm event.

REASONS FOR LIMITED USE

- Cannot alone achieve the 80% TSS removal target

KEY CONSIDERATIONS

- Can be used as part of the runoff conveyance system to provide pretreatment
- Grass channels 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%; slopes between 1% and 2% recommended
- Ineffective unless carefully designed to achieve low flow rates in the channel (<1.0 ft/s)
- Potential for bottom erosion and resuspension
- Standing water may not be acceptable in some areas

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality**
- [ ] **Channel/Flood Protection**

SPECIAL APPLICATIONS

- [x] **Pretreatment**
- [ ] **High Density/Ultra-Urban**
- [x] **Other:** Curb and gutter replacement

Residential Subdivision Use: *Yes*

3.3.2.1 General Description

Grass channels, also termed “biofilters,” are typically designed to provide nominal treatment of runoff as well as meet runoff velocity targets for the water quality design storm. 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 differ from the enhanced dry swale design in that they do not have an engineered filter media to enhance pollutant removal capabilities and therefore have a lower pollutant removal rate than for a dry or wet (enhanced) swale. Grass channels can partially infiltrate runoff from small storm events in areas with pervious soils. When properly incorporated into an overall site design, grass channels can reduce impervious cover, accent the natural landscape, and provide aesthetic benefits.

When designing a grass channel, the two primary considerations are channel capacity and minimization of 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.

3.3.2.2 Pollutant Removal Capabilities

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 – 50%**
- **Total Phosphorus – 25%**
- **Total Nitrogen – 20%**
- **Fecal Coliform – insufficient data**
- **Heavy Metals – 30%**

3.3.2.3 Design Criteria and Specifications

- Grass channels should generally be used to treat small drainage areas of less than 5 acres. If the practices are used on larger drainage areas, the flows and volumes through the channel become too large to allow for filtering and infiltration of runoff.

- Grass channels should be designed on relatively flat slopes of less than 4%; channel slopes between 1% and 2% are recommended.

- 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 0.27 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 2.1.7).

- Grass channels should have a trapezoidal or parabolic cross section with relatively flat side slopes (generally 3:1 or flatter).

- The bottom of the channel should be between 2 and 6 feet wide. The minimum width ensures a minimum filtering surface for water quality treatment, and the maximum width prevents braiding, which is the formation of small channels within the swale 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.

- Runoff velocities must be nonerosive. The full-channel design velocity will typically govern.

- 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).

- The depth from the bottom of the channel to the groundwater should be at least 2 feet to prevent a moist swale bottom, or contamination of the groundwater.

- Incorporation of check dams within the channel will maximize retention time.

- Designers should choose a grass that can withstand relatively high velocity flows at the entrances, and both wet and dry periods. See Appendix F for a list of appropriate grasses for use in Georgia.

See Section 4.4 (*Open Channel Design*) for more information and specifications on the design of grass channels.

Grass Channels for Pretreatment

A number of other structural controls, 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 3.3.2-1 provides sizing guidance for grass channels for a 1-acre drainage area. The minimum grassed channel length should be 20 feet.

Table 3.3.2-1 Bioretention 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

3.3.2.4 Inspection and Maintenance Requirements

Table 3.3.2-2 Typical Maintenance Activities for Grass Channels
(Source: Adapted from CWP, 1996)

Activity	Schedule
• Mow grass to maintain a height of 3 to 4 inches.	As needed (frequently/seasonally)
• Remove sediment build-up within the bottom of the grass channel once it has accumulated to 25% of the original design volume.	As needed (Infrequently)
• Inspect grass along side slopes for erosion and formation of rills or gullies and correct. • Remove trash and debris accumulated in the channel. • Based on inspection, plant an alternative grass species if the original grass cover has not been successfully established.	Annually (Semi-annually the first year)

3.3.2.5 Example Schematics

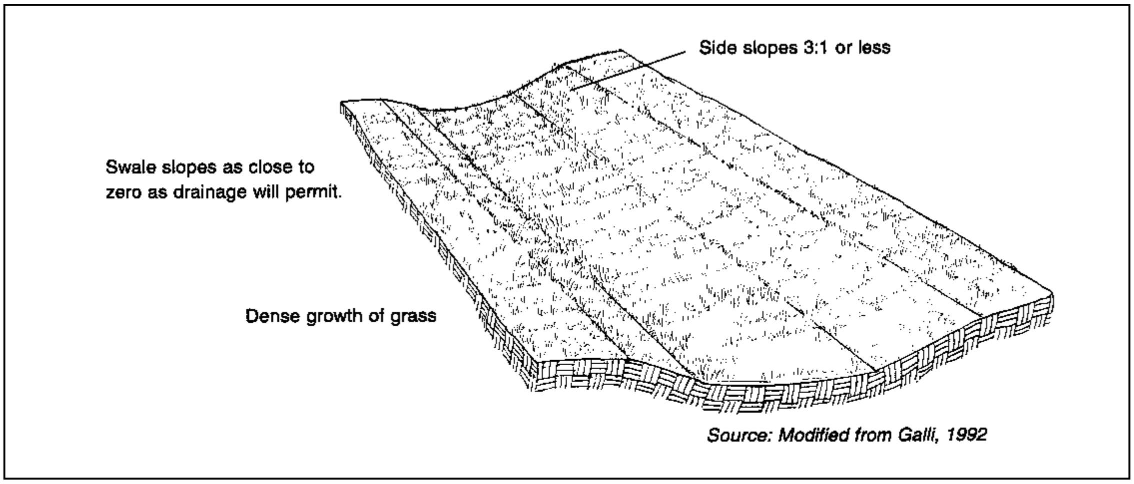


Figure 3.3.2-1 Typical Grass Channel

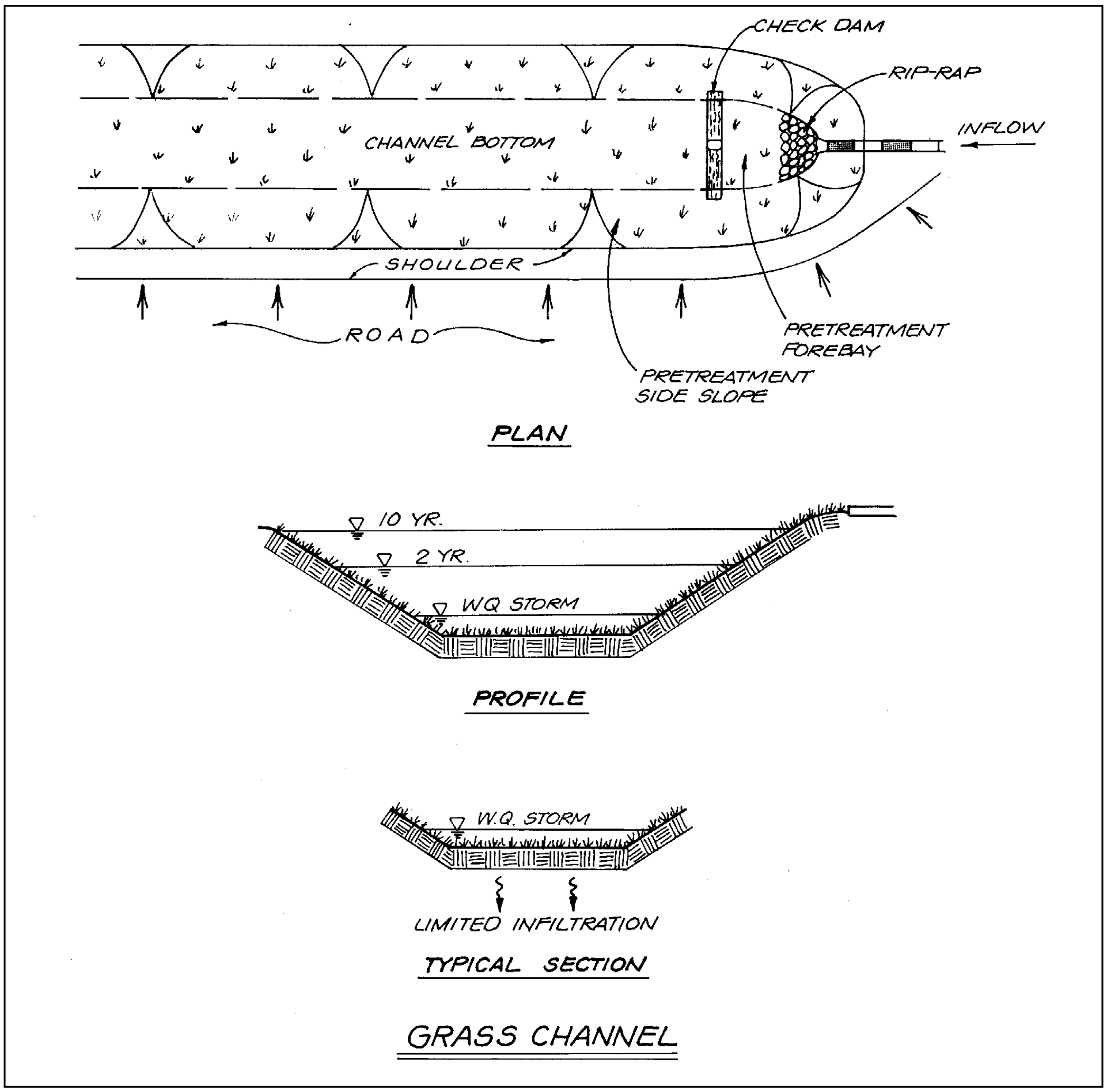


Figure 3.3.2-2 Schematic of Grass Channel

3.3.2.6 Design Example

Basic Data

Small commercial lot 300 feet deep x 145 feet wide located in Athens

- Drainage area (A) = 1.0 acres
- Impervious percentage (I) = 70%

Water Quality Peak Flow

See subsection 2.1.7 for details

Compute the Water Quality Volume in inches:

WQ_v = 1.2 (0.05 + 0.009 * 70) = 0.82 inches

Compute modified CN for 1.2-inch rainfall (P=1.2):

$$
\begin{aligned}
CN &= 1000/[10+5P+10Q-10(Q^2+1.25*Q*P)^{½}] \\
&= 1000/[10+5*1.2+10*0.82-10(0.82^2+1.25*0.82*1.2)^{½}] \\
&= 96.09 \text{ (Use CN = 96)}
\end{aligned}
$$

For CN = 96 and an estimated time of concentration (T_c) of 8 minutes (0.13 hours), compute the Q_{wq} for a 1.2-inch storm.

From Figure 2.1.5-3, I_a = 0.083, therefore I_a/P = 0.083/1.2 = 0.069.

From Figure 2.1.5-6 for a Type II storm (using the limiting values) q_u = 950 csm/in, and therefore:

Q_{wq} = (950 csm/in) (1.0ac/640ac/mi^2) (0.82") = 1.22 cfs

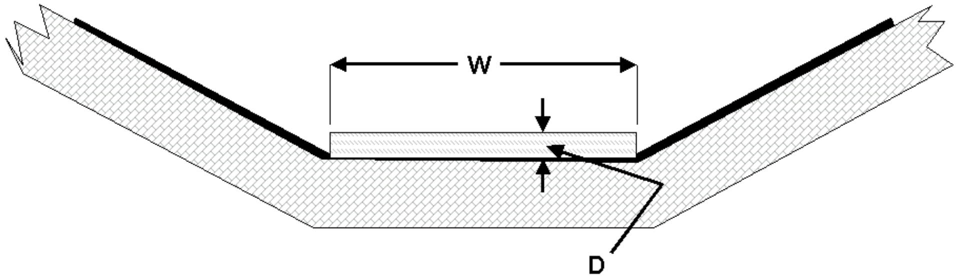
Utilize Q_{wq} to Size the Channel

The maximum flow depth for water quality treatment should be approximately the same height of the grass. A maximum flow depth of 4 inches is allowed for water quality design. A maximum flow velocity of 1.0 foot per second for water quality treatment is required. For Manning's n use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass. Site slope is 2%.

Input variables:
n = 0.15
S = 0.02 ft/ft
D = 4/12 = 0.33 ft

Then: $Q_{wq} = Q = VA = 1.49/n\ D^{2/3}\ S^{1/2}\ DW$

Where:
Q = peak flow (cfs)
V = velocity (ft/sec)
A = flow area (ft^2) = WD
W = channel bottom width (ft)
D = flow depth (ft)
S = slope (ft/ft)
(Note: D approximates hydraulic radius for shallow flows)



Then for a known n, Q, D and S minimum width can be calculated.

$(nQ)/(1.49\ D^{5/3}\ S^{1/2}) = W = (0.15*1.22)/(1.49*0.33^{5/3}*0.02^{1/2}) = 5.5$ feet minimum

$V = Q/(WD) = 1.22/(4.0 * 4/12) = 0.92$ fps (okay)

(Note: WD approximates flow area for shallow flows.)

Minimum length for 5-minute residence time, $L = V * (5*60) = 201$ feet

Depending on the site geometry, the width or slope or density of grass (Manning's n value) might be adjusted to slow the velocity and shorten the channel in the next design iteration. For example, using a 9.3-foot bottom width* of flow and a Manning's n of 0.25, solve for new depth and length.

$$Q = VA = 1.49/n\ D^{5/3}\ S^{1/2}\ W$$

$$D = [(Q * n)/(1.49 * S^{1/2} * W)]^{3/5}$$

$$= [(1.22 * 0.25)/(1.49 * 0.02^{1/2} * 9.3)]^{3/5} = 0.33 \text{ ft} = 4\text{" (okay)}$$

$$V = Q/WD = 1.22/(9.3 * 0.33) = 0.40 \text{ feet per second}$$

$$L = V * 5 * 60 = 120 \text{ feet}$$

** In this case a dividing berm should be used to control potential braiding.*

Refer to Section 4.4 (*Open Channel Design*) to complete the grass channel design for a specified design storm event.

3.3.3 Organic Filter

Limited Application
Structural Stormwater Control

Description: Design variant of the surface sand filter using organic materials in the filter media.

REASONS FOR LIMITED USE	STORMWATER MANAGEMENT SUITABILITY
• Intended for hotspot or space-limited applications, or for areas requiring enhanced pollutant removal capability • High maintenance requirements	☑ **Water Quality** ☐ **Channel/Flood Protection**
KEY CONSIDERATIONS	**SPECIAL APPLICATIONS**
• High pollutant removal capability • Removal of dissolved pollutants is greater than sand filters due to cation exchange capacity • Filter may require more frequent maintenance than most of the other stormwater controls • Minimum head requirement of 5 to 8 feet • Severe clogging potential if exposed soil surfaces exist upstream	☐ **Pretreatment** ☑ **High Density/Ultra-Urban** ☑ **Other:** Hotspot areas **Residential Subdivision Use**: *No*

3.3.3.1 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.

3.3.3.2 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%**

3.3.3.3 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 3.3.3-1). 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 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.

- Consult the design criteria for the surface sand filter (see subsection 3.2.4, *Sand Filters*) for the organic filter sizing and design steps.

3.3.3.4 Inspection and Maintenance Requirements

The inspection and maintenance requirements for organic filters are similar to those for surface sand filter facilities (see subsection 3.2.4)

3.3.3.5 Example Schematic

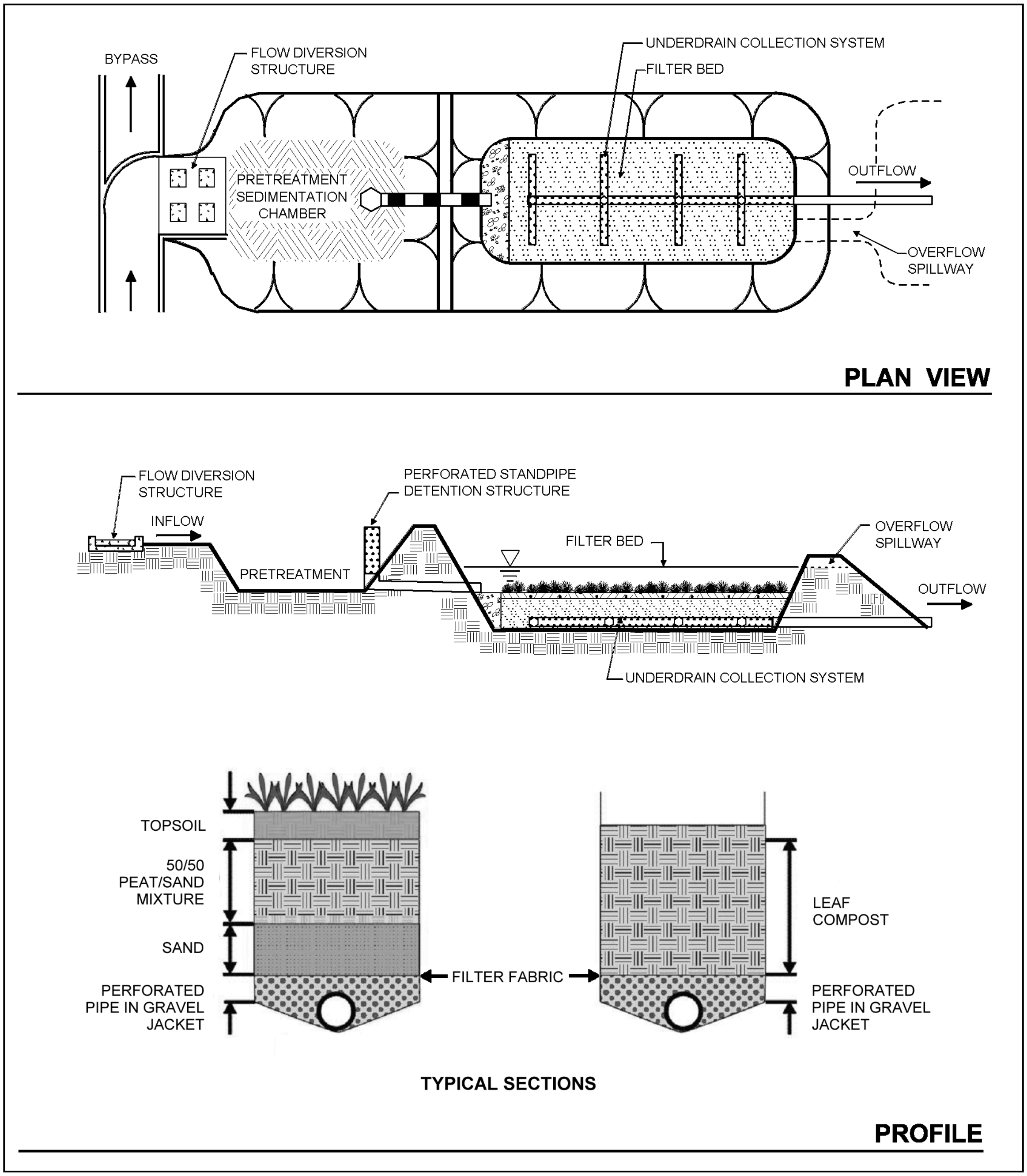


Figure 3.3.3-1 Schematic of Organic Filter

(Source: Center for Watershed Protection)

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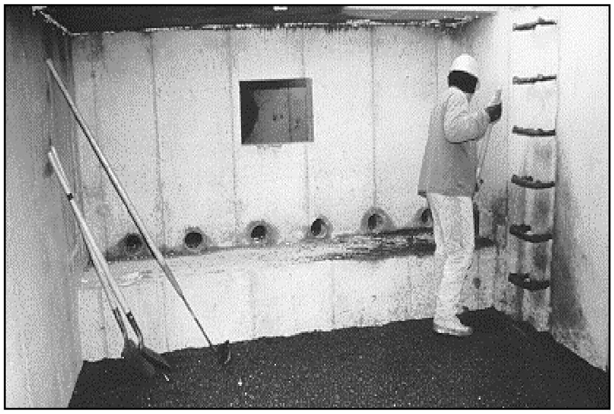
3.3.4 Underground Sand Filter

Limited Application
Structural Stormwater Control



Description: Design variant of the sand filter located in an underground vault.

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REASONS FOR LIMITED USE

- Intended for space-limited applications
- High maintenance requirements

KEY CONSIDERATIONS

- High pollutant removal capability
- Filter may require more frequent maintenance than most of the other stormwater controls
- High removal rates for sediment, BOD, and fecal coliform bacteria
- Precast concrete shells available, which decrease construction costs

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STORMWATER MANAGEMENT SUITABILITY

☑ **Water Quality**
☐ **Channel/Flood Protection**

SPECIAL APPLICATIONS

☐ **Pretreatment**
☑ **High Density/Ultra-Urban**
☑ **Other:** Hotspot areas

Residential Subdivision Use: *No*

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3.3.4.1 General Description

The underground sand filter is a design variant of the sand filter located in an underground vault designed for high-density land use or ultra-urban applications where there is not enough space for a surface sand filter or other structural stormwater controls.

The underground sand filter is a three-chamber system. The initial chamber is a sedimentation (pretreatment) chamber that temporarily stores runoff and utilizes a wet pool to capture sediment. The sedimentation chamber is connected to the sand filter chamber by a submerged wall that protects the filter bed from oil and trash. The filter bed is 18 to 24 inches deep and may have a protective screen of gravel or permeable geotextile to limit clogging. The sand filter chamber also includes an underdrain system with inspection and clean out wells. Perforated drain pipes under the sand filter bed extend into a third chamber that collects filtered runoff. Flows beyond the filter capacity are diverted through an overflow weir.

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.

3.3.4.2 Pollutant Removal Capabilities

Underground sand filter pollutant removal rates are similar to those for surface and perimeter sand filters (see subsection 3.2.4, *Sand Filters*).

3.3.4.3 Design Criteria and Specifications

- Underground sand filters are typically used on highly impervious sites of 1 acre or less. The maximum drainage area that should be treated by an underground sand filter is 5 acres.

- Underground sand filters are typically constructed on-line, but can be constructed off-line. For off-line construction, the overflow between the second and third chambers is not included.

- The underground vault should be tested for water tightness prior to placement of filter layers.

- Adequate maintenance access must be provided to the sedimentation and filter bed chambers.

- Compute the minimum wet pool volume required in the sedimentation chamber as:

$$V_w = A_s * 3 \text{ feet minimum}$$

- Consult the design criteria for the perimeter sand filter (see Section 3.2.4) for the rest of the underground filter sizing and design steps.

3.3.4.4 Inspection and Maintenance Requirements

Table 3.3.4-1 Typical Maintenance Activities for Underground Sand Filters
(Source: CWP, 1996)

Activity	Schedule
• Monitor water level in sand filter chamber.	Quarterly and following large storm events
• Sedimentation chamber should be cleaned out when the sediment depth reaches 12 inches.	As needed
• Remove accumulated oil and floatables in sedimentation chamber.	As needed, (typically every 6 months)

Additional inspection and maintenance requirements for organic filters are similar to those for surface sand filter facilities (see subsection 3.2.4)

3.3.4.5 Example Schematic

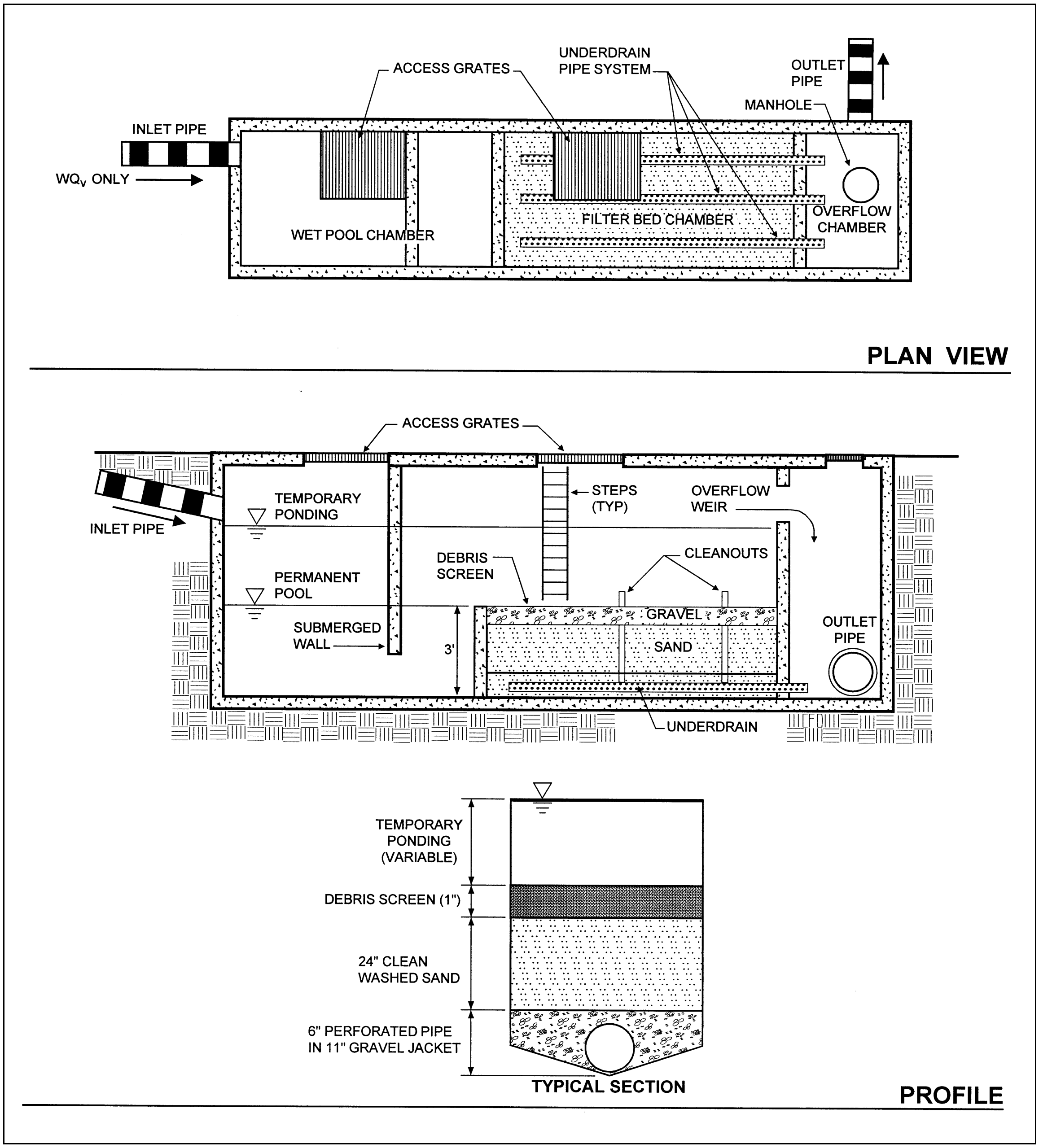

Figure 3.3.4-1 Schematic of Underground Sand Filter

(Source: Center for Watershed Protection)

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3.3.5 Submerged Gravel Wetlands

Limited Application
Structural Stormwater Control

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.

REASONS FOR LIMITED USE	STORMWATER MANAGEMENT SUITABILITY
• Intended for space-limited applications • High maintenance requirements	☑ **Water Quality** ☐ **Channel/Flood Protection**
KEY CONSIDERATIONS	**SPECIAL APPLICATIONS**
• Generally requires low land consumption, and can fit within an area that is typically devoted to landscaping • High pollutant removal capabilities are expected; however, limited performance data exist • Can be located in low-permeability soils with a high water table • Periodic sediment removal required to prevent clogging of gravel base	☐ **Pretreatment** ☑ **High Density/Ultra-Urban** ☑ **Other:** Hotspot areas **Residential Subdivision Use**: *No*

3.3.5.1 General Description

The submerged gravel wetland system consists of one or more treatment cells that are filled with crushed rock or gravel and is 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 structural control relies on the pollutant-stripping ability of plants and soils to remove pollutants from runoff.

3.3.5.2 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 60% 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 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 – 50%**
- **Total Nitrogen – 20%**
- **Fecal Coliform – 70%**
- **Heavy Metals – 50%**

3.3.5.3 Design Criteria and Specifications

- Submerged gravel wetlands should be designed as off-line systems designed to handle only water quality volume.

- Submerged gravel wetland systems need sufficient drainage area to maintain vegetation. See subsection 2.1.8 for guidance on performing a water balance calculation.

- The local slope should be relatively flat (<2%). 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 3 to 5 feet).

- All submerged gravel wetland designs should include a sediment forebay or other equivalent pretreatment measures to prevent sediment or debris from entering and clogging the gravel bed.

- Unless they receive hotspot runoff, submerged gravel wetland systems can be allowed to intersect the groundwater table.

- Guidance on establishing wetland vegetation can be found in Appendix F, *Landscaping and Aesthetics Guidance*.

- See subsection 3.2.2 (*Stormwater Wetlands*) for additional planning and design guidance.

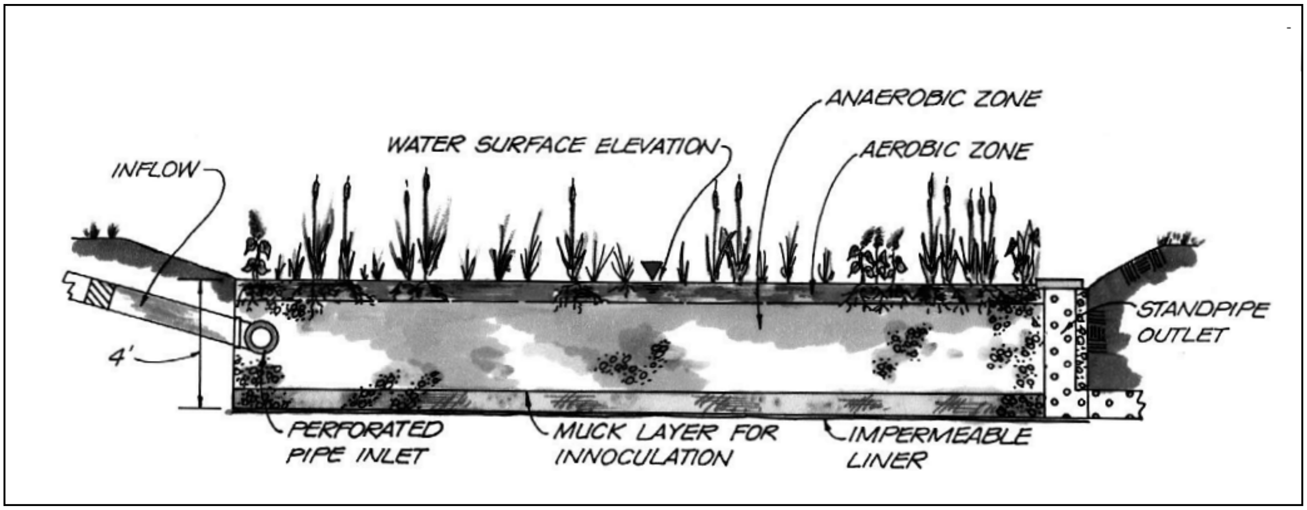
3.3.5.4 Inspection and Maintenance Requirements

Table 3.3.5-1 Typical Maintenance Activities for Submerged Gravel Wetland Systems

Activity	Schedule
• Ensure that inlets and outlets to each submerged gravel wetland cell are free from debris and not clogged.	Monthly
• Check for sediment buildup in gravel bed .	Annual inspection
• If sediment buildup is preventing flow through the wetland, remove gravel and sediment from cell. Replace with clean gravel and replant vegetation.	As needed

Additional inspection and maintenance requirements for submerged gravel wetland systems are similar to those for stormwater wetlands (see subsection 3.2.2).

3.3.5.5 Example Schematic



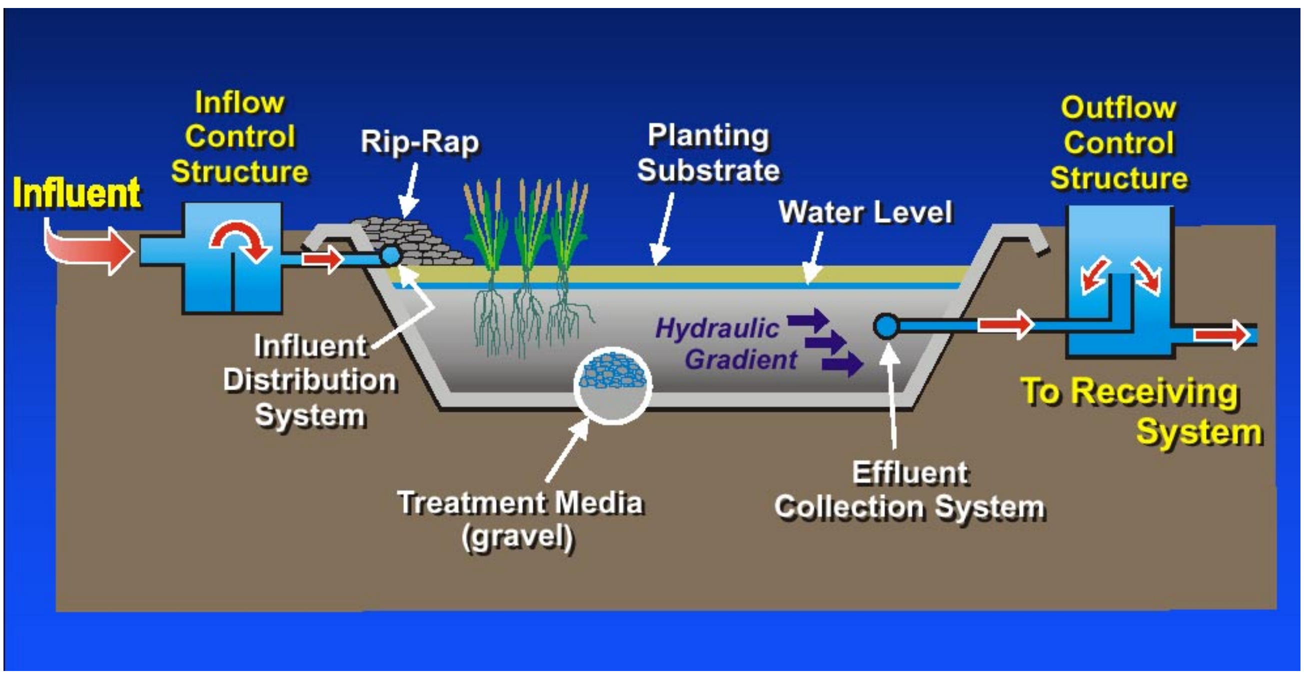


Figure 3.3.5-1 Schematics of Submerged Gravel Wetland System

(Sources: Center for Watershed Protection; Roux Associates Inc.)

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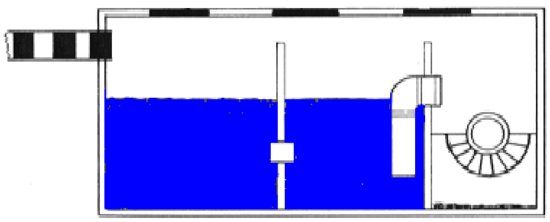
3.3.6 Gravity (Oil-Grit) Separator

Limited Application
Structural Stormwater Control



Description: Hydrodynamic separation device designed to remove settleable solids, oil and grease, debris and floatables from stormwater runoff through gravitational settling and trapping of pollutants.

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REASONS FOR LIMITED USE

- Cannot alone achieve the 80% TSS removal target
- Intended for hotspot, space-limited or pretreatment applications
- Limited performance data

KEY CONSIDERATIONS

- Intended for the removal of settleable solids (grit and sediment) and floatable matter, including oil and grease
- Dissolved pollutants are not effectively removed
- Frequent maintenance required
- Performance dependent on design and frequency of inspection and cleanout of unit

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STORMWATER MANAGEMENT SUITABILITY

☑ **Water Quality**
☐ **Channel/Flood Protection**

SPECIAL APPLICATIONS

☑ **Pretreatment**
☑ **High Density/Ultra-Urban**
☑ **Other:** Hotspot areas

Residential Subdivision Use: *No*

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3.3.6.1 General Description

Gravity separators (also known as oil-grit separators) are hydrodynamic separation devices that are designed to remove grit and heavy sediments, oil and 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/storage chamber, a bypass chamber, and an access port for maintenance purposes. Runoff enters the inlet chamber where heavy sediments and solids drop out. 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 the specific gravity of water and the specific gravities of petroleum compounds. 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 into the unit.

Gravity separators are best 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 pollutnants 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.

3.3.6.2 Pollutant Removal Capabilities

Testing of gravity separators has shown that they can remove between 40 and 50% of the TSS loading when used in an off-line configuration (Curran, 1996 and Henry, 1999). Gravity 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.

3.3.6.3 Design Criteria and Specifications

- The use of gravity (oil-grit) separators should be limited to the following applications:
  - Pretreatment for other structural stormwater controls
  - High-density, ultra urban or other space-limited development sites
  - Hotspot areas where the control of grit, floatables, and/or oil and grease are required

- Gravity separators are typically used for 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.

- Gravity separator systems can be installed in almost any soil or terrain. Since these devices are underground, appearance is not an issue and public safety risks are low.

- Gravity separators are rate-based devices. This contrasts with most other stormwater structural controls, which are sized based on capturing and treating a specific volume.

- 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.

- The separation chamber should provide for three separate storage volumes:

  (1) A volume for separated oil storage at the top of the chamber

  (2) A volume for settleable solids accumulation at the bottom of the chamber

(3) A volume required to give adequate flow-through detention time for separation of oil and sediment from the stormwater flow

- The total wet storage of the gravity separator unit should be at least 400 cubic feet per contributing impervious acre.
- The minimum depth of the permanent pools should be 4 feet.
- Horizontal velocity through the separation chamber should be 1 to 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 to prevent debris from becoming oil impregnated.
- Ideally, a gravity separator design will provide an oil draw-off mechanism to a separate chamber or storage area.
- Adequate maintenance access to each chamber must be provided for inspection and cleanout of a gravity separator unit.
- Gravity separator units should be watertight to prevent possible groundwater contamination.
- The design criteria and specifications of a proprietary gravity separator unit should be obtained from the manufacturer.

3.3.6.4 Inspection and Maintenance Requirements

Table 3.3.6-1 Typical Maintenance Activities for Gravity Separators

Activity	Schedule
• Inspect the gravity separator unit.	Regularly (quarterly)
• Clean out sediment, oil and grease, and floatables, using catch basin cleaning equipment (vacuum pumps). Manual removal of pollutants may be necessary.	As Needed

Additional Maintenance Considerations and Requirements

- Additional maintenance requirements for a proprietary system should be obtained from the manufacturer.
- Failure to provide adequate inspection and maintenance can result in the resuspension of accumulated solids. Frequency of inspection and maintenance is dependent on land use, climatological conditions, and the design of gravity separator.
- Proper disposal of oil, solids and floatables removed from the gravity separator must be ensured.

3.3.6.5 Example Schematic

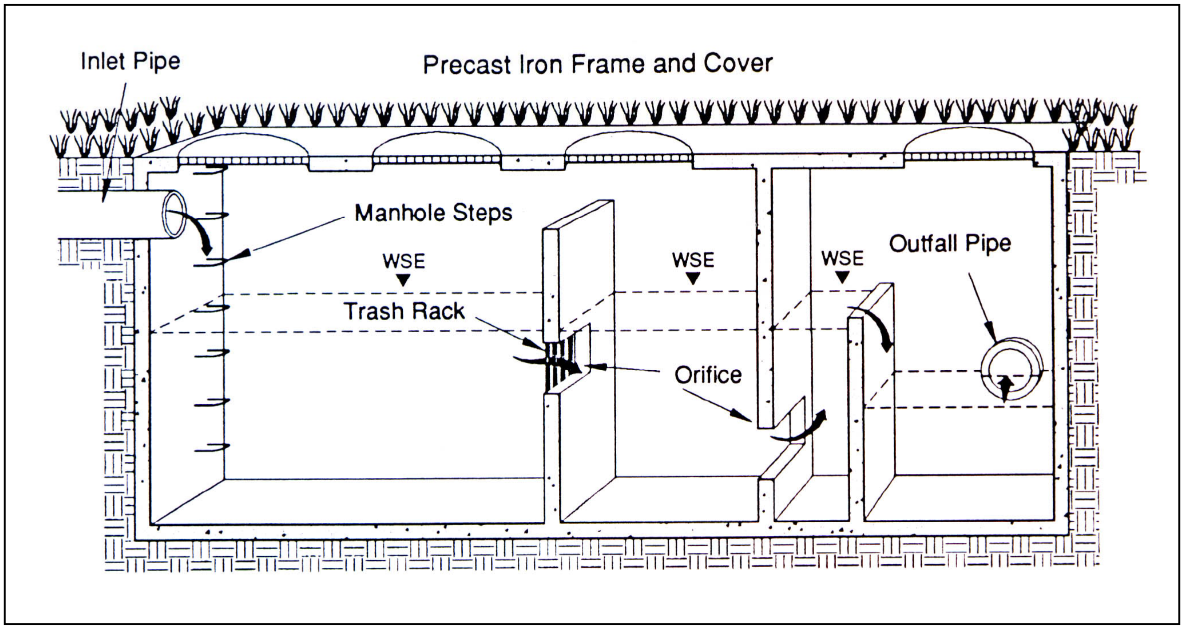
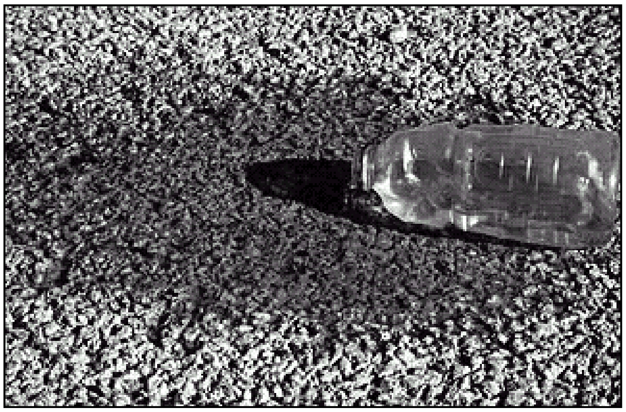


Figure 3.3.6-1 Schematic of an Example Gravity (Oil-Grit) Separator
(Source: NVRC, 1992[1])

3.3.7 Porous Concrete

Limited Application
Structural Stormwater Control



Description: Porous concrete is the term for 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.

REASONS FOR LIMITED USE

- Traditionally high failure rate and short life span
- Intended for low volume auto traffic areas, or for overflow parking applications
- High maintenance requirements
- Special attention to design and construction needed
- Should not be used in areas of soils with low permeability, wellhead protection zones, or recharge areas of water supply aquifer recharge areas.
- Restrictions on use by heavy vehicles

KEY CONSIDERATIONS

- Soil infiltration rate of 0.5 in/hr or greater required
- Excavated area filled with stone media; gravel and sand filter layers with observation well
- Pre-treat runoff if sediment present
- Provides reduction in runoff volume
- Somewhat higher cost when compared to conventional pavements
- Potential for high failure rate if poorly designed, poorly constructed, not adequately maintained or used in unstabilized areas
- Potential for groundwater contamination

STORMWATER MANAGEMENT SUITABILITY

☑ **Water Quality**

✪ **Channel/Flood Protection**

SPECIAL APPLICATIONS

☐ **Pretreatment**

☑ **High Density/Ultra-Urban**

☑ **Other: Overflow Parking, Driveways & related uses**

Residential Subdivision Use: *Yes*
(in common areas that are maintained)

✪ in certain situations

3.3.7.1 General Description

Porous 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. Porous concrete is thought to have a greater ability than porous asphalt to maintain its porosity in hot weather and thus is provided as a limited application control. Although, porous concrete has seen growing use in Georgia, there is still very limited practical experience with this measure. According to the U.S. EPA, porous pavement sites have had a high failure rate – approximately 75 percent. Failure has been attributed to poor design, inadequate construction techniques, soils with low permeability, heavy vehicular traffic and poor maintenance. This measure, if used, should be carefully monitored over the life of the development.

Porous concrete consists of a specially formulated mixture of portland cement, uniform, open graded course aggregate, and water. The concrete layer has a high permeability, often many times that of the underlying permeable soil layer, and allows rapid percolation of rainwater through the surface and into the layers beneath. The void space in porous concrete is in the 15% to 22% range compared to three to five percent 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.

Porous concrete is designed primarily for stormwater quality, i.e. the removal of stormwater pollutants. However, they can provide limited runoff quantity control, particularly for smaller storm events. For some smaller sites, trenches can be designed to capture and infiltrate the channel protection volume (Cp_v) in addition to WQ_v. Porous concrete will need to be used in conjunction with another structural control to provide overbank and extreme flood protection, if required.

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 or treatment. These include:

- 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

The infiltration rate of the soils in the subgrade should be adequate to support drawdown of the entire runoff capture volume within 24 to 48 hours. Special care must be taken during construction to avoid undue compaction of the underlying soils which could affect the soils' infiltration capability.

Porous concrete systems are typically used in low-traffic areas such as the following types of applications:

- 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

Slopes should be flat or gentle to facilitate infiltration versus runoff and the seasonally high water table or bedrock should be a minimum of two feet below the bottom of the gravel layer if infiltration is to be relied on to remove the stored volume.

Porous 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, a fact that may offset the greater placement cost somewhat.

A drawback is the cost and complexity of porous concrete systems compared to conventional pavements. Porous concrete systems require a very high level of construction workmanship to ensure that they function as designed. They experience a high failure rate if they are not designed, constructed and maintained properly.

Like other infiltration controls, porous concrete should not be used in areas that experience high rates of wind erosion or in drinking water aquifer recharge areas.

3.3.7.2 Pollutant Removal Capabilities

As they provide for the infiltration of stormwater runoff, porous concrete systems have a high removal of both soluble and particulate pollutants, where they become trapped, absorbed or broken down in the underlying soil layers. Due to the potential for clogging, porous concrete surfaces should not be used for the removal of sediment or other coarse particulate pollutants.

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 – not applicable**
- **Total Phosphorus – 50%**
- **Total Nitrogen – 65%**
- **Fecal Coliform – insufficient data**
- **Heavy Metals – 60%**

Pollutant removal can be improved through routine vacuum sweeping and high pressure washing, insuring a drainage time of at least 24 hours, pretreating the runoff, having organic material in the subsoil, and using clean washed aggregate (EPA, 1999).

3.3.7.3 Design Criteria and Specifications

- Porous concrete systems can be used where the underlying in-situ subsoils have an infiltration rate greater than 0.5 inches per hour. Therefore, porous concrete 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 the soils.

- Porous concrete systems should typically be used in applications where the pavement receives tributary runoff only from impervious areas. Actual pervious surface area sizing will depend on achieving a 24 hour minimum and 48 hour maximum draw down time for the design storm volume.

- 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 paver surface. Pretreatment using filter strips or vegetated swales for removal of course sediments is recommended. (see sections 3.3.1 and 3.3.2)

- Porous concrete systems should not be used on slopes greater than 5% with slopes of no greater than 2% recommended. For slopes greater than 1% barriers perpendicular to the direction of drainage should be installed in sub-grade material to keep it from washing away, or filter fabric should be placed at the bottom and sides of the aggregate to keep soil from migrating into the aggregate and reducing porosity.

- 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.

- Porous 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. Porous concrete should not be used for manufacturing and industrial sites, where there is a potential for high concentrations of soluble pollutants and heavy metals. In addition, porous concrete should not be considered for areas with a high pesticide concentration. Porous concrete is also not suitable in areas with karst geology without adequate geotechnical testing by qualified individuals and in accordance with local requirements.

- Porous concrete system designs 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 concrete system, or if the surface clogs.

- For the purpose of sizing downstream conveyance and structural control system, porous concrete surface areas can be assumed to 35% impervious. In addition, credit can be taken for the runoff volume infiltrated from other impervious areas using the methodology in Section 3.1.

- 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, the aggregate reservoir, and the sub-grade above the seasonal high water table – if the sub-grade is sandy. The storm duration (fill time) is normally short compared to the infiltration rate of the sub-grade, a duration of <u>two hours</u> can be used for design purposes. The total storage volume in a layer is equal to the percent of voids times the volume of the layer. Alternately storage may be created on the surface through temporary ponding, though this would tend to accelerate clogging if course sediment or mud settles out on the surface.

- The cross-section typically consists of four layers, as shown in Figure 3.3.7-1. 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. Descriptions of each of the layers is presented below:

  <u>Porous Concrete Layer</u> – The porous concrete layer consists of an open-graded concrete mixture usually ranging from depths of 2 to 4 inches depending on required bearing strength and pavement design requirements. Porous concrete can be assumed to contain 18 percent voids (porosity = 0.18) for design purposes. Thus, for example, a 4 inch thick porous concrete layer would hold 0.72 inches of rainfall. The omission of the fine aggregate provides the porosity of the porous pavement. To provide a smooth riding surface and to enhance handling and placement a coarse aggregate of 3/8 inch maximum size is normally used. Use GDOT No. 8 coarse aggregate (3/8 to No. 16) per ASTM C 33 or No. 89 coarse aggregate (3/8 to No. 50) per ASTM D 448. See the GCPA specifications (referenced).

  <u>Top Filter Layer</u> – Consists of a 0.5 inch diameter crushed stone to a depth of 1 to 2 inches. This layer serves to stabilize the porous asphalt layer. Can be combined with reservoir layer using suitable stone.

  <u>Reservoir Layer</u> – The reservoir gravel base course consists of washed, bank-run gravel, 1.5 to 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 a function of the soil infiltration rate and void spaces, but typically ranges from two to four feet. The layer must have a minimum depth of nine inches. The layer should be designed to drain completely in 48 hours. layer should be designed to store at a minimum the water quality volume (WQ_v). 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. .

  <u>Bottom Filter Layer</u> – The surface of the subgrade should be an 6 inch layer of sand (ASTM C-33 concrete sand or GADOT Fine Aggregate Size No. 10) or a 2 inch thick layer of 0.5 inch crushed stone, and be completely flat to promote infiltration across the entire surface. This layer serves to stabilize the reservoir layer, to protect the underlying soil from compaction, and act as the interface between the reservoir layer and the filter fabric covering the underlying soil.

  <u>Filter Fabric</u> – It is very important to line the entire trench area, including the sides, with filter fabric prior to placement of the aggregate. The filter fabric serves a very important function by inhibiting soil from migrating into the reservoir layer and reducing storage capacity. Fabric should be MIRFI # 14 N or equivalent.

Underlying Soil – The underlying soil should have an infiltration capacity of at least 0.5 in/hr, but preferably greater than 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). Infiltration trenches cannot be used in fill soils. Soils at the lower end of this range may not be suited for a full infiltration system. Test borings are recommended to determine the soil classification, seasonal high ground water table elevation, and 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 as not to jeopardize 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 sand. The sides of the trench shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling. All infiltration trench facilities should be protected during site construction, and should be constructed after upstream areas have been stabilized.

- An observation well consisting of perforated PVC pipe 4 to 6 inches in diameter should be placed at the downstream end of the facility and protected. The well should be used to determine actual infiltration rates.

- A warning sign should be placed at the facility that states, “Porous Paving used on this site to reduce pollution. Do not resurface with non-porous material. Call XXX-XXXX for more information.”

- Details of construction of the concrete layer are beyond the scope of this manual. However, construction of porous concrete is exacting, and requires special handling, timing, and placement to perform adequately (LACDPW, 2000, Paine, 1992, Maryland, 1984).

3.3.7.4 Inspection and Maintenance Requirements

Table 3.3.7-1 Typical Maintenance Activities for Porous Concrete Systems

Activity	Schedule
• Initial inspection	Monthly for three months after installation
• Ensure that the porous paver surface is free of sediment	Monthly
• Ensure that the contributing and adjacent area is stabilized and mowed, with clippings removed	As needed, based on inspection
• Vacuum sweep porous concrete surface followed by high pressure hosing to keep pores free of sediment	Four times a year
• Inspect the surface for deterioration or spalling • Check to make sure that the system dewaters between storms	Annually
• Spot clogging can be handled by drilling half-inch holes through the pavement every few feet • Rehabilitation of the porous concrete system, including the top and base course as needed	Upon failure

To ensure proper maintenance of porous pavement, a carefully worded maintenance agreement is essential. It should include specific the specific requirements and establish the responsibilities of the property owner and provide for enforcement.

3.3.7.5 Example Schematics

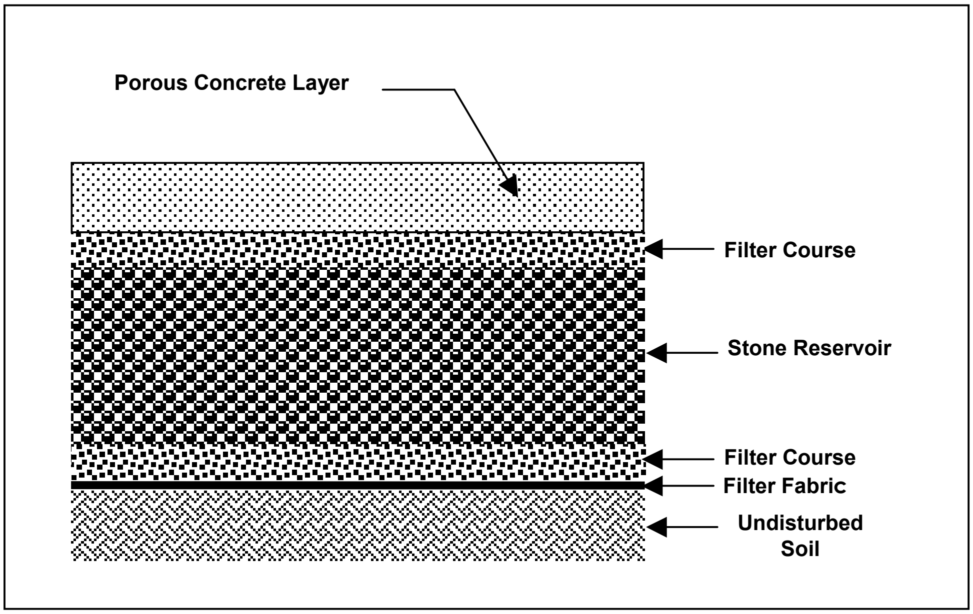


Figure 3.3.7-1 Porous Concrete System Section
(Modified From: LAC 2000)

Figure 3.3.7-2 Porous Concrete System Installation

Figure 3.3.7-3 Typical Porous Concrete System Applications
(Photos by Bruce Ferguson, Don Wade)

3.3.7.6 Design Example

Data

A 1.5 acre overflow parking area is to be designed to provide water quality treatment using porous concrete for at least part of the site to handle the runoff from the whole overflow parking area. Initial data shows:

- Borings show depth to water table is 5.0 feet
- Boring and infiltrometer tests show sand-loam with percolation rate (k) of 1.02 inches/hr
- Structural design indicates the thickness of the porous concrete must be at least three inches

Water Quality Volume

R_v = 0.05 + 0.009 I (where I = 100 percent)
= 0.95

WQ_v = 1.2 R_v A / 12 = 1.2 * 0.95 * 1.5/12 converted to cubic feet from acre-feet
= 6,207 cubic feet

Surface Area

A porosity value n = 0.32 should be used for the gravel and 0.18 for the concrete layer.

All infiltration systems should be designed to fully de-water the entire WQ_v within 24 to 48 hours after the rainfall event at the design percolation rate.

A fill time T=2 hours can be used for most designs

Chose a depth of gravel pit of three feet (including layer under concrete) which fits the site with a two foot minimum to water table (other lesser depths could be chosen, making the surface area larger). The minimum surface area of the trench can be determined, in a manner similar to the infiltration trench, from the following equation:

$$A = WQ_v/(n_g d_g + kT/12 + n_p d_p)$$

$$= 6{,}207/(0.32*3 + 1.02*2/12 + 0.18 * 3/12)$$

$$= 5{,}283 \text{ square feet}$$

Where:

A = Surface Area
WQv = Water Quality Volume (or total volume to be infiltrated)
n = porosity (g of the gravel, p of the concrete layer)
d = depth or gravel layer (feet) (g of the gravel, p of the concrete layer)
k = percolation (inches/hour)
T= Fill Time (time for the practice to fill with water), in hours

Check of drain time:
depth = 3*12 + 3 inches to sand layer = 39 inches @ 1.02 in/hr = 38 hours (ok)

Overflow will be carried across the porous concrete and tied into the drainage system for the rest of the site.

3.3.8 Modular Porous Paver Systems

Limited Application
Structural Stormwater Control

Description: A pavement surface composed of structural units with void areas that are filled with pervious materials such as sand or grass turf. Porous pavers are installed over a gravel base course that provides storage as runoff infiltrates through the porous paver system into underlying permeable soils.

REASONS FOR LIMITED USE

- Intended for low traffic areas, or for residential or overflow parking applications
- High maintenance requirements

KEY CONSIDERATIONS

- Soil infiltration rate of 0.5 in/hr or greater required
- High level of pollutant removal
- Provides reduction in runoff volume
- High cost compared to conventional pavements
- Potential for high failure rate if not adequately maintained or used in unstabilized areas
- Potential for groundwater contamination
- Available from commercial vendors

STORMWATER MANAGEMENT SUITABILITY

✓ **Water Quality**

✪ **Channel/Flood Protection**

SPECIAL APPLICATIONS

☐ **Pretreatment**

✓ **High Density/Ultra-Urban**

✓ **Other: Overflow Parking, Driveways & related uses**

Residential Subdivision Use: *Yes*

✪ in certain situations

3.3.8.1 General Description

Modular porous pavers are structural units, such as concrete blocks, bricks, or reinforced plastic mats, with regularly interdispersed void areas used to create a load-bearing pavement surface. The void areas are filled with pervious materials (gravel, sand, or grass turf) to create a system that allows for the infiltration of stormwater runoff. Porous paver systems provide water quality benefits in addition to groundwater recharge and a reduction in stormwater volume. The use of porous paver systems results in a reduction of the effective impervious area on a site.

There are many different types of modular porous pavers 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 3.3.8-1).

Modular porous pavers are typically placed on a gravel (stone aggregate) base course. Runoff infiltrates through the porous paver surface 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 to 48 hours. Special care must be taken during construction to avoid undue compaction of the underlying soils, which could affect the soils' infiltration capability.

Modular porous paver systems are typically used in low-traffic areas such as the following types of applications:

- 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 porous paver systems compared to conventional pavements. Porous 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 should they become clogged. Therefore, consideration of porous paver systems should include the construction and maintenance requirements and costs.

3.3.8.2 Pollutant Removal Capabilities

As they provide for the infiltration of stormwater runoff, porous paver systems have a high removal of both soluble and particulate pollutants, where they become trapped, absorbed or broken down in the underlying soil layers. Due to the potential for clogging, porous paver surfaces should not be used for the removal of sediment or other coarse particulate pollutants.

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 – not applicable**
- **Total Phosphorus – 80%**
- **Total Nitrogen – 80%**
- **Fecal Coliform – insufficient data**
- **Heavy Metals – 90%**

3.3.8.3 Design Criteria and Specifications

- Porous paver systems can be used where the underlying in-situ subsoils have an infiltration rate of between 0.5 and 3.0 inches per hour. Therefore, porous 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 the soils.

- Porous 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 porous 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 porous paver surface.

- Porous paver systems are not recommended on sites with a slope greater than 2%.

- 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.

- Porous paver systems should be sited at least 10 feet downgradient from buildings and 100 feet away from drinking water wells.

- An appropriate modular porous 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 porous 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 porous 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 shall not be used. A porosity value (void space/total volume) of 0.32 should be used in calculations.

- The gravel base course must have a minimum depth of 9 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:

$$\mathbf{d = V / A * n}$$

Where:

d = Gravel Layer Depth (feet)
V = Water Quality Volume –or– Total Volume to be Infiltrated
A = Surface Area (square feet)
n = Porosity (use n=0.32)

- The 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.

- Porous paver system designs 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 paver system, or if the surface clogs.

- For the purpose of sizing downstream conveyance and structural control system, porous paver surface areas can be assumed to be 35% impervious. In addition, credit can taken for the runoff volume infiltrated from other impervious areas using the methodology in Section 3.1.

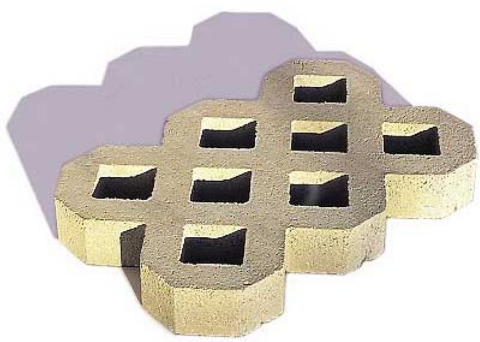
3.3.8.4 Inspection and Maintenance Requirements

Table 3.3.8-1 Typical Maintenance Activities for Modular Porous Paver Systems

Activity	Schedule
• Ensure that the porous paver surface is free of sediment. • Check to make sure that the system dewaters between storms.	Monthly
• Ensure that contributing area and porous paver surface are clear of debris. • Ensure that the contributing and adjacent area is stabilized and mowed, with clippings removed.	As needed, based on inspection
• Vacuum sweep porous paver surface to keep free of sediment.	Typically three to four times a year
• Inspect the surface for deterioration or spalling.	Annually
• Totally rehabilitate the porous paver system, including the top and base course, as needed.	Upon failure

3.3.8.5 Example Schematics

Figure 3.3.8-1 Examples of Modular Porous Pavers



Concrete Paver Block

Castellated Block

Lattice Block

Grass / Gravel Paver Mat

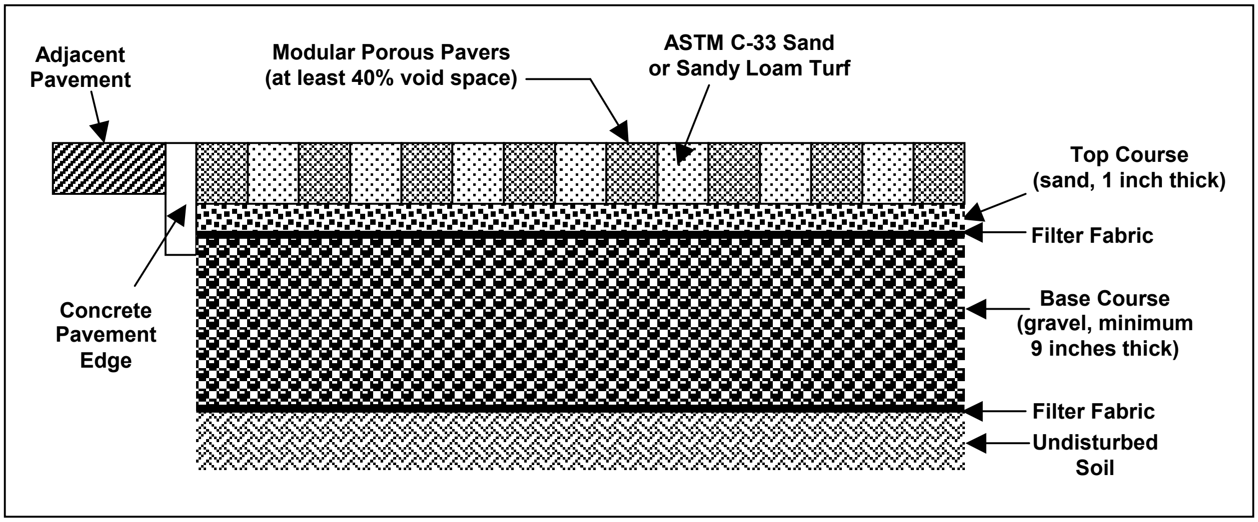


Figure 3.3.8-2 Modular Porous Paver System Section

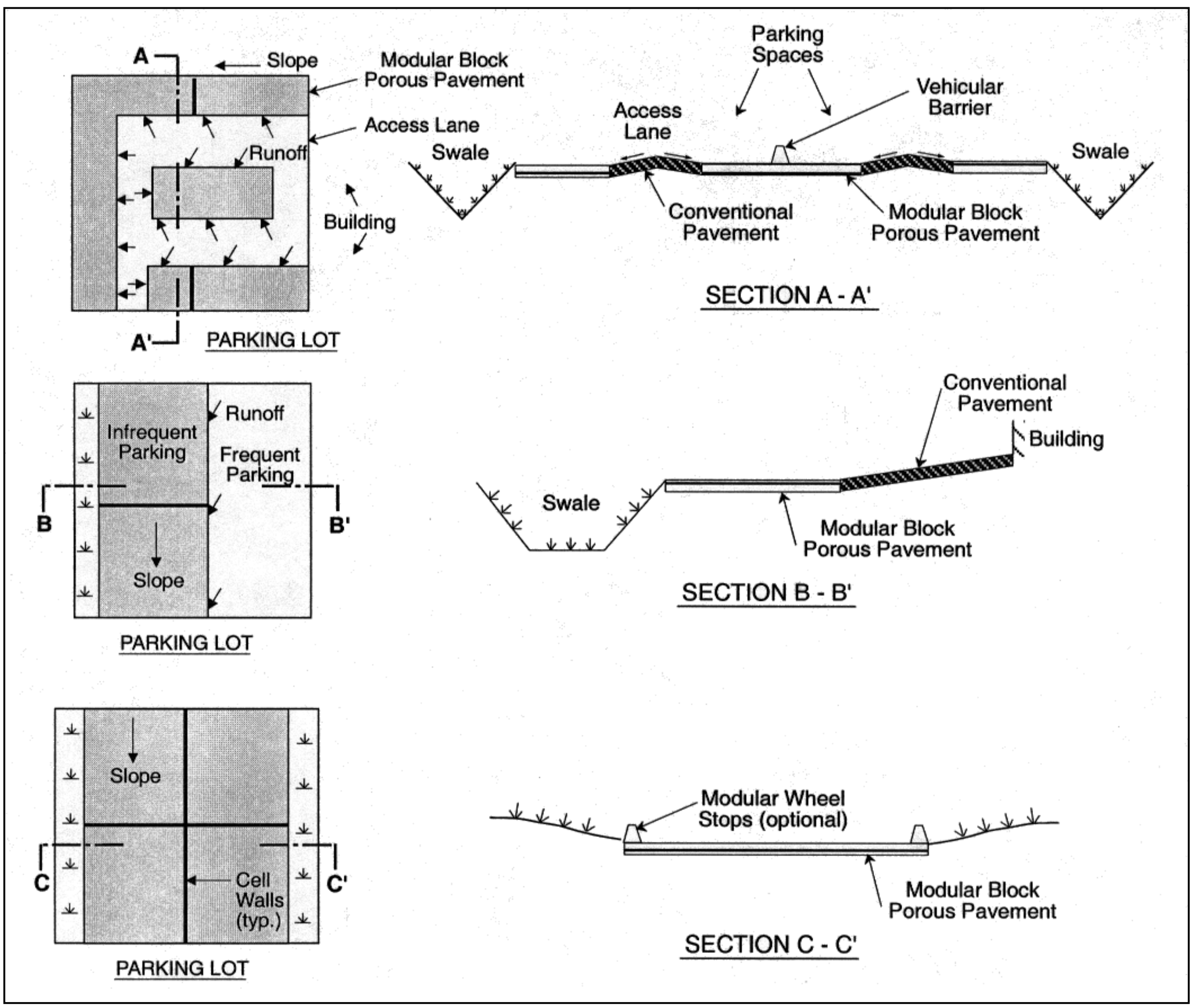


Figure 3.3.8-3 Typical Modular Porous Paver System Applications
(Source: UDFCD, 1999)

Figure 3.3.8-4 Examples of Porous Paver Surfaces
(Sources: Invisible Structures, Inc.; EP Henry Corp.)

3.3.9 Alum Treatment System

Limited Application
Structural Stormwater Control

Description: Chemical treatment of stormwater runoff entering a wet pond by injecting liquid alum into storm sewer lines on a flow-weighted basis during rain events.

REASONS FOR LIMITED USE

- Intended for areas requiring regional stormwater treatment from a piped stormwater drainage system
- High maintenance requirements

KEY CONSIDERATIONS

- Requires no additional land purchase
- Reduces concentrations of total phosphorus, total aluminum and heavy metals
- Dependent on pH level ranging from 6.0 to 7.5 during treatment process
- High capital and operations and maintenance costs

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality**
- [ ] **Channel/Flood Protection**

SPECIAL APPLICATIONS

- [ ] **Pretreatment**
- [x] **High Density/Ultra-Urban**
- [x] **Other:** Regional Treatment

Residential Subdivision Use: *Yes*

3.3.9.1 General Description

The process of alum (aluminum sulfate) treatment provides treatment of stormwater runoff from a piped stormwater drainage system entering a wet pond by injecting liquid alum into storm sewer lines on a flow-weighted basis during rain events. When added to runoff, liquid alum forms nontoxic precipitates of aluminum hydroxide [$Al(OH)_3$] and aluminum phosphate [$AlPO_4$], which combine with phosphorus, suspended solids and heavy metals, causing them to be deposited into the sediments of the receiving waters in a stable, inactive state.

The alum precipitate formed during coagulation of stormwater can be allowed to settle in receiving water or collected in small settling basins. Alum precipitates are stable in sediments and will not re-dissolve due to changes in redox potential or pH under conditions normally found in surface water bodies. Laboratory or field testing may be necessary to verify feasibility and to establish design, maintenance, and operational parameters, such as the optimum coagulant dose required to achieve the desired water quality goals, chemical pumping rates and pump sizes.

Construction costs for existing alum stormwater treatment facilities in Florida have ranged from $135,000 to $400,000. The capital construction costs of alum stormwater treatment systems is independent of watershed size and depends primarily on the number of outfall locations treated.

Estimated annual operations and maintenance (O&M) costs for chemicals and routine inspections range from approximately $6,500 to $25,000 per year. O&M costs include chemical, power, manpower for routine inspections, equipment renewal and replacement costs.

3.3.9.2 Pollutant Removal Capabilities

Alum treatment has consistently achieved a 85 to 95% reduction in total phosphorus, 90 to 95% reduction in orthophosphorus, 60 to 70% reduction in total nitrogen, 50 to 90% reduction in heavy metals, 95 to 99% reduction in turbidity and TSS, 60% reduction in BOD, and >99% reduction in fecal coliform bacteria compared with raw stormwater characteristics.

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 – 90%**
- **Total Phosphorus – 80%**
- **Total Nitrogen – 60%**
- **Fecal Coliform – 90%**
- **Heavy Metals – 75%**

3.3.9.3 Design Criteria and Specifications

Alum treatment systems are fairly complex, and design details are beyond the scope of this Manual. However, further information can be obtained from the Internet and by contacting local municipalities and engineers who have designed and implemented successful systems. The following are general guidelines for alum treatment systems:

- Injection points should be 100 feet upstream of discharge points.
- Alum concentration is typically 10 μg/l.
- Alum treatments systems may need to control pH.
- For new pond design, the required size is approximately 1% of the drainage basin size, as opposed to 10 to 15% of the drainage basin area for a standard detention pond.
- No volume requirement is required when discharging to existing lakes.

3.3.9.4 Inspection and Maintenance Requirements

Table 3.3.9-1 Typical Maintenance Activities for Alum Treatment
(Source: Harper, Herr, and Livingston)

Activity	Schedule
• Perform routine inspection. • Monitor of water quality and pH.	Monthly
• Perform maintenance of pump equipment, chemical supplies and delivery system.	As Needed

Figure 3.3.9-1. Alum Treatment System and Injection Equipment

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3.3.10 Proprietary Structural Controls

Limited Application
Structural Stormwater Control

Description: Manufactured structural control systems available from commercial vendors designed to treat stormwater runoff and/or provide water quantity control.

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REASONS FOR LIMITED USE

Depending on the proprietary system, there may be:

- Limited performance data
- Application constraints
- High maintenance requirements
- Higher costs than other structural control alternatives

KEY CONSIDERATIONS

- Independent performance data must be available to prove a demonstrated capability of meeting stormwater management goal(s)
- System or device must be appropriate for use in Georgia conditions, and specifically for the community in question
- Installation and operations/maintenance requirements must be understood by all parties approving and using the system or device in question

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STORMWATER MANAGEMENT SUITABILITY

[*] **Water Quality**

[*] **Channel/Flood Protection**

SPECIAL APPLICATIONS

[*] **Pretreatment**

[*] **High Density/Ultra-Urban**

[*] **Other:**

Residential Subdivision Use: *

* Depends on the specific proprietary structural control

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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.

3.3.10.1 General Description

There are many types of commercially-available proprietary stormwater structural controls available 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 structural control alternatives. Proprietary systems can often be used in pretreatment applications 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.

3.3.10.2 Guidelines for Using Proprietary Systems

In order for use as a limited application control, a proprietary system must have a demonstrated capability of meeting the stormwater management goals for which it is being intended. This means that the system must 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) Proven record of longevity in the field

(3) Proven ability to function in Georgia conditions (e.g., climate, rainfall patterns, soil types, etc.)

For a propriety system to meet (1) above for water quality goals, the following monitoring criteria should be met for supporting studies:

- At least 15 storm events must be sampled
- The study must be independent or independently verified (i.e., may not be conducted by the vendor or designer without third-party verification)
- The study must be conducted in the field, as opposed to laboratory testing
- Field monitoring must be conducted using standard protocols which require proportional sampling both upstream and downstream of the device
- Concentrations reported in the study must be flow-weighted
- The propriety system or device must 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 the 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 in regards to the use of proprietary structural stormwater controls.

SECTION

3.4

DETENTION STRUCTURAL STORMWATER CONTROLS

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3.4.1 Dry Detention / Dry ED Basins

Detention
Structural Stormwater Control

Description: A surface storage basin or facility designed to provide water quantity control through detention and/or extended detention of stormwater runoff.

REASONS FOR LIMITED USE

- Controls for stormwater quantity only – not intended to provide water quality treatment

KEY CONSIDERATIONS

- Applicable for drainage areas up to 75 acres
- Typically less costly than stormwater (wet) ponds for equivalent flood storage, as less excavation is required
- Used in conjunction with water quality structural control
- Recreational and other open space opportunities between storm runoff events

STORMWATER MANAGEMENT SUITABILITY

☐ **Water Quality**
☑ **Channel/Flood Protection**

SPECIAL APPLICATIONS

☐ **Pretreatment**
☐ **High Density/Ultra-Urban**
☐ **Other:**

Residential Subdivision Use: *Yes*

3.4.1.1 General Description

Dry detention and dry extended detention (ED) 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.

Dry detention basins are intended to provide overbank flood protection (peak flow reduction of the 25-year storm, Q_{p25}) and can be designed to control the extreme flood (100-year, Q_f) storm event. Dry ED basins provide downstream channel protection through extended detention of the channel protection volume (CP_v), and can also provide Q_{p25} and Q_f control.

Both dry detention and dry ED basins provide limited pollutant removal benefits and are not intended for water quality treatment. Detention-only facilities must be used in a treatment train approach with other structural controls that provide treatment of the WQ_v (see Section 3.1).

Compatible multi-objective use of dry detention facilities in strongly encouraged.

3.4.1.2 Design Criteria and Specifications

Location

- Dry detention and dry ED basins are to be located downstream of other structural stormwater controls providing treatment of the water quality volume (WQ_v). See Section 3.1 for more information on the use of multiple structural controls in a treatment train.

- The maximum contributing drainage area to be served by a single dry detention or dry ED basin is 75 acres.

General Design

- Dry detention basins are sized to temporarily store 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.

  Dry ED basins are sized to provide extended detention of the channel protection volume over 24 hours and can also provide additional storage volume for normal detention (peak flow reduction) of Q_{p25} and Q_f.

  Routing calculations must be used to demonstrate that the storage volume is adequate. See Section 2.2 (*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 (see Appendix H) unless the facility is excavated to this depth.

- Vegetated embankments shall be less than 20 feet in height and shall have side slopes no steeper than 2:1 (horizontal to vertical) although 3:1 is preferred. Riprap-protected embankments shall 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 (see Appendix H).

- The maximum 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 to 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. A low flow or pilot channel across the facility bottom from the inlet to the outlet (often constructed with riprap) is recommended to convey low flows and prevent standing water conditions.

- Adequate maintenance access must be provided for all dry detention and dry ED basins.

Inlet and Outlet Structures

- Inflow channels are to be stabilized with flared riprap aprons, or the equivalent. A sediment forebay sized to 0.1 inches per impervious acre of contributing drainage should be provided for dry detention and dry ED basins that are in a treatment train with off-line water quality treatment structural controls.

- For a dry detention 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.

  For a dry ED basin, 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 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.

See Section 2.3 (*Outlet Structures*) for more information on the design of outlet works.

- 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. See Section 4.5, *Energy Dissipation Design*, for more guidance.

- An emergency spillway is to 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 designed to State of Georgia guidelines for dam safety (see Appendix H) 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.

3.4.1.3 Inspection and Maintenance Requirements

Table 3.4.1-1 Typical Maintenance Activities for Dry Detention / Dry ED Basins
(Source: Denver Urban Storm Drainage Manual, 1999)

Activity	Schedule
• Remove debris from basin surface to minimize outlet clogging and improve aesthetics.	Annually and following significant storm events
• Remove sediment buildup. • Repair and revegetate eroded areas. • Perform structural repairs to inlet and outlets.	As needed based on inspection
• Mow to limit unwanted vegetation.	Routine

3.4.1.4 Example Schematics

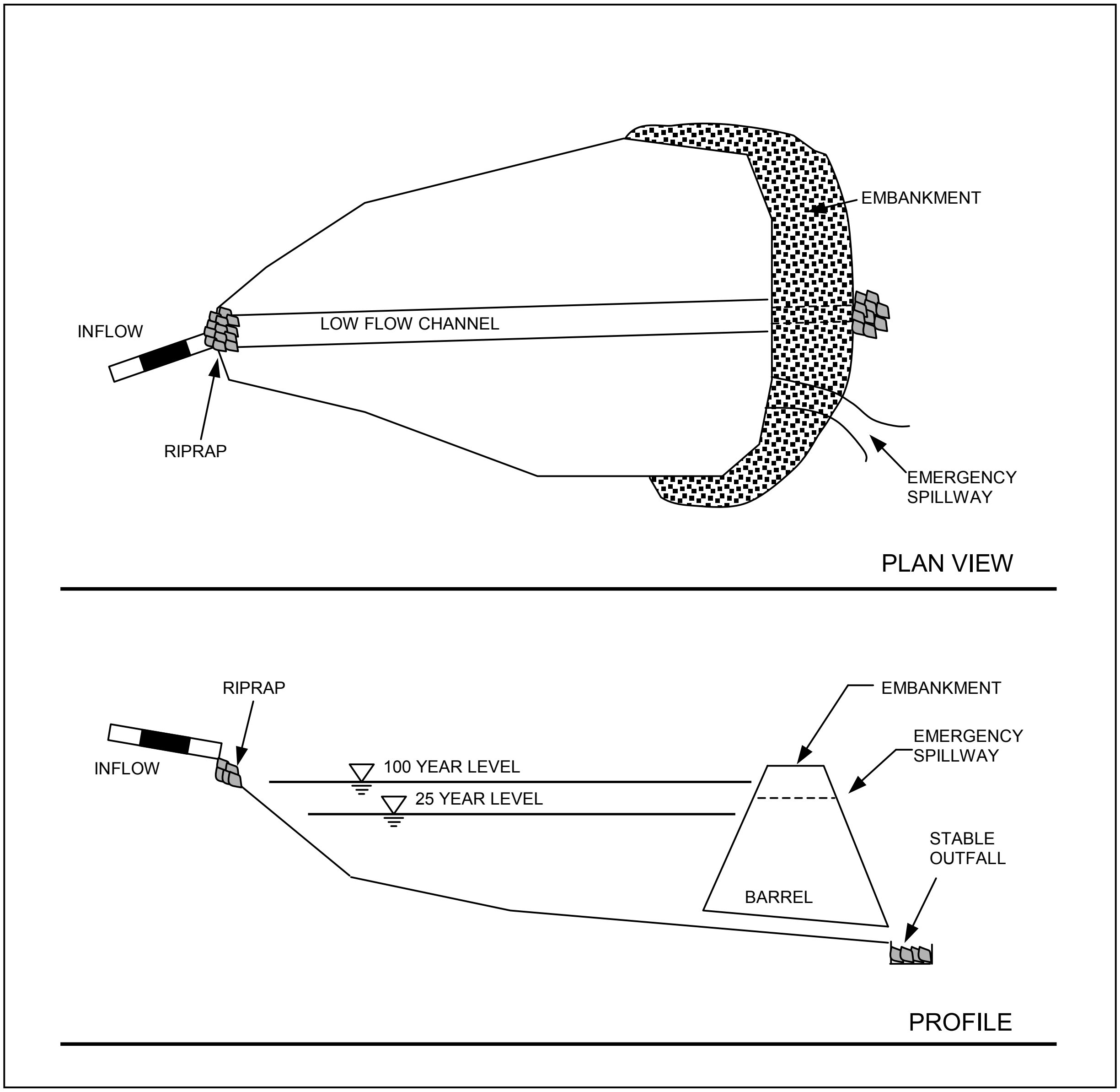

Figure 3.4.1-1 Schematic of Dry Detention Basin

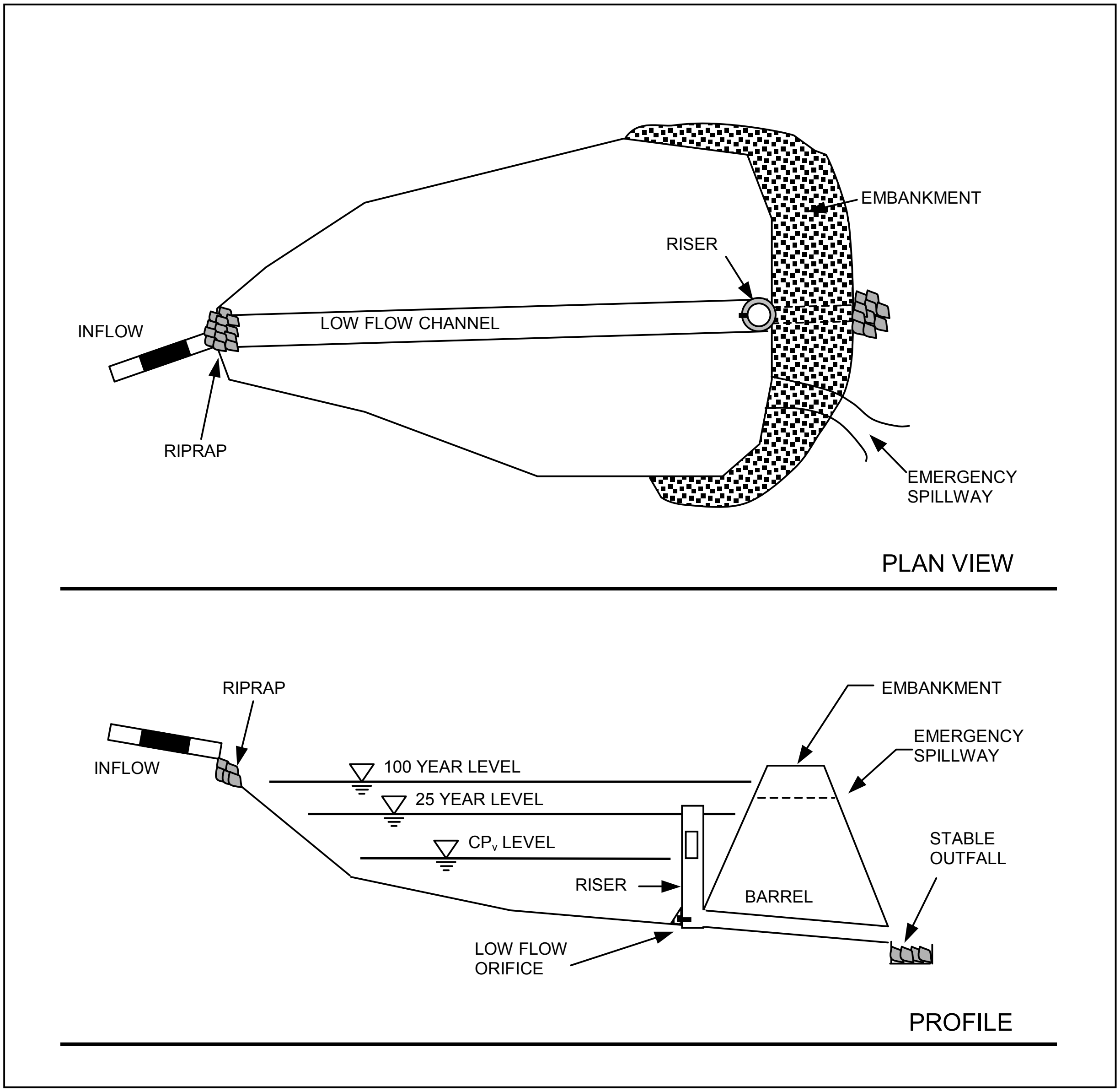

Figure 3.4.1-2 Schematic of Dry Extended Detention Basin

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3.4.2 Multi-Purpose Detention Areas

Detention
Structural Stormwater Control

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.

REASONS FOR LIMITED USE

- Controls for stormwater quantity only – not intended to provide water quality treatment

KEY CONSIDERATIONS

- Allows for multiple uses of site areas and reduces the need for downstream detention facilities
- Used in conjunction with water quality structural control
- Adequate grading and drainage must be provided to allow full use of facility's primary purposes following a storm event

STORMWATER MANAGEMENT SUITABILITY

- ☐ **Water Quality**
- ☑ **Channel/Flood Protection**

SPECIAL APPLICATIONS

- ☐ **Pretreatment**
- ☑ **High Density/Ultra-Urban**
- ☐ **Other:**

Residential Subdivision Use: *Yes*

3.4.2.1 General Description

Multi-purpose detention areas 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 (CP_v control).

Multi-purpose detention areas are not intended for water quality treatment and must be used in a treatment train approach with other structural controls that provide treatment of the WQ_v (see Section 3.1).

3.4.2.2 Design Criteria and Specifications

Location

- Multi-purpose detention areas can be located upstream or downstream of other structural stormwater controls providing treatment of the water quality volume (WQ_v). See Section 3.1 for more information on the use of multiple structural controls in a treatment train.

General Design

- Multi-purpose detention areas are 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 2.2 (*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 a significant 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 storm, and 9 inches for a 100-year storm. The maximum depth of ponding at a flow control structure is 12 inches for a 100-year 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.

Sports Fields

- Athletic facilities such as football and 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 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.

3.4.2.3 Inspection and Maintenance Requirements

Table 3.4.2-1. Typical Maintenance Activities for Multi-Purpose Detention Areas
(Based on: Denver Urban Storm Drainage Manual, 1999)

Activity	Schedule
• Remove debris from ponding area to minimize outlet clogging and improve aesthetics.	Annually and following significant storm events
• Remove sediment buildup. • Repair and revegetate eroded areas. • Perform structural repairs to inlet and outlets.	As needed based on inspection
• Perform additional maintenance activities specific to the type of facility.	As required

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3.4.3 Underground Detention

Detention
Structural Stormwater Control

Description: Detention storage located in underground tanks or vaults designed to provide water quantity control through detention and/or extended detention of stormwater runoff.

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REASONS FOR LIMITED USE

- Controls for stormwater quantity only – not intended to provide water quality treatment
- Intended for space-limited applications

KEY CONSIDERATIONS

- Does not take up surface space
- Used in conjunction with water quality structural control
- Concrete vaults or pipe/tank systems can be used

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STORMWATER MANAGEMENT SUITABILITY

☐ **Water Quality**
☑ **Channel/Flood Protection**

SPECIAL APPLICATIONS

☐ **Pretreatment**
☑ **High Density/Ultra-Urban**
☐ **Other:**

Residential Subdivision Use: *No*

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3.4.3.1 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 dry 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 extreme flood Q_f) control through normal detention. Basic storage design and routing methods are the same as for detention basins except that the bypass for high flows is typically included.

Underground detention vaults and tanks are not intended for water quality treatment and must be used in a treatment train approach with other structural controls that provide treatment of the WQ_v (see Section 3.1). 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.

3.4.3.2 Design Criteria and Specifications

Location

- Underground detention systems are to be located downstream of other structural stormwater controls providing treatment of the water quality volume (WQ_v). See Section 3.1 for more information on the use of multiple structural controls in a treatment train.

- The maximum contributing drainage area to be served by a single underground detention vault or tank is 25 acres.

General Design

- Underground detention systems are sized to provide extended detention of the channel protection volume over 24 hours and temporarily store 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). 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).

  Routing calculations must be used to demonstrate that the storage volume is adequate. See Section 2.2 (*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. The maximum depth from finished grade to the vault invert should be 20 feet.

- Detention Tanks: 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.

- 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.

Inlet and Outlet Structures

- A separate sediment sump or vault chamber sized to 0.1 inches per impervious acre 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 structural controls.

- 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 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 (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 this equivalent diameter.

  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 2.3 (*Outlet Structures*) for more information on the design of outlet works.

- Riprap, plunge pools or pads, or other energy dissipators are to be placed at the end of the outlet to prevent scouring and erosion. See Section 4.5, *Energy Dissipation Design*, for more guidance.

- A high flow bypass is to be included in the underground detention system design to safely pass the extreme flood flow.

3.4.3.3 Inspection and Maintenance Requirements

Table 3.4.3-1 Typical Maintenance Activities for Underground Detention Systems

Activity	Schedule
• Remove any trash/debris and sediment buildup in the underground vaults or tanks.	Annually
• Perform structural repairs to inlet and outlets.	As needed, based on inspection

3.4.3.4 Example Schematics

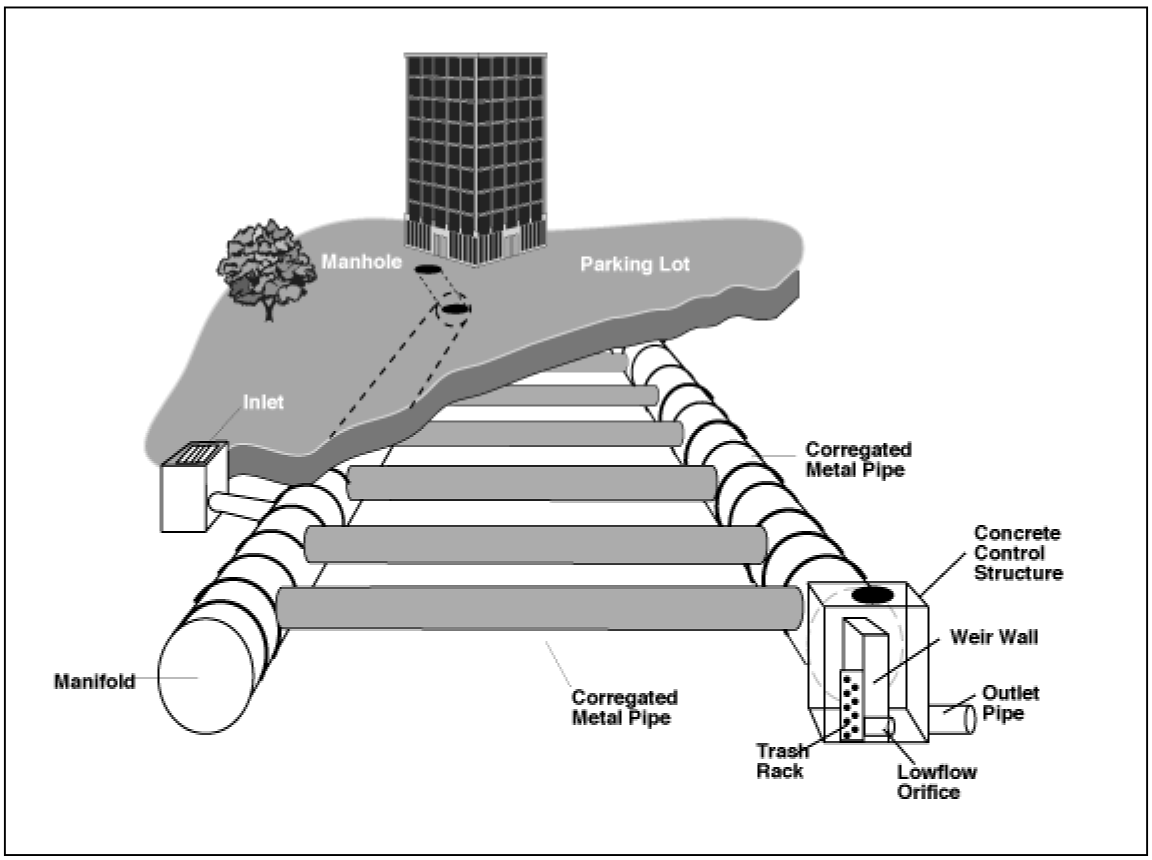

Figure 3.4.3-1 Example Underground Detention Tank System

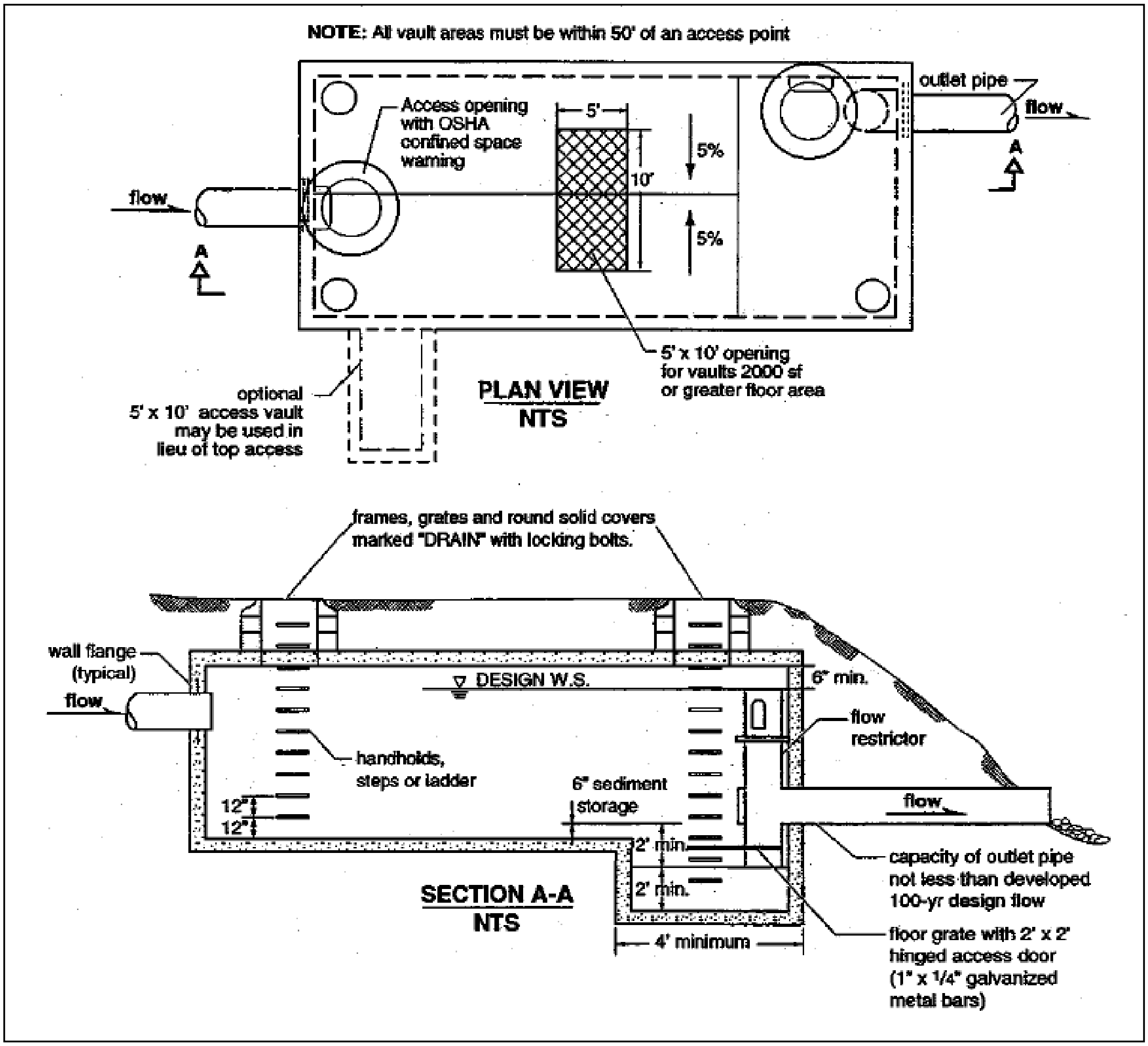

Figure 3.4.3-2 Schematic of Typical Underground Detention Vault
(Source: WDE, 2000)

CHAPTER 4

STORMWATER DRAINAGE SYSTEM DESIGN

GEORGIA STORMWATER MANAGEMENT MANUAL
FIRST EDITION – AUGUST 2001

CHAPTER 4

STORMWATER DRAINAGE SYSTEM DESIGN

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SECTION

4.1

STORMWATER DRAINAGE DESIGN OVERVIEW

4.1.1 Stormwater Drainage System Design

4.1.1.1 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 water quality, streambank channel protection, habitat protection and groundwater recharge.

4.1.1.2 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 (see Section 1.5).

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 4.2); culverts (Section 4.3); vegetated and lined open channels (Section 4.4); and energy dissipation devices for outlet protection (Section 4.5). The rest of this section covers important considerations to keep in mind in the planning and design of stormwater drainage facilities.

4.1.1.3 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)

(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.

4.1.2 Key Issues in Stormwater Drainage Design

4.1.2.1 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.

4.1.2.2 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.

4.1.2.3 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.

- 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 4.1-1 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.

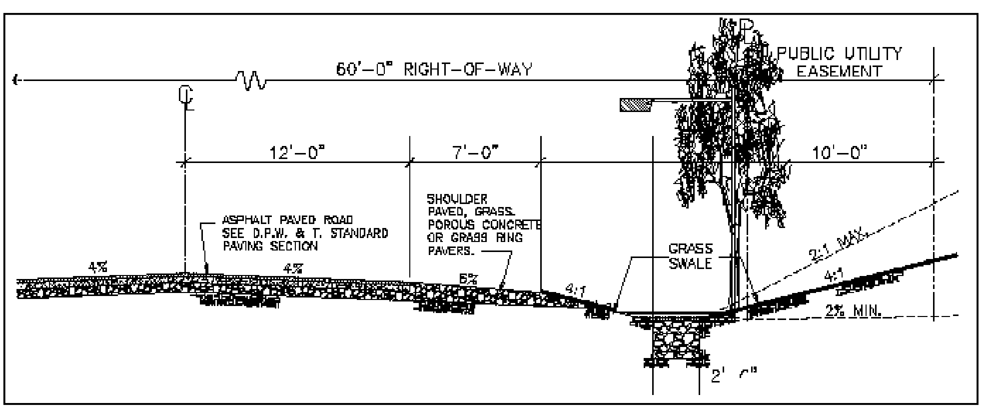

Figure 4.1-1 Alternate Roadway Section without Gutters
(Source: Prince George's County, MD, 1999)

4.1.2.4 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.

4.1.2.5 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.

4.1.2.6 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.

4.1.2.7 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 4.1-2 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.

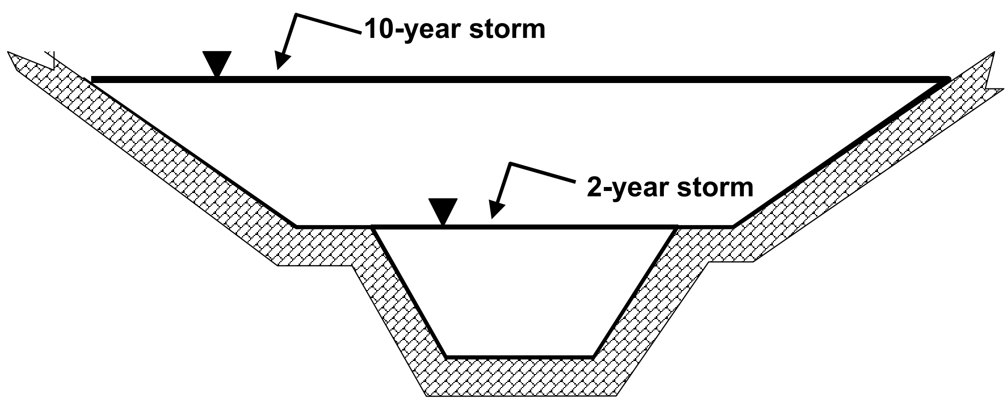

Figure 4.1-2 Compound Channel

- 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.

4.1.2.8 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.

4.1.3 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 GaDOT 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.

References

Department of Irrigation and Drainage Malaysia, River Engineering Division, 2000. Urban Stormwater Management Manual for Malaysia (Draft).

Prince George's County, MD, 1999. Low-Impact Development Design Strategies, An Integrated Design Approach.

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SECTION

4.2

MINOR DRAINAGE SYSTEM DESIGN

4.2.1 Overview

4.2.1.1 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 4.4, *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.

4.2.1.2 General Criteria

Design Frequency

See Section 4.1 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

4.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.2-1 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 4.2-1 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 gutter	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 cross slope	-

4.2.3 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.

4.2.3.1 Formula

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$\mathbf{Q = [0.56 / n]\ S_x^{5/3}\ \ S^{1/2}\ \ T^{8/3}} \qquad \textbf{(4.2.1)}$$

Where: Q= gutter flow rate, cfs
n = Manning's roughness coefficient
T = width of flow or spread, ft
S_x =pavement cross slope, ft/ft
S = longitudinal slope, ft/ft

4.2.3.2 Nomograph

Figure 4.2-1 is a nomograph for solving Equation 4.2.1. Manning's n values for various pavement surfaces are presented in Table 4.2-2 below.

4.2.3.3 Manning's n Table

Table 4.2-2 Manning's n Values for Street and Pavement Gutters

Type of Gutter or Pavement	Range of Manning's n
Concrete gutter, troweled finish	0.012
Asphalt 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).

4.2.3.4 Uniform Cross Slope

The nomograph in Figure 4.2-1 is used with the following procedures to find gutter capacity for uniform cross slopes:

<u>Condition 1</u>: 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.

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 Qn 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 (Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

4.2.3.5 Composite Gutter Sections

Figure 4.2-2 in combination with Figure 4.2-1 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 4.2-3 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 4.2-3 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: $$Q_W = Q - Q_S \qquad \textbf{(4.2.2)}$$

(Step 3) Calculate the ratios Q_W/Q or E_O and S_W/S_X and use Figure 4.2-2 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 T_S from Step 5 along with Manning's n, S, and S_X to find the actual value of Q_S from Figure 4.2-1.

(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 Q_S 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 4.2-1 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 4.2-2, find the appropriate value of E_O (the ratio of Q_W/Q).

(Step 4) Calculate the total gutter flow using the equation:

$$\mathbf{Q = Q_S / (1 - E_O)} \qquad \textbf{(4.2.3)}$$

Where: Q = gutter flow rate, cfs
Q_S = flow capacity of the gutter section above the depressed section, cfs
E_O = ratio of frontal flow to total gutter flow (Q_W/Q)

(Step 5) Calculate the gutter flow in width (W), using Equation 4.2.2.

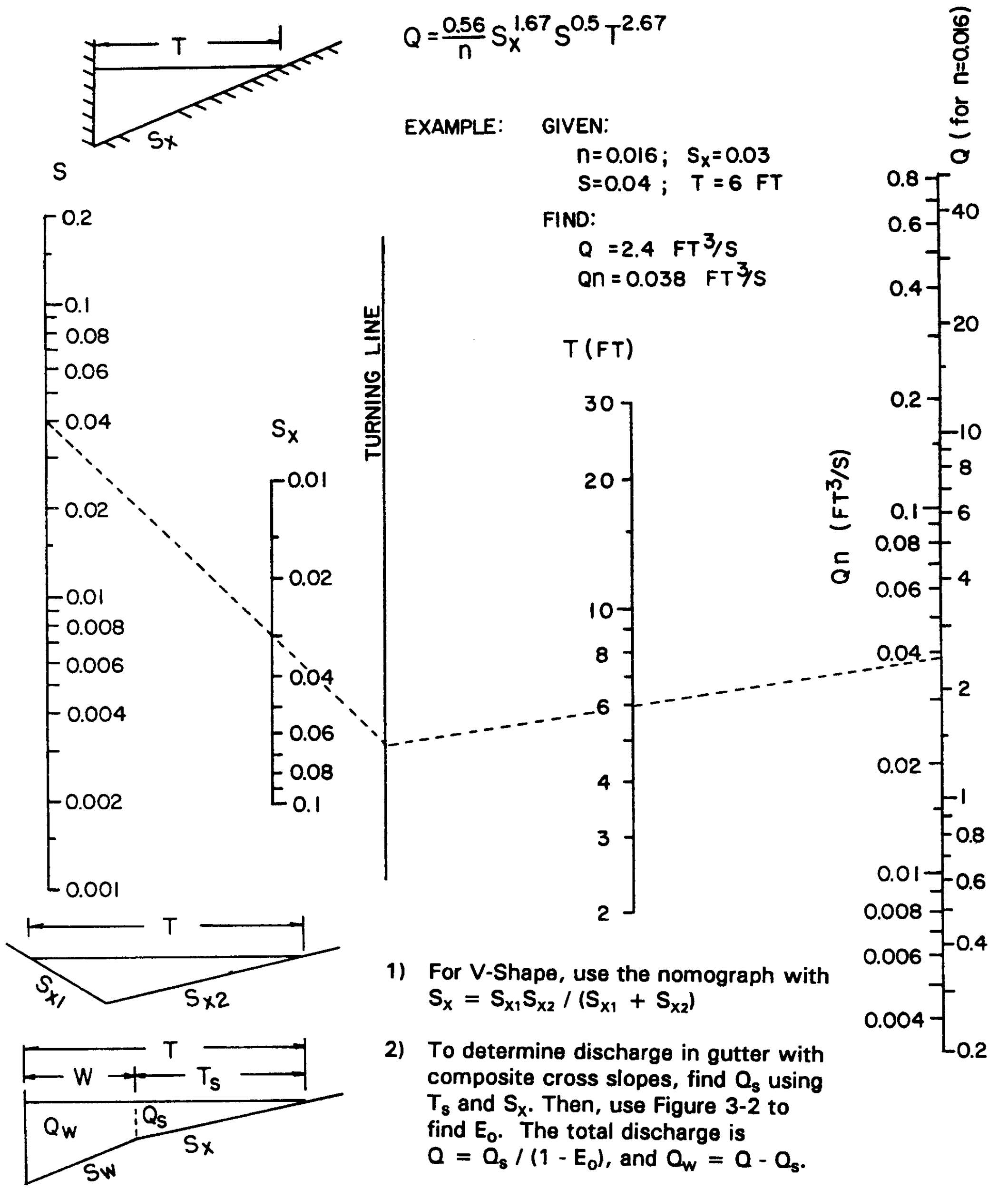


Figure 4.2-1 Flow in Triangular Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

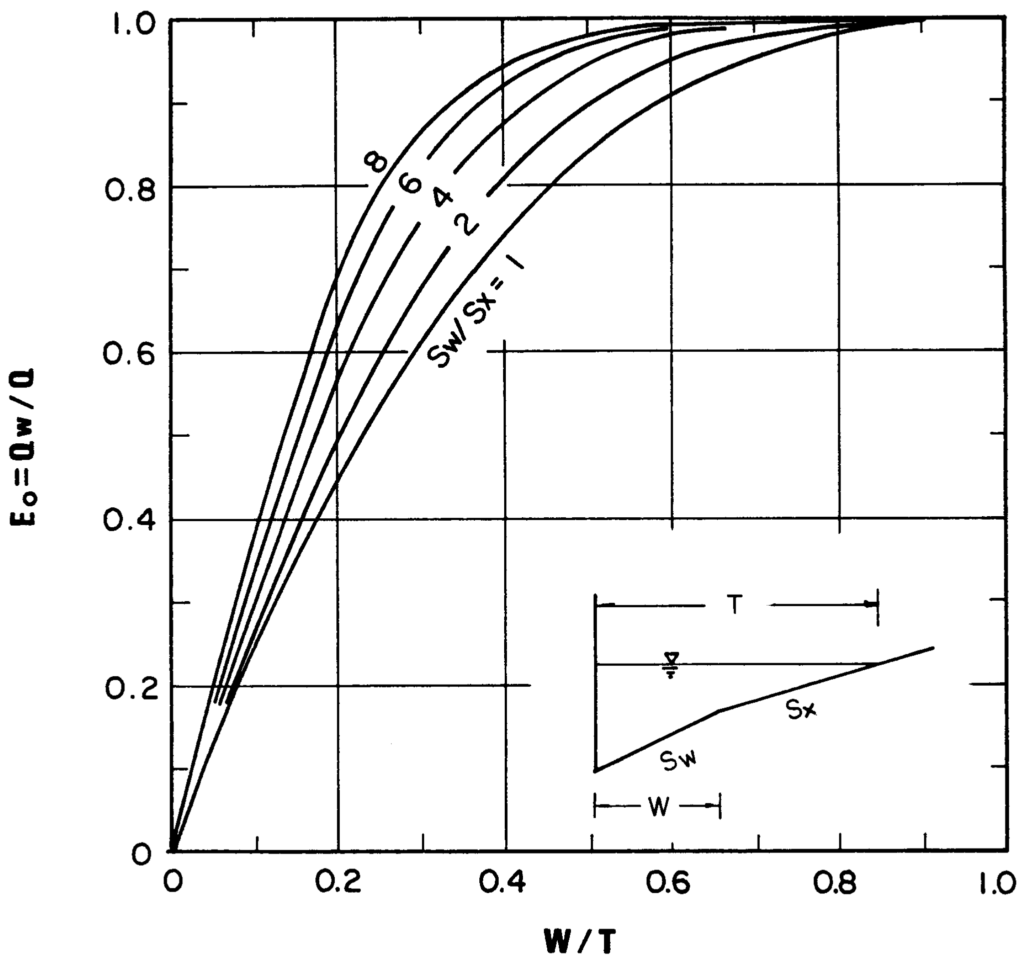


Figure 4.2-2 Ratio of Frontal Flow to Total Gutter Flow
(Source: AASHTO Model Drainage Manual, 1991)

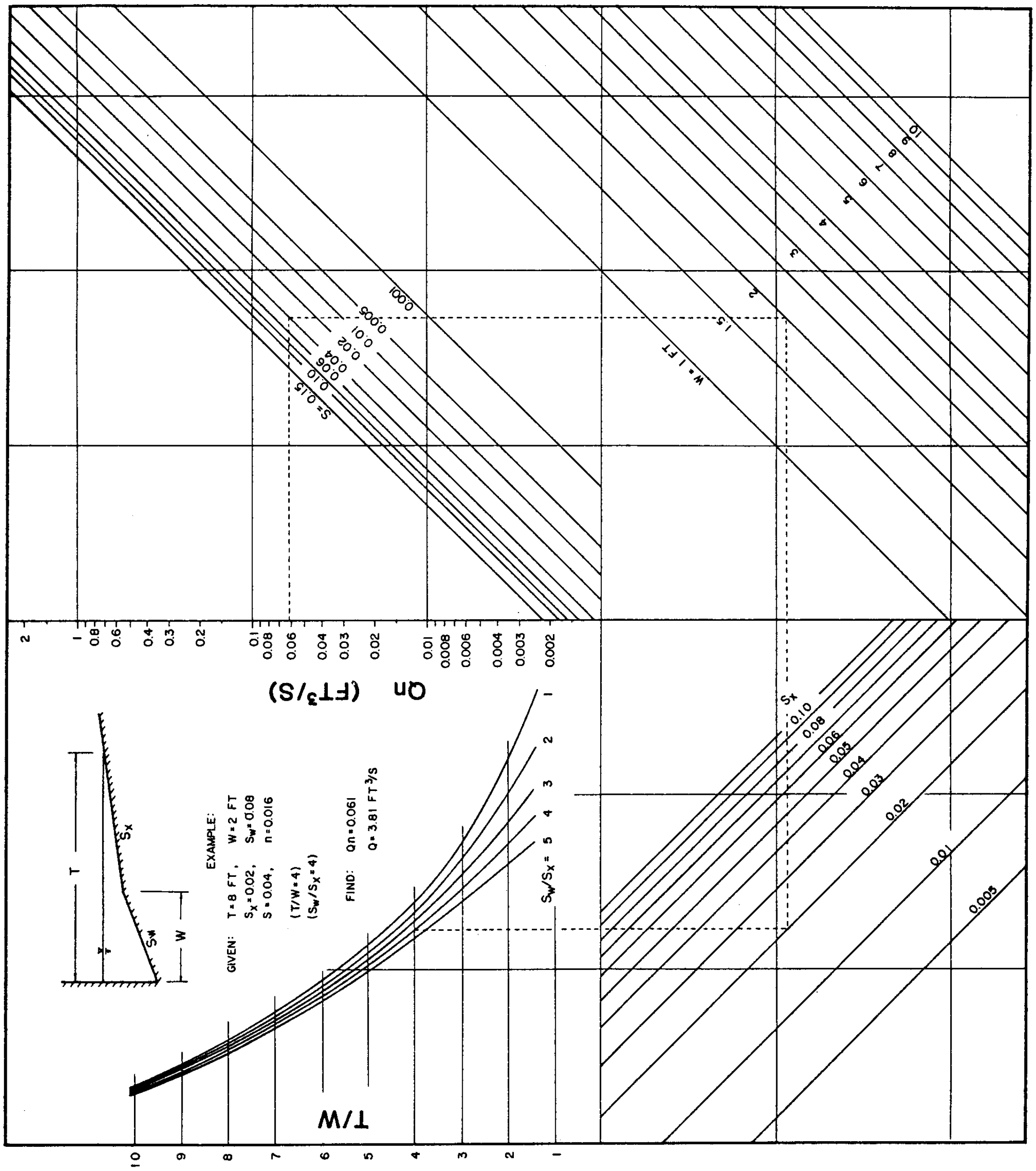


Figure 4.2-3 Flow in Composite Gutter Sections
(Source: AASHTO Model Drainage Manual, 1991)

4.2.3.6 Examples

Example 1

Given: $T = 8$ ft, $S_x = 0.025$ 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 4.2-1, Qn = 0.03
$Q = Qn/n = 0.03/0.015 = 2.0$ cfs

(b) $T = 8 - 2 = 6$ ft
$(Qn)_2 = 0.014$ (Figure 4.2-1) (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: $T = 6$ ft, $S_w = 0.0833$ ft/ft
$T_s = 6 - 1.5 = 4.5$ ft, $W = 1.5$ ft
$S_x = 0.03$ ft/ft, $n = 0.014$
$S = 0.04$ ft/ft

Find: Flow in the composite gutter

Solution:
(1) Use Figure 4.2-1 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 4.2-2 to find $E_o = 0.64$

(3) Calculate the gutter flow using Equation 4.2.3
$Q = 2.7/(1 - 0.64) = 7.5$ cfs

(4) Calculate the gutter flow in width, W, using Equation 4.2.2
$Q_w = 7.5 - 2.7 = 4.8$ cfs

4.2.4 Stormwater Inlets

Inlets are drainage structures used to collect surface water through grate or curb openings 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:

- 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.

- Curb-Opening Inlets – These inlets are vertical openings in the curb covered by a top slab.

- 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 subsection 4.2.5, curb inlet design in Section 4.2.6, and combination inlets in Section 4.2.7.

4.2.5 Grate Inlet Design

4.2.5.1 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. 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 qualitatively simulate field conditions. Table 4.2-3 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 expressed by the following equation:

$$E_O = Q_W/Q = 1 - (1 - W/T)^{2.67} \qquad (4.2.4)$$

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

Table 4.2-3 Grate Debris Handling Efficiencies

Rank	Grate	Longitudinal Slope (0.005)	Longitudinal Slope (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.

Figure 4.2-2 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:

$$Q_S/Q = 1 - Q_W/Q = 1 - E_O \qquad (4.2.5)$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$R_f = 1 - 0.09\,(V - V_0) \qquad (4.2.6)$$

Where: V = velocity of flow in the gutter, ft/s (using Q from Figure 4.2-1)
V_O = gutter velocity where splash-over first occurs, ft/s (from Figure 4.2-4)

This ratio is equivalent to frontal flow interception efficiency. Figure 4.2-4 provides a solution of equation 4.2.6, which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 4.2-4 is total gutter flow divided by the area of flow. The ratio of side flow intercepted to total side flow, R_S, or side flow interception efficiency, is expressed by:

$$R_S = 1 / [1 + (0.15V^{1.8}/S_X L^{2.3})] \qquad (4.2.7)$$

Where: L = length of the grate, ft

Figure 4.2-5 provides a solution to equation 4.2.7.

The efficiency, E, of a grate is expressed as:

$$\mathbf{E = R_f E_o + R_s(1 - E_o)} \qquad \textbf{(4.2.8)}$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$\mathbf{Q_i = EQ = Q[R_f E_o + R_s(1 - E_o)]} \qquad \textbf{(4.2.9)}$$

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 4.2-4 for Curved Vane Grate, R_f = 1.0
From Figure 4.2-4 for Reticuline Grate, R_f = 1.0
From Figure 4.2-5 R_s = 0.1 for both grates
From Equation 4.2.9:

$$Q_i = 3.0[1.0 \text{ X } 0.69 + 0.1(1 - 0.69)] = 2.2 \text{ cfs}$$

For this example, the interception capacity of a curved vane grate is the same as that for a reticuline grate for the sited conditions.

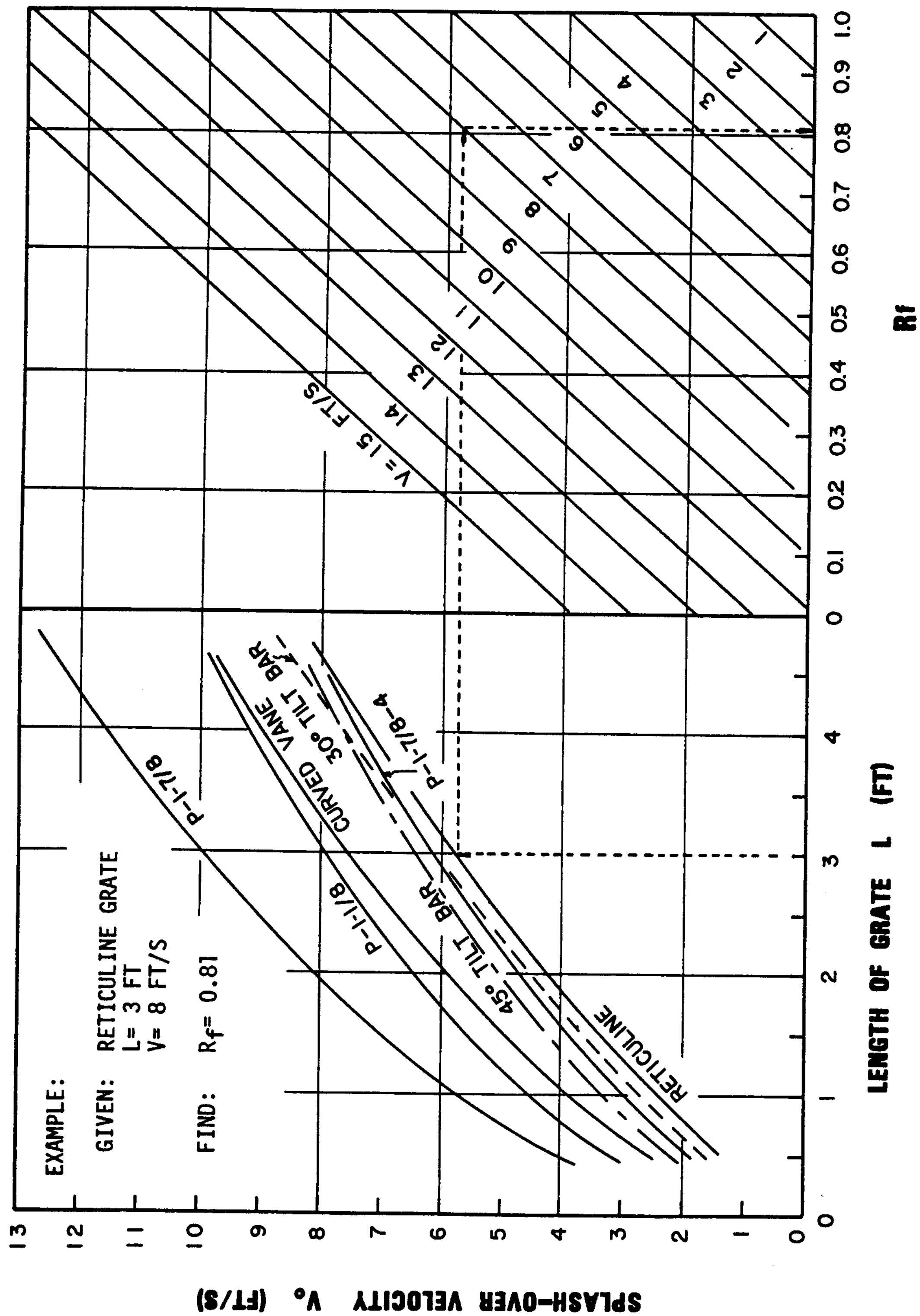


Figure 4.2-4 Grate Inlet Frontal Flow Interception Efficiency
(Source: HEC-12, 1984)

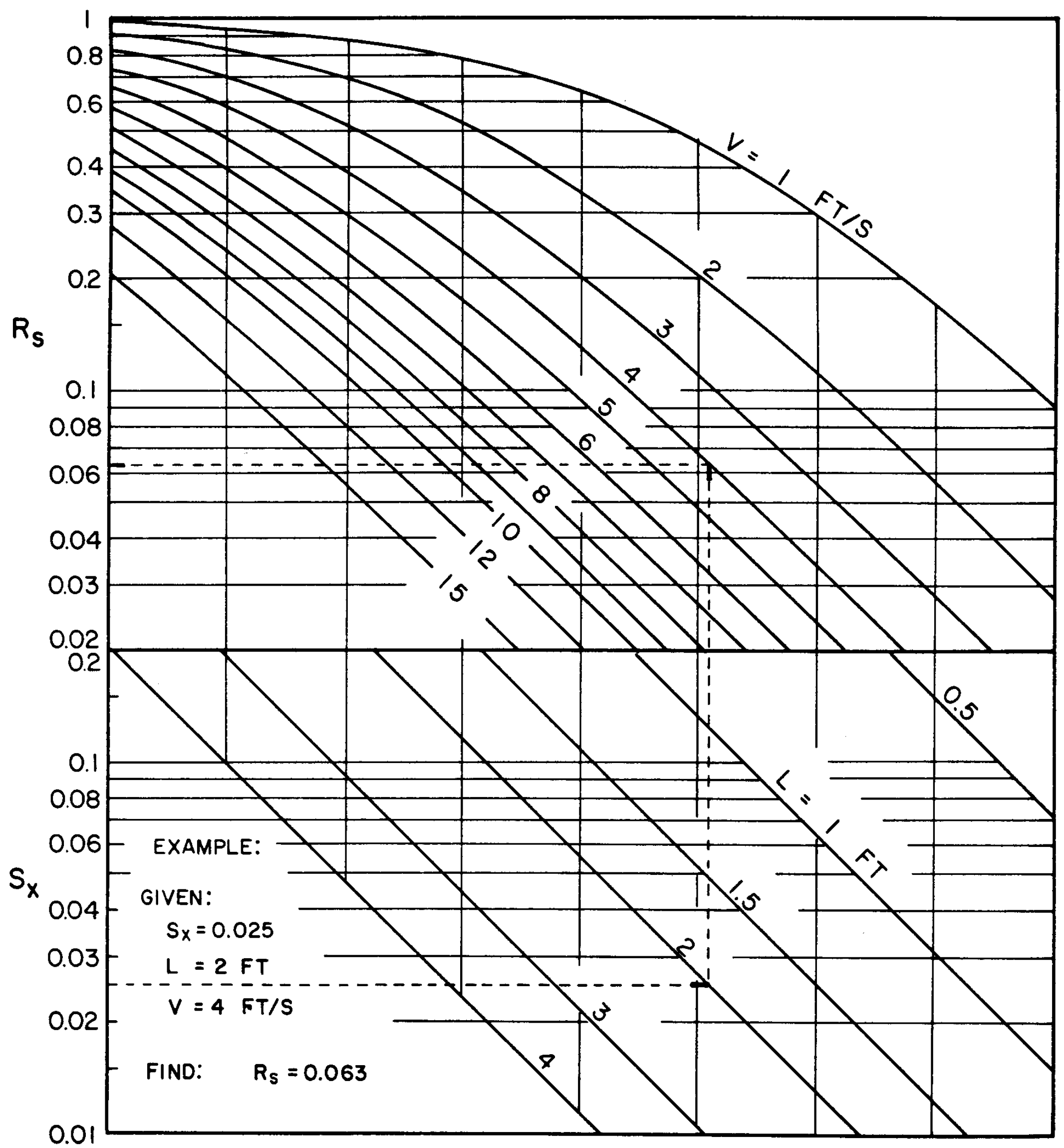


Figure 4.2-5 Grate Inlet Side Flow Interception Efficiency
(Source: HEC-12, 1984)

4.2.5.2 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:

$$Q_i = CPd^{1.5} \qquad \textbf{(4.2.10)}$$

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:

$$Q_i = CA(2gd)^{0.5} \qquad \textbf{(4.2.11)}$$

Where: C = 0.67 orifice coefficient
A = clear opening area of the grate, ft^2
g = 32.2 ft/s^2

Figure 4.2-6 is a plot of equations 4.2.10 and 4.2.11 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.

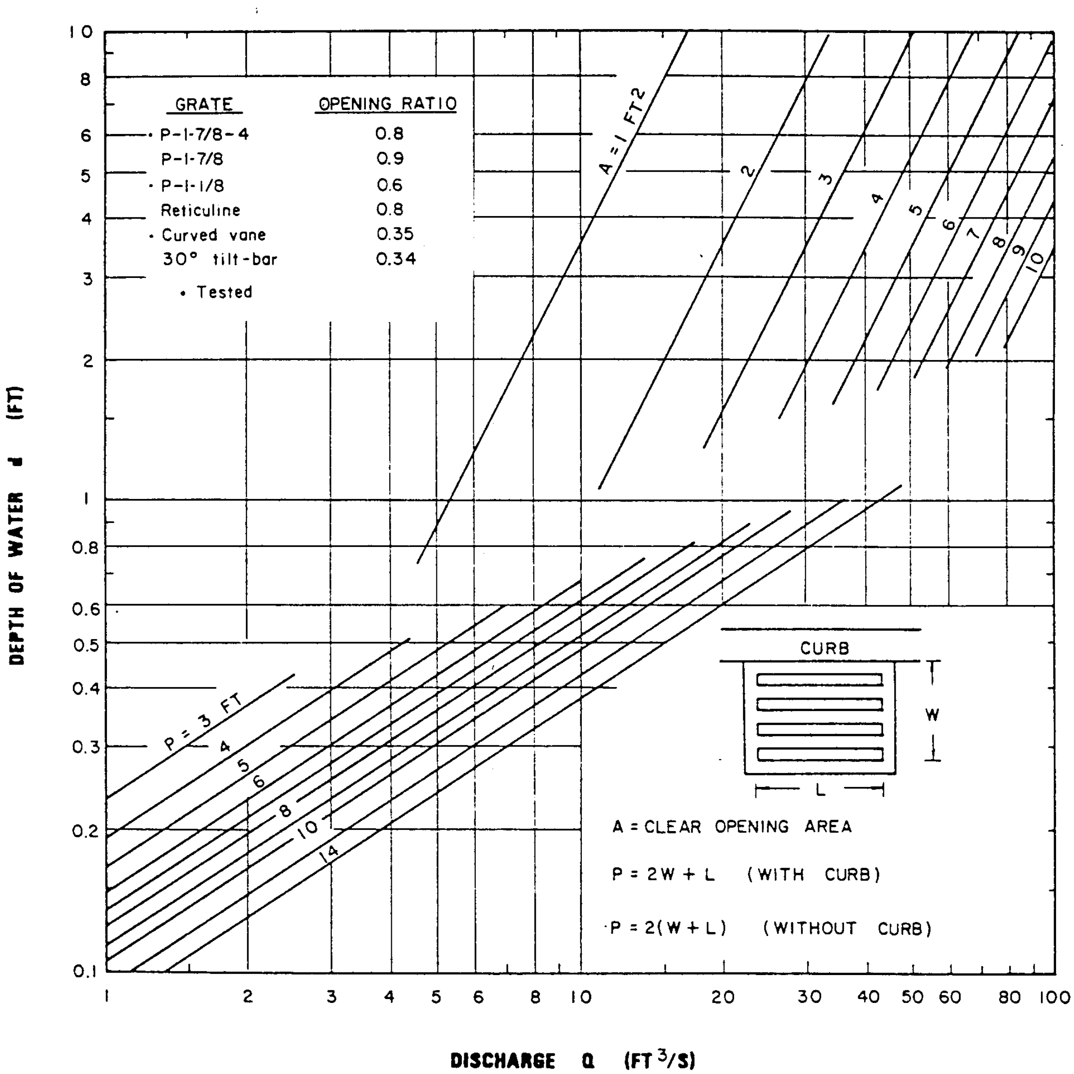
Given: A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point; allow for 50% clogging of the grate.

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 4.2-6, 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 compute 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%.



Reference: USDOT, FHWA, HEC-12 (1984).

Figure 4.2-6 Grate Inlet Capacity in Sag Conditions
(Source: HEC-12, 1984)

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 4.2-6)

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:

Q = 3.6 cfs, T = 8.2 ft (25-year storm) (from Figure 4.2-1)

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.

4.2.6 Curb Inlet Design

4.2.6.1 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 pavements 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 pavement section with a straight cross slope is determined using Figure 4.2-7. The efficiency of curb-opening inlets shorter than the length required for total interception is determined using Figure 4.2-8.

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:

$$\mathbf{S_e = S_x + S'_w E_o} \qquad \textbf{(4.2.12)}$$

Where: E_o = ratio of flow in the depressed section to total gutter flow

S'_w= cross slope of gutter measured from the cross slope of the pavement, S_x

S'_w= (a/12W)

Where: a = gutter depression, in

W = width of depressed gutter, ft

It is apparent from examination of Figure 4.2-7 that the length of curb opening 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.

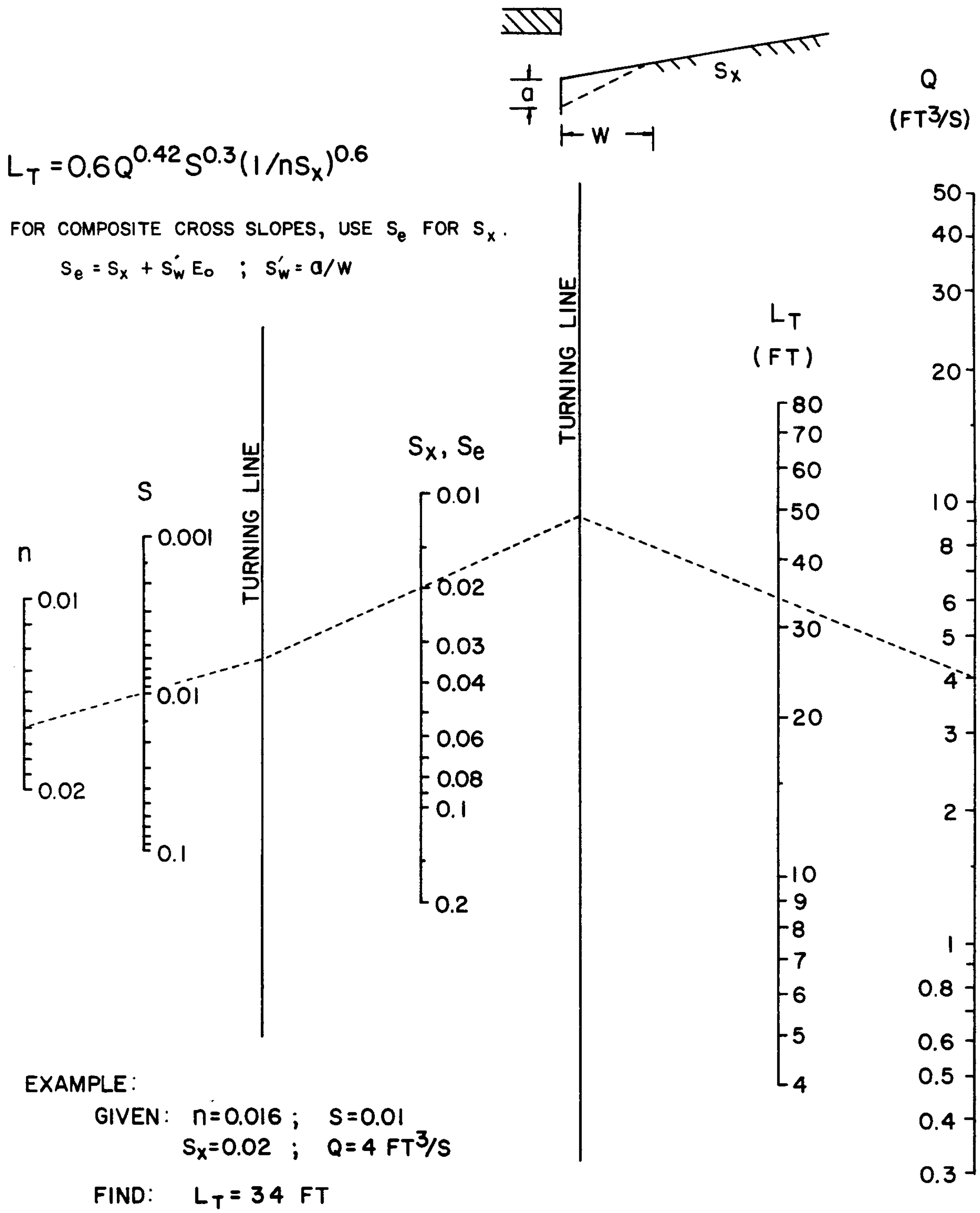


Figure 4.2-7 Curb-Opening and Slotted Drain Inlet Length for Total Interception
(Source: HEC-12, 1984)

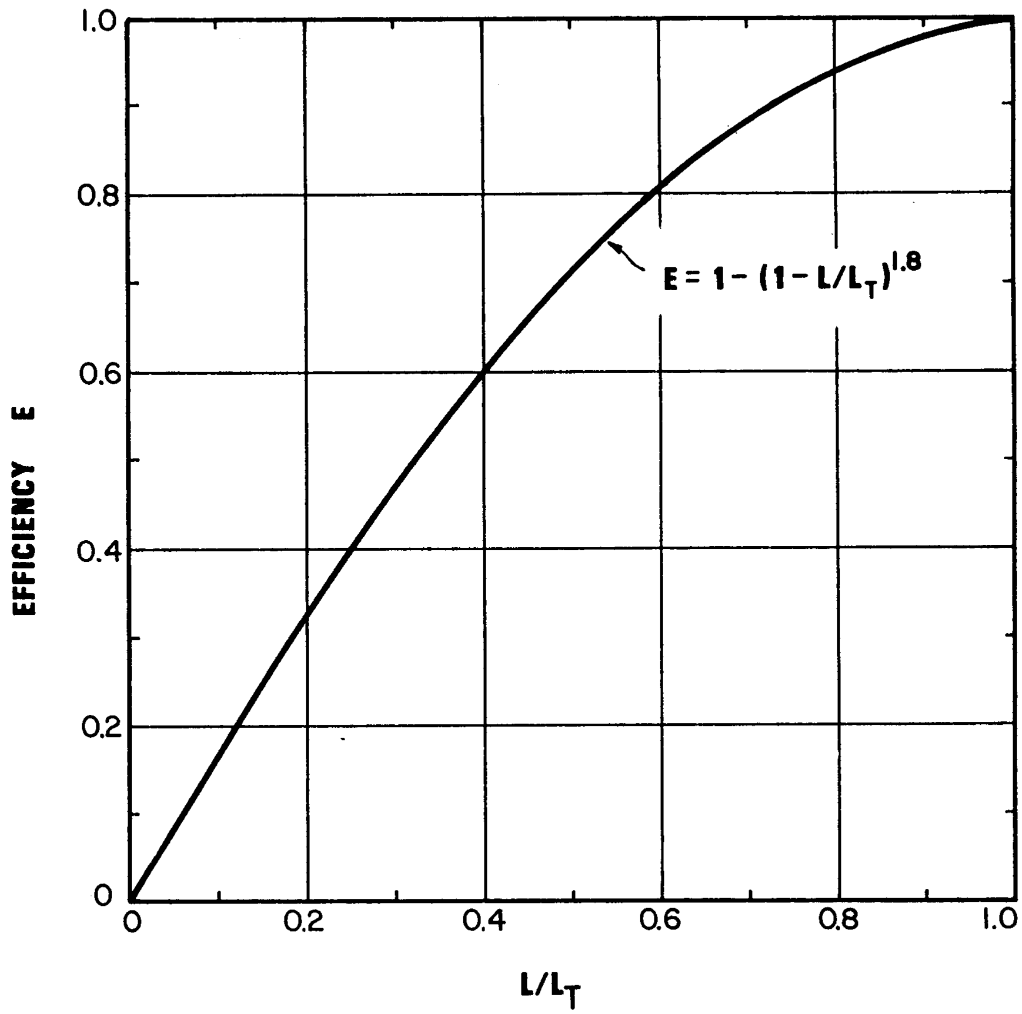


Figure 4.2-8 Curb-Opening and Slotted Drain Inlet Interception Efficiency
(Source: HEC-12, 1984)

Design Steps

Steps for using Figures 4.2-7 and 4.2-8 in the design of curb inlets on grade are given below.

(Step 1) Determine the following input parameters:

Cross slope = S_x (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 4.2-1

(Step 2) Enter Figure 4.2-7 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 required from the vertical line labeled L_T.

(Step 5) If the curb-opening inlet is shorter than the value obtained in Step 4, Figure 4.2-8 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 4.2-1)

Solution:

(1) From Figure 4.2-7, L_T = 41 ft, L/L_T = 10/41 = 0.24
From Figure 4.2-8, E = 0.39, Q_i = 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 4.2-3)
T = 3.5 x 2 = 7 ft
W/T = 2/7 = 0.29 ft
E_o = 0.72 (from Figure 4.2-2)
Therefore, $S_e = S_x + S'_w E_o$ = 0.03 + 0.083(0.72) = 0.09

From Figure 4.2-7, L_T = 23 ft, L/L_T = 10/23 = 0.4
From Figure 4.2-8, 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.

4.2.6.2 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 4.2-9, 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 4.2-9). The weir portion of Figure 4.2-9 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 4.2-10. The capacity of curb-opening inlets in a sump location with other than vertical orifice openings can be determined by using Figure 4.2-11.

Design Steps

Steps for using Figures 4.2-9, 4.2-10, and 4.2-11 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 4.2-1
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 (4.2-9, 4.2-10, or 4.2-11 depending on whether the inlet is in a depression 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 horizontal 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.

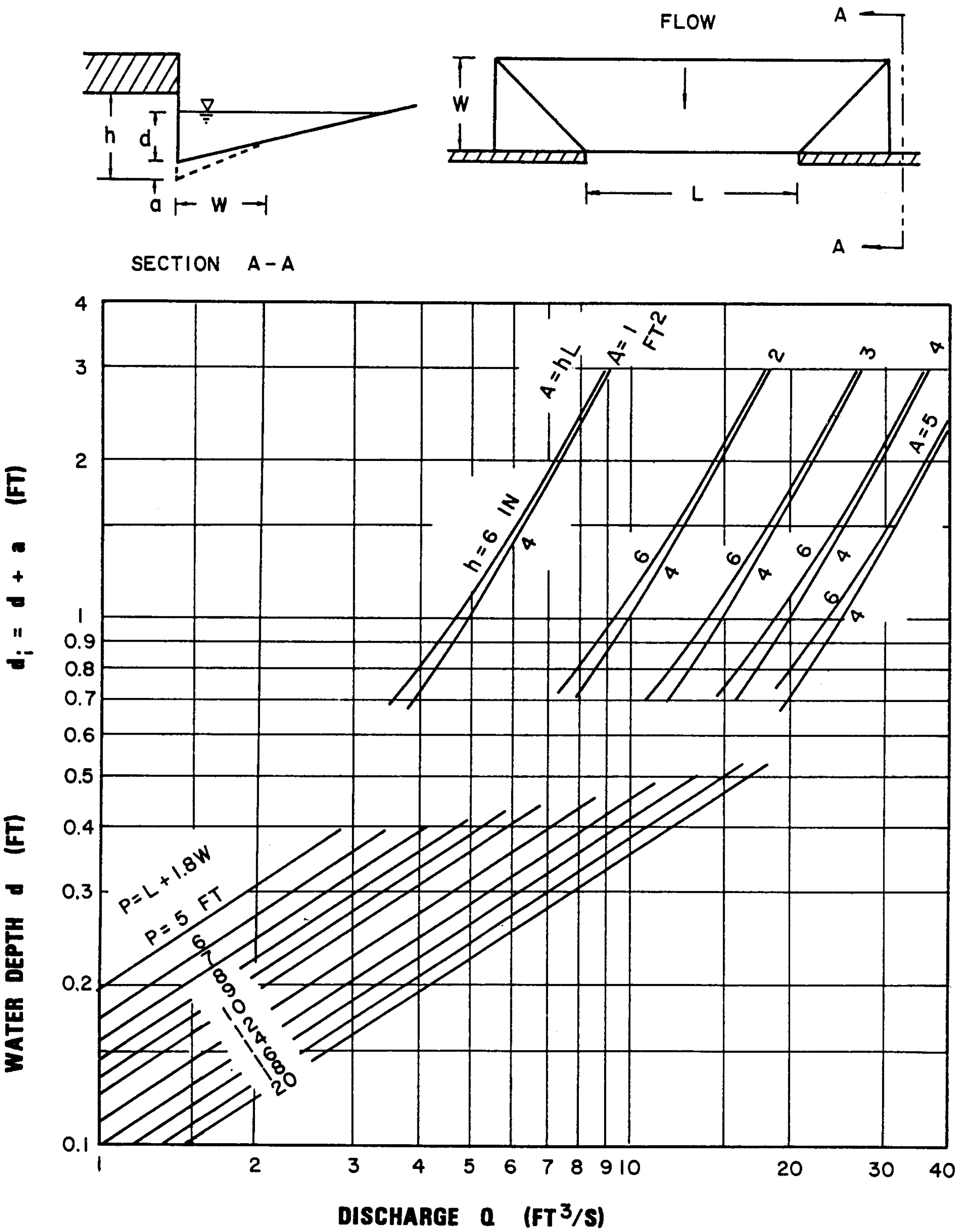


Figure 4.2-9 Depressed Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

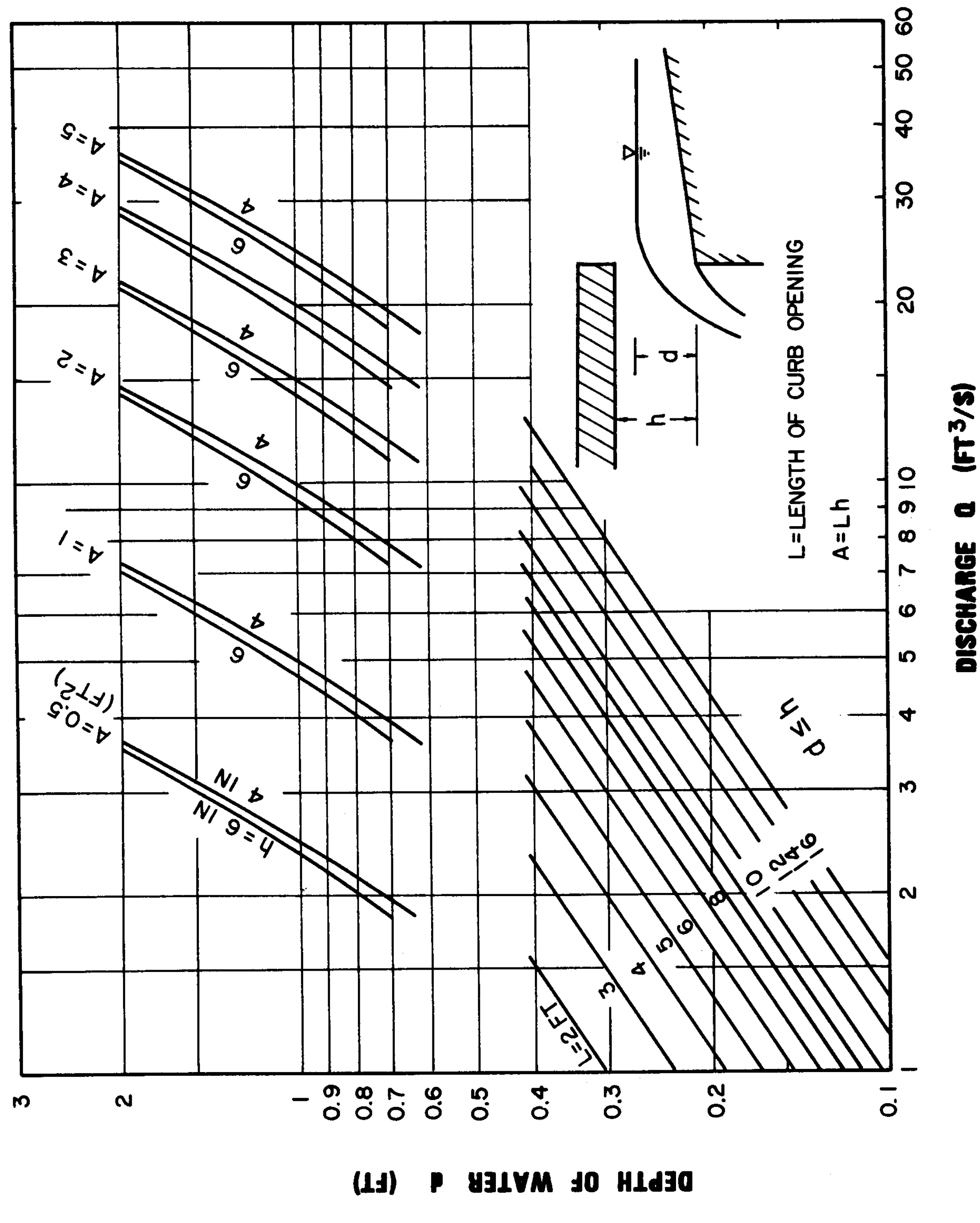


Figure 4.2-10 Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

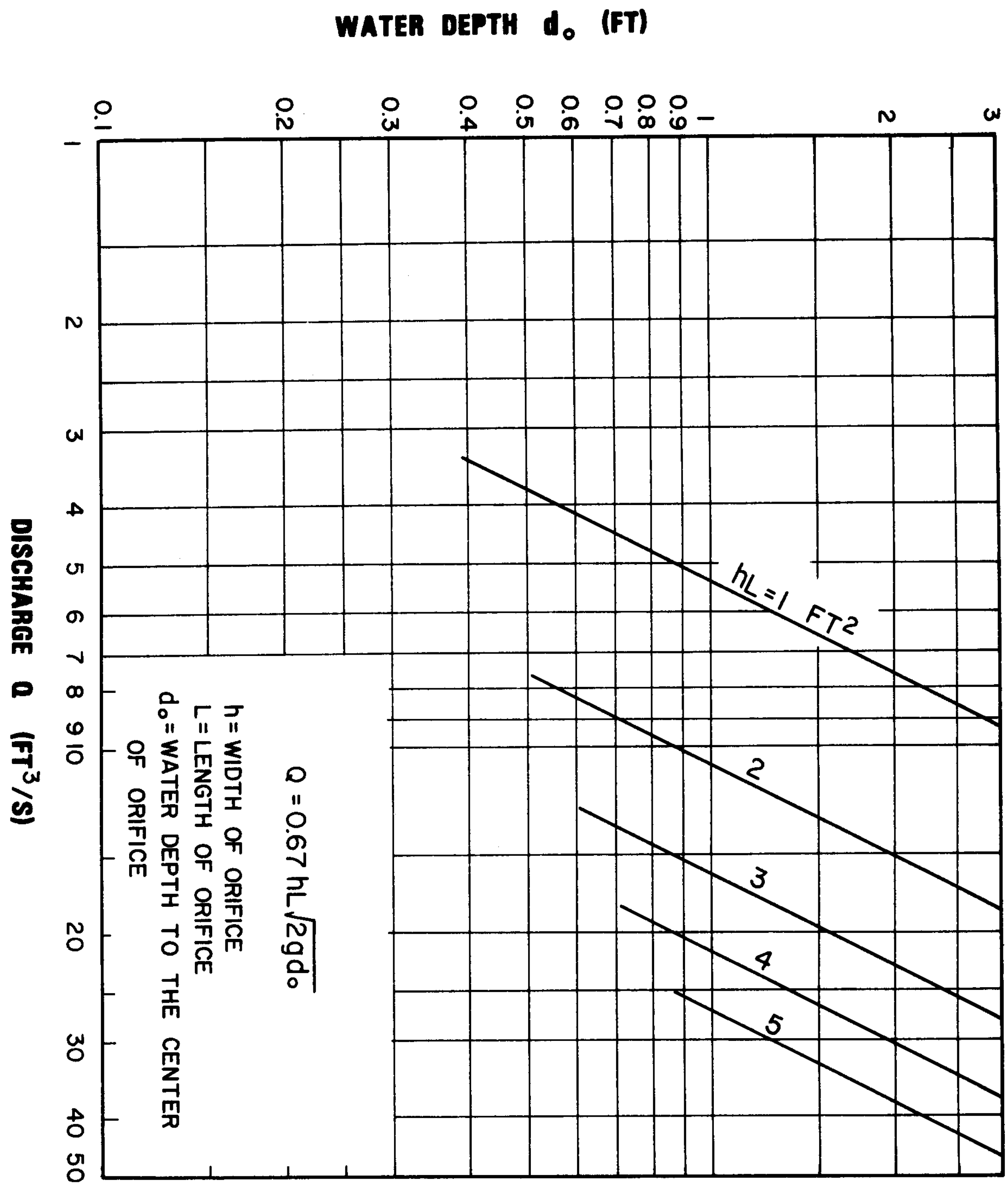


Figure 4.2-11 Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats
(Source: AASHTO Model Drainage Manual, 1991)

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 4.2-10, $Q_i = 3.8$ 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 4.2-9 is applicable (lower portion of the figure).

$P = L + 1.8W = 5 + 3.6 = 8.6$ ft

From Figure 4.2-9, $Q_i = 5$ cfs

At d = 0.4 ft, the depressed curb-opening inlet has about 30% more capacity than an inlet without depression.

4.2.7 Combination Inlets

4.2.7.1 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 4.2-4, 4.2-5 and 4.2-6.

4.2.7.2 Combination Inlets In Sump

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be concentrated 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 4.2-9, 4.2-10, and 4.2-11 for curb-opening inlets should be used for design.

4.2.8 Storm Drain Pipe Systems

4.2.8.1 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.

4.2.8.2 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 4.2-12) 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.

4.2.8.3 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.

STORM DRAIN COMPUTATION SHEET

ROUTE ______
SECTION ______
COUNTY ______

COMPUTED ______ DATE ______
CHECKED ______ DATE ______

STATION		LENGTH FEET	DRAINAGE AREA "A" (ACRES)		RUNOFF COEFFICIENT "C"	"A" • "C"		FLOW TIME MIN.		RAINFALL INTENSITY "I" in/hour	TOTAL RUNOFF Q = C.I.A. (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		

Figure 4.2-12 Storm Drain System Computation Form
(Source: AASHTO Model Drainage Manual, 1991)

4.2.8.4 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:

$$V = [1.486\ R^{2/3}S^{1/2}]/n \qquad (4.2.13)$$

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:

$$Q = [1.486\ AR^{2/3}S^{1/2}]/n \qquad (4.2.14)$$

Where: Q = rate of flow, cfs
A = cross sectional area of flow, ft^2

For pipes flowing full, the above equations become:

$$V = [0.590\ D^{2/3}S^{1/2}]/n \qquad (4.2.15)$$
$$Q = [0.463\ D^{8/3}S^{1/2}]/n \qquad (4.2.16)$$

Where: D = diameter of pipe, ft

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$$H_f = [2.87\ n^2V^2L]/[S^{4/3}] \qquad (4.2.17)$$
$$H_f = [29\ n^2V^2L]/[(R^{4/3})\ (2g)] \qquad (4.2.18)$$

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

4.2.8.5 Nomographs and Table

The nomograph solution of Manning's formula for full flow in circular storm drain pipes is shown in Figures 4.2-13, 4.2-14, and 4.2-15. Figure 4.2-16 has been provided to solve the Manning's equation for partially full flow in storm drains.

4.2.8.6 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 4.2-17 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 submerged 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:

$$\mathbf{S_f = (Q^2)/K} \qquad \mathbf{(4.2.19)}$$

Where: S_f = friction slope
$K = [1.486\ AR^{2/3}]/n$

(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 Δ = 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:

$H_o = [0.25\ V_o^2)]/2g$, where g = 32.2 ft/s^2

(Step 10) Enter in Column 10 the design discharge (Q_i) 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:

$$H_i = [0.35 (V_1^2)]/2g$$

(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_Δ).

(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 manholes, 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.

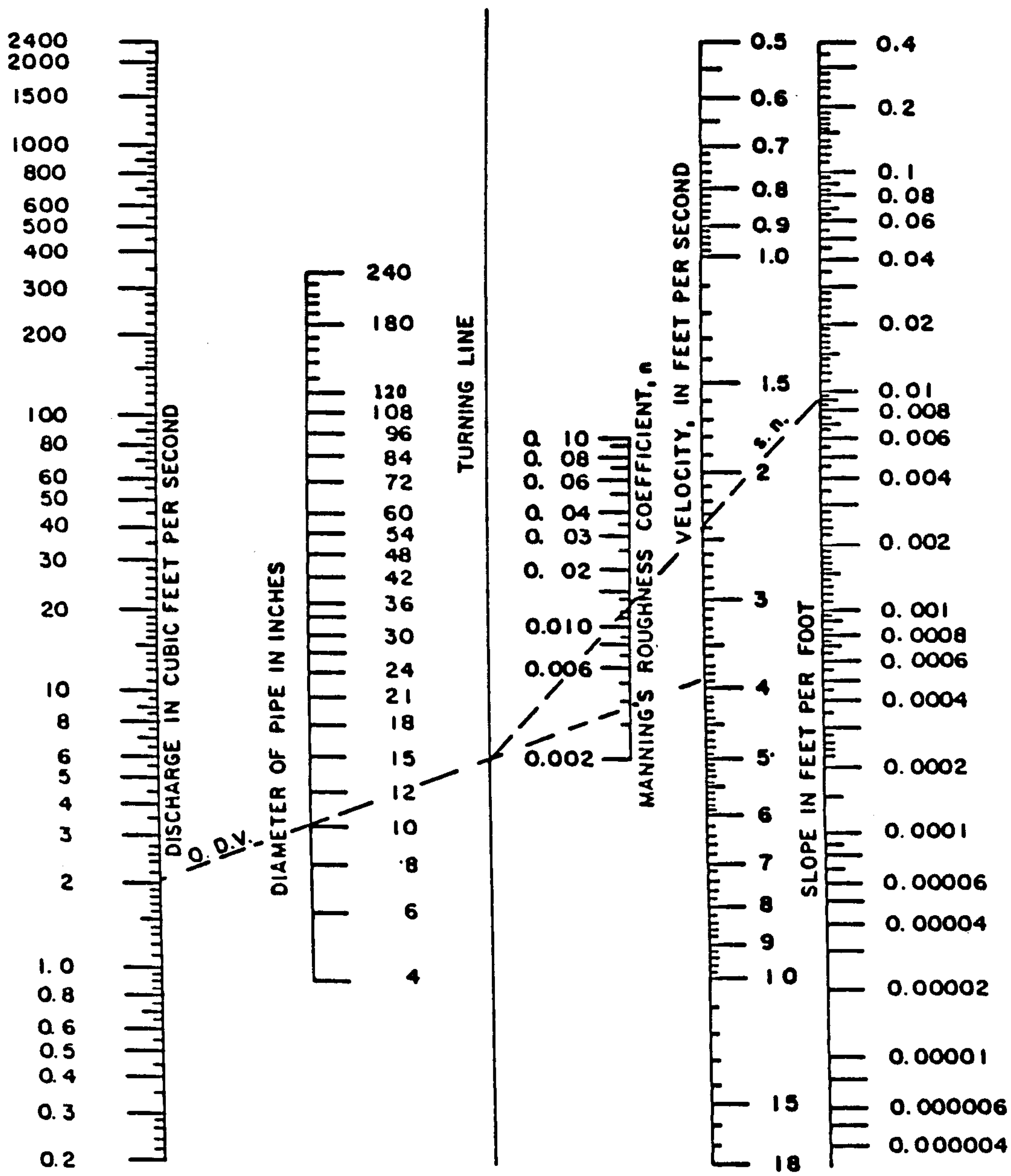


Figure 4.2-13 Nomograph for Solution of Manning's Formula for Flow in Storm Sewers
(Source: AASHTO Model Drainage Manual, 1991)

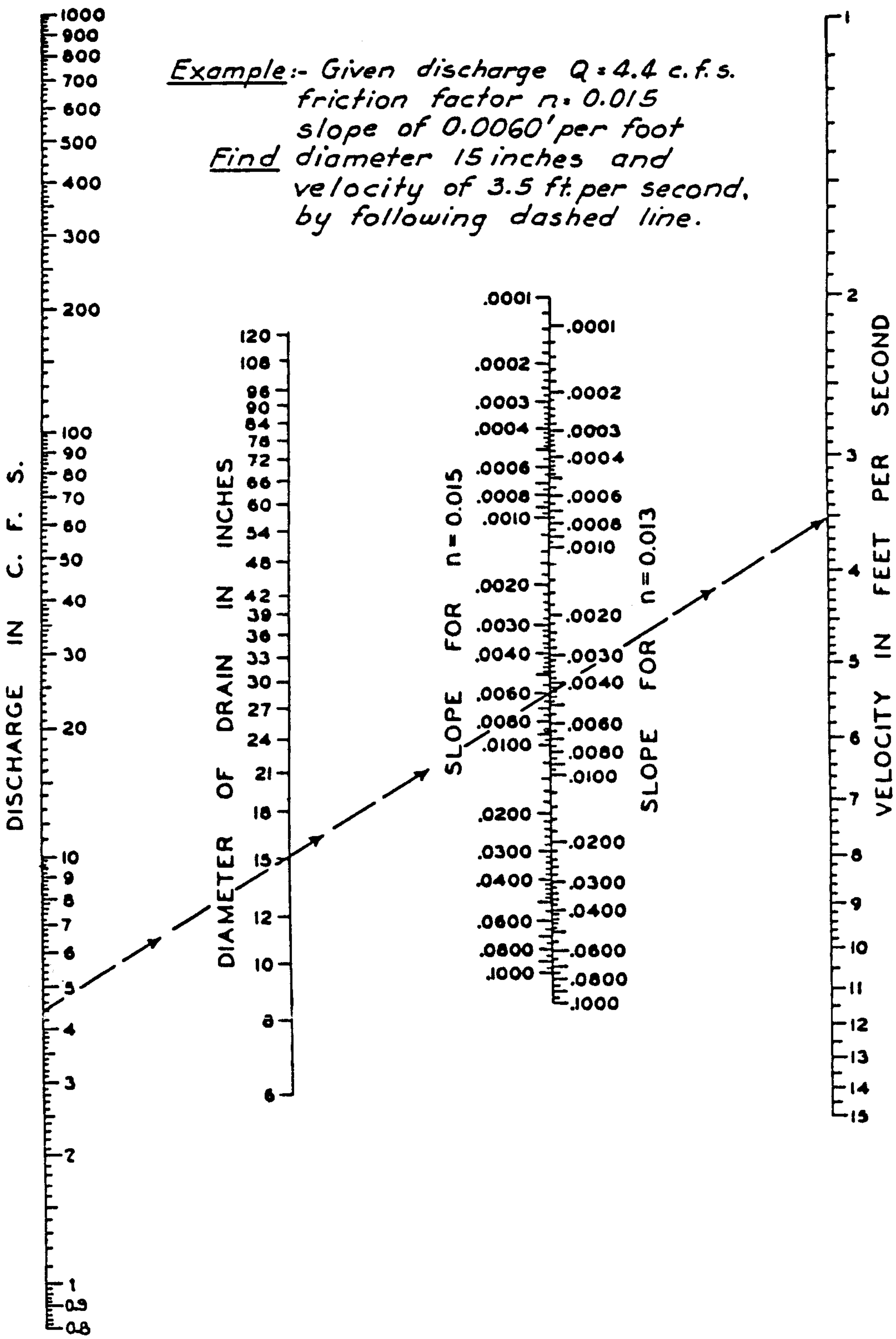


Figure 4.2-14 Nomograph for Computing Required Size of Circular Drain, Flowing Full n = 0.013 or 0.015

(Source: AASHTO Model Drainage Manual, 1991)

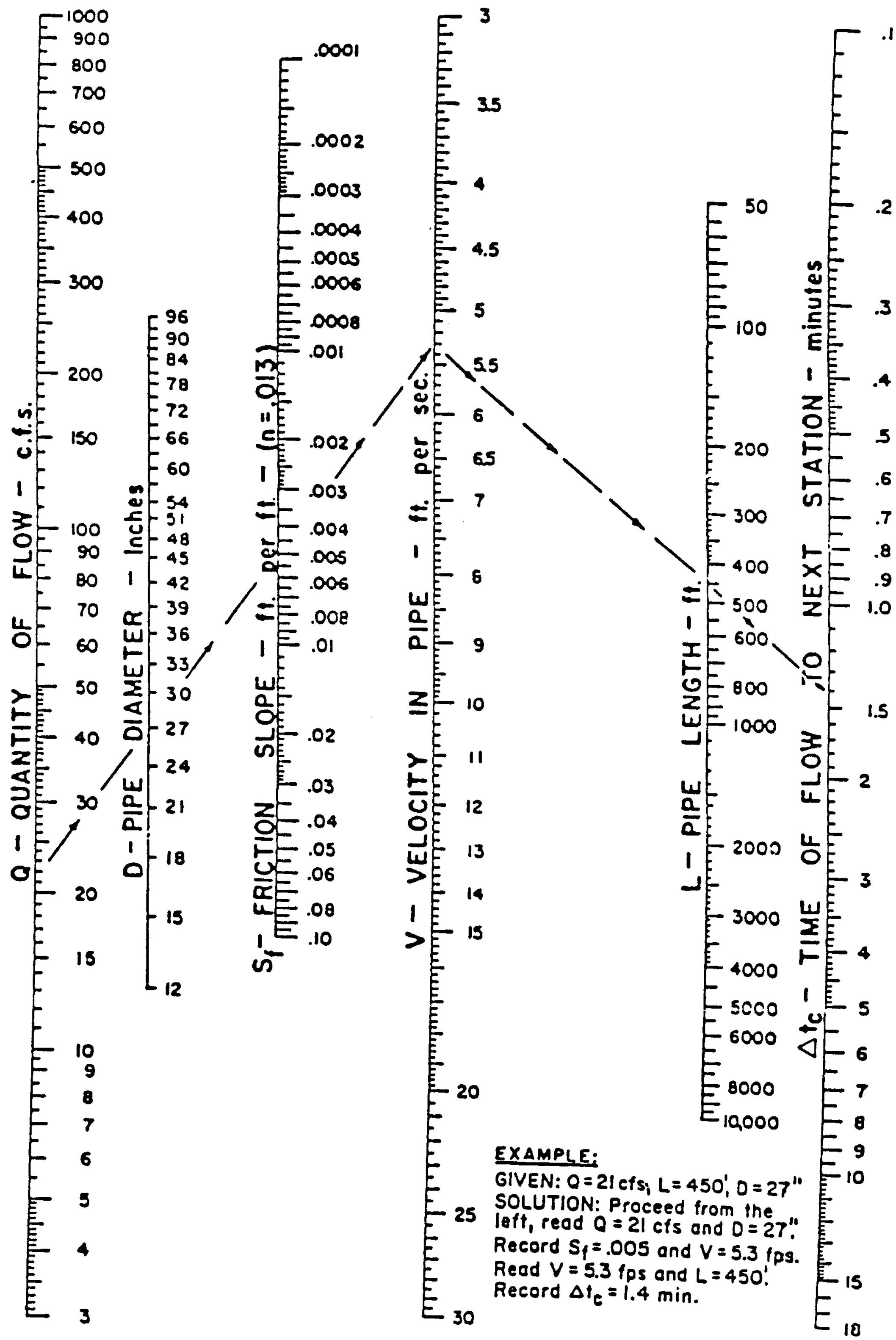
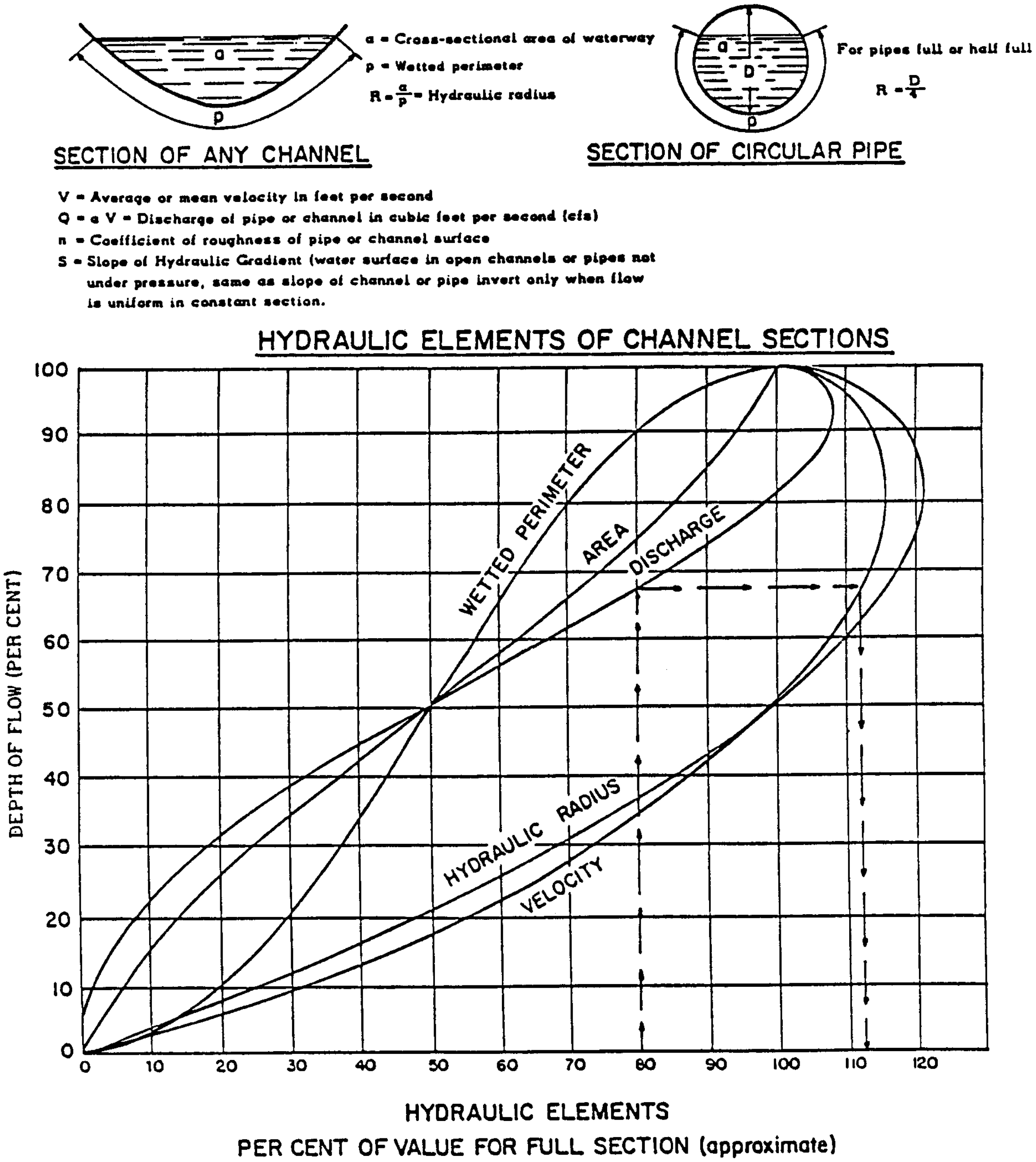


Figure 4.2-15 Concrete Pipe Flow Nomograph

(Source: AASHTO Model Drainage Manual, 1991)



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 4.2-16 Values of Various Elements of Circular Section for Various Depths of Flow
(Source: AASHTO Model Drainage Manual, 1991)

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_1	V_1	Q_1V_1	$\frac{V_1^2}{2g}$	H_1	ANGLE	H_Δ	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)

$H_1 = 0.35 \frac{V_1^2}{2g}$ $H_o = 0.25 \frac{V_o^2}{2g}$ $H_\Delta = K \frac{V_1^2}{2g}$

FINAL $H = H_f + H_t$

$H_t = H_o + H_1 + H_\Delta$

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 4.2-17 Hydraulic Grade Line Computation Form
(Source: AASHTO Model Drainage Manual, 1991)

4.2.8.7 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:

$$S = [(nV)^2]/[2.208R^{4/3}] \qquad (4.2.20)$$

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)

4.2.8.8 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 2.2 for more information).

References

U.S. Department of Transportation, Federal Highway Administration, 1984. Drainage of Highway Pavements. Hydraulic Engineering Circular No. 12.

American Association of State Highway and Transportation Officials, 1981 and 1998. Model Drainage Manual.

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SECTION

4.3

CULVERT DESIGN

4.3.1 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.

4.3.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual the symbols listed in Table 4.3-1 will be used. These symbols were selected because of their wide use.

Table 4.3-1 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
H_f	Depth of pool or head, above the face 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/f
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow	ft/s
V_c	Critical velocity	ft/s

4.3.3 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.

4.3.3.1 Frequency Flood

See Section 4.1 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.

4.3.3.2 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

4.3.3.3 Buoyancy Protection

Headwalls, endwalls, slope paving or other means of anchoring to provide buoyancy protection should be considered for all flexible culverts.

4.3.3.4 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.

4.3.3.5 Debris Control

In designing debris control structures it is recommended that the Hydraulic Engineering Circular No. 9 entitled *Debris Control Structures* be consulted.

4.3.3.6 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 4.4.3).
- Either the headwater should be set to produce acceptable velocities, or stabilization or energy dissipation should be provided where these velocities are exceeded.
- 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.

4.3.3.7 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 equivalent 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 4.4, *Open Channel Design*.
- If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.
- 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.

4.3.3.8 Storage

If storage is being assumed or will occur upstream of the culvert, refer to subsection 4.3.4.6 regarding storage routing as part of the culvert design.

4.3.3.9 Culvert Inlets

Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient K_e, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in Table 4.3-2.

4.3.3.10 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.

4.3.3.11 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.

4.3.3.12 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.

4.3.3.13 Material Selection

Reinforced concrete pipe (RCP) is recommended for use (1) under a roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams. RCP and fully coated corrugated metal pipe can be used in all other cases. High-density polyethylene (HDPE) pipe may also be used as specified in the municipal regulations. Table 4.3-3 gives recommended Manning's n values for different materials.

4.3.3.14 Culvert Skews

Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval.

4.3.3.15 Culvert Sizes

The minimum allowable pipe diameter shall be 18 inches.

4.3.3.16 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.

Table 4.3-2 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 (groove-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^o or 45^o 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^o or 45^o 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^o to 75^o 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^o or 25^o 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, 1985

Table 4.3-3 Manning's n Values

Type of Conduit	Wall & Joint Description	Manning's n
Concrete Pipe	Good joints, smooth walls	0.012
	Good joints, rough walls	0.016
	Poor joints, rough walls	0.017
Concrete Box	Good joints, smooth finished walls	0.012
	Poor joints, rough, unfinished walls	0.018
Corrugated Metal Pipes and Boxes Annular Corrugations	2 2/3- by ½-inch corrugations	0.024
	6- by 1-inch corrugations	0.025
	5- by 1-inch corrugations	0.026
	3- by 1-inch corrugations	0.028
	6-by 2-inch structural plate	0.035
	9-by 2-1/2 inch structural plate	0.035
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	2 2/3-by ½-inch corrugated 24-inch plate width	0.012
Spiral Rib Metal Pipe	3/4 by 3/4 in recesses at 12 inch spacing, good joints	0.013
High Density Polyethylene (HDPE)	Corrugated Smooth Liner	0.015
	Corrugated	0.020
Polyvinyl Chloride (PVC)		0.011

Source: HDS No. 5, 1985

Note: For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS No. 5, page 163

4.3.3.17 Outlet Protection

See Section 4.5 for information on the design of outlet protection. Outlet protection should be provided for the 25-year storm.

4.3.3.18 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.

4.3.3.19 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.

4.3.4 Design Procedures

4.3.4.1 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.

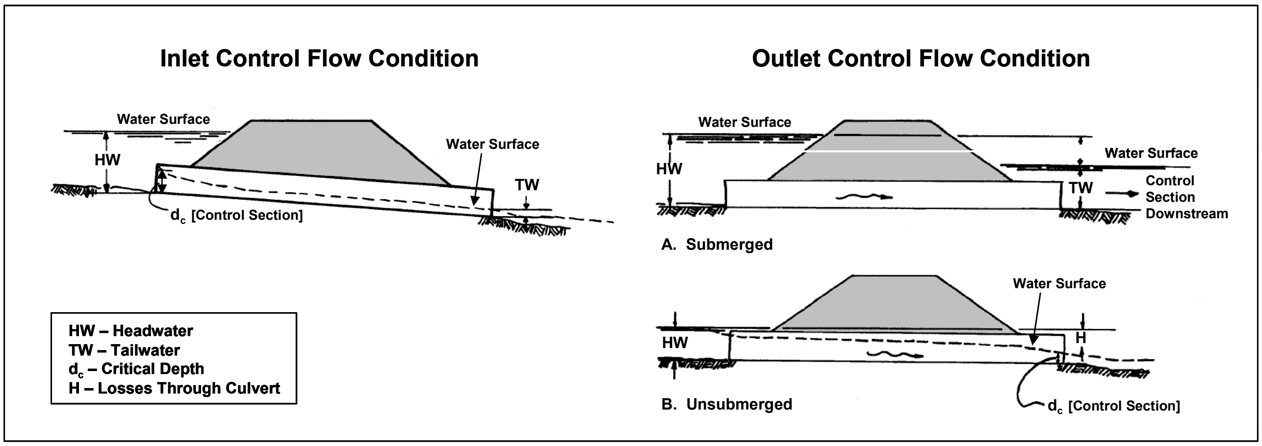


Figure 4.3-1 Culvert Flow Conditions
(Adapted from: HDS-5, 1985)

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, 1985.

4.3.4.2 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.

4.3.4.3 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 outlet scour require additional, separate computations beyond what can be obtained from the nomographs. Figures 4.3-2(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 subsection 4.3.8.

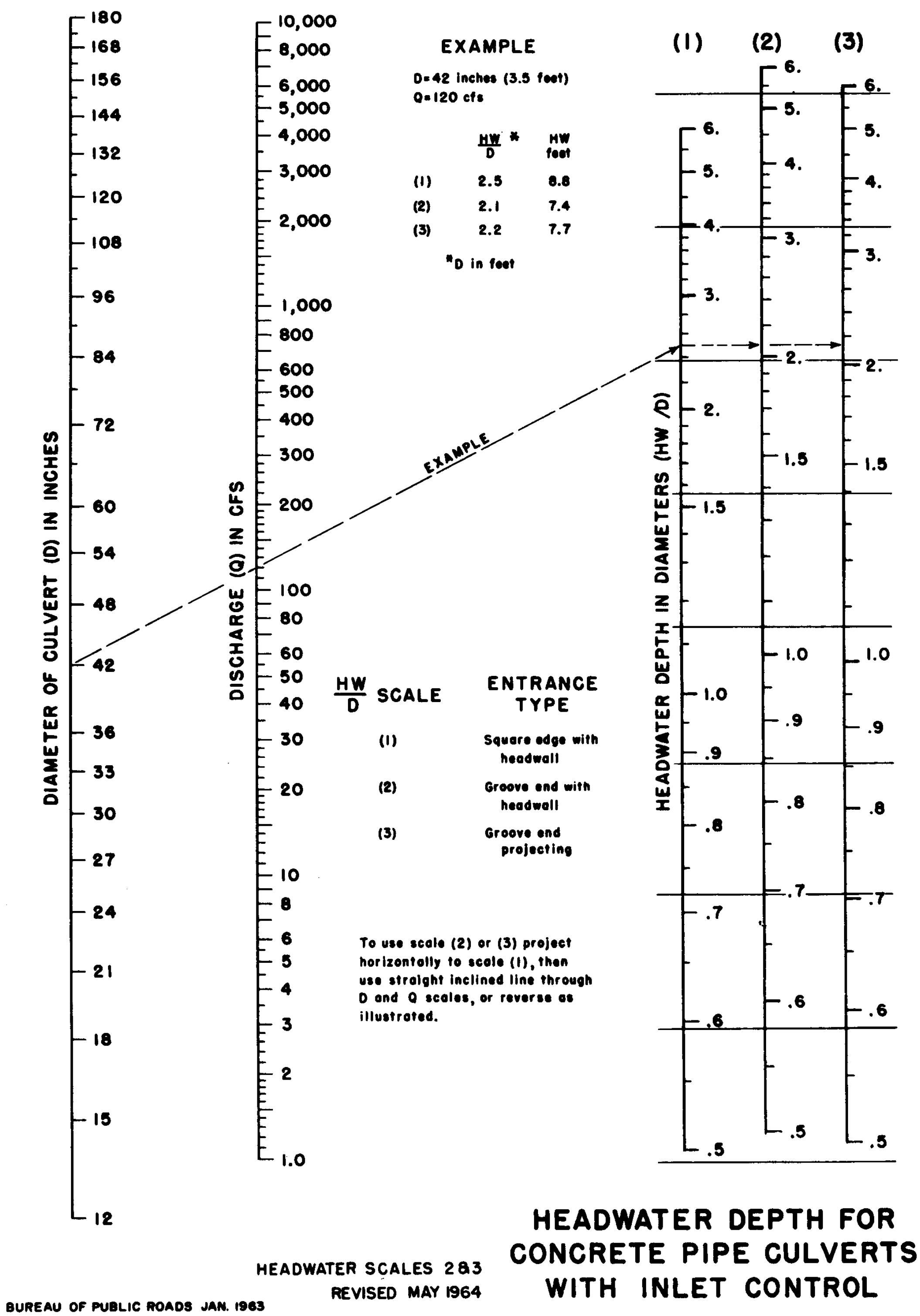


Figure 4.3-2(a) Headwater Depth for Concrete Pipe Culvert with Inlet Control

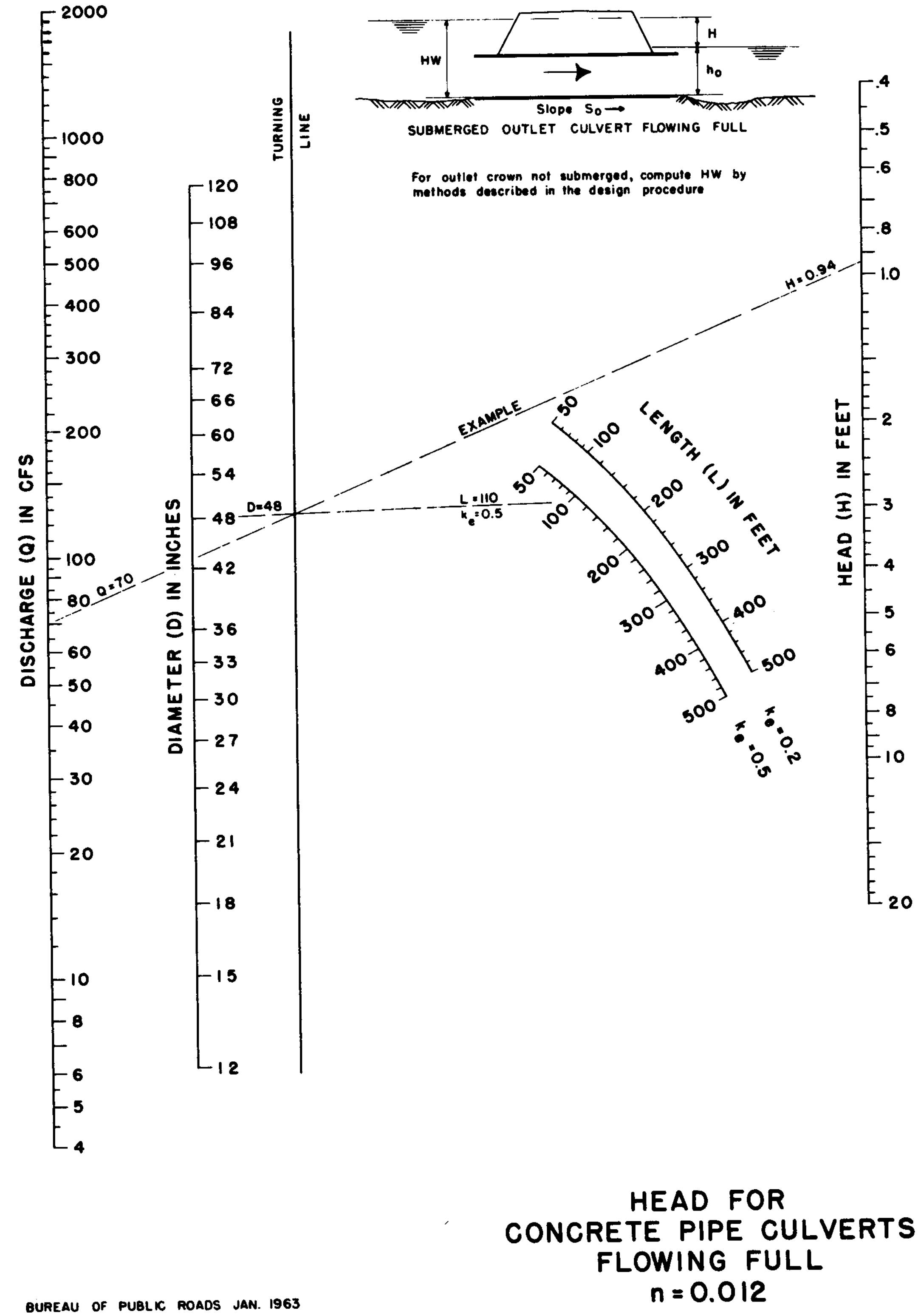


Figure 4.3-2(b) Head for Concrete Pipe Culverts Flowing Full

4.3.4.4 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:

$$HW = H + h_o - LS \qquad (4.3.1)$$

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 4.5 for appropriate energy dissipation designs.

4.3.4.5 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 4.3.2 to calculate flow rates across the roadway.

$$Q = C_d L(HW)^{1.5} \qquad (4.3.2)$$

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 4.3-3 on the next page 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.

4.3.4.6 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 4.3.7 and Section 2.2 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.

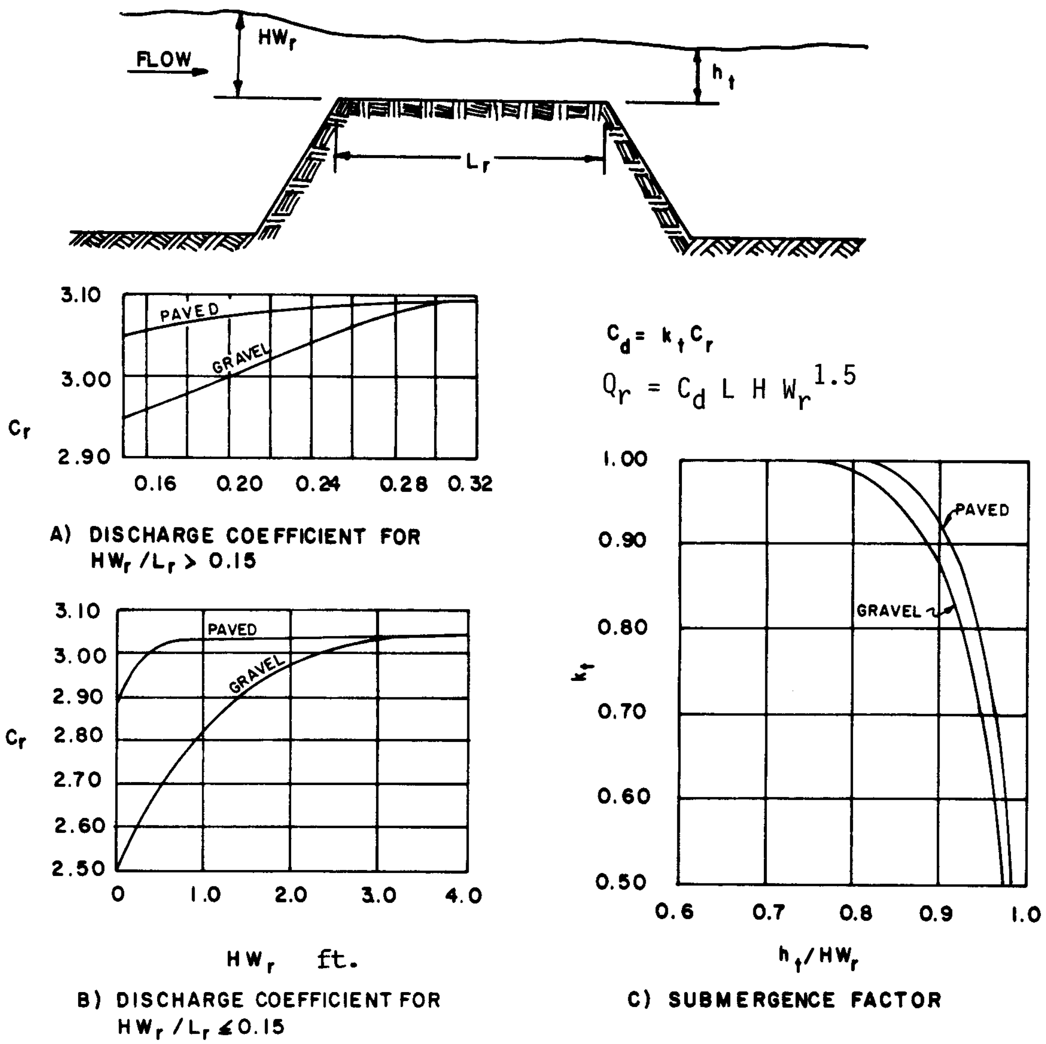


Figure 4.3-3 Discharge Coefficients for Roadway Overtopping
(Source: HDS No. 5, 1985)

4.3.5 Culvert Design Example

4.3.5.1 Introduction

The following example problem illustrates the procedures to be used in designing culverts using the nomographs.

4.3.5.2 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.

4.3.5.3 Example Data

Input Data

Discharge for 2-yr flood = 35 cfs
Discharge for 25-yr flood = 70 cfs
Allowable H_W 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

4.3.5.4 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 = $(\text{Area times } 4/3.14)^{0.5}$.
Therefore: D = $((14 \text{ sq ft} \times 4)/3.14)^{0.5} \times 12$ in/ft) = 50.7 in

(3) A grooved end culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 4.3-1), 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 4.3-2.

With an entrance loss coefficient K_e 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

Note: critical depth is obtained from Chart 4 on page 4.3-24

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 4.5 (*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 4.3-4 provides a convenient form to organize culvert design calculations.

PROJECT: ____________________	STATION: ____________	CULVERT DESIGN FORM
____________________	SHEET ______ OF ______	DESIGNER / DATE: ______ OF ______ REVIEWER / DATE: ______ OF ______

SEE ADD'L SHTS.

HYDROLOGICAL DATA

- ☐ METHOD: ____________
- ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______
- ☐ CHANNEL SHAPE: ____________
- ☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

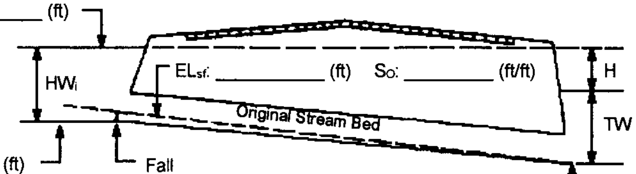
R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION: __________ (ft)

EL_{hi}: ______ (ft)

EL_i: ______ (ft)

EL_{sf}: ______ (ft) S_O: ______ (ft/ft)



EL_o: ______ (ft)

$S = S_o - Fall/L_o$

S = ______ (ft/ft)

L_o = ______ (ft)

CULVERT DESCRIPTION: MATERIAL-SHAPE-SIZE-ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW/D (2)	HW_i	FALL (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) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL 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) $H = [1+k_e+(29\ n^2L)/R^{1.33}]\ V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUSCRIPT DEFINITIONS:

a. Approximate
f. Culvert Face
hd. Design Headwater
hi. Headwater in Inlet Control
ho. Headwater in Outlet Control
i. Inlet Control Section
o. Outlet
sf. Streamed at Culvert Face
tw. Tailwater

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ____________

SHAPE: ____________

MATERIAL: ____________ n ______

ENTRANCE: ____________

Figure 4.3-4 Culvert Design Calculation Form
(Source: HDS No. 5, 1985)

4.3.6 Design Procedures for Beveled-Edged Inlets

4.3.6.1 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 interested 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.

4.3.6.2 Design Figures

Four inlet control figures for culverts with beveled edges are included in subsection 4.3.8.

Chart	Page	Use for :
3	A-3	circular pipe culverts with beveled rings
10	A-10	90^{o} headwalls (same for 90^{o} 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

4.3.6.3 Design Procedure

The figures for bevel-edged inlets are used for design in the same manner as the conventional inlet design nomographs discussed earlier. Note that Charts 10, 11, and 12 in subsection 4.3.8 apply only to bevels having either a 33^{o} angle (1.5:1) or a 45^{o} 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.

4.3.6.4 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.

4.3.6.5 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.

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.

4.3.6.6 Skewed Inlets

It is recommended that Chart 11 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.

4.3.7 Flood Routing and Culvert Design

4.3.7.1 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.

4.3.7.2 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

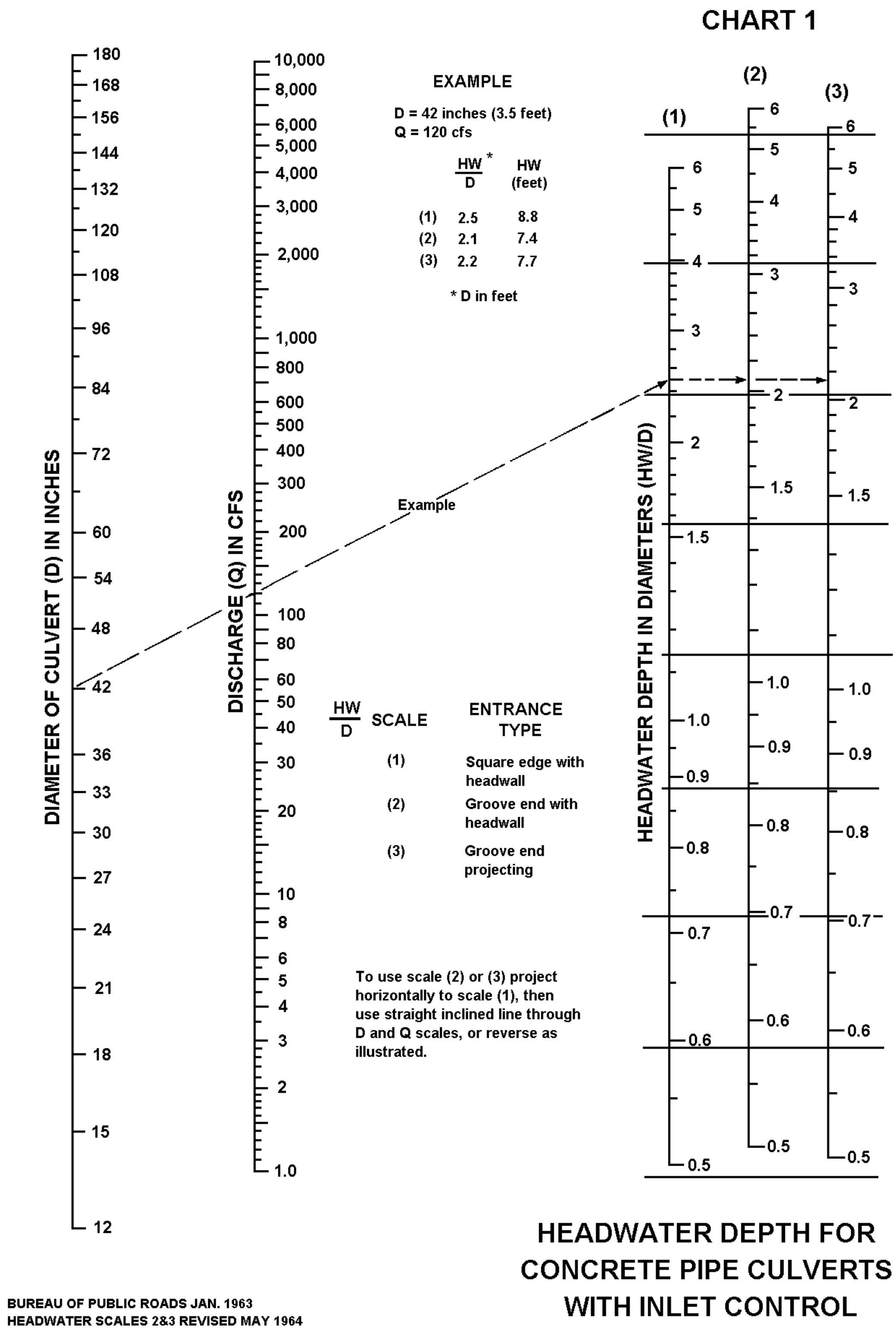
(Step 3) Flood routing computations are made with successive trial discharges until the flood routing equation is satisfied

(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.

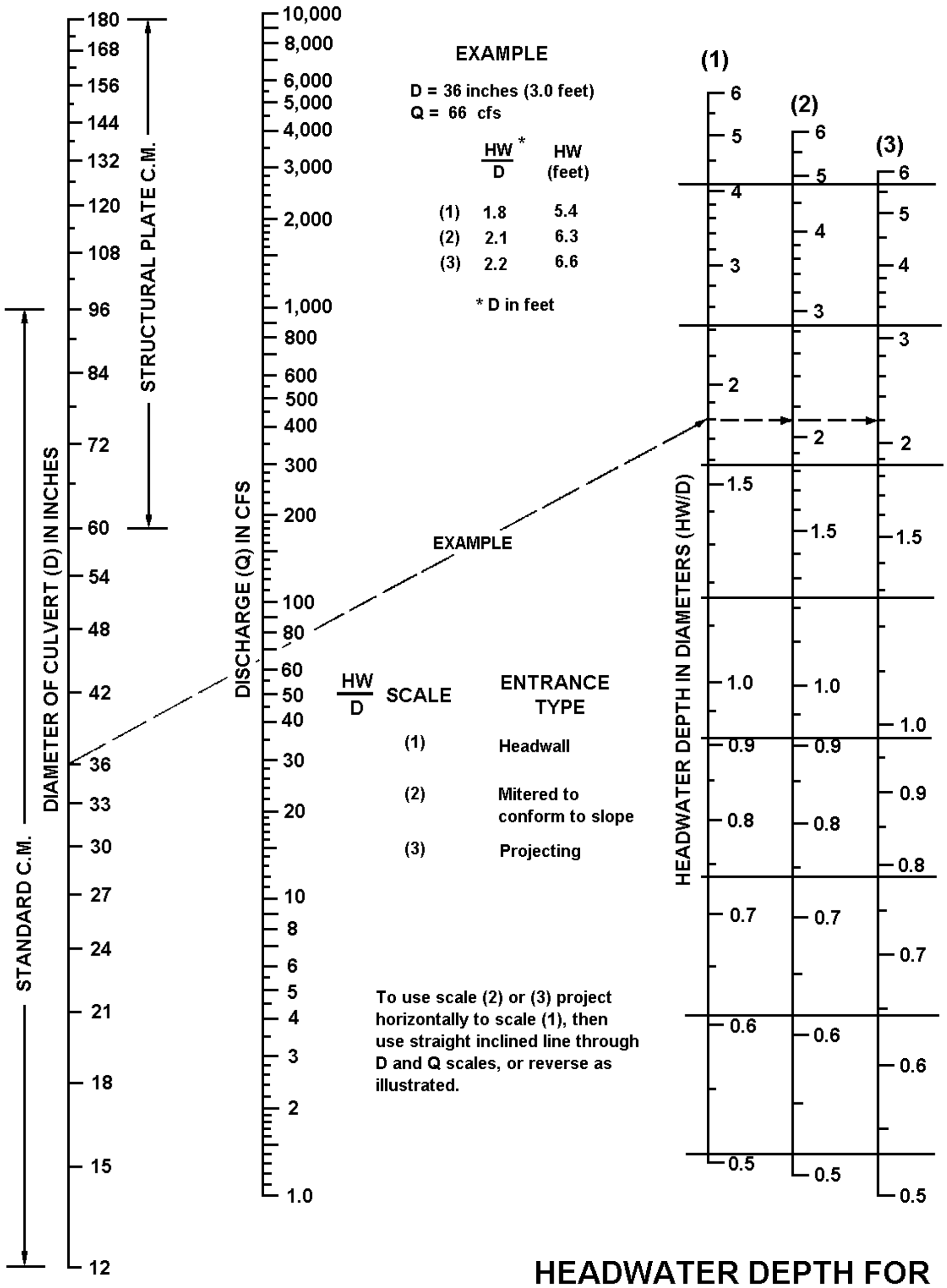
4.3.8 Culvert Design Charts and Nomographs

All of the figures in this section are from the AASHTO Model Drainage Manual, 1991.



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

CHART 2

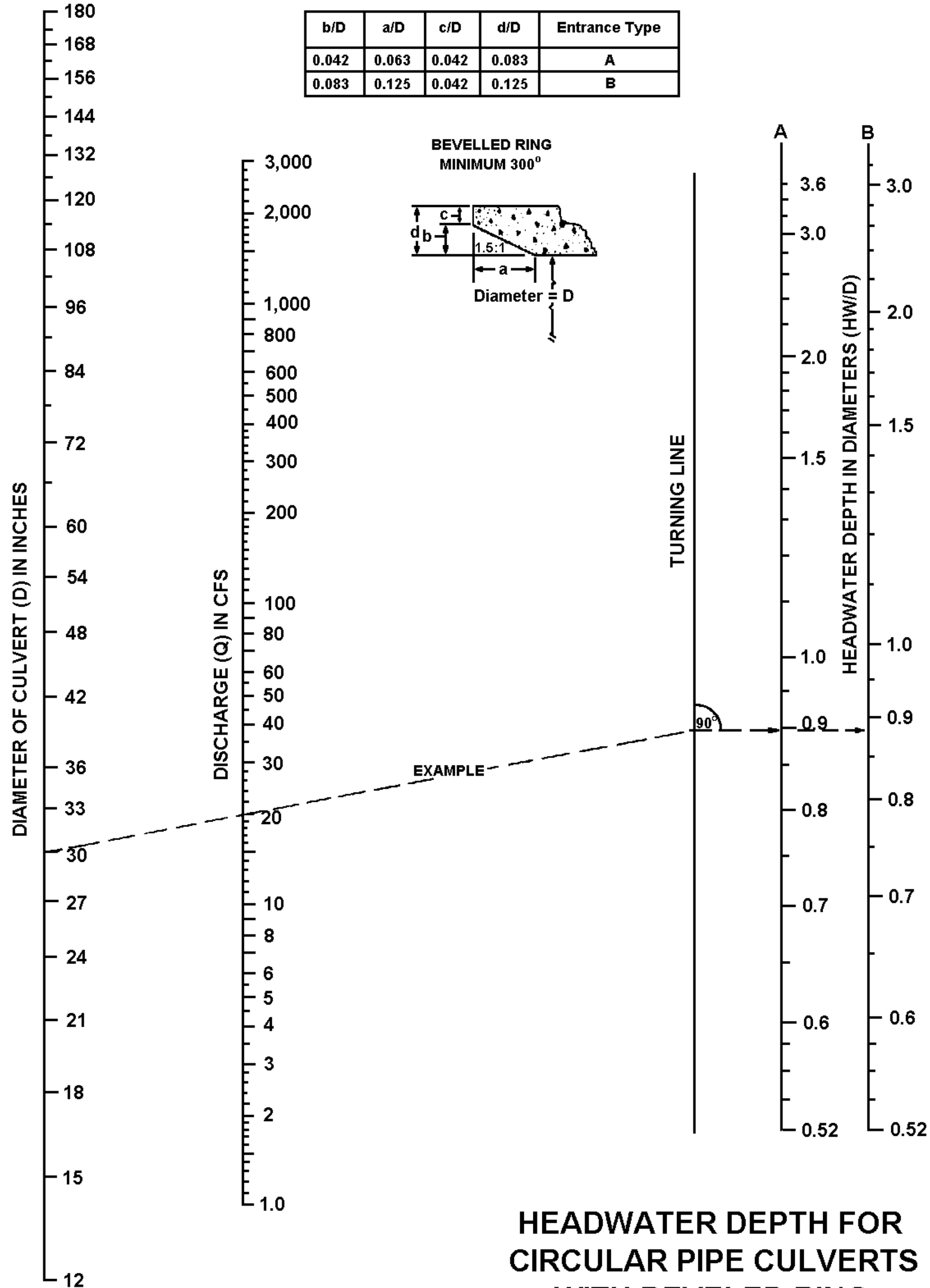


HEADWATER DEPTH FOR C.M. PIPE CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 3

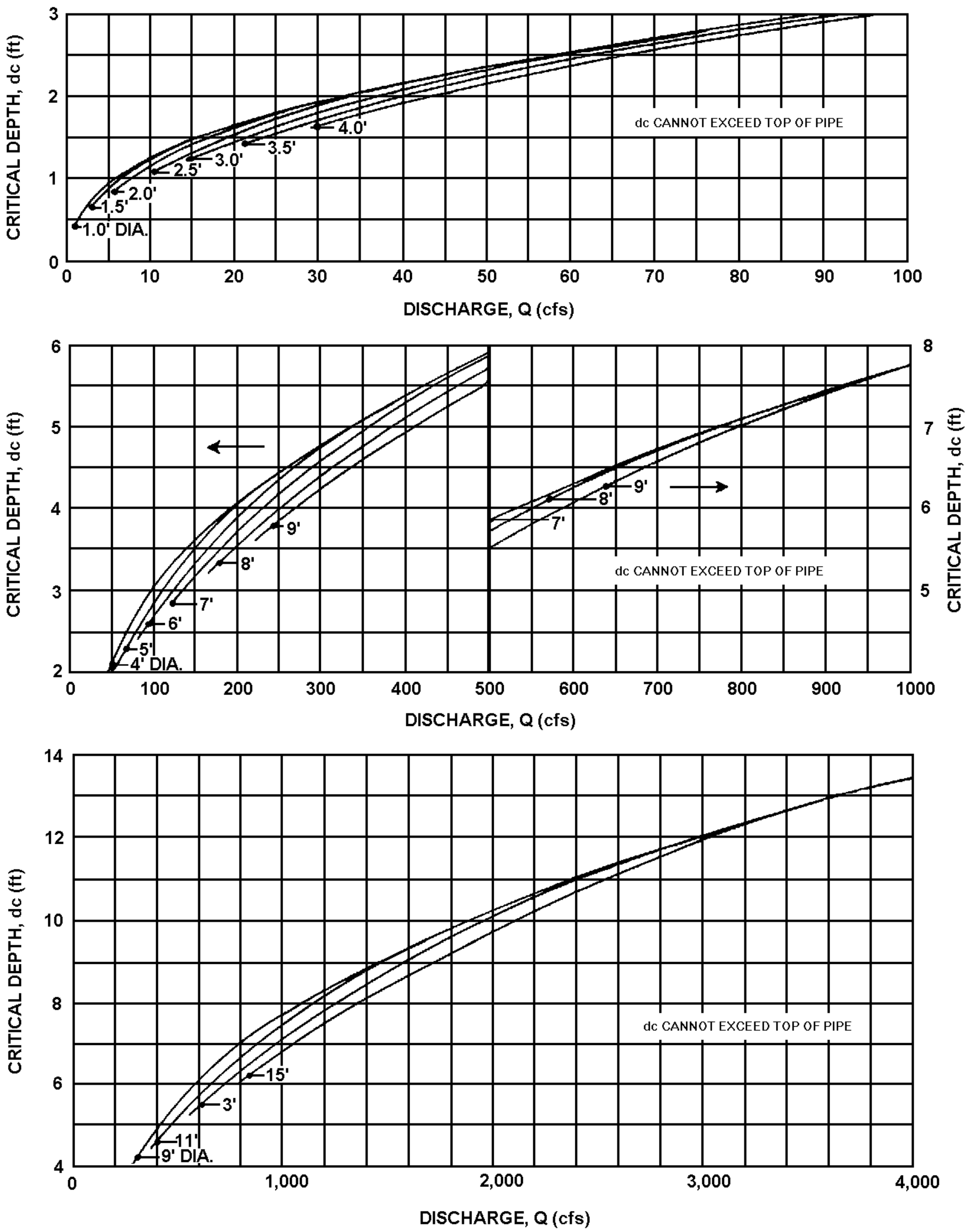
b/D	a/D	c/D	d/D	Entrance Type
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



FEDERAL HIGHWAY ADMINISTRATION MAY 1973

HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL

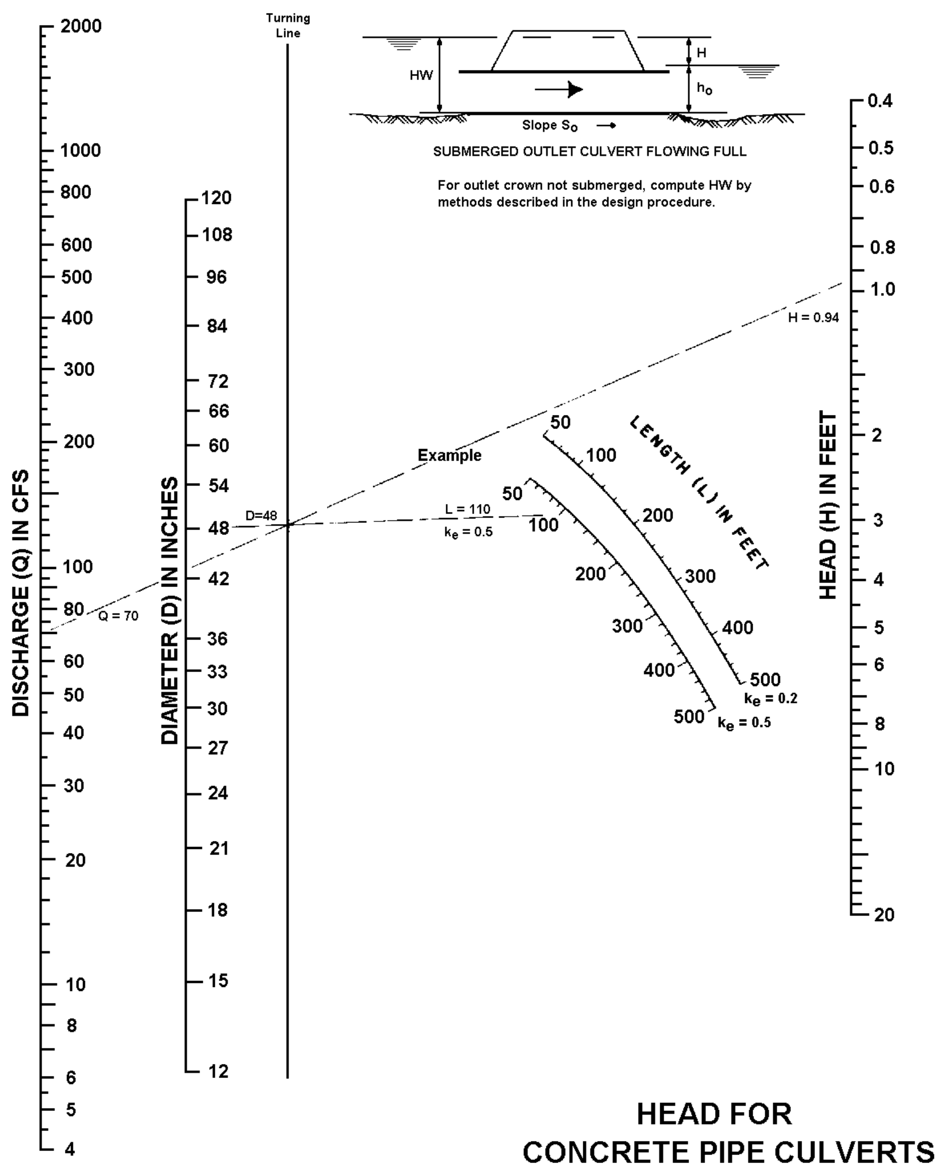
CHART 4



BUREAU OF PUBLIC ROADS JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

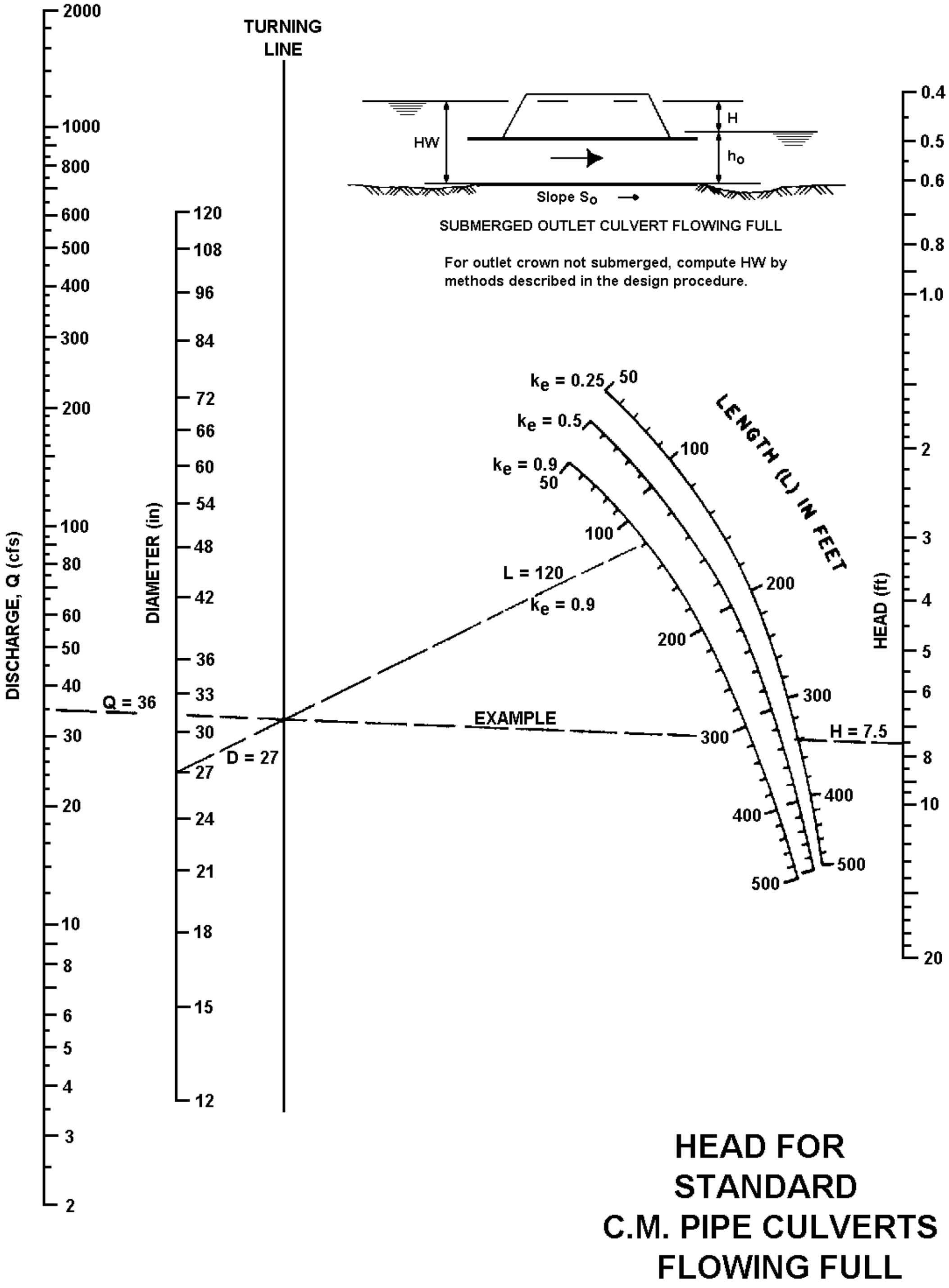
CHART 5



HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

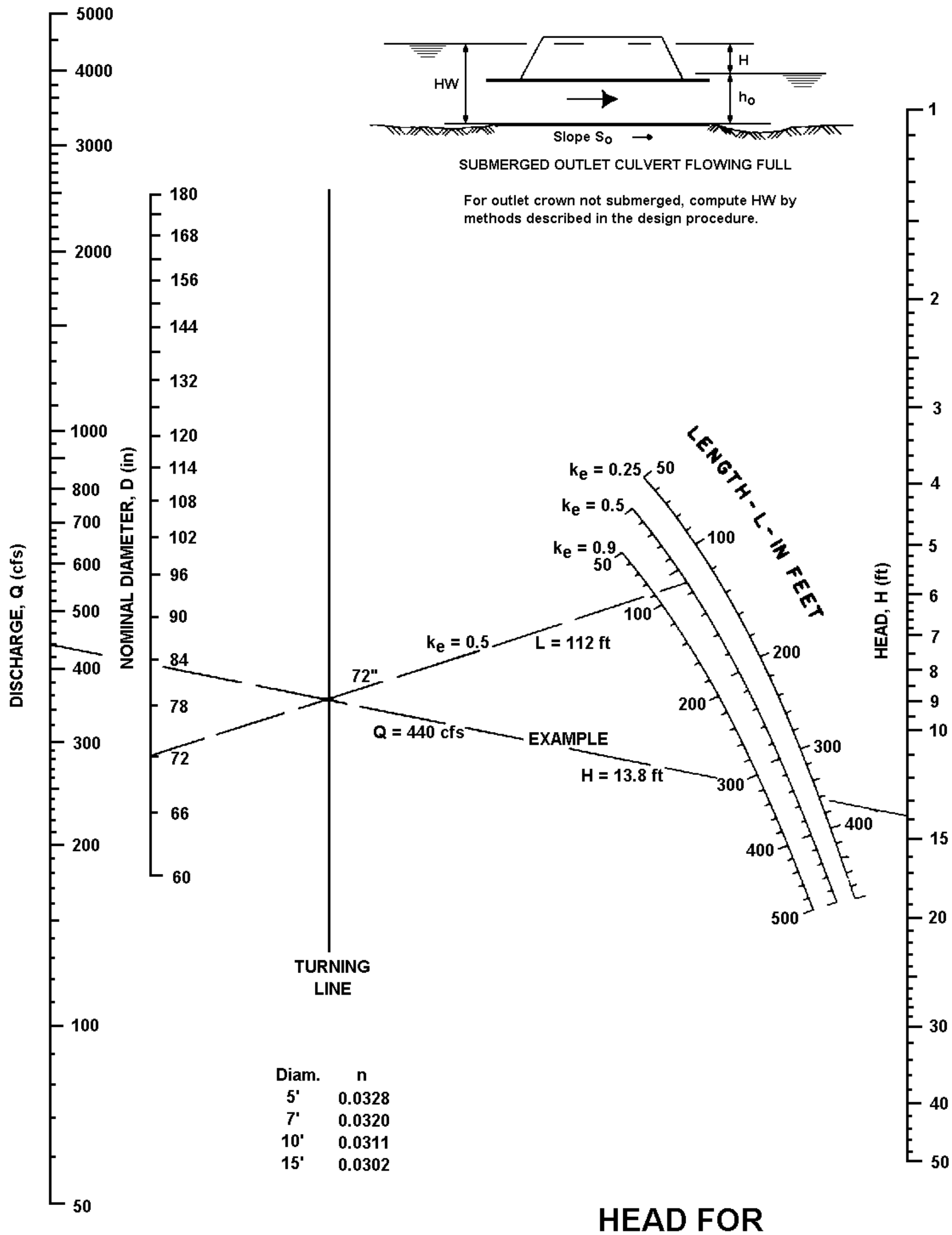
CHART 6



HEAD FOR STANDARD C.M. PIPE CULVERTS FLOWING FULL n = 0.024

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 7

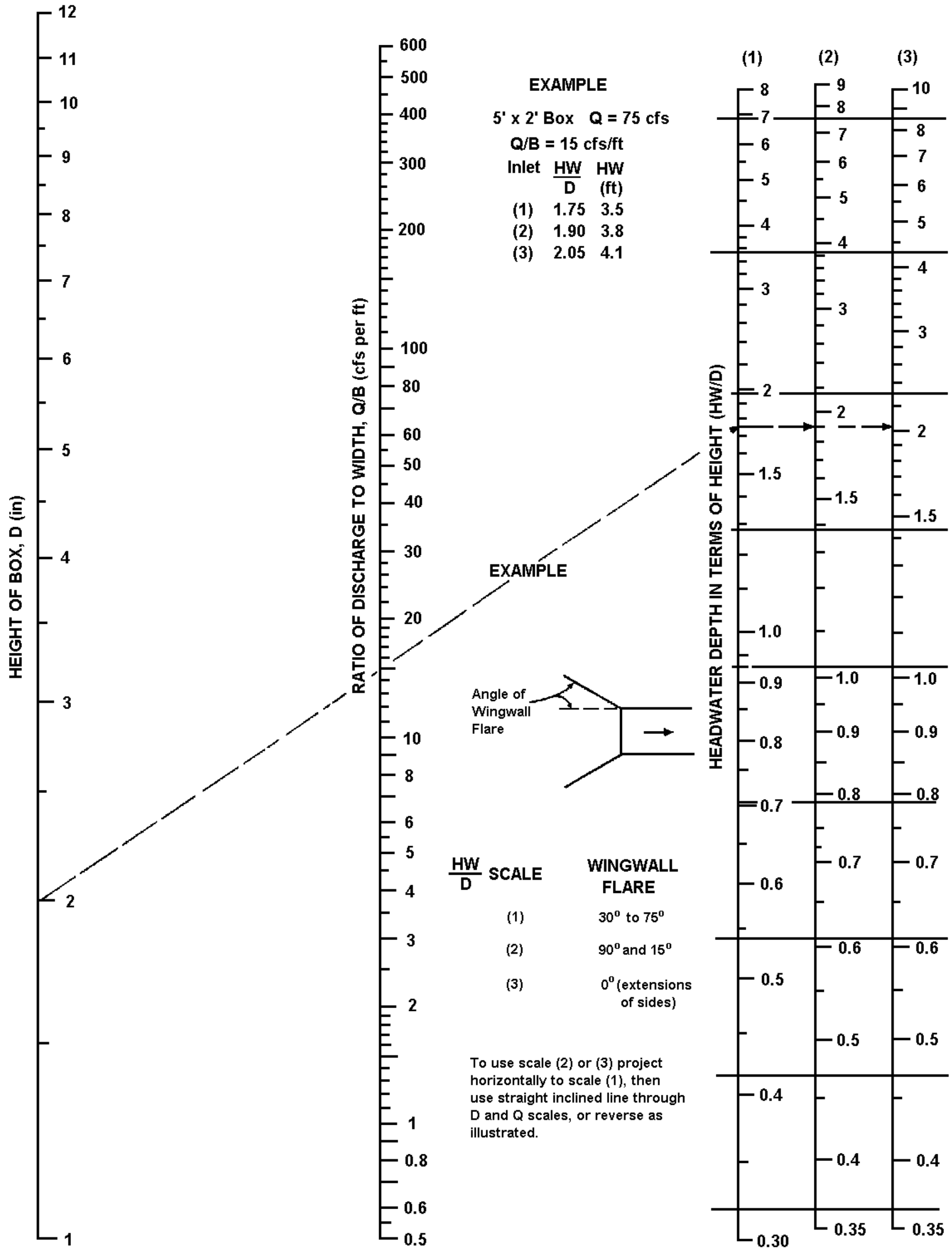


Diam.	n
5'	0.0328
7'	0.0320
10'	0.0311
15'	0.0302

BUREAU OF PUBLIC ROADS JAN. 1963

HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL
n = 0.0328 TO 0.0302

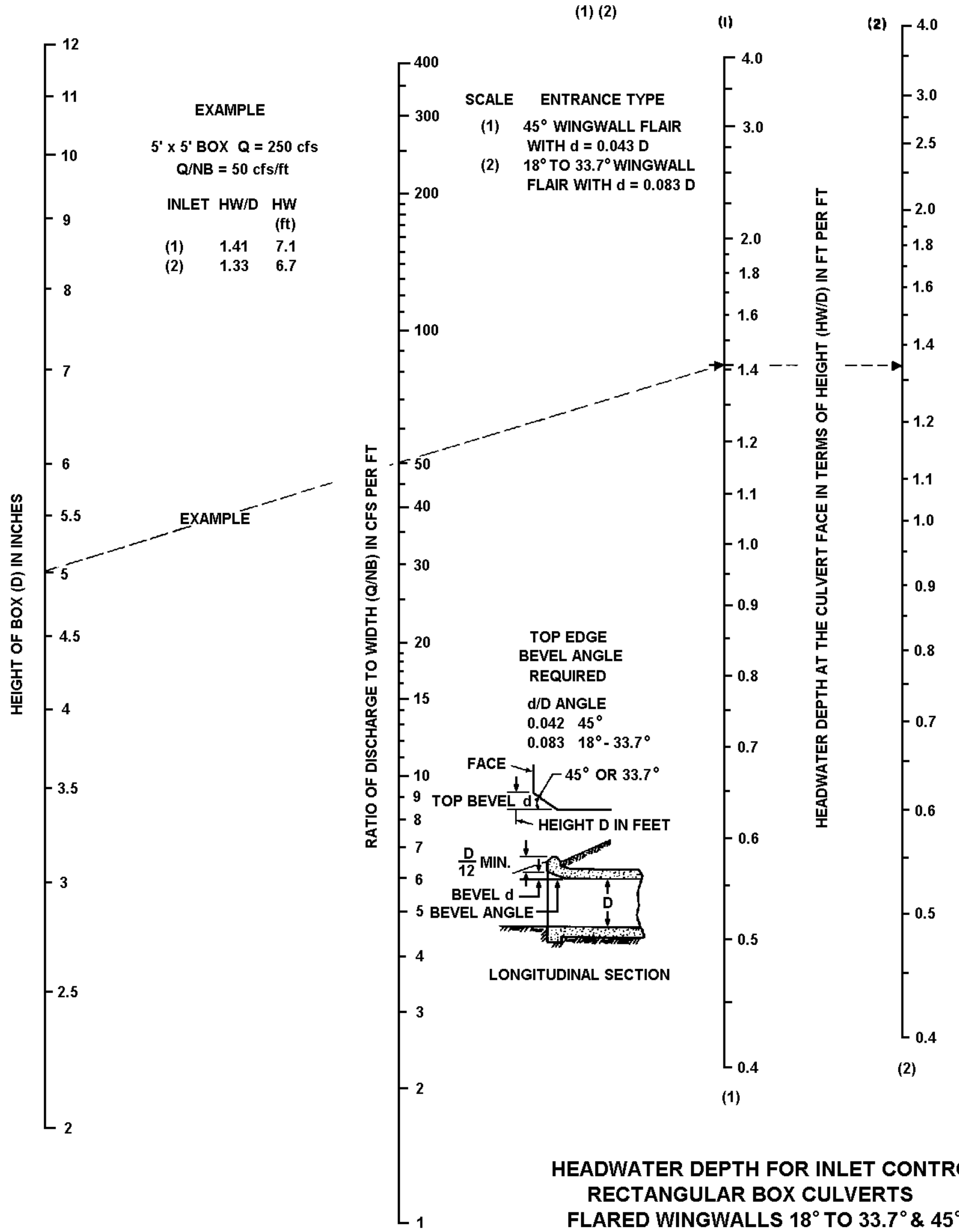
CHART 8



BUREAU OF PUBLIC ROADS JAN. 1963

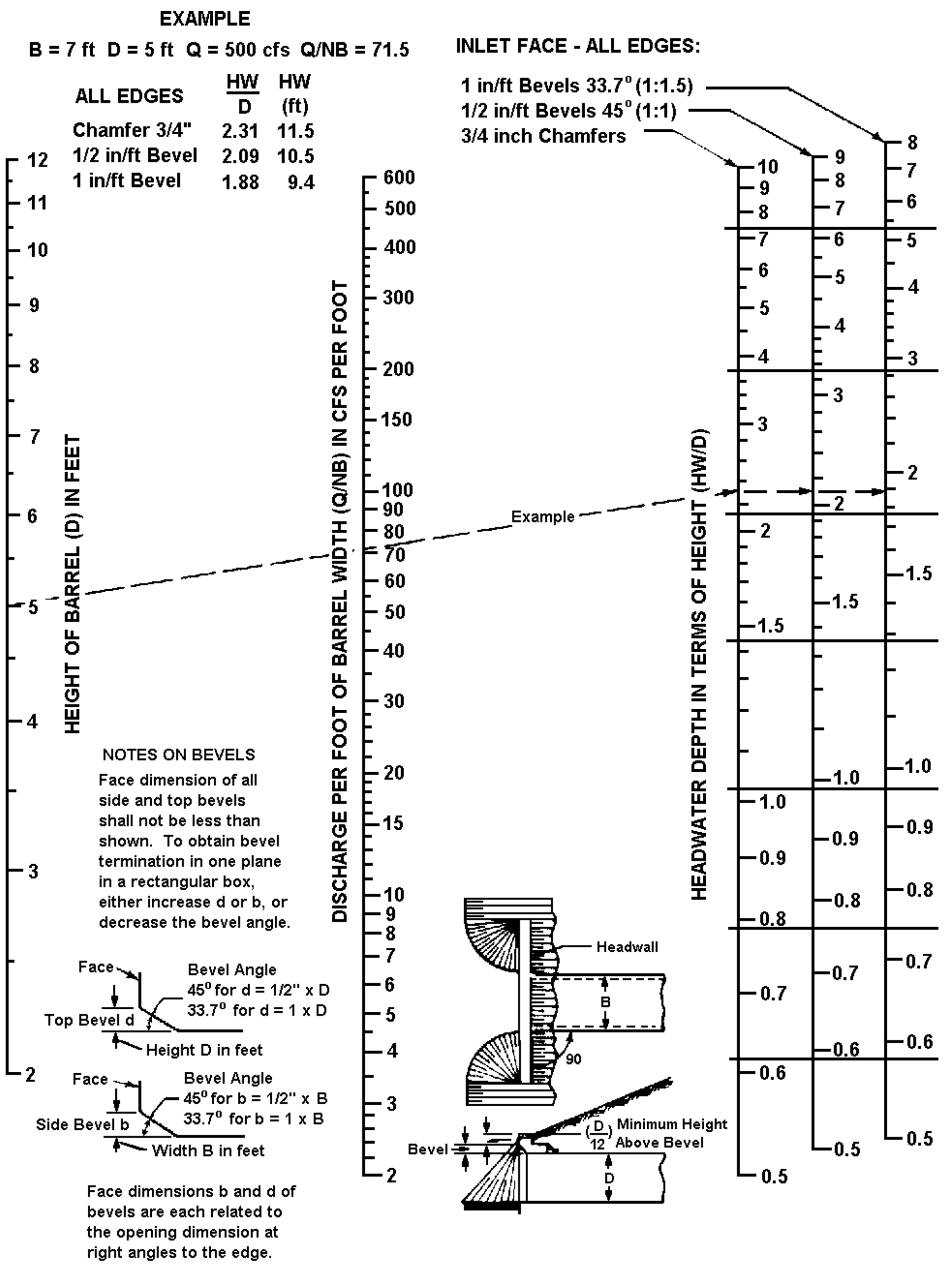
HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

CHART 9



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° & 45°
WITH BEVELED EDGE AT TOP OF INLET

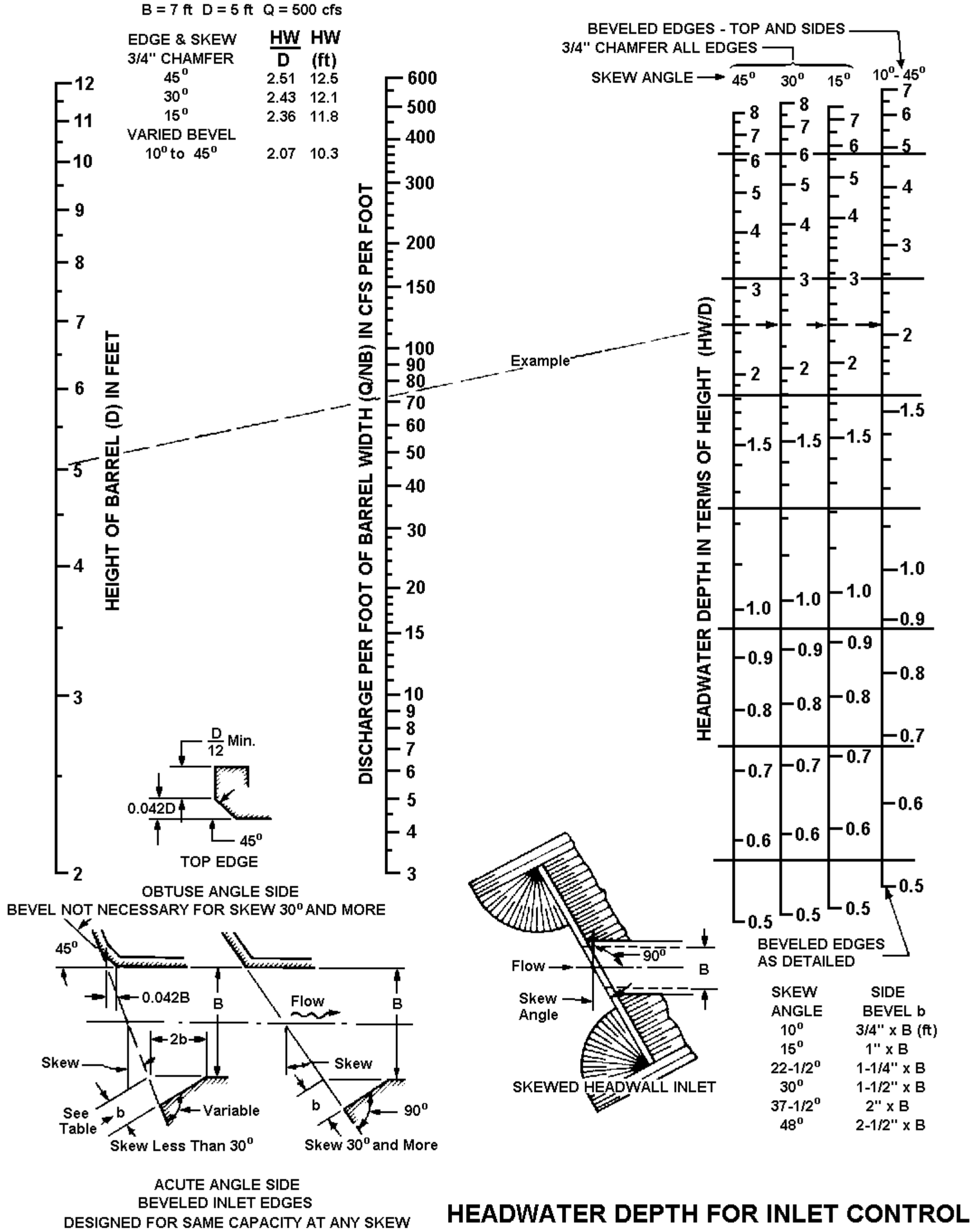
CHART 10



FEDERAL HIGHWAY ADMINISTRATION MAY 1973

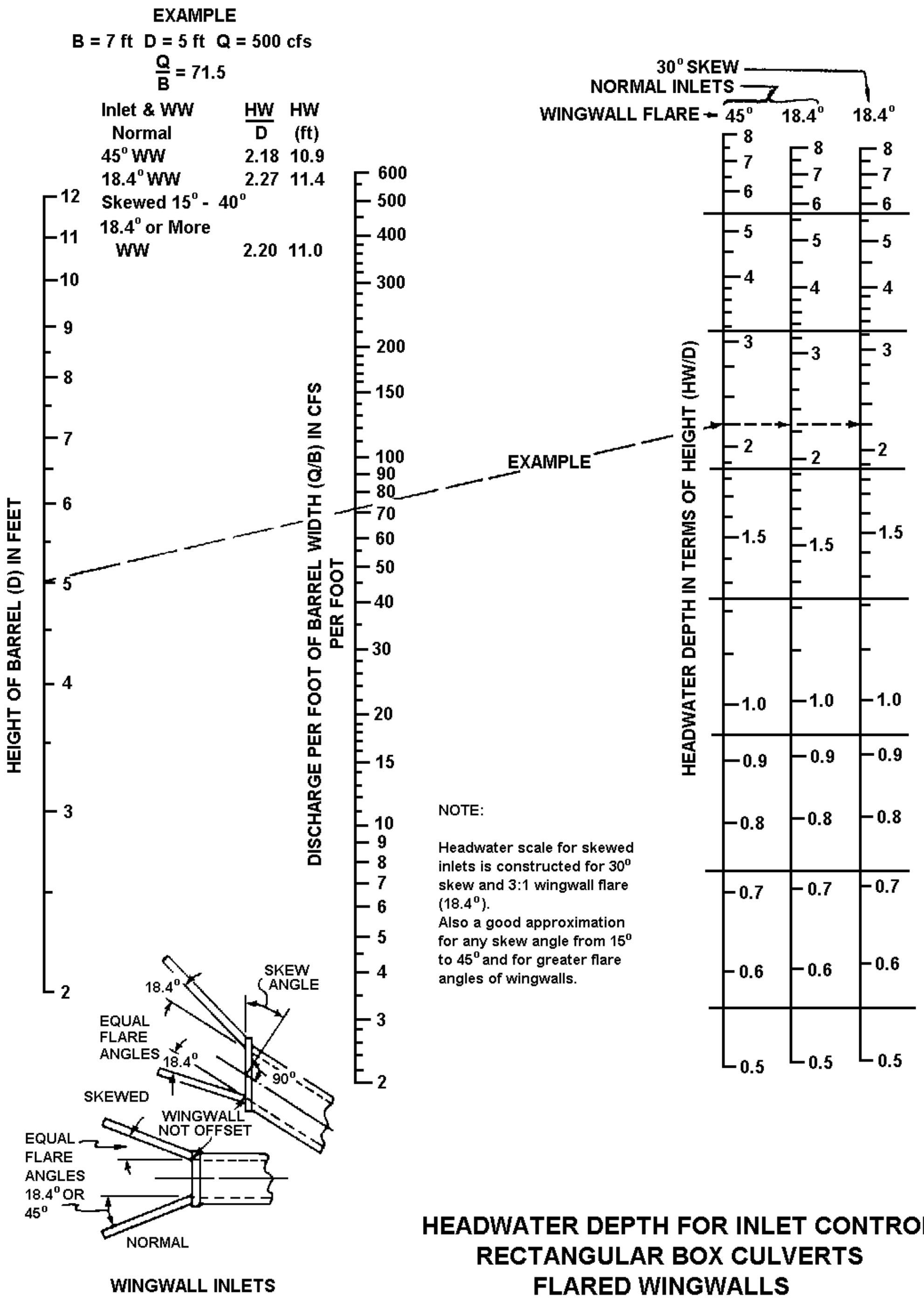
HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
90° HEADWALL
CHAMFERED OR BEVELED INLET EDGES

CHART 11



HEADWATER DEPTH FOR INLET CONTROL
SINGLE BARREL BOX CULVERTS
SKEWED HEADWALLS
CHAMFERED OR BEVELED INLET EDGES

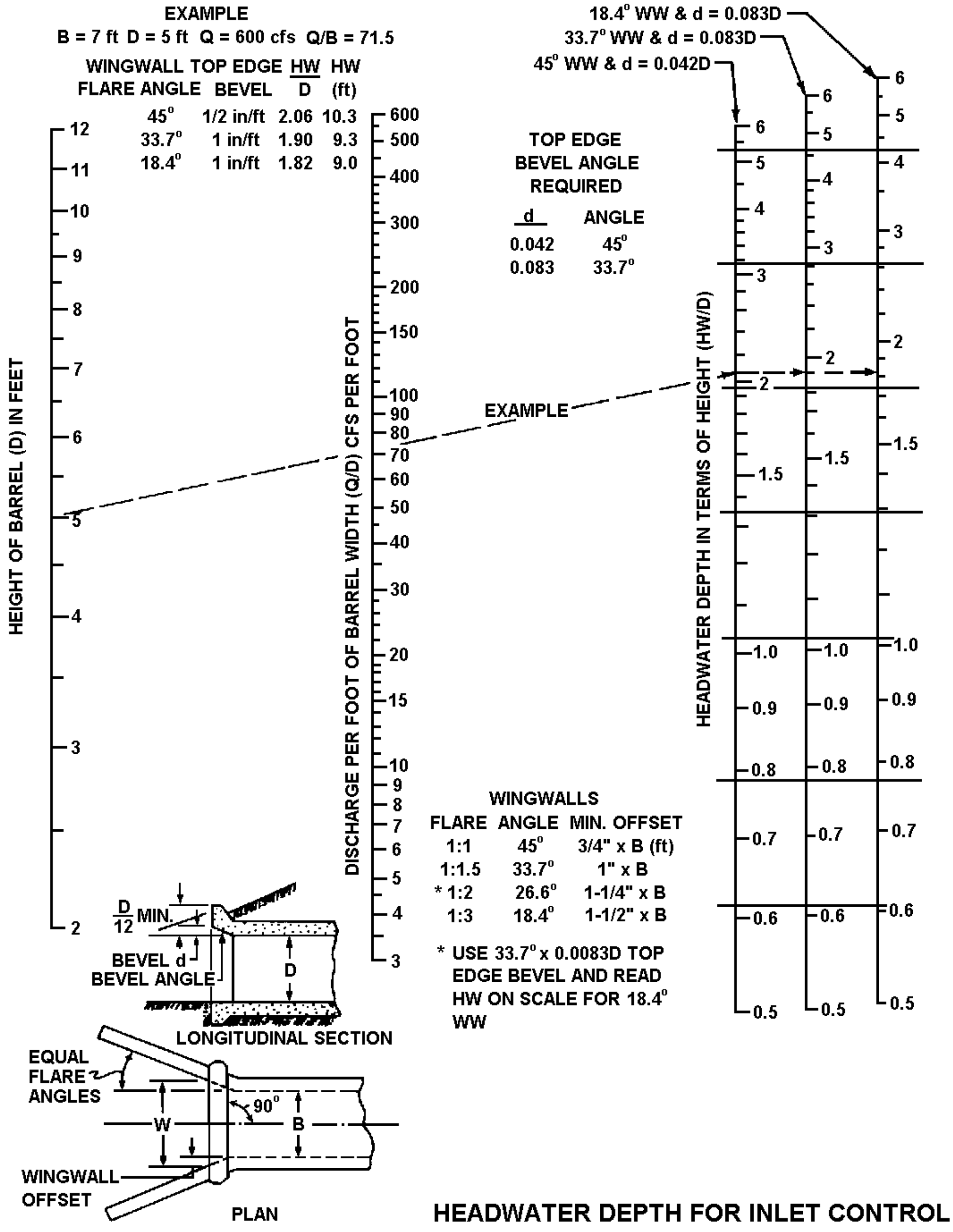
CHART 12



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS
NORMAL AND SKEWED INLETS
3/4" CHAMFER AT TOP OF OPENING

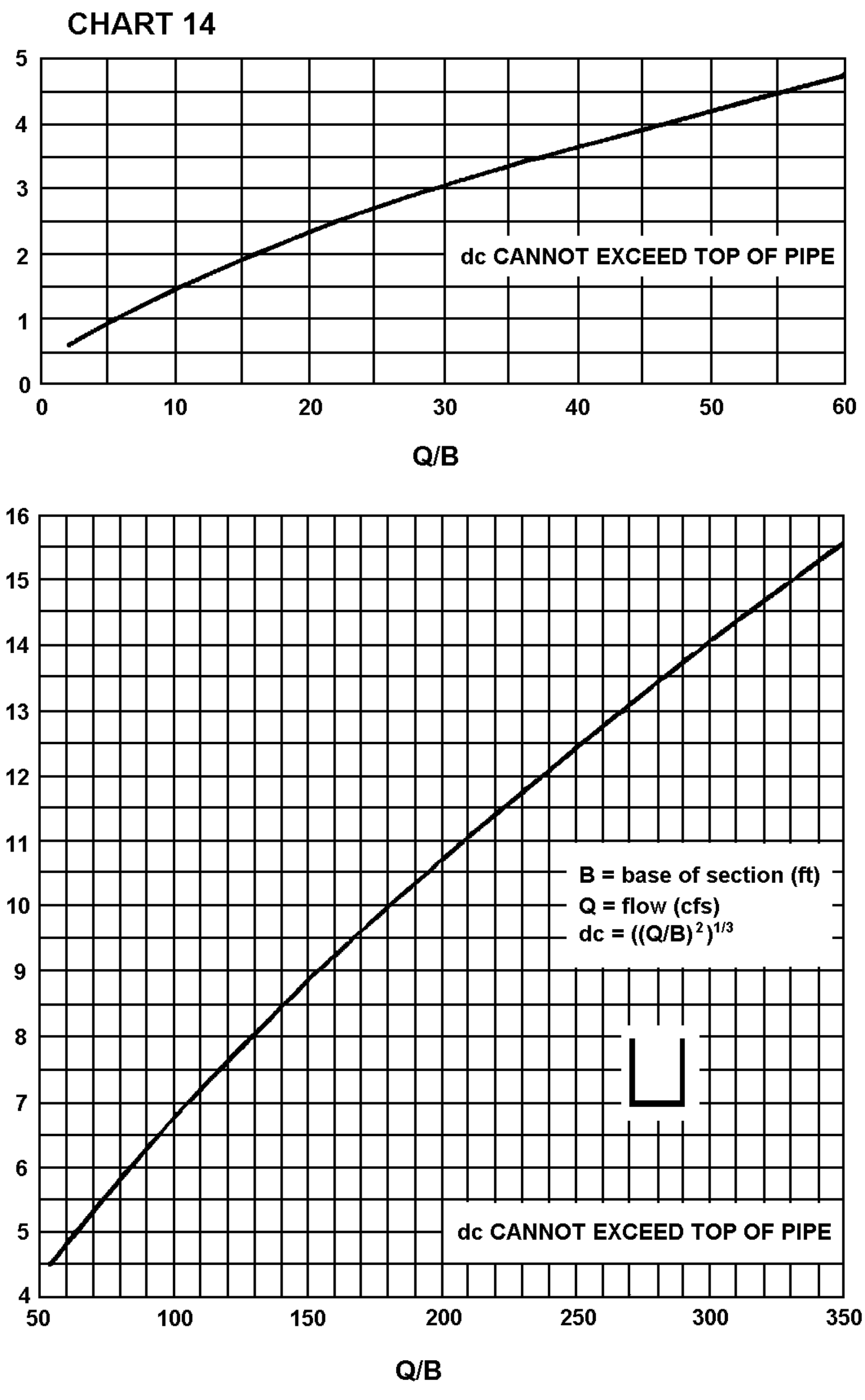
BUREAU OF PUBLIC ROADS
OFFICE OF R&D AUGUST 1968

CHART 13



HEADWATER DEPTH FOR INLET CONTROL RECTANGULAR BOX CULVERTS OFFSET FLARED WINGWALLS AND BEVELED EDGE AT TOP OF INLET

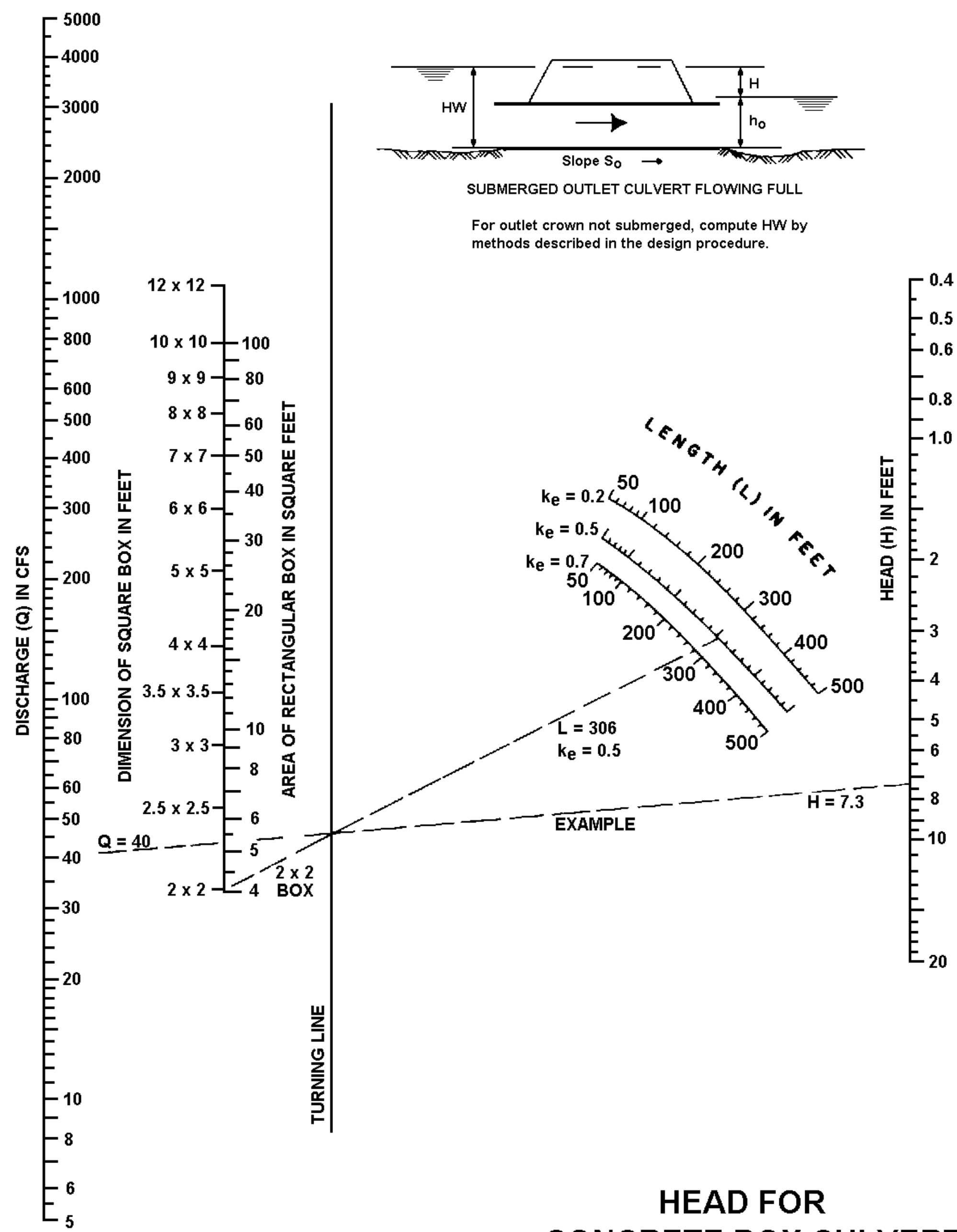
BUREAU OF PUBLIC ROADS
OFFICE OF R&D AUGUST 1968



BUREAU OF PUBLIC ROADS JAN. 1963

CRITICAL DEPTH
RECTANGULAR SECTION

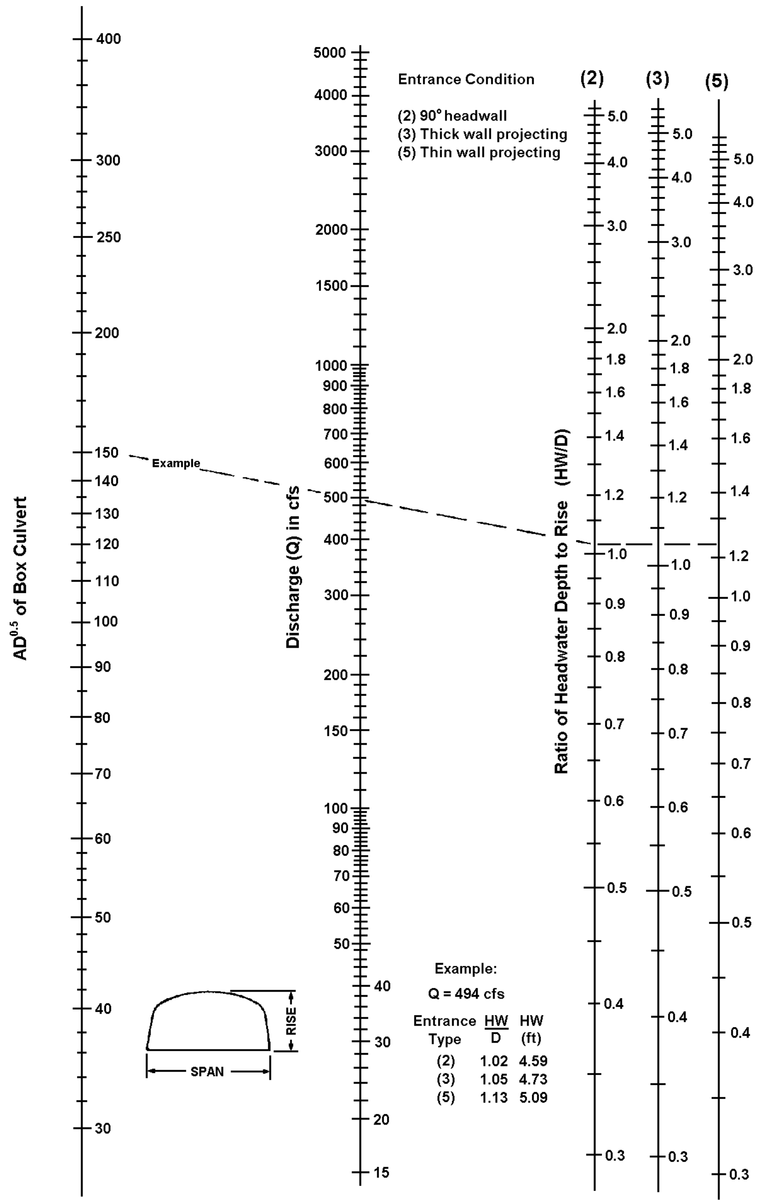
CHART 15



HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL n = 0.012

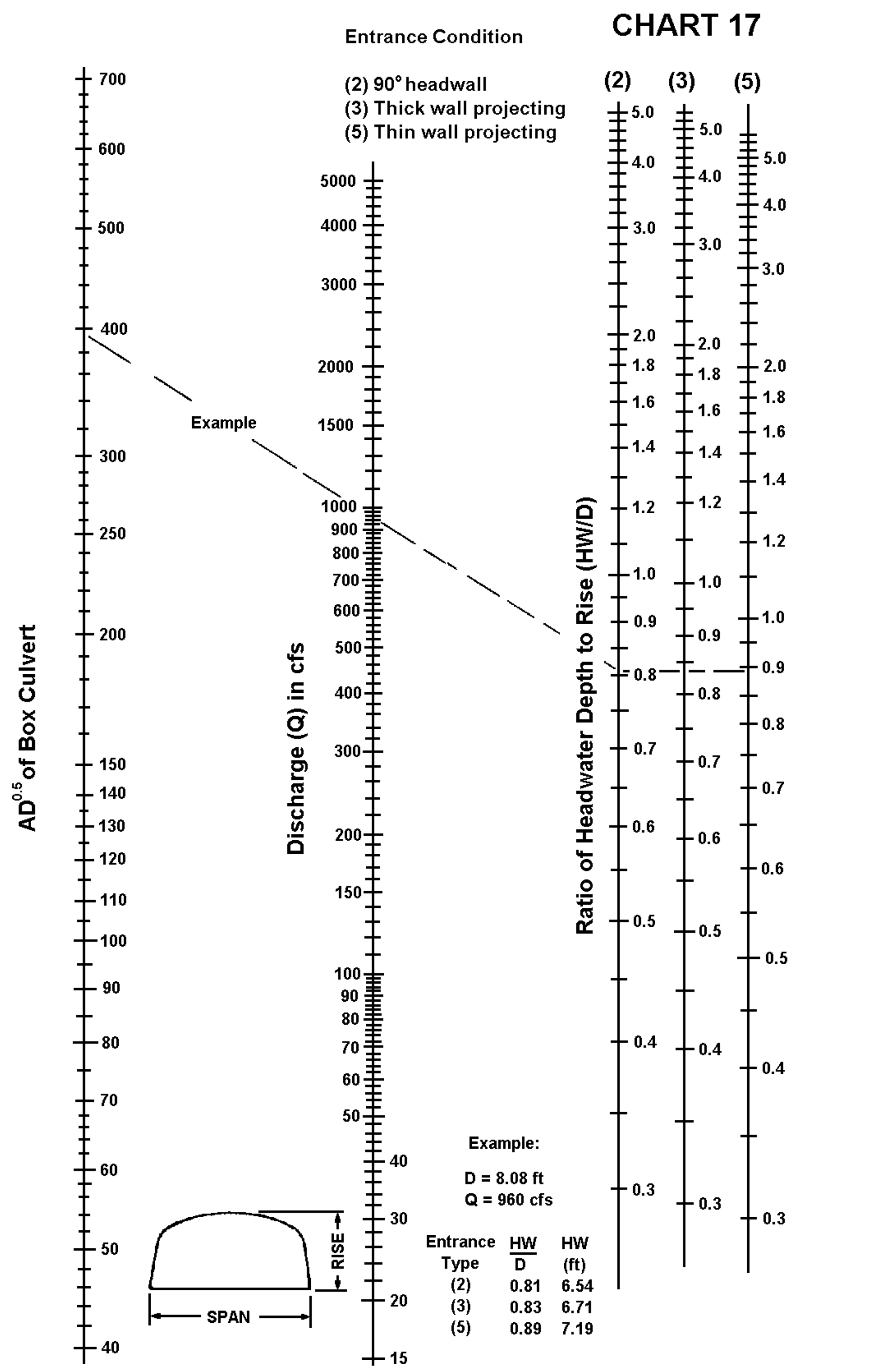
BUREAU OF PUBLIC ROADS JAN. 1963

CHART 16



HEADWATER DEPTH
FOR C.M. BOX CULVERTS
RISE/SPAN < 0.3
WITH INLET CONTROL

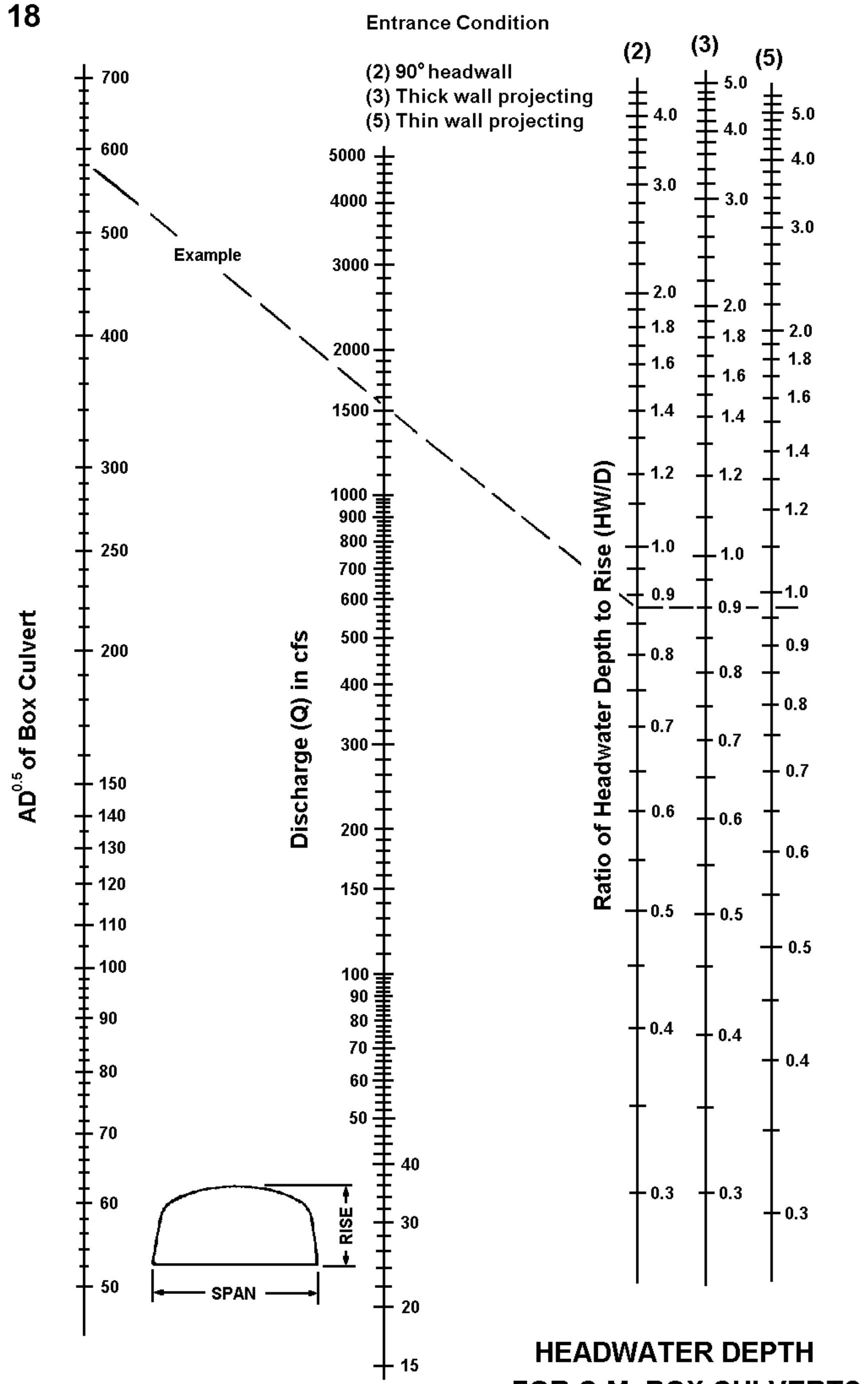
Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.



HEADWATER DEPTH
FOR C.M. BOX 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.

CHART 18



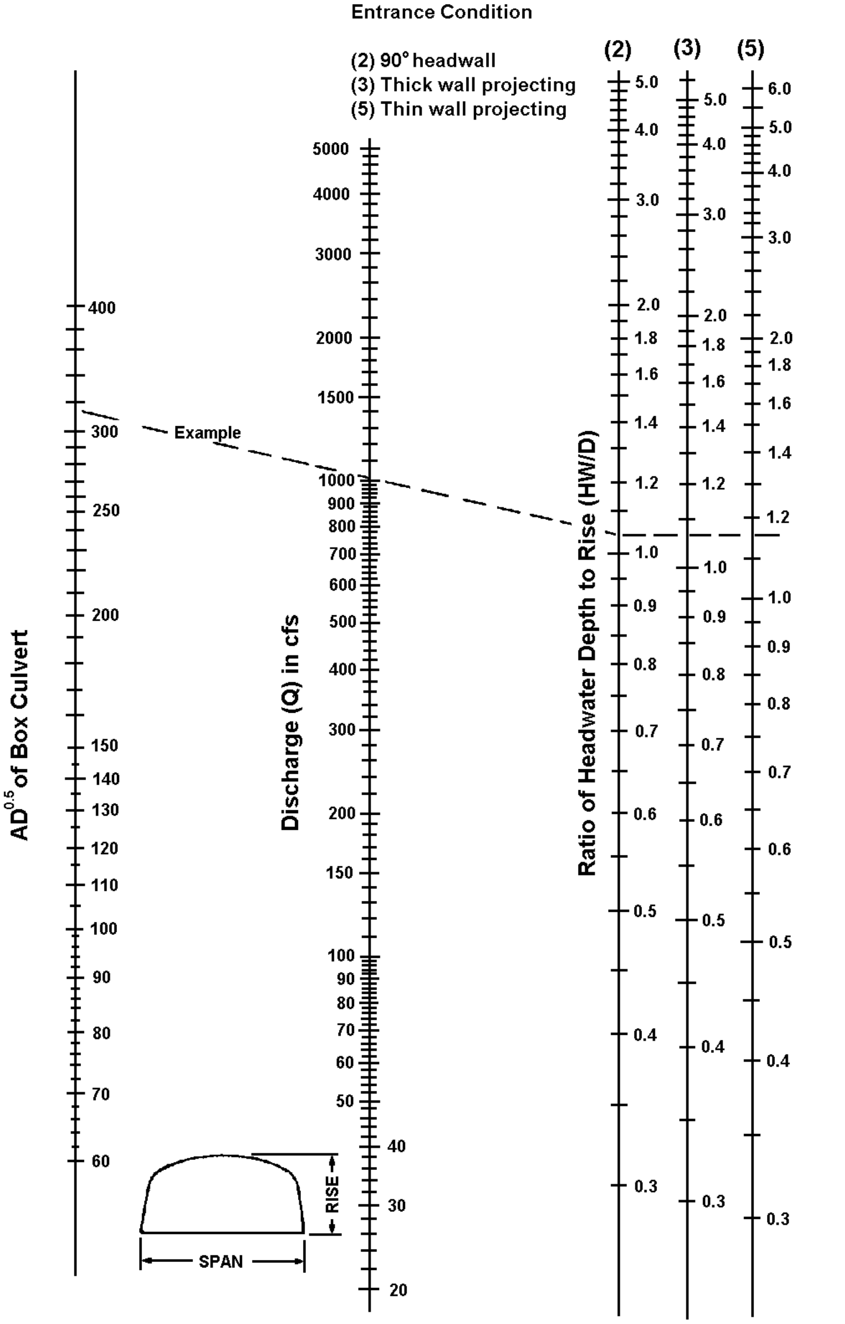
Example:
D = 9.67 ft
Q = 1520 cfs

Entrance Type	HW/D	HW (ft)
(2)	0.88	8.51
(3)	0.90	9.38
(5)	0.97	9.38

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 $0.4 \le$ RISE/SPAN < 0.5 WITH INLET CONTROL

CHART 19



Example:

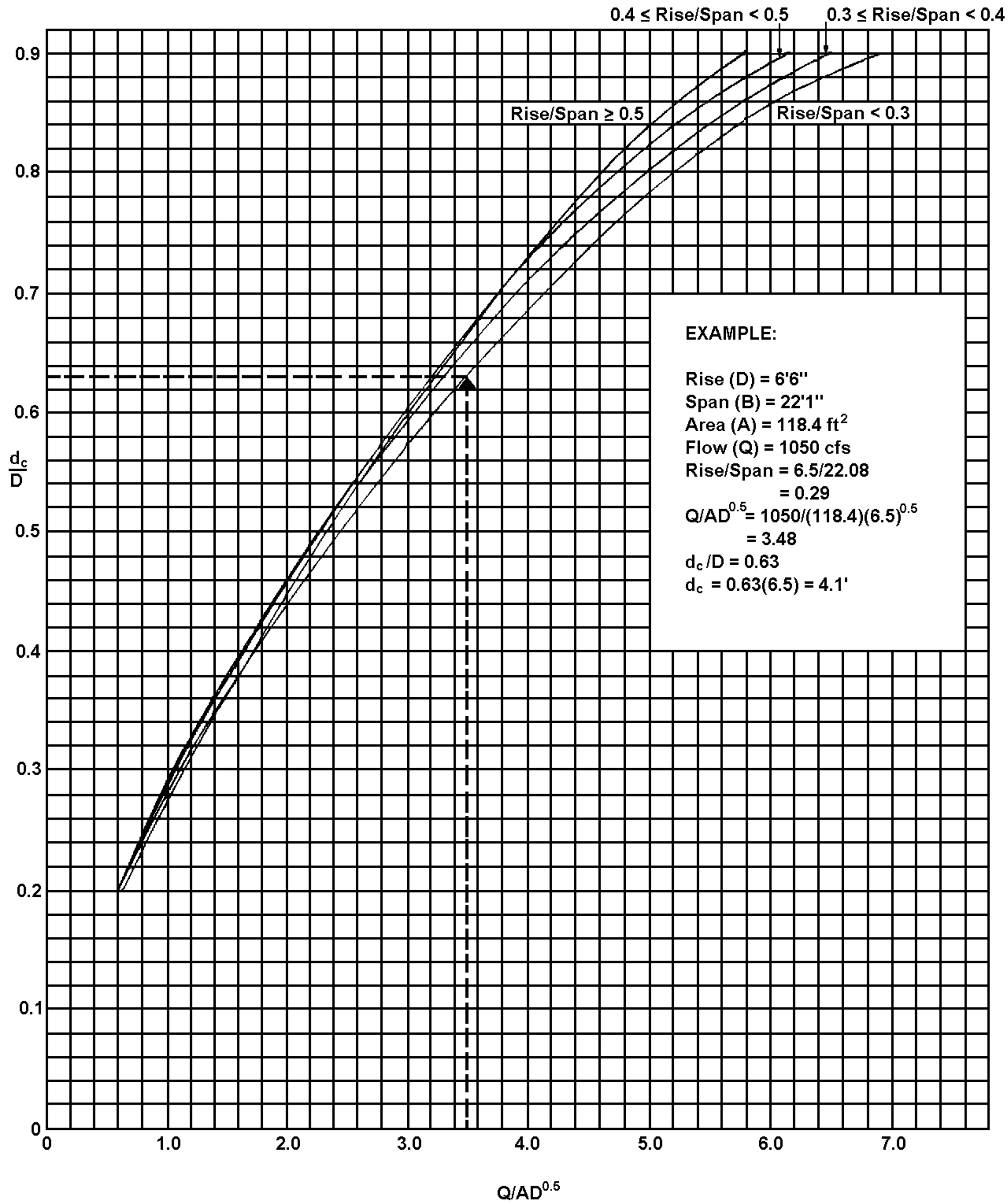
D = 8.0 ft
Q = 1004 cfs

Entrance Type	HW/D	HW (ft)
(2)	1.04	8.32
(3)	1.07	8.58
(5)	1.15	9.20

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.

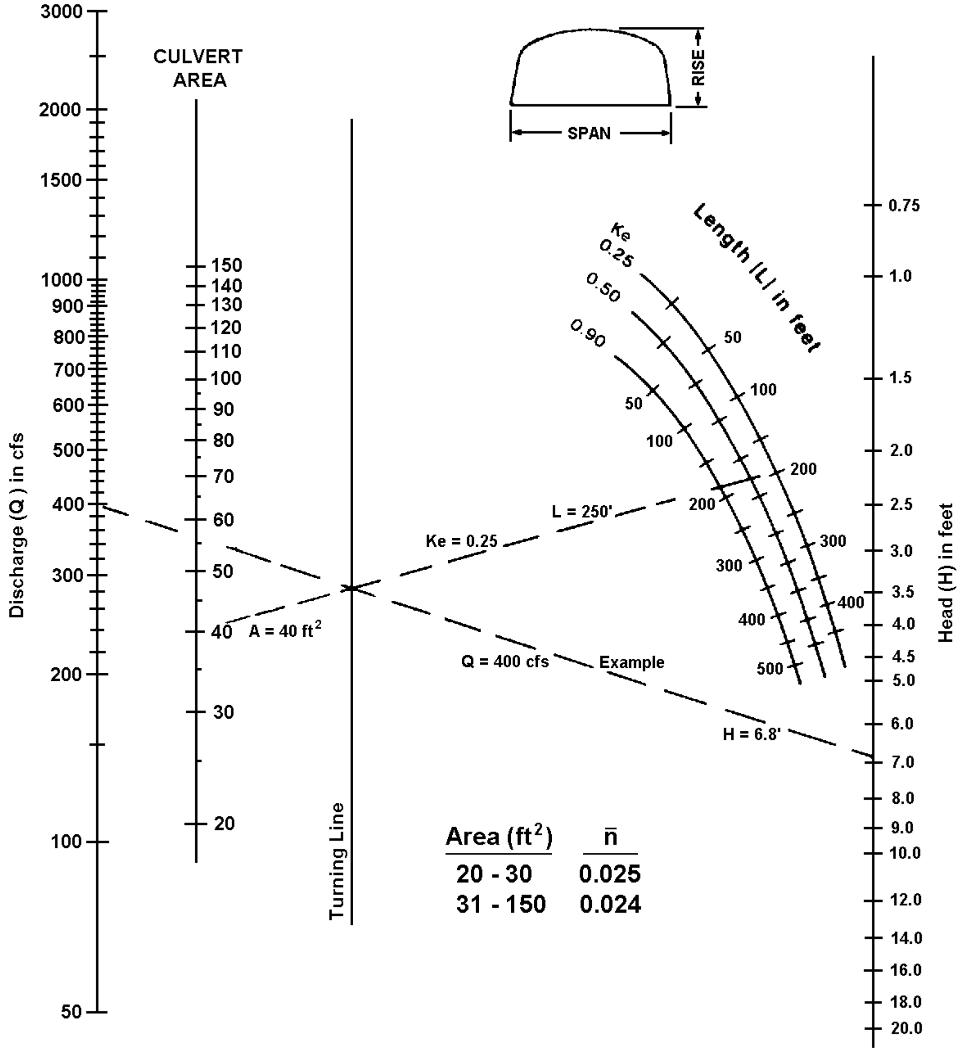
HEADWATER DEPTH FOR C.M. BOX CULVERTS $0.5 \leq$ RISE/SPAN WITH INLET CONTROL

CHART 20

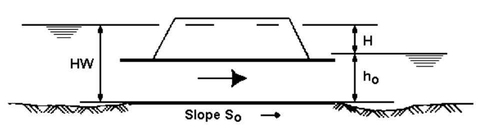


DIMENSIONLESS CRITICAL DEPTH
FOR C.M. BOX CULVERTS

CHART 21



Area (ft^2)	$\bar{n}$
20 - 30	0.025
31 - 150	0.024



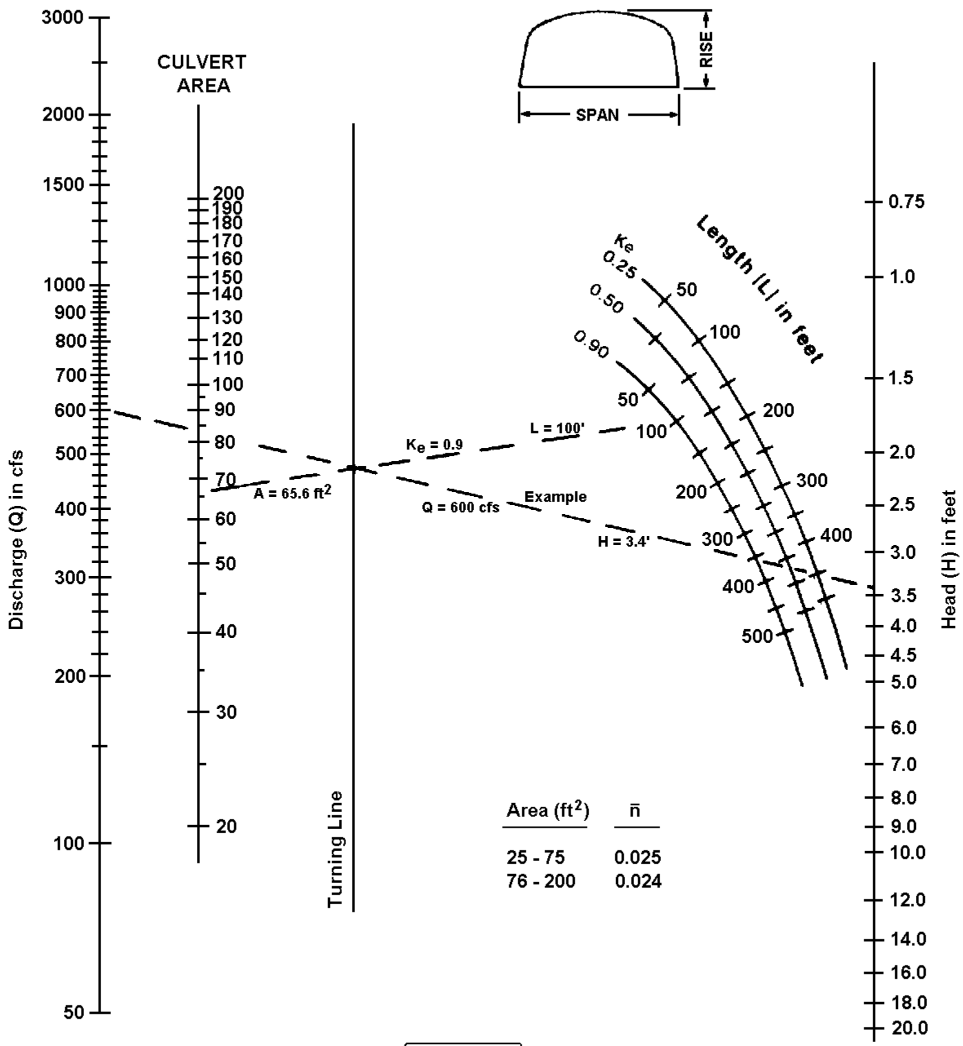
SUBMERGED OUTLET CULVERT FLOWING FULL

For outlet crown not submerged, compute HW by methods described in the design procedure.

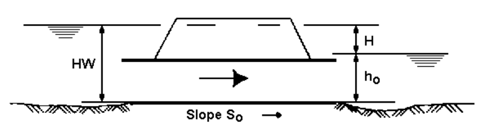
HEAD FOR C.M. BOX CULVERTS FLOWING FULL CONCRETE BOTTOM RISE/SPAN < 0.3

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

CHART 22



Area (ft^2)	$\bar{n}$
25 - 75	0.025
76 - 200	0.024



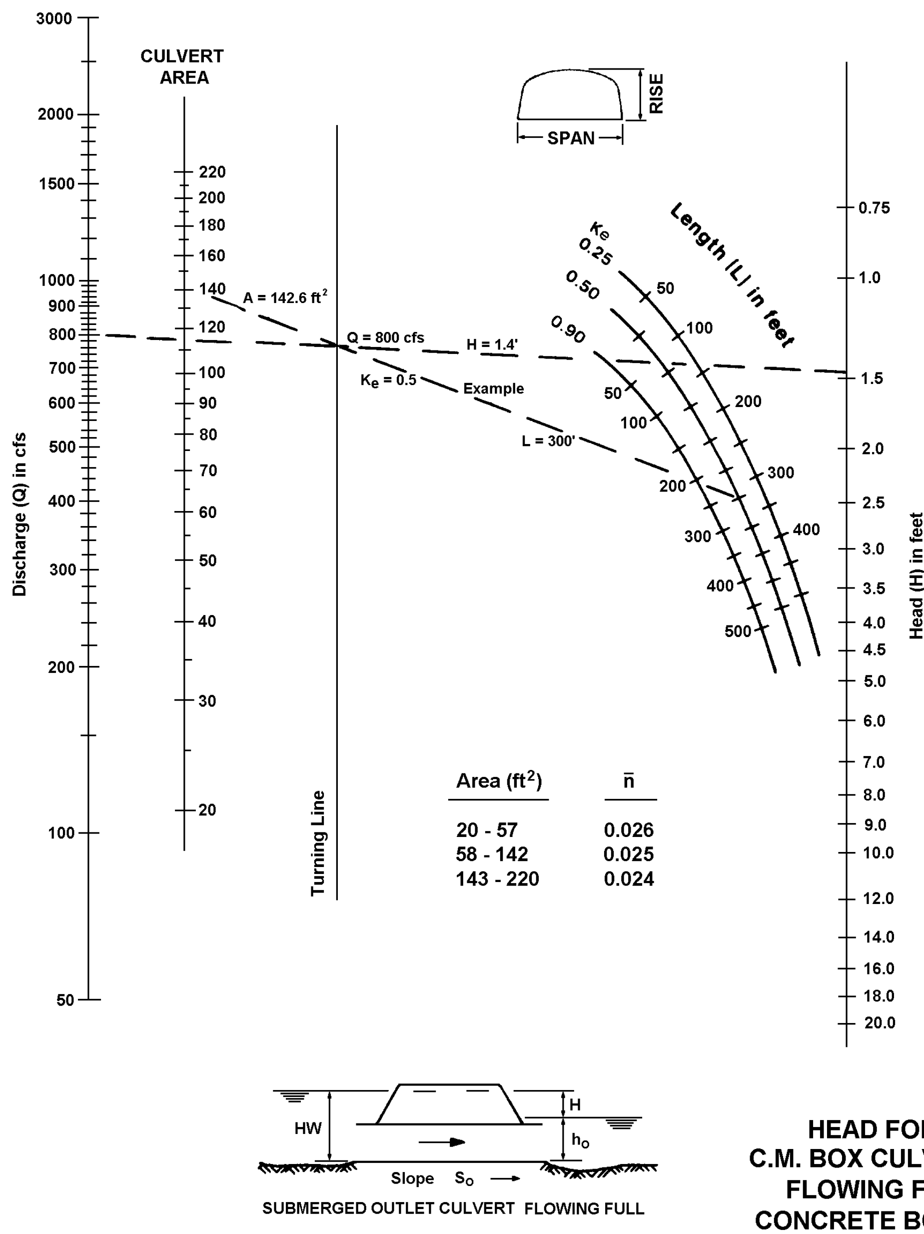
SUBMERGED OUTLET CULVERT FLOWING FULL

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 ≤ RISE/SPAN < 0.4

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

CHART 23

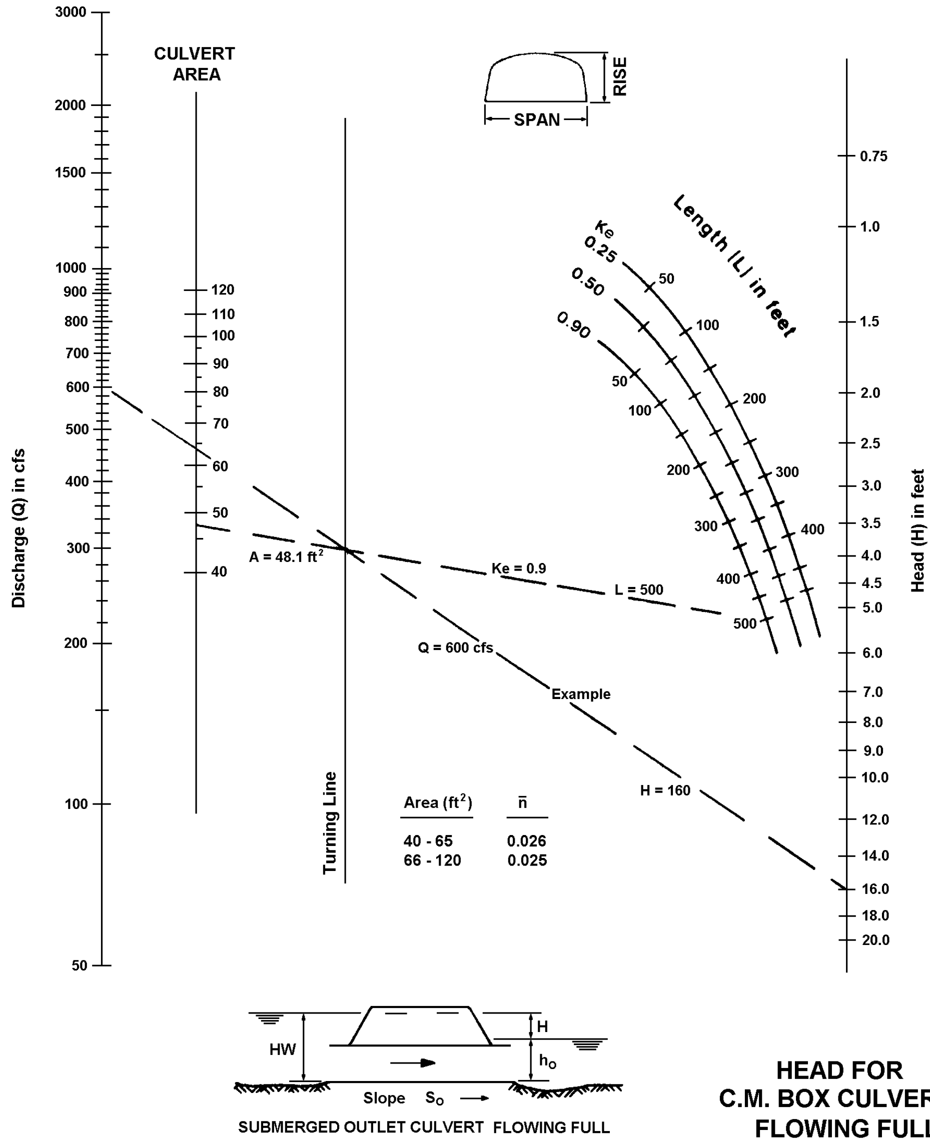


Area (ft²)	n̄
20 - 57	0.026
58 - 142	0.025
143 - 220	0.024

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.

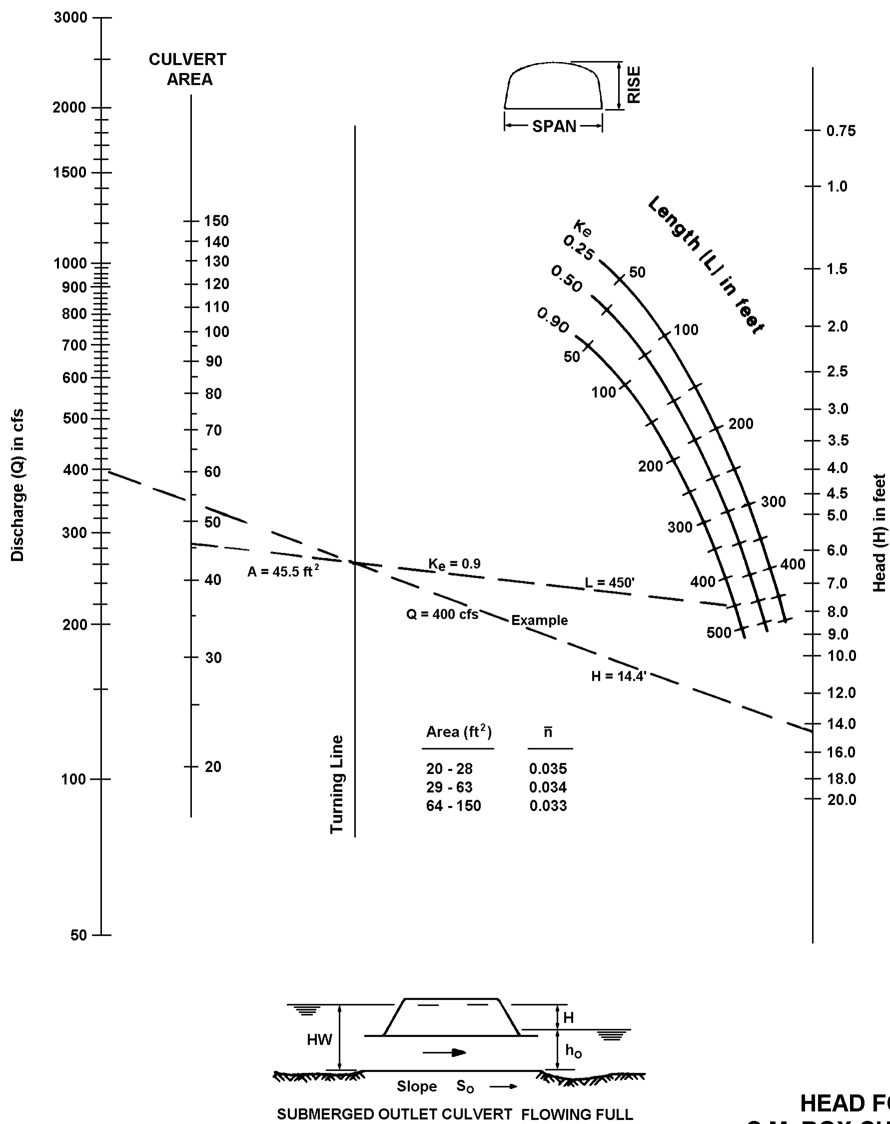
CHART 24



HEAD FOR
C.M. BOX 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.

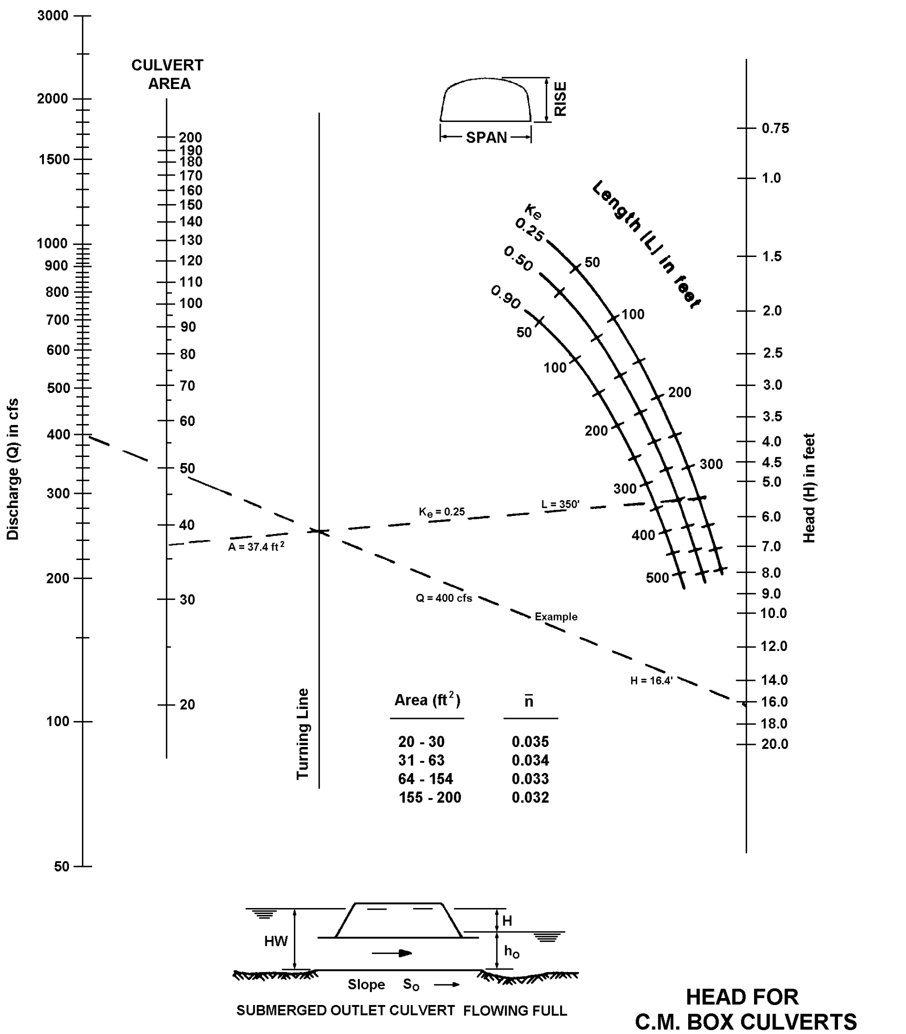
CHART 25



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

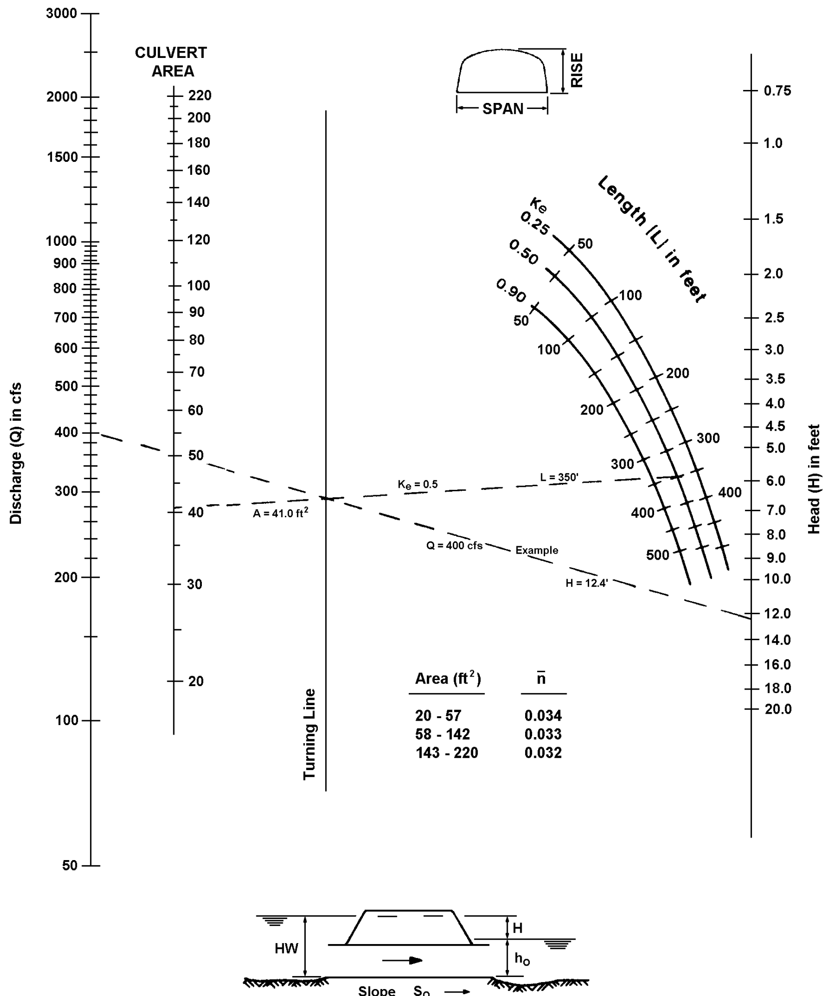
CHART 26



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

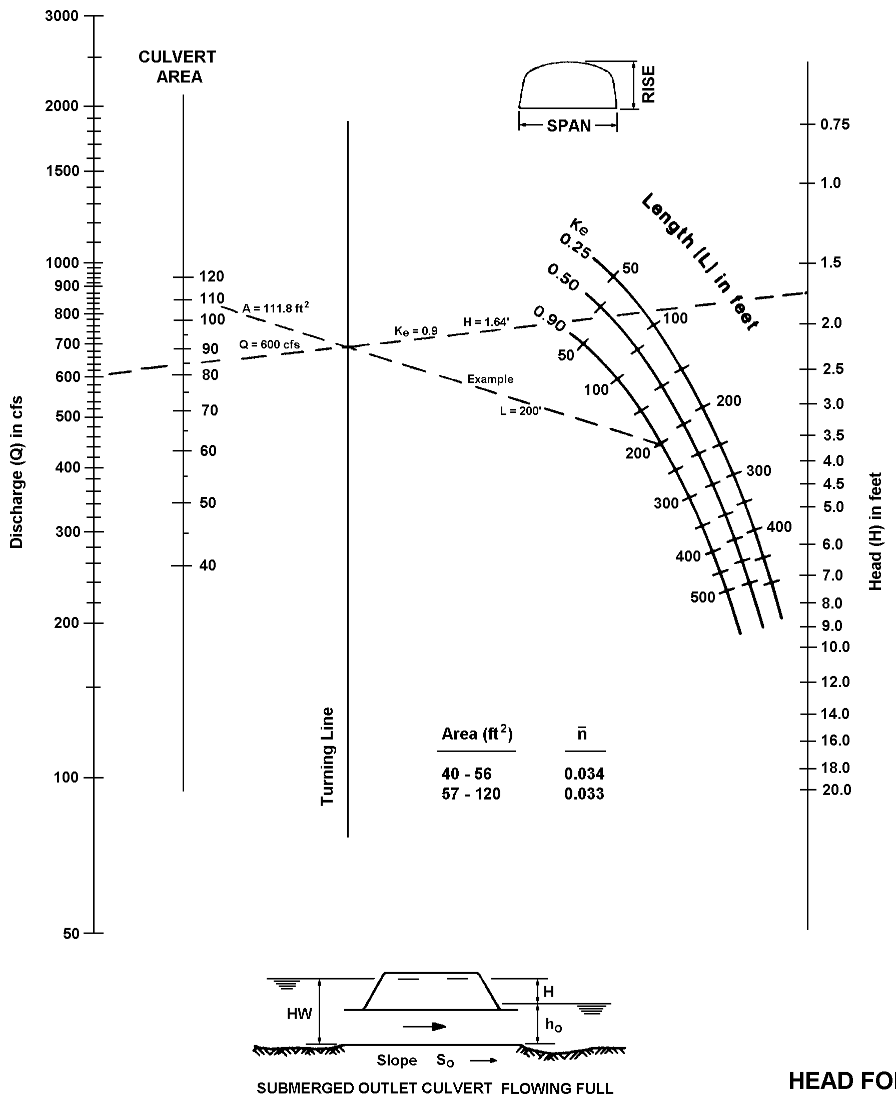
CHART 27



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 ≤ RISE/SPAN < 0.5**

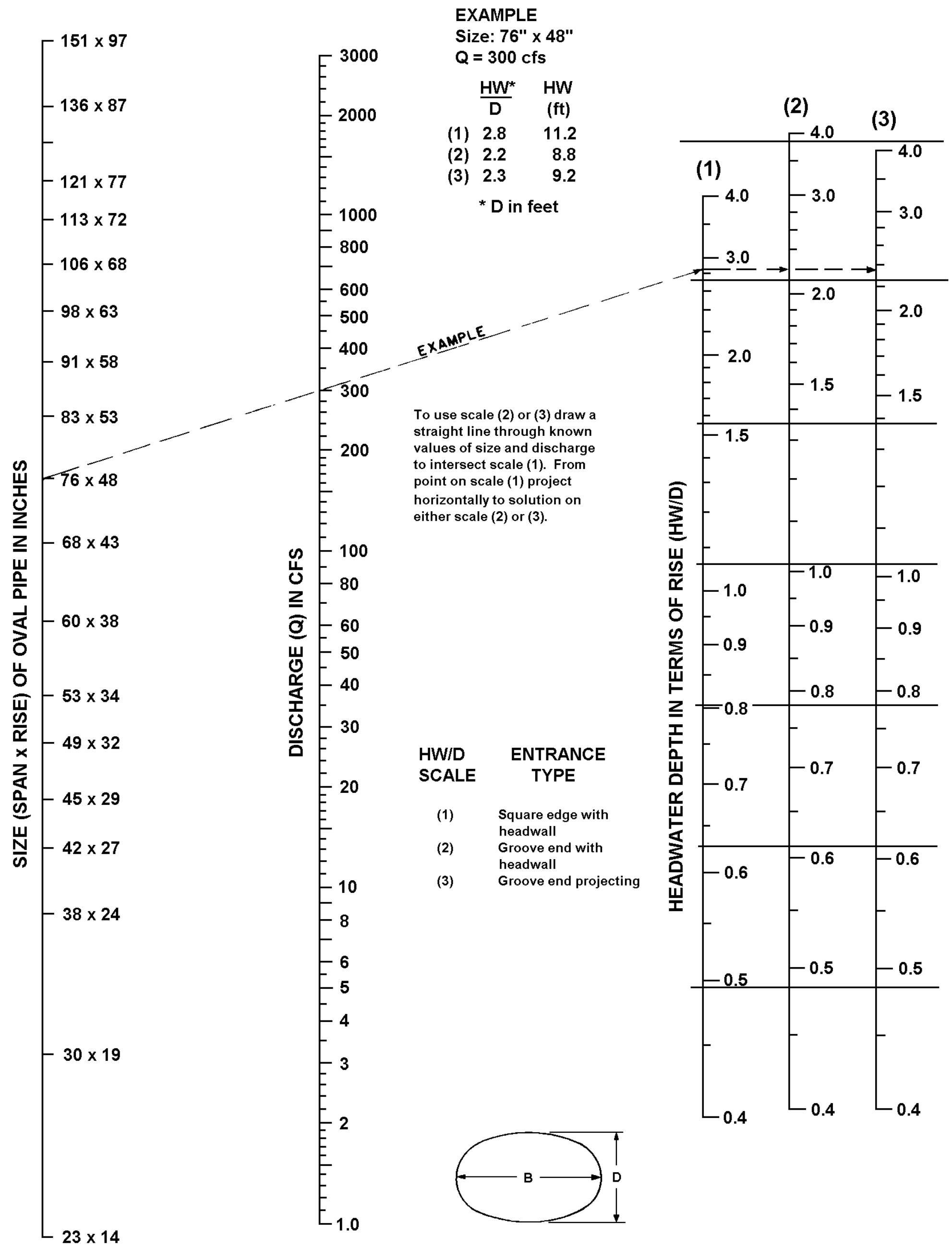
CHART 28



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
0.5 ≤ RISE/SPAN

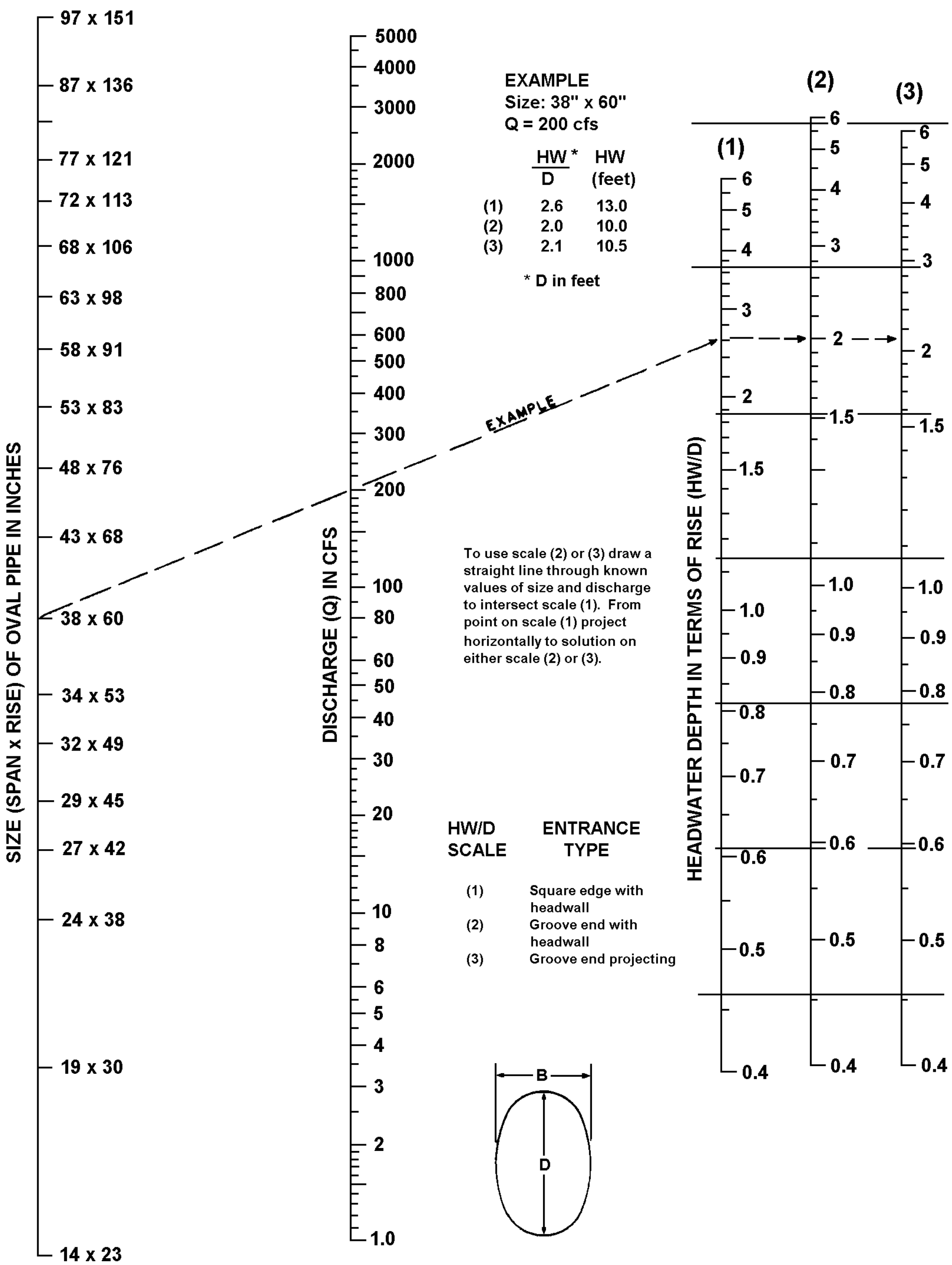
CHART 29



HEADWATER DEPTH FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

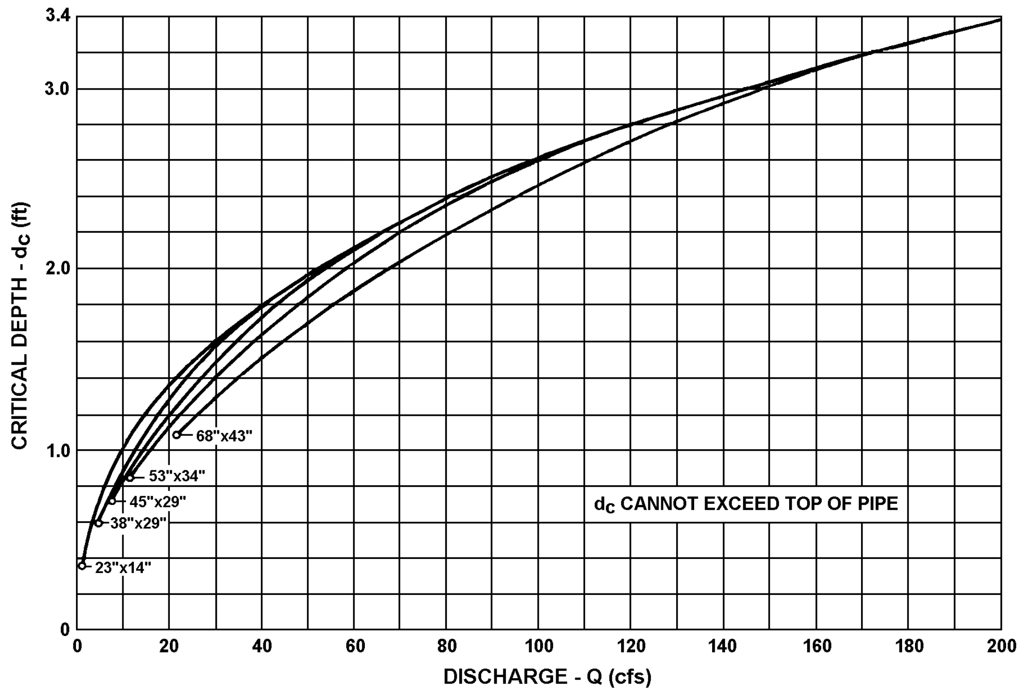
CHART 30

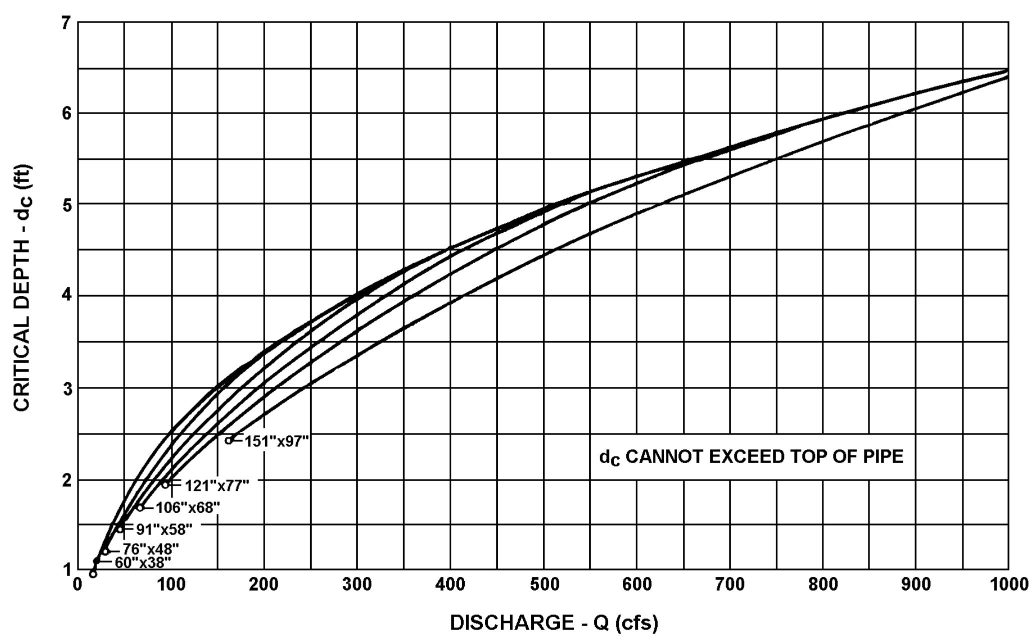


HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS VERTICAL
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 31

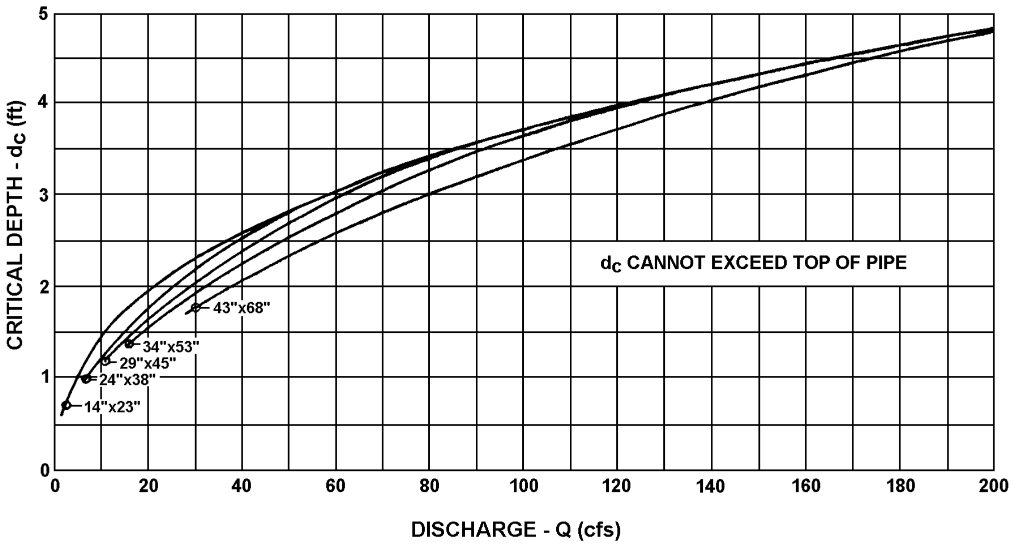


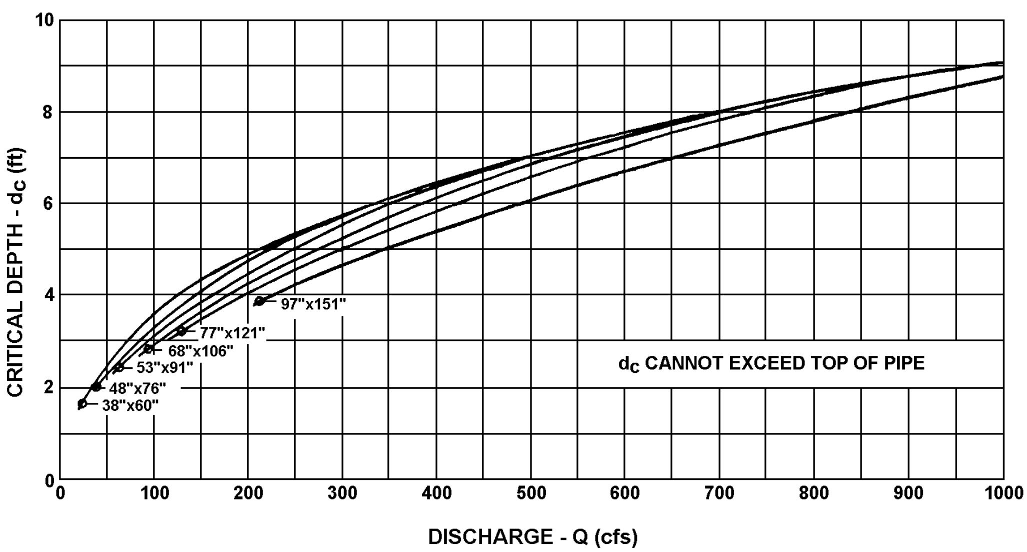


BUREAU OF PUBLIC ROADS JAN. 1964

CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL

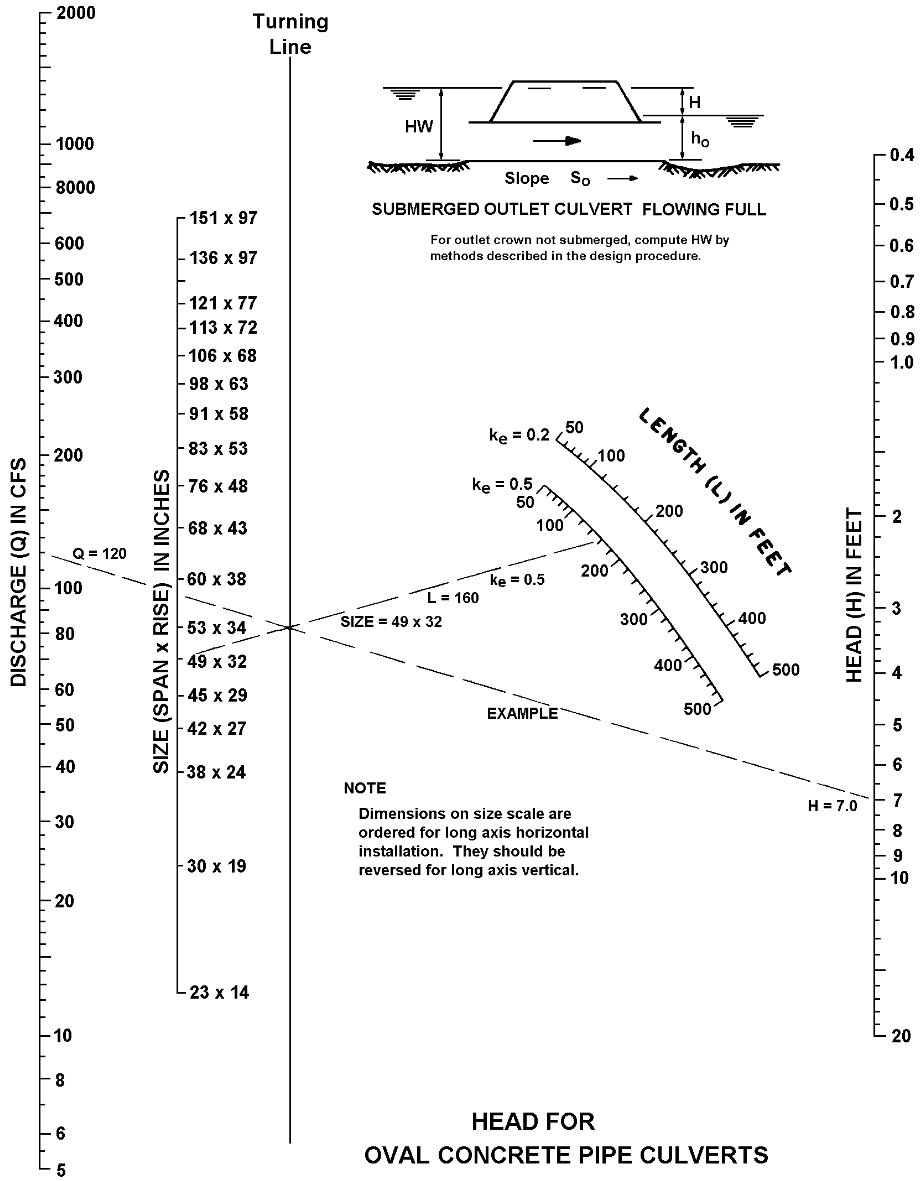
CHART 32





BUREAU OF PUBLIC ROADS JAN. 1964

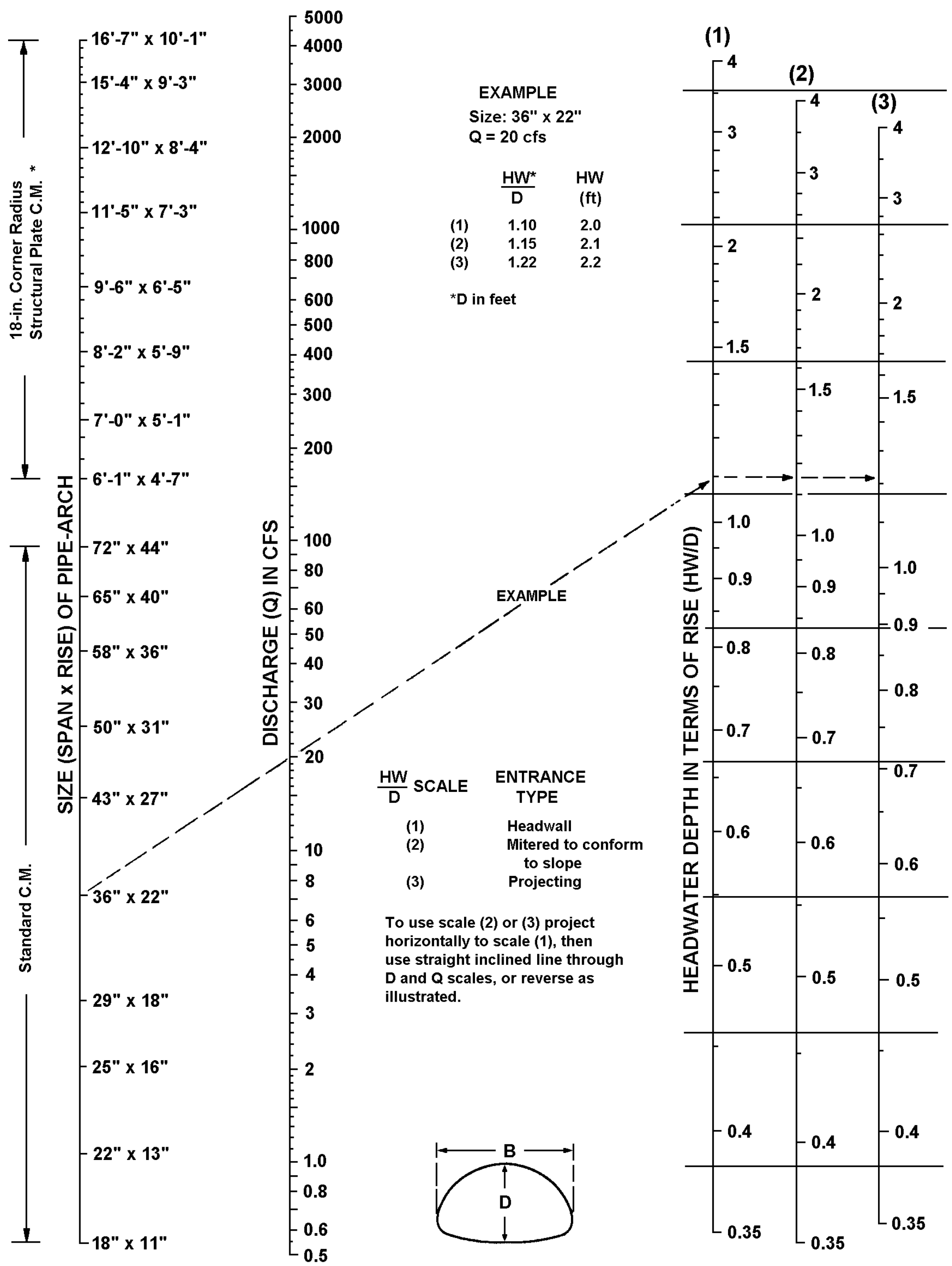
CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS VERTICAL



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.

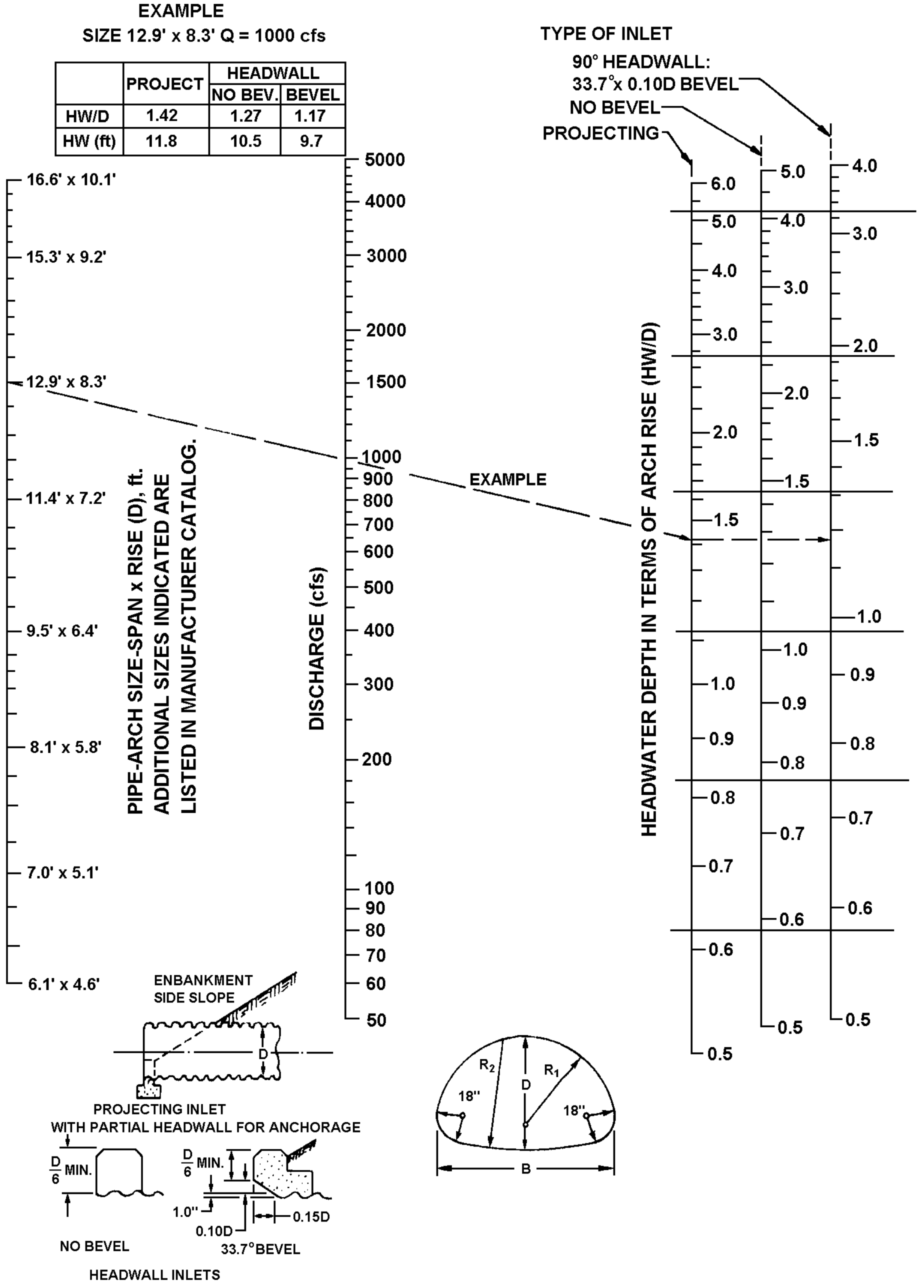
HEADWATER DEPTH FOR
C.M. PIPE-ARCH CULVERTS
WITH INLET CONTROL

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

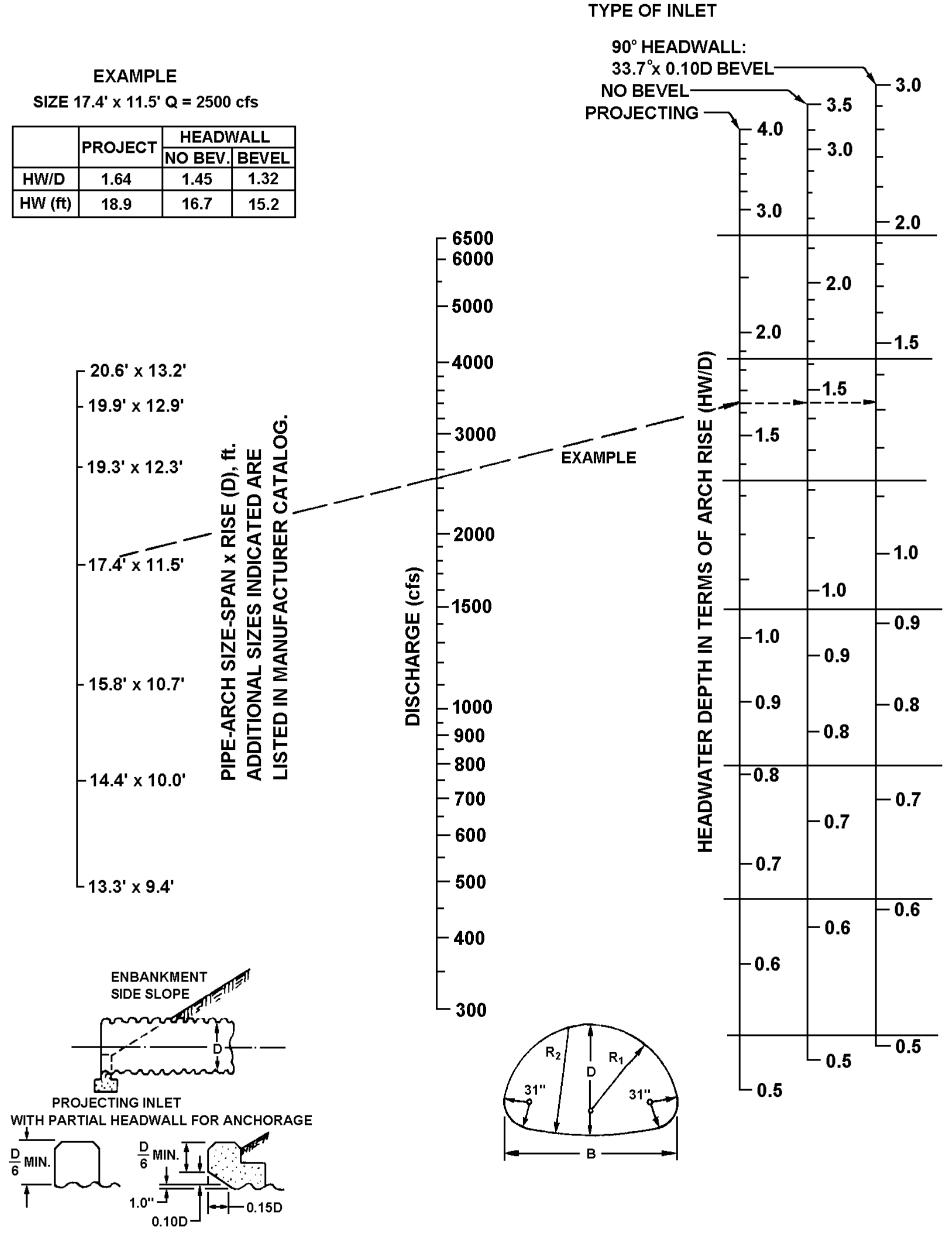
HEADWATER DEPTH FOR INLET CONTROLS
STRUCTURAL PLATE PIPE-ARCH CULVERTS
18 in. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

CHART 36

EXAMPLE

SIZE 17.4' x 11.5' Q = 2500 cfs

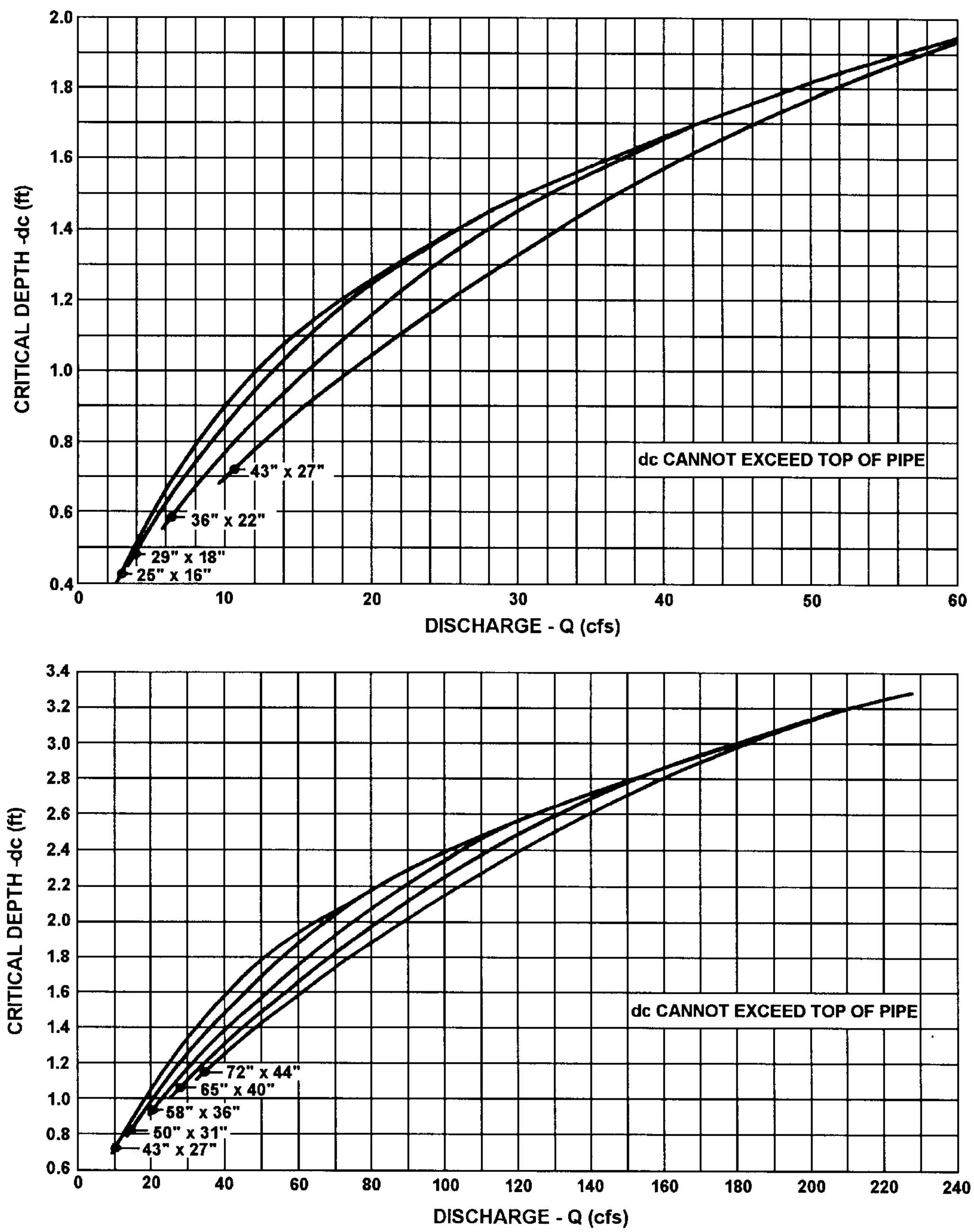
	PROJECT	HEADWALL	
		NO BEV.	BEVEL
HW/D	1.64	1.45	1.32
HW (ft)	18.9	16.7	15.2



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

BUREAU OF PUBLIC ROADS
OFFICE OF R&D JULY 1966

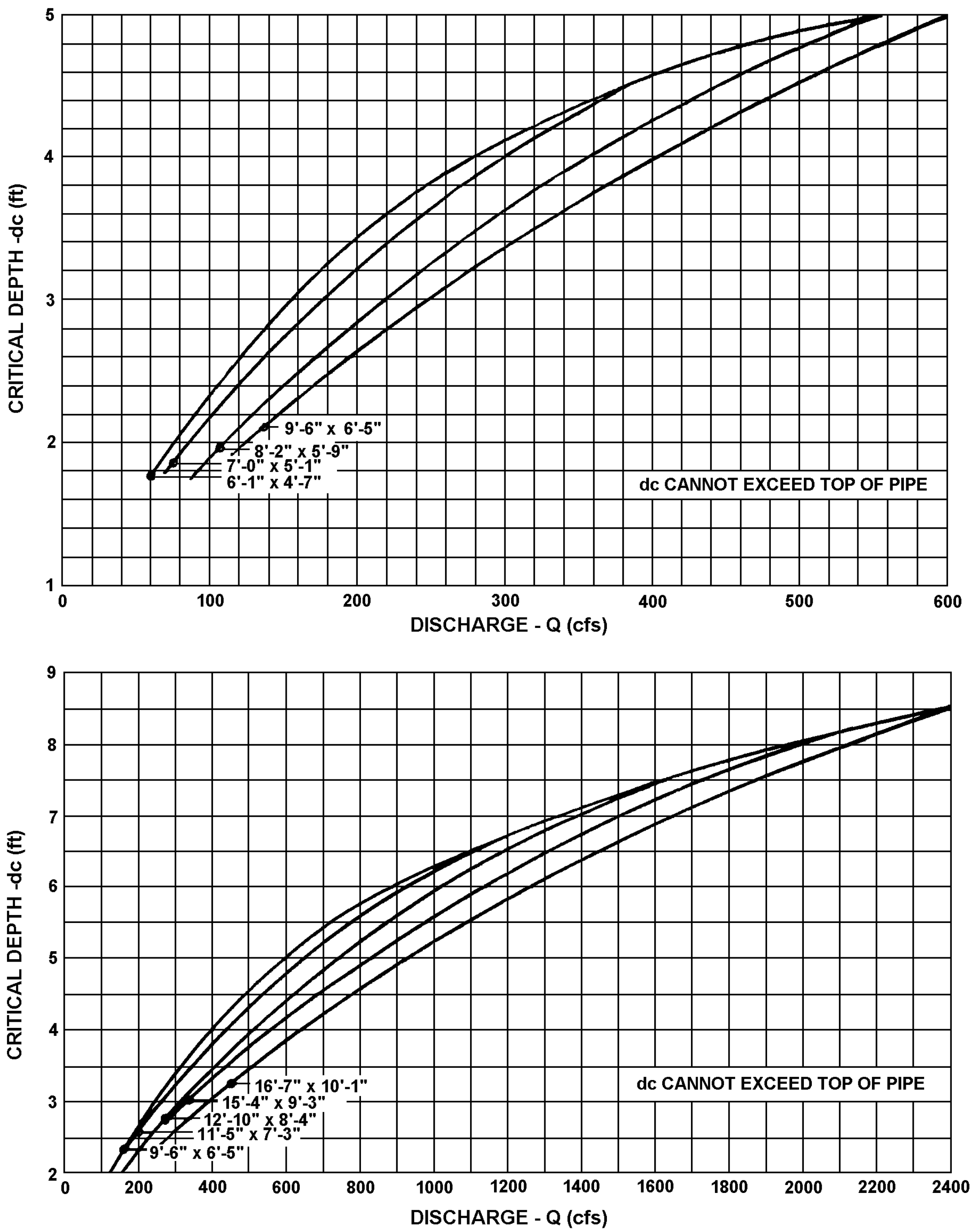
CHART 37



BUREAU OF PUBLIC ROADS
JAN. 1964

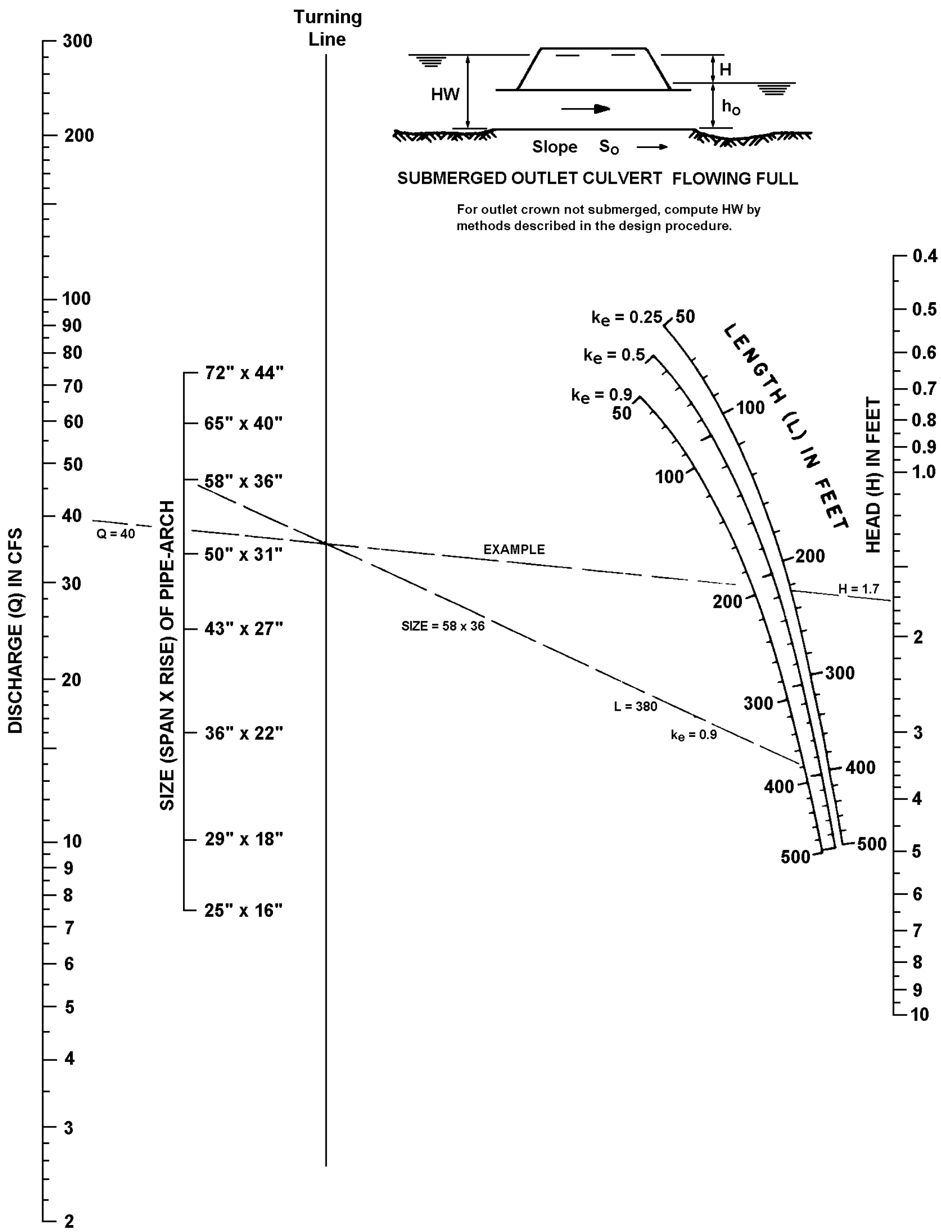
CRITICAL DEPTH
STANDARD C.M. PIPE ARCH

CHART 38



BUREAU OF PUBLIC ROADS
JAN. 1964

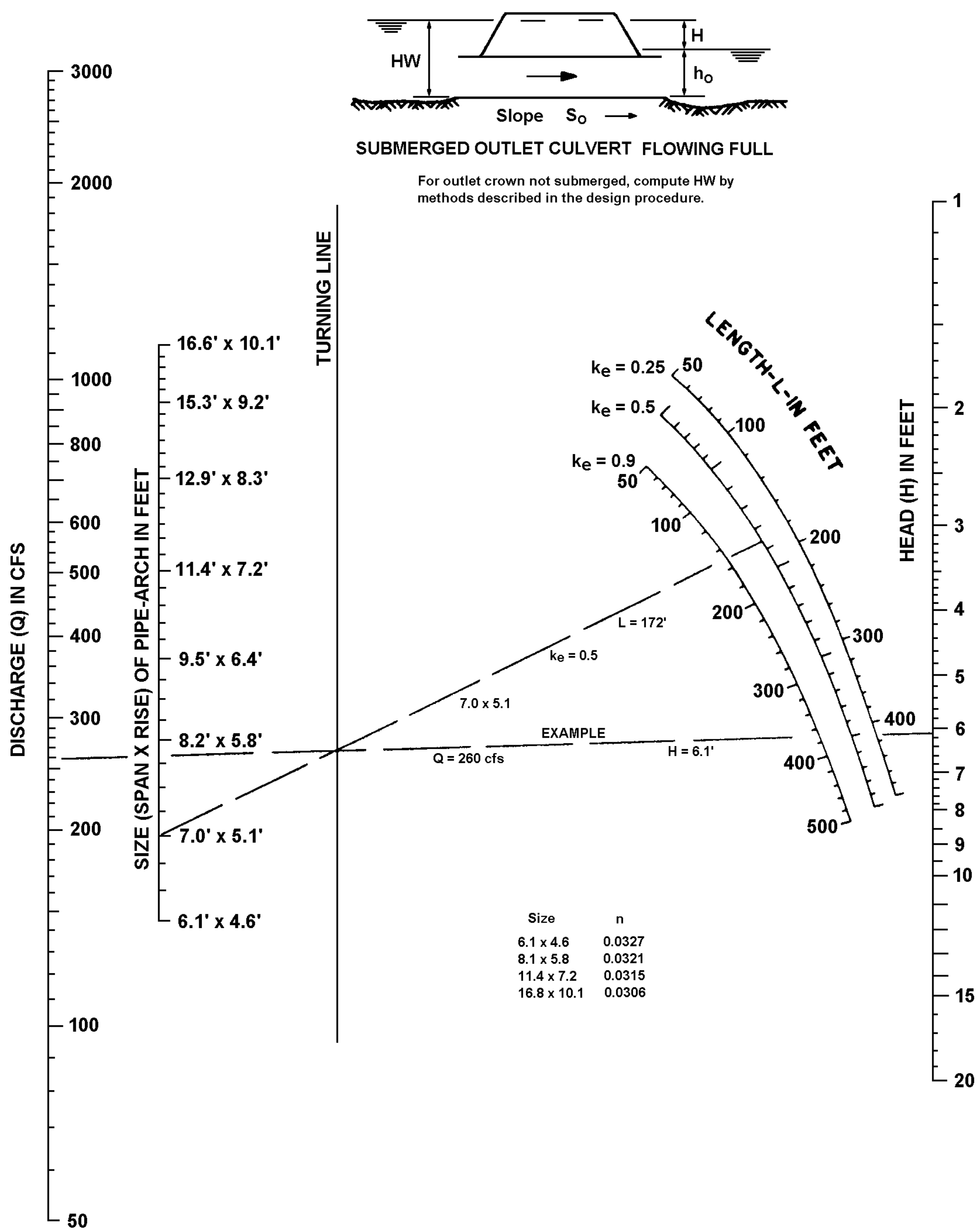
CRITICAL DEPTH
STRUCTURAL PLATE
C.M. PIPE ARCH
18 in. CORNER RADIUS



BUREAU OF PUBLIC ROADS JAN. 1963

HEAD FOR
STANDARD C.M. PIPE-ARCH CULVERTS
FLOWING FULL
n = 0.024

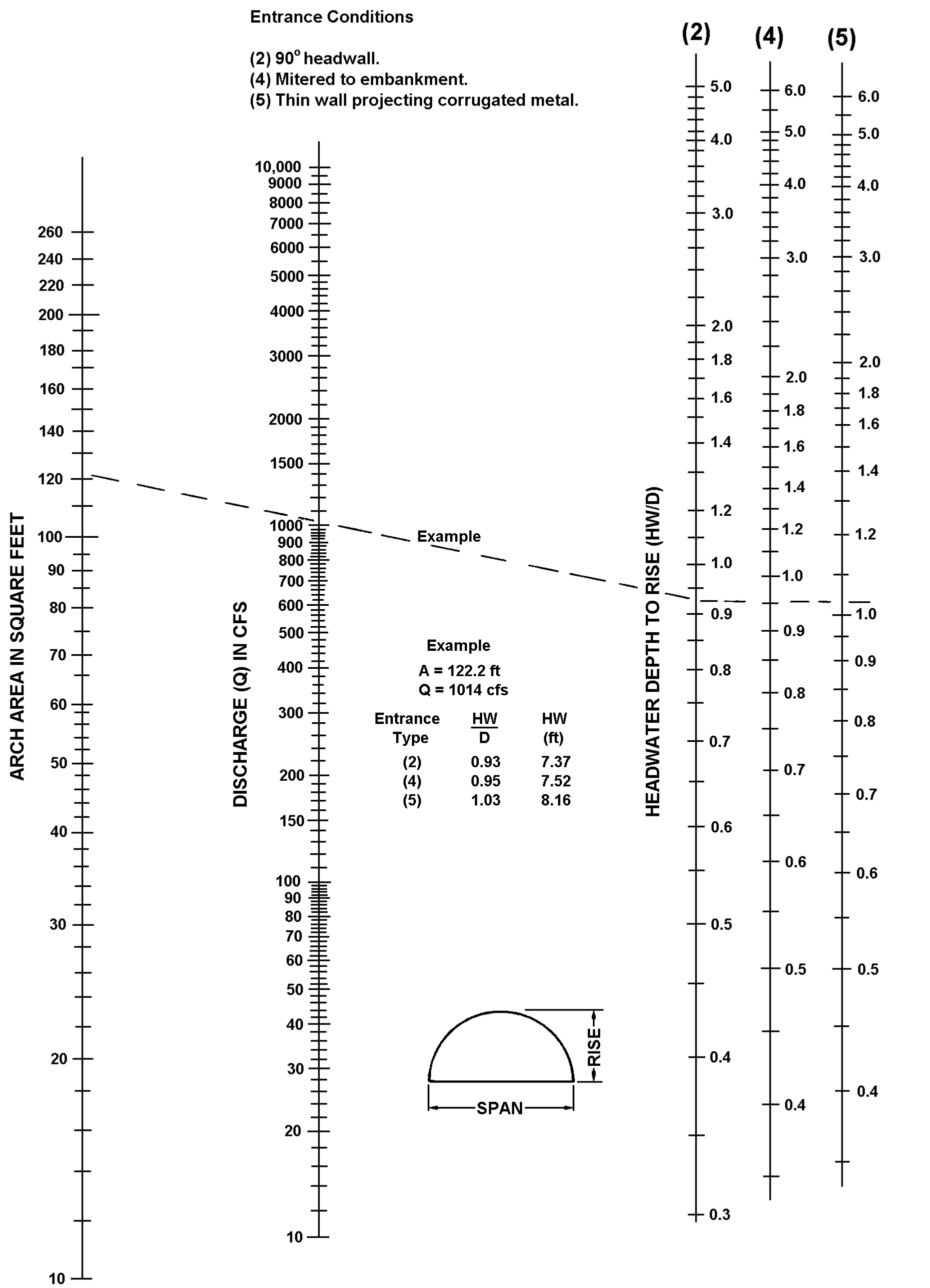
CHART 40



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

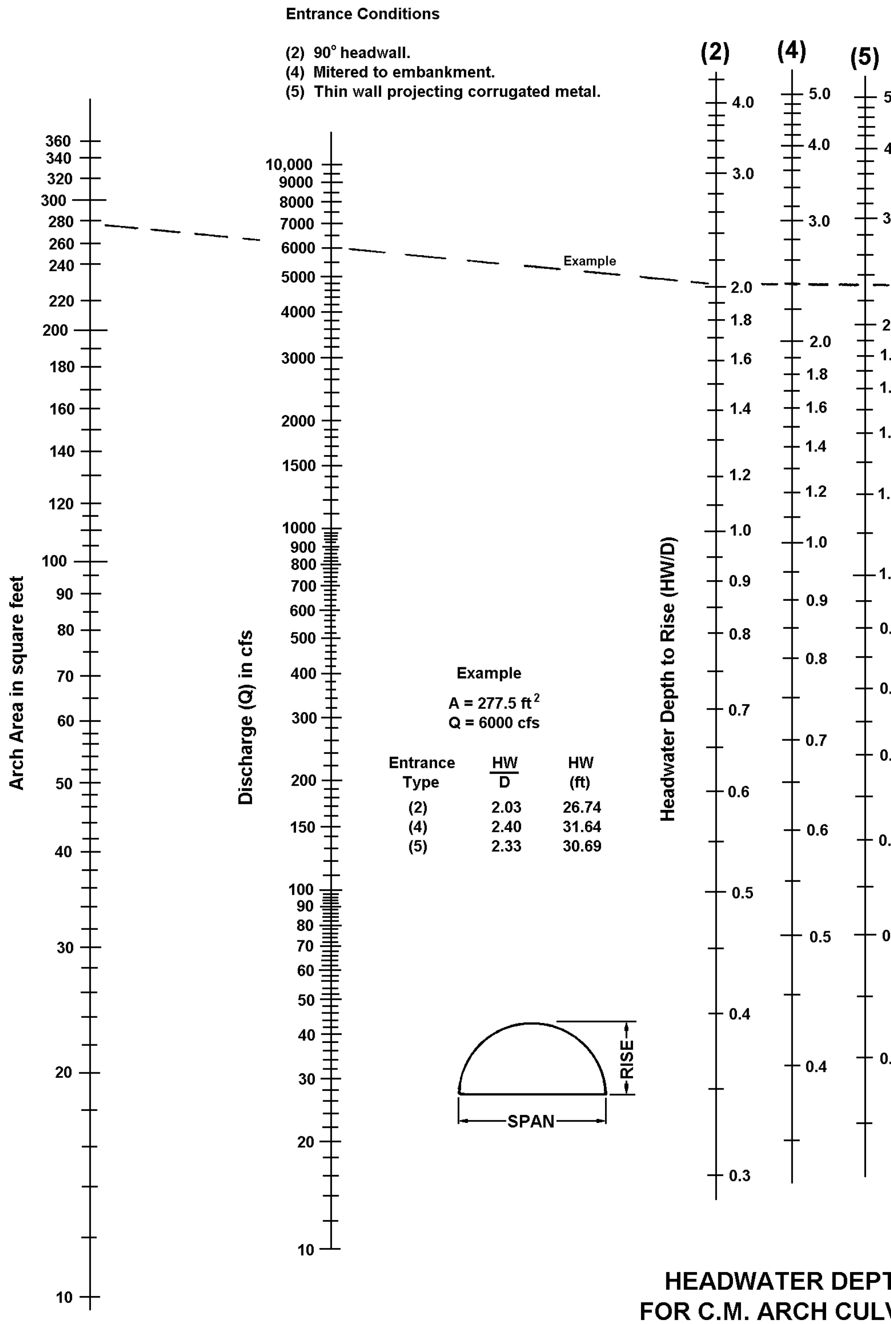
CHART 41



HEADWATER DEPTH FOR C.M. ARCH CULVERTS $0.3 \leq$ 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.

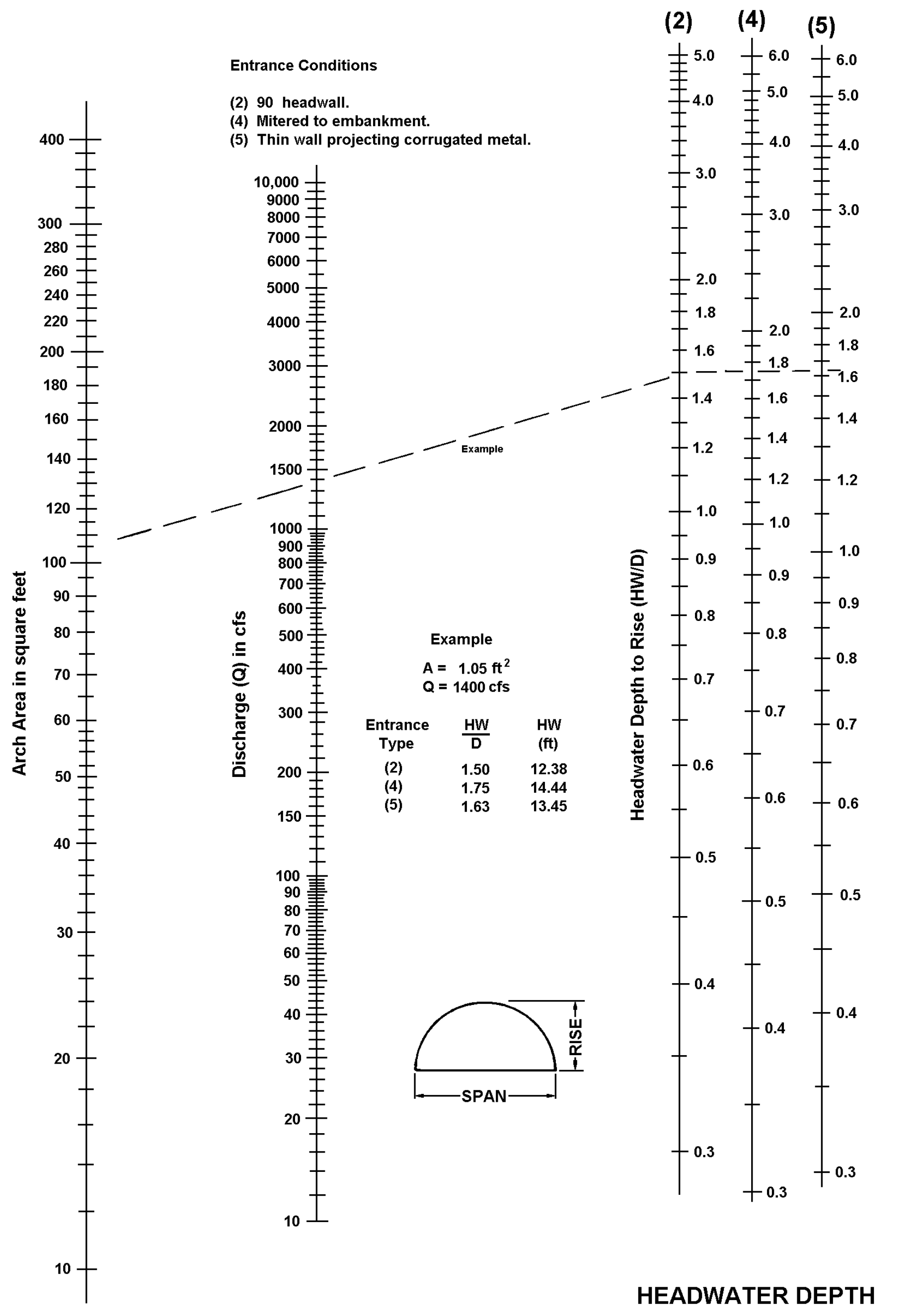
CHART 42



HEADWATER DEPTH FOR C.M. ARCH 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.

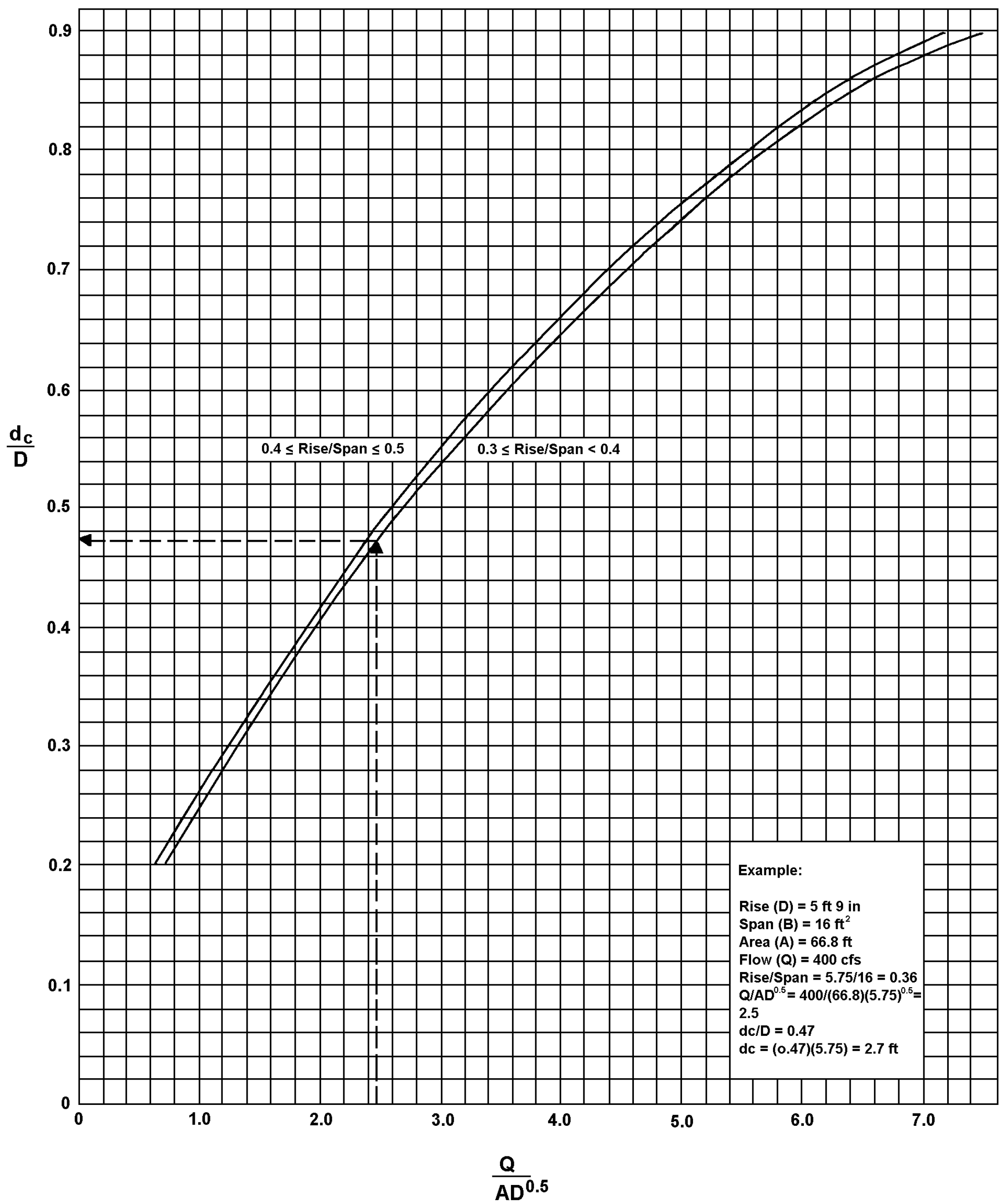
CHART 43



HEADWATER DEPTH
FOR C.M. ARCH CULVERTS
0.5 ≤ RISE/SPAN
WITH INLET CONTROL

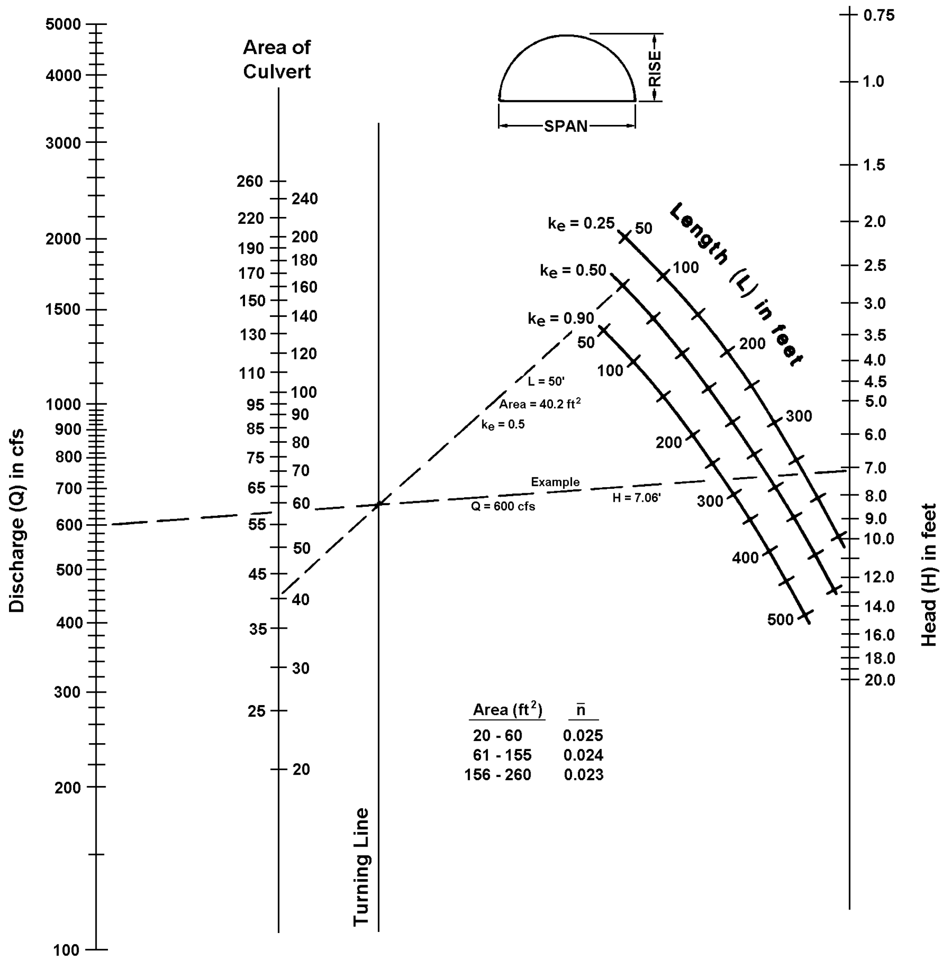
Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.
Duplication of this nomograph may distort scale.

CHART 44

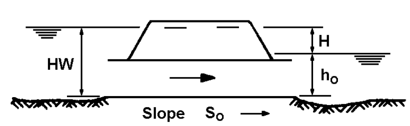


DIMENSIONLESS CRITICAL DEPTH CHART
FOR C.M. ARCH CULVERTS

CHART 45



Area (ft^2)	$\bar{n}$
20 - 60	0.025
61 - 155	0.024
156 - 260	0.023

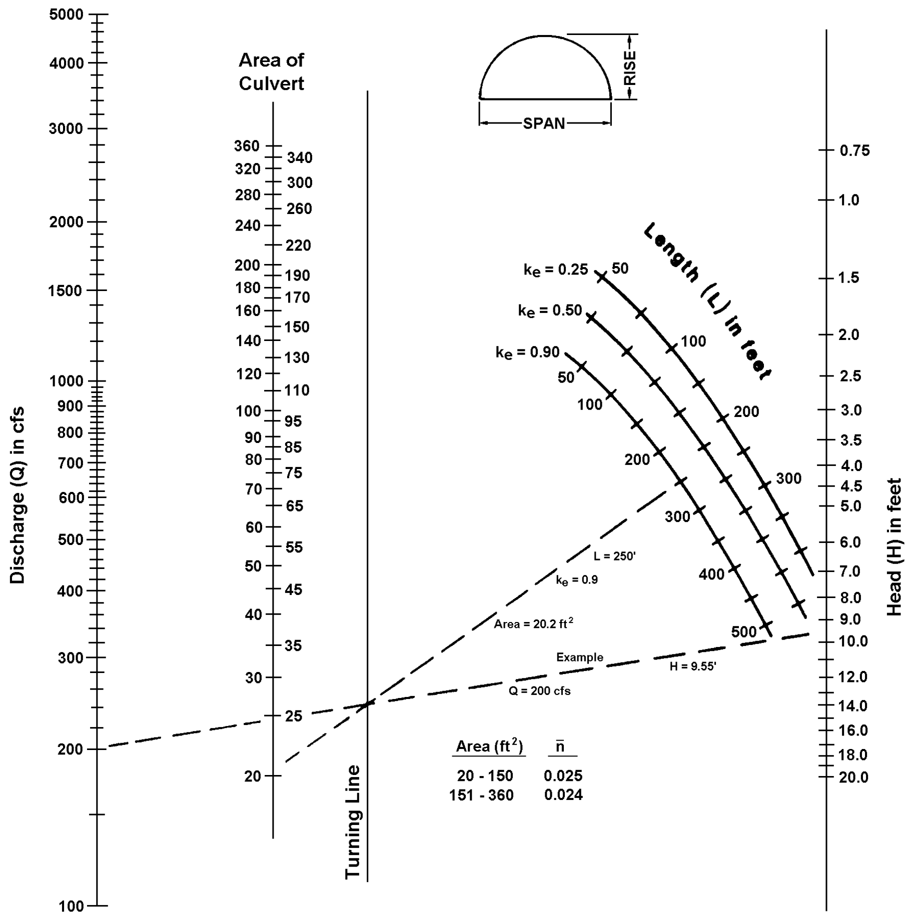


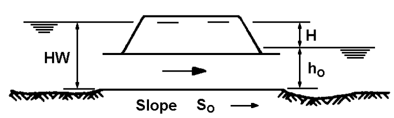
SUBMERGED OUTLET CULVERT FLOWING FULL

HEAD FOR C.M. ARCH CULVERTS FLOWING FULL CONCRETE BOTTOM 0.3 ≤ RISE/SPAN < 0.4

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

CHART 46



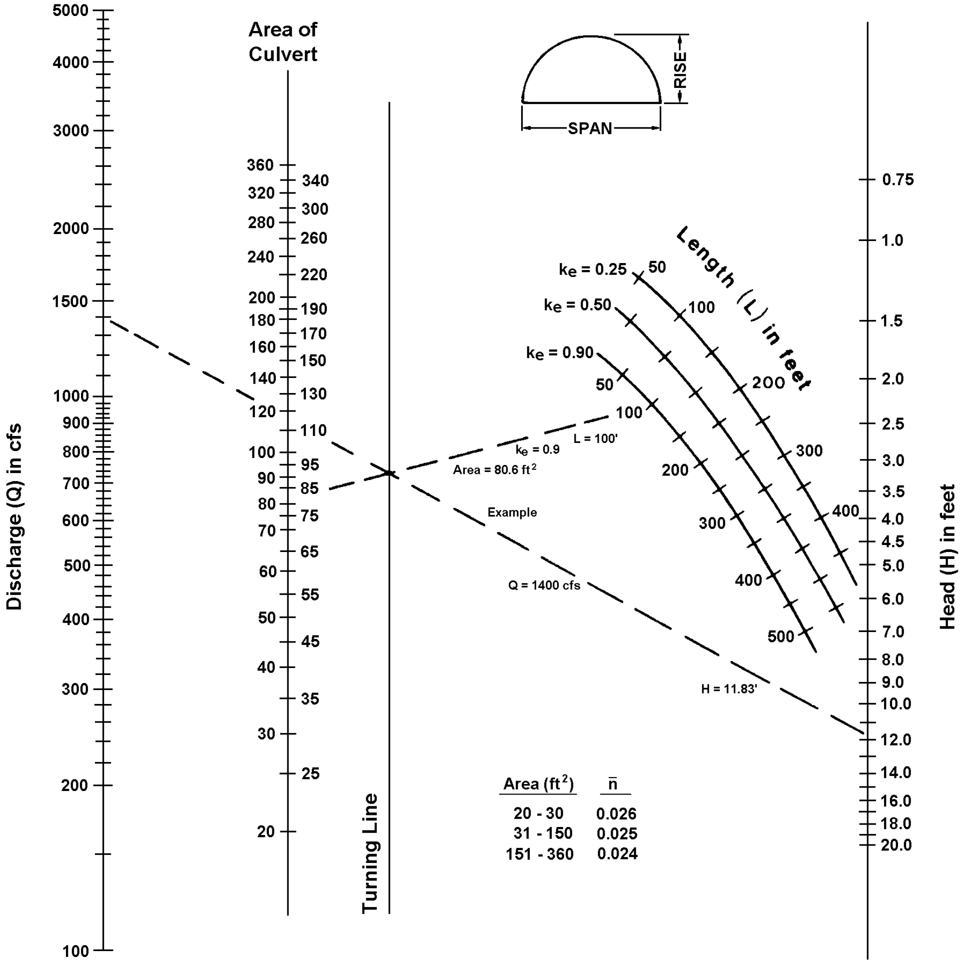


SUBMERGED OUTLET CULVERT FLOWING FULL

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.

CHART 47



Area (ft²)	n̄
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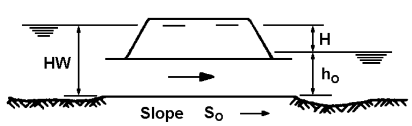
31 - 150	0.025
151 - 360	0.024

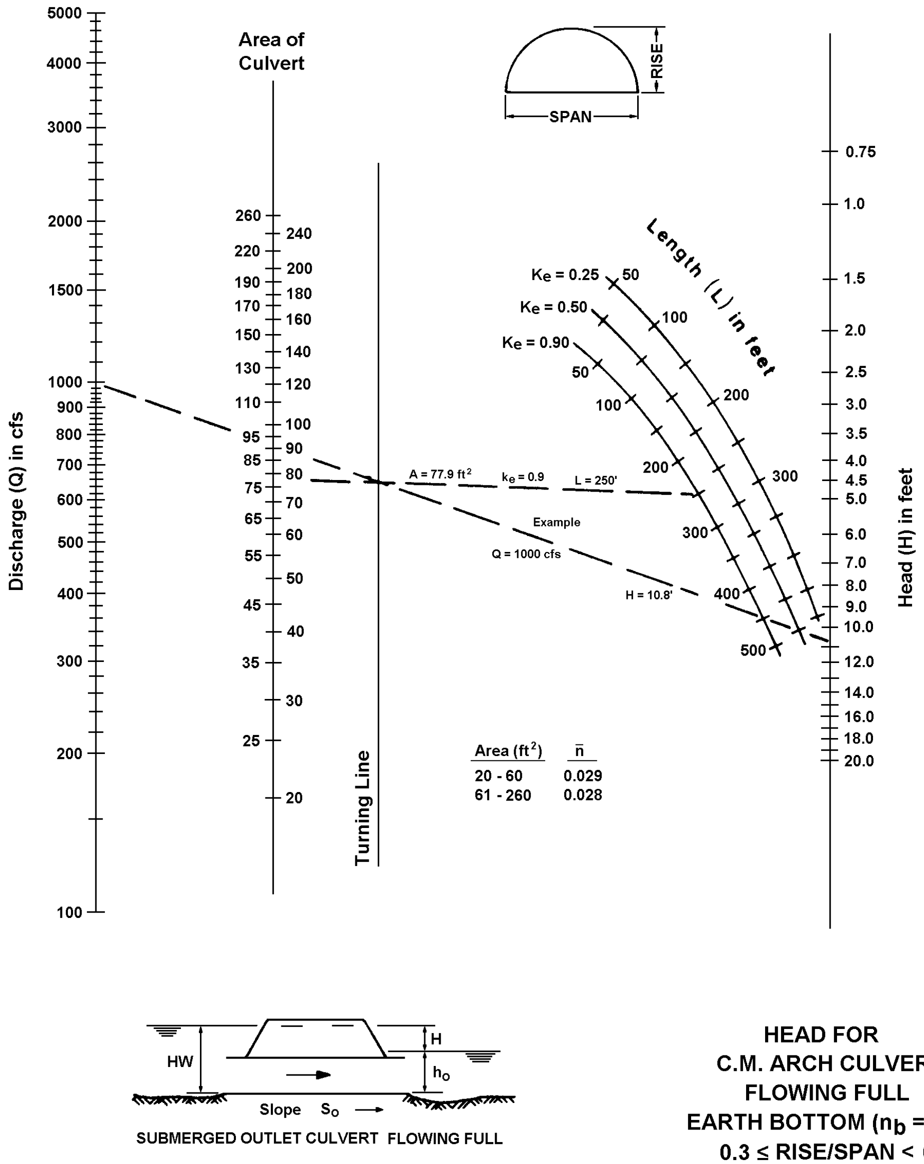


SUBMERGED OUTLET CULVERT FLOWING FULL

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.

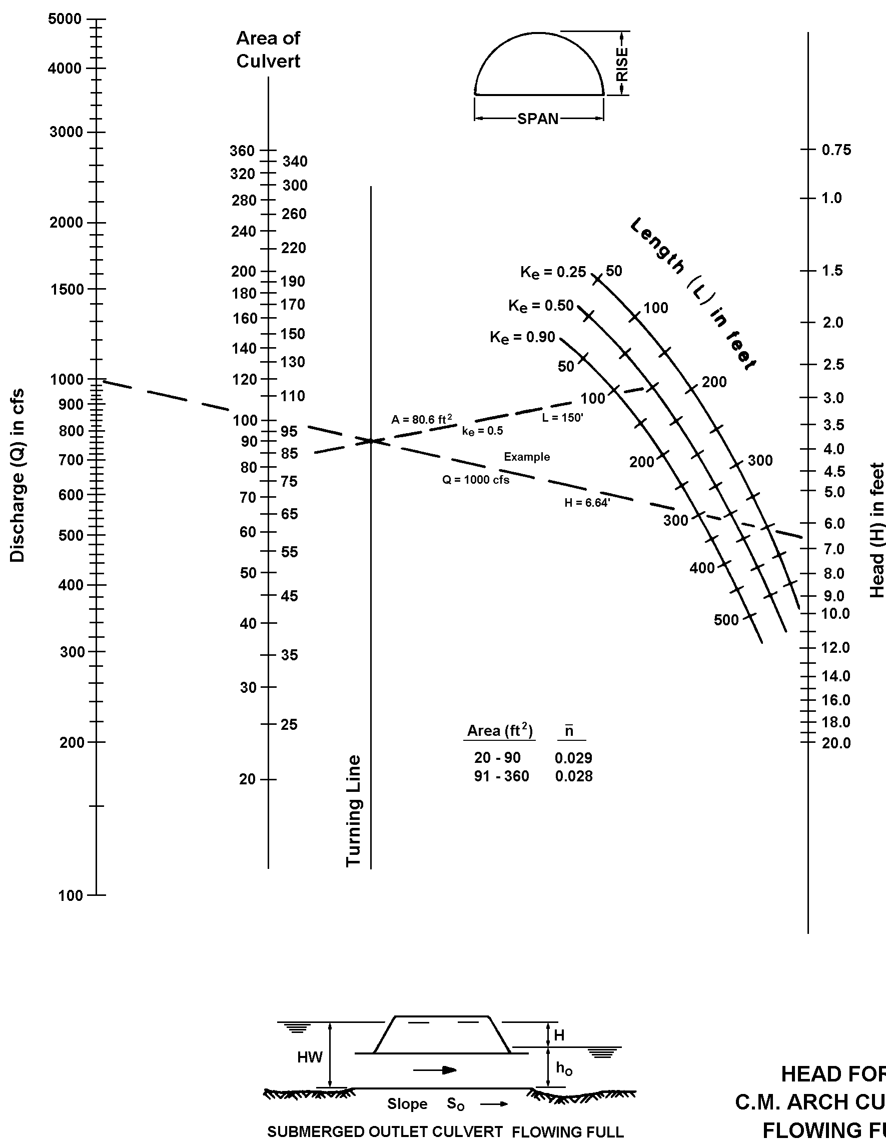
CHART 48



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.

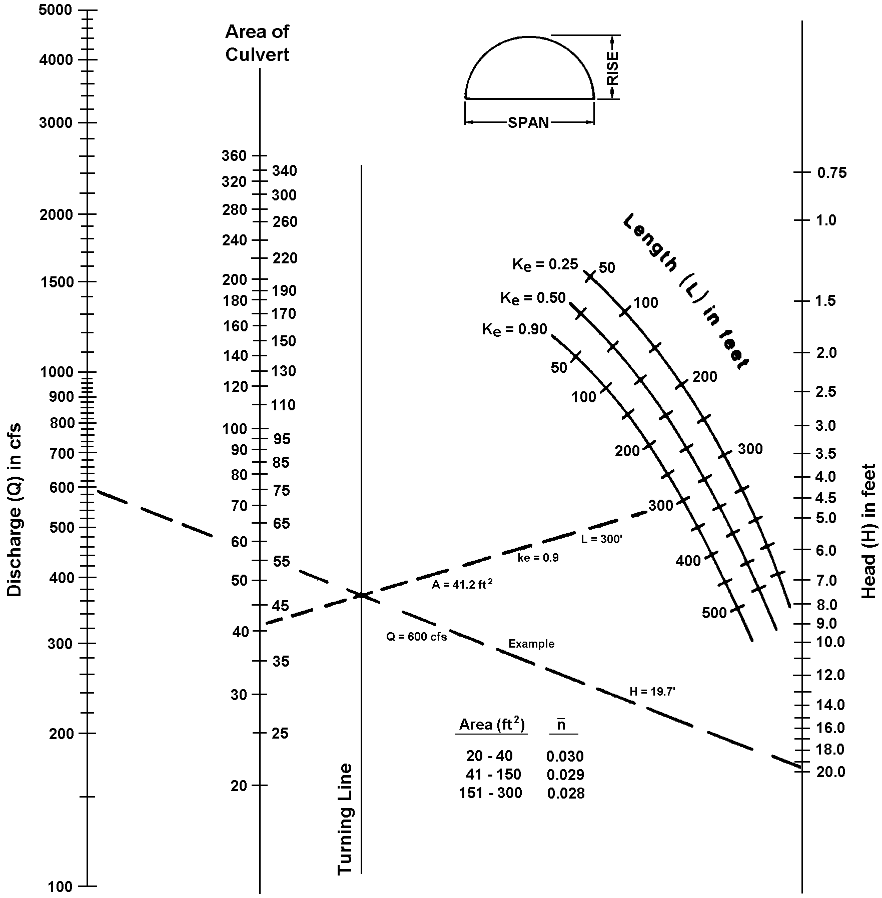
CHART 49

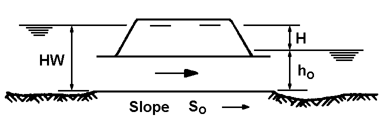


HEAD FOR C.M. ARCH CULVERTS FLOWING FULL EARTH BOTTOM (n_b = 0.022) 0.4 ≤ RISE/SPAN < 0.5

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

CHART 50



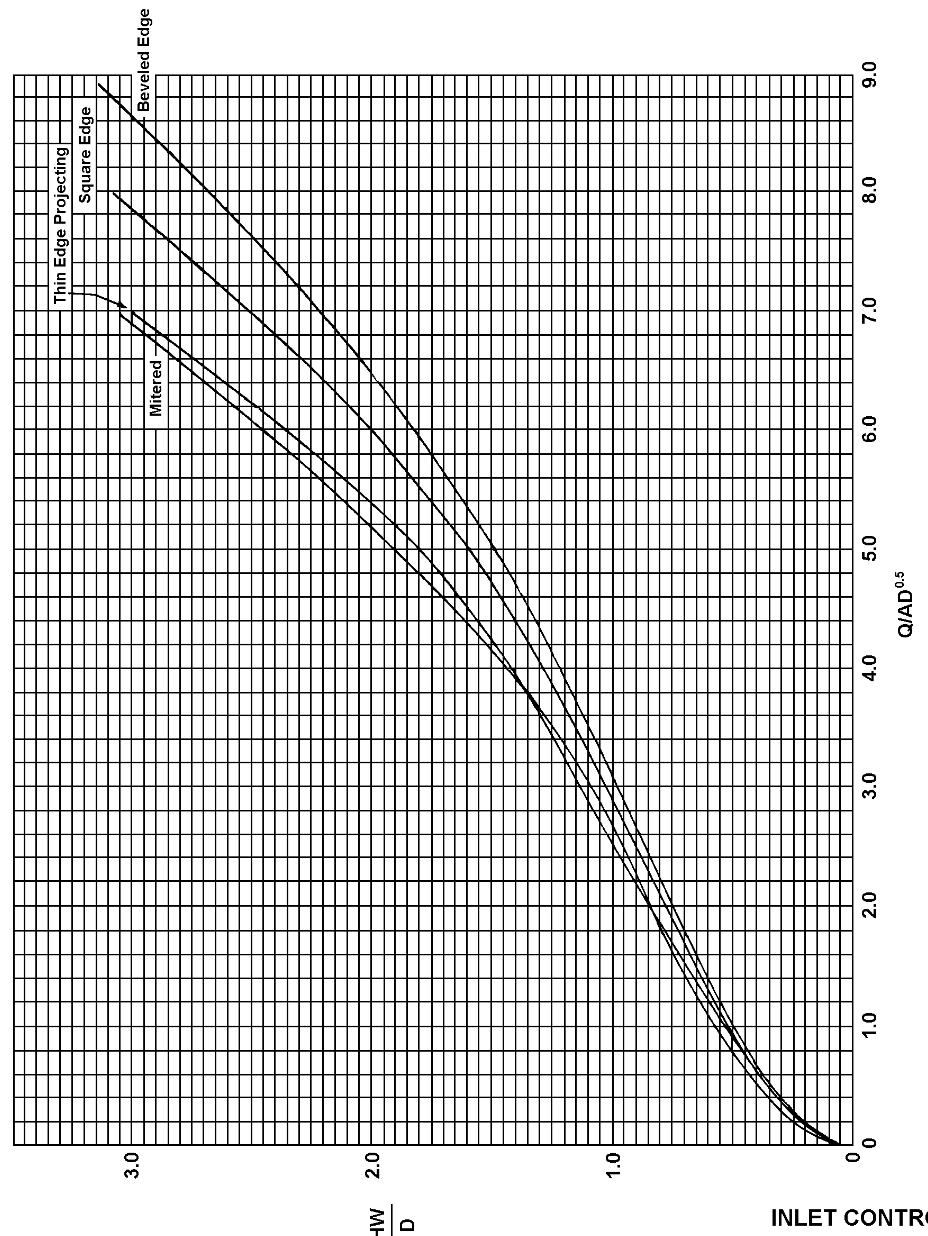


SUBMERGED OUTLET CULVERT FLOWING FULL

HEAD FOR
C.M. ARCH CULVERTS
FLOWING FULL
EARTH BOTTOM (n_b = 0.022)
0.5 ≤ RISE/SPAN

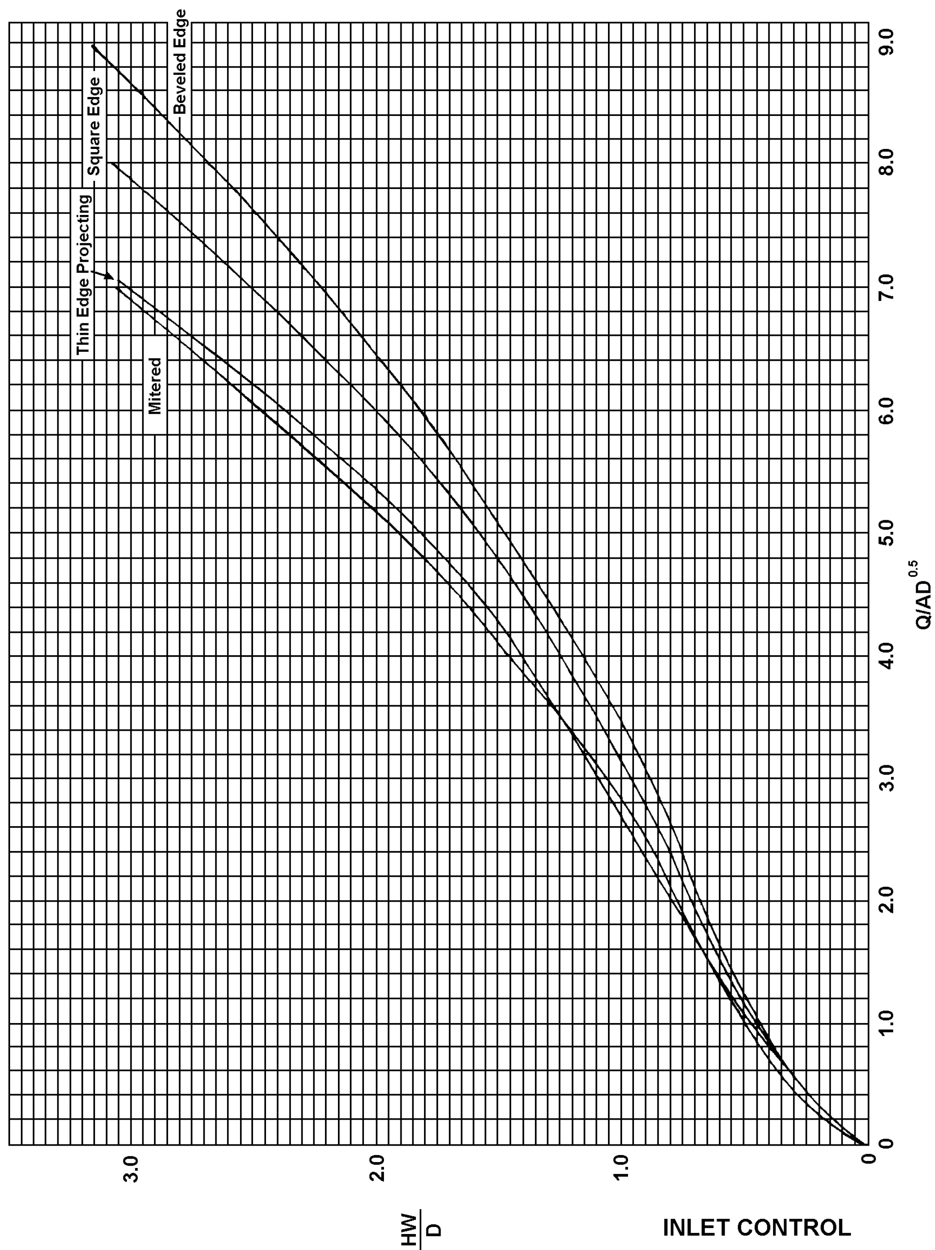
Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

CHART 51



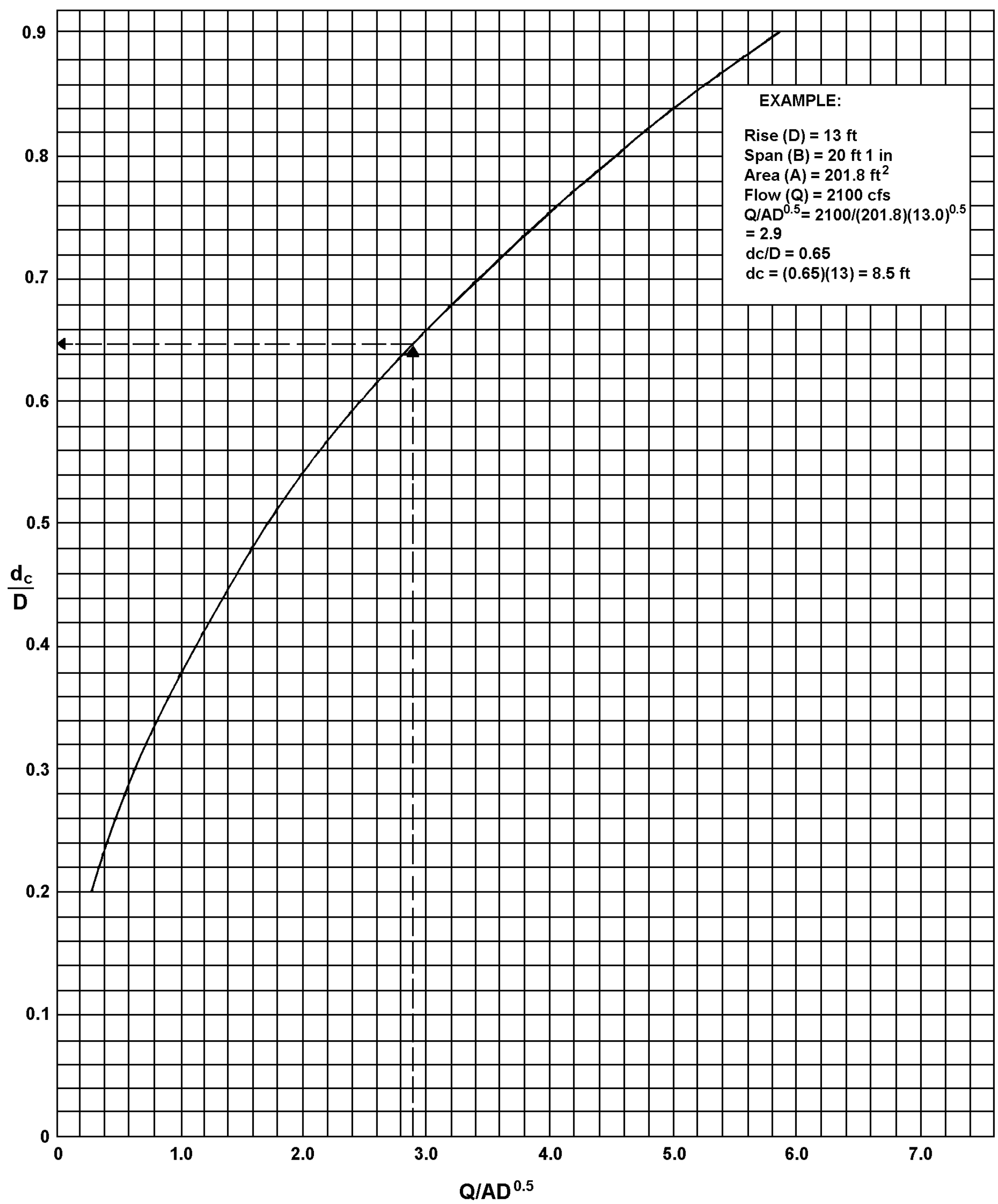
INLET CONTROL
HEADWATER DEPTH FOR
CIRCULAR OR ELLIPTICAL
STRUCTURAL PLATE
C.M. CONDUITS

CHART 52



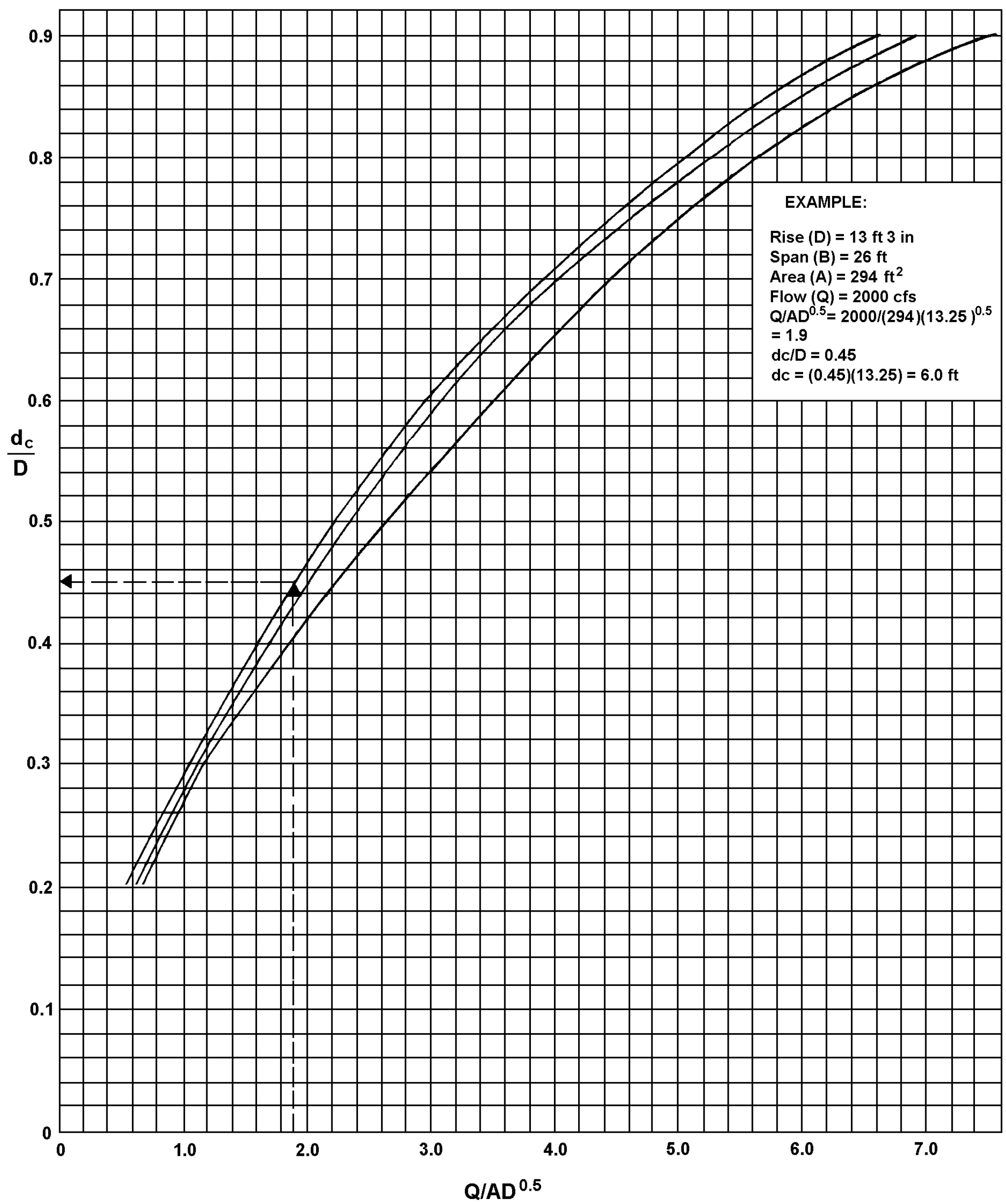
INLET CONTROL
HEADWATER DEPTH FOR
HIGH AND LOW PROFILE
STRUCTURAL PLATE
C.M. ARCH

CHART 53



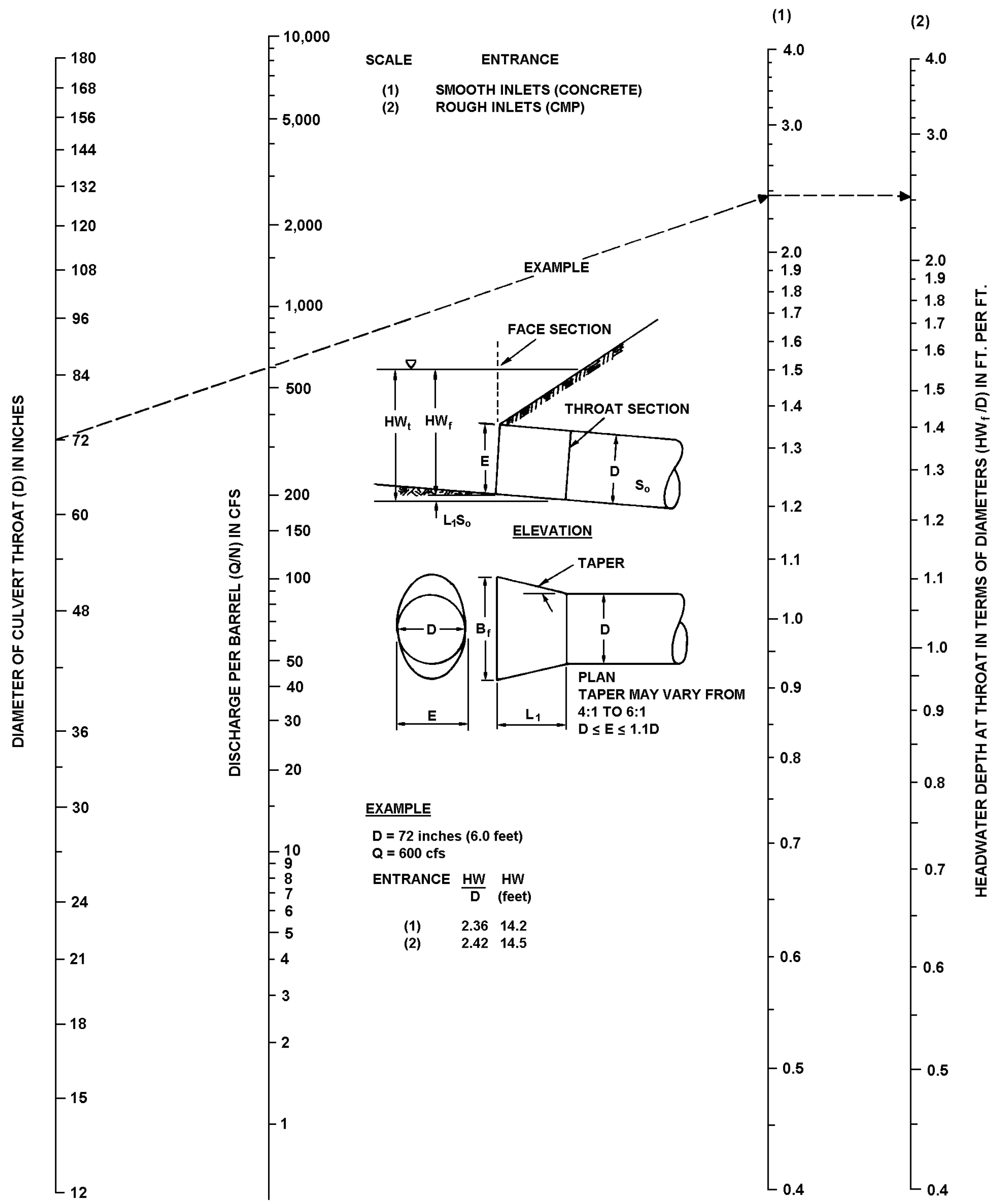
DIMENSIONLESS CRITICAL DEPTH CHART
FOR STRUCTURAL PLATE
ELLIPSE LONG AXIS HORIZONTAL

CHART 54



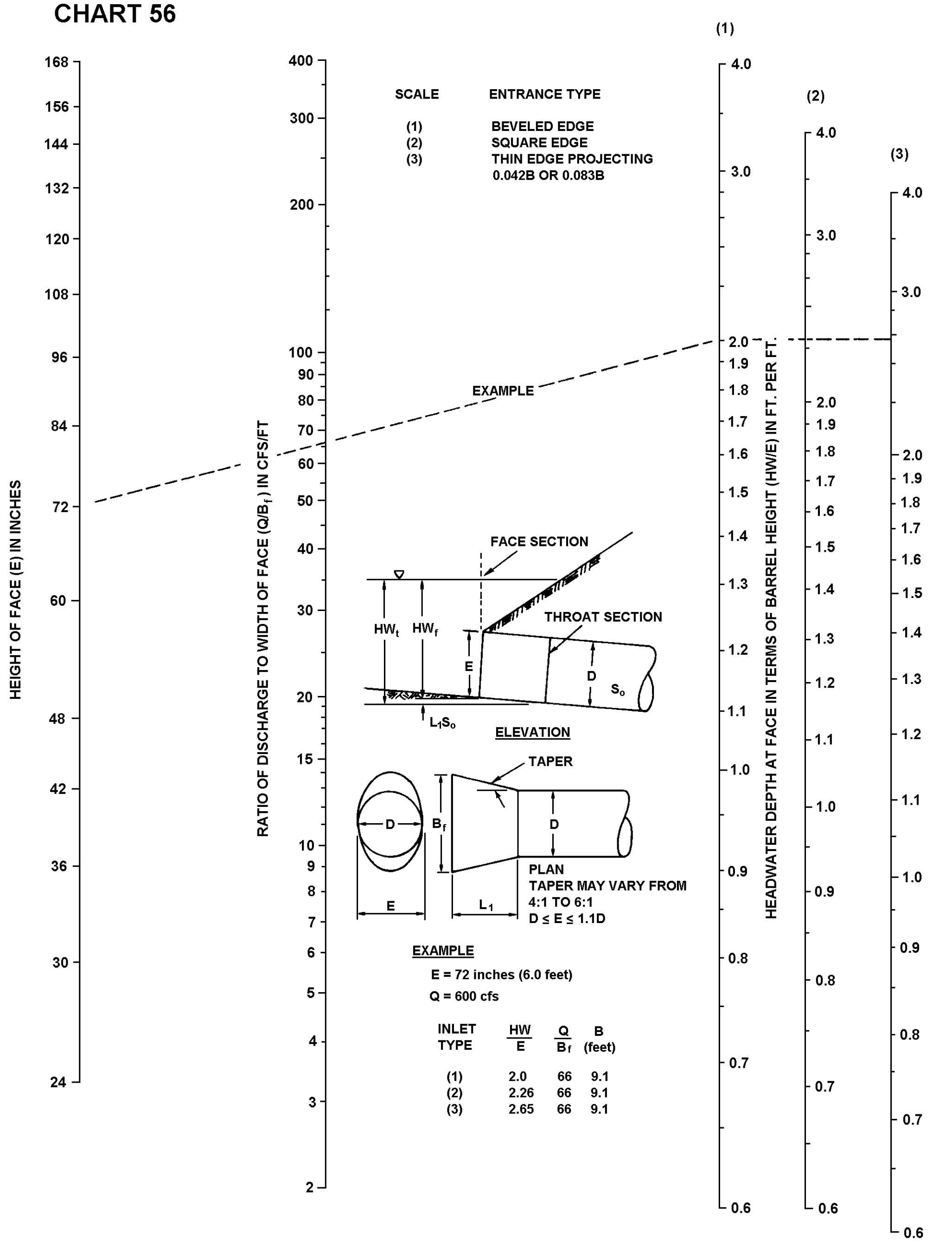
DIMENSIONLESS CRITICAL DEPTH CHART
FOR STRUCTURAL PLATE
LOW- AND HIGH-PROFILE ARCHES

CHART 55



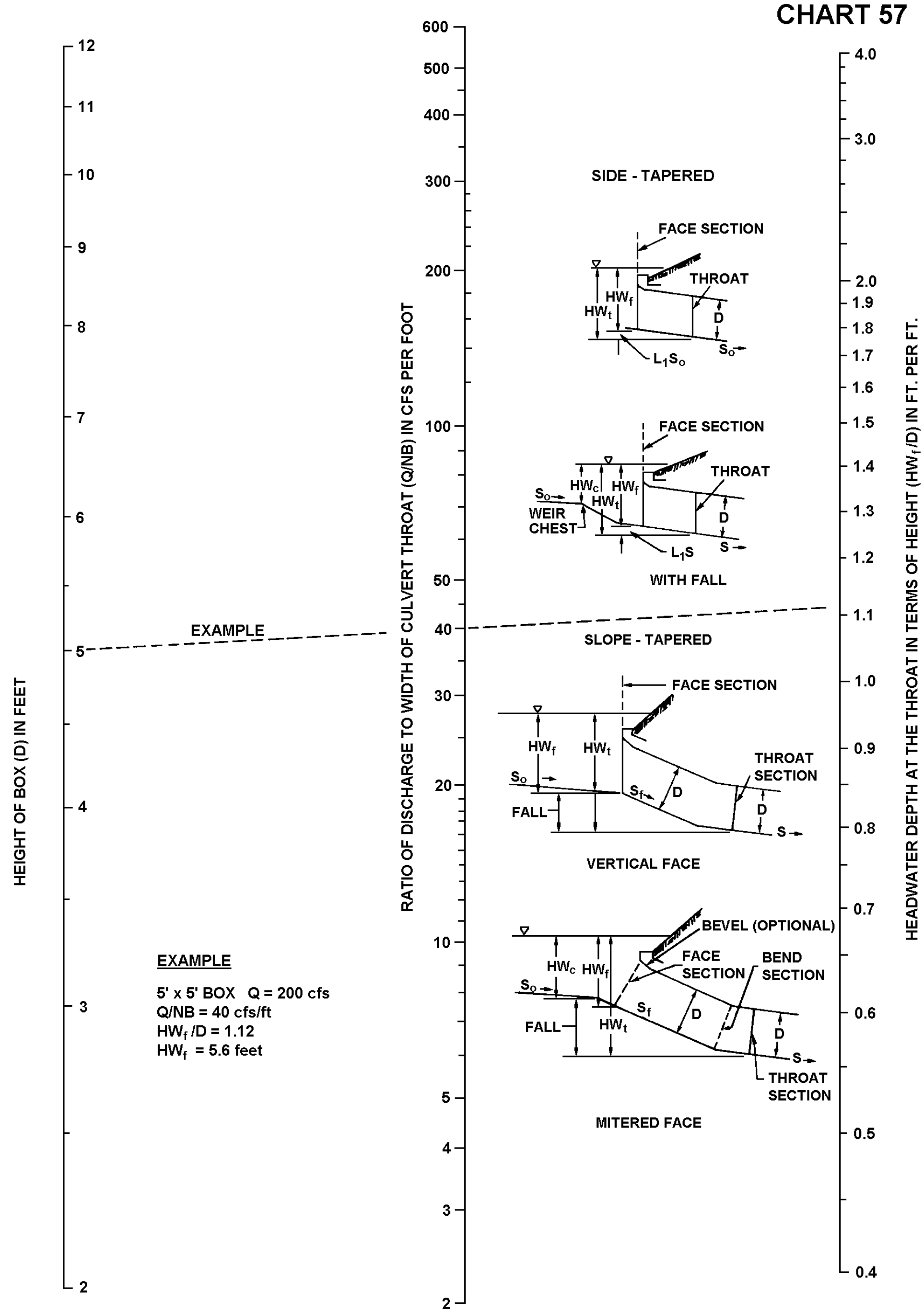
THROAT CONTROL
FOR SIDE-TAPERED INLETS
TO PIPE CULVERT
(CIRCULAR SECTION ONLY)

CHART 56



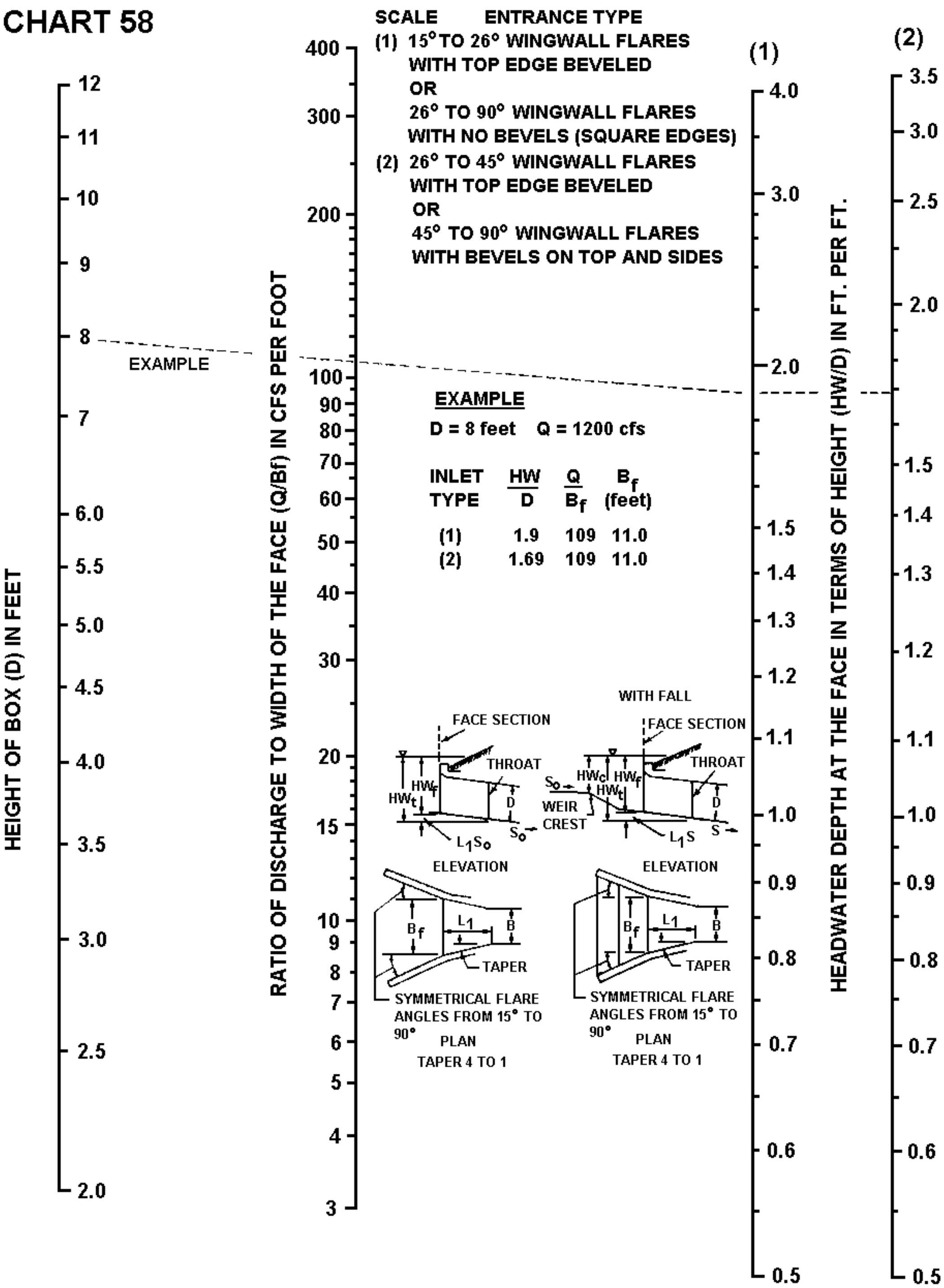
FACE CONTROL FOR SIDE-TAPERED
INLETS TO PIPE CULVERTS
(NON-RECTANGULAR SECTIONS ONLY)

CHART 57



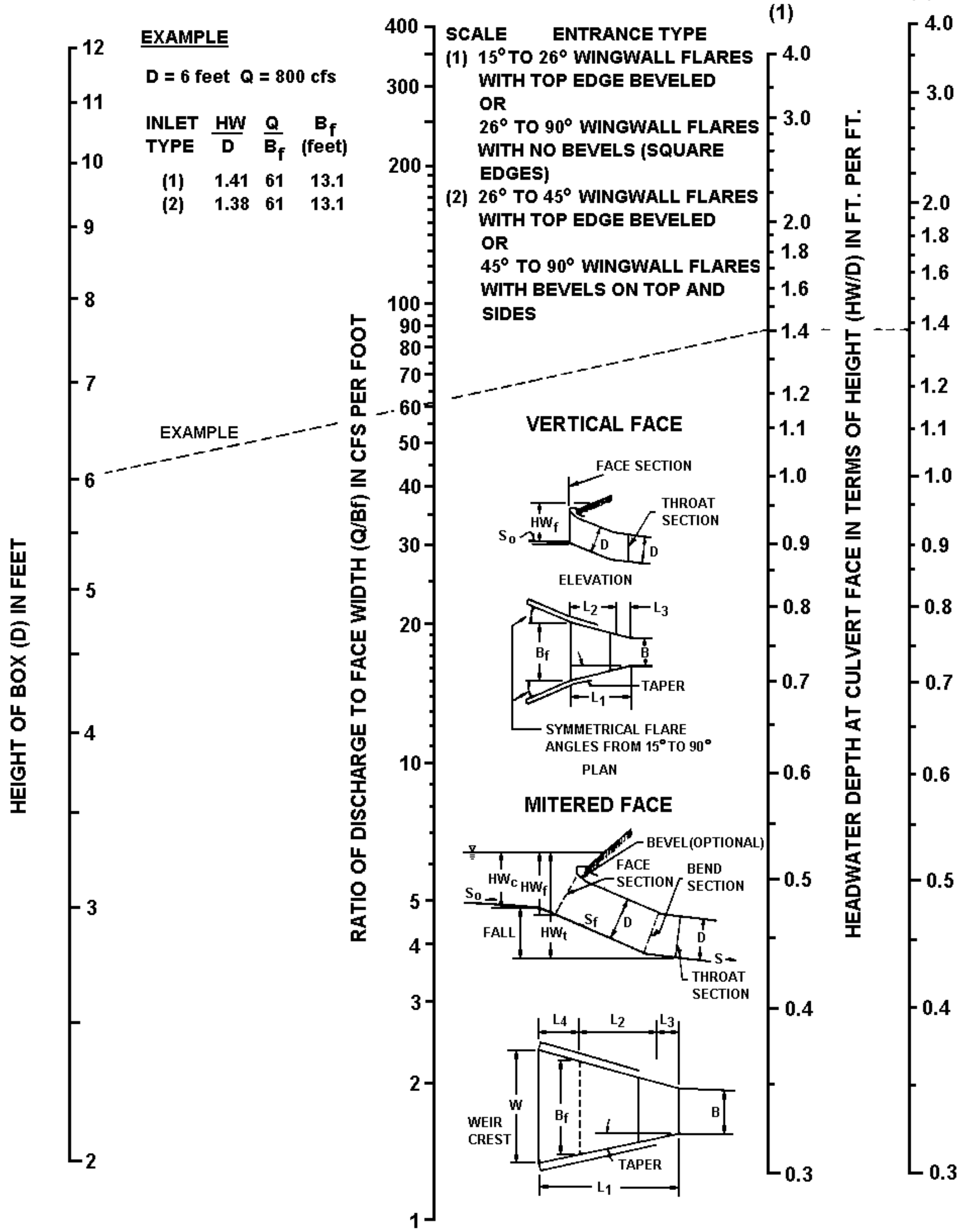
THROAT CONTROL FOR BOX CULVERTS WITH TAPERED INLETS

CHART 58



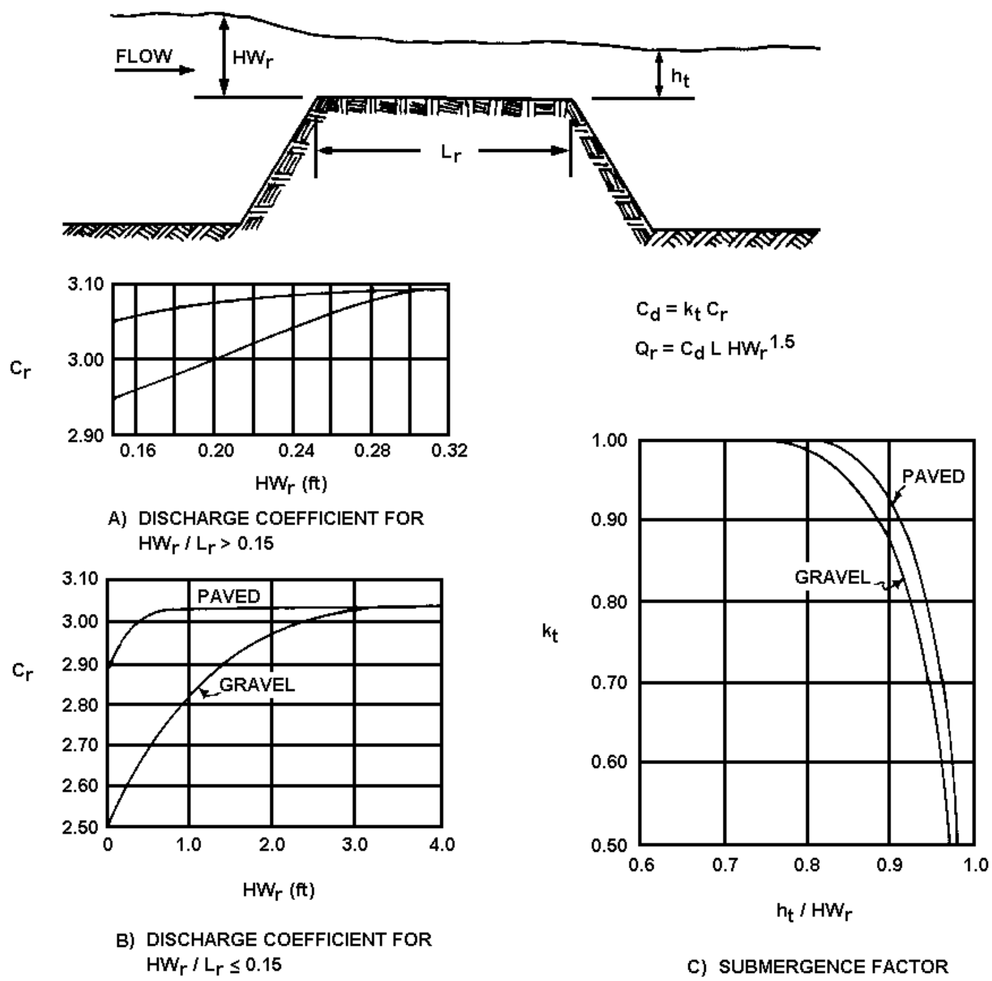
FACE CONTROL FOR BOX CULVERTS
WITH SIDE-TAPERED INLETS

CHART 59



FACE CONTROL FOR BOX CULVERTS WITH SLOPE TAPERED INLETS

CHART 60



DISCHARGE COEFFICIENTS FOR ROADWAY OVERTOPPING

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Debo, Thomas N. and Andrew J. Reese, 1995. Municipal Storm Water Management. Lewis Publishers.

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OPEN CHANNEL DESIGN

4.4.1 Overview

4.4.1.1 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.

4.4.1.2 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.

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 and provides water quality benefits (see Section 1.4 and Chapter 3 for more details on using enhanced swales and grass channels for 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.

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.

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.

4.4.2 Symbols And Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.4-1 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 4.4-1 Symbols and Definitions

Symbol	Definition	Units
α	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	Diameter 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	-

4.4.3 Design Criteria

4.4.3.1 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 4.4-2 and 4.4-3.

- 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.

4.4.3.2 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 4.4-2. 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 4.4-3. Vegetative lining calculations are presented in Section 4.4.7 and riprap procedures are presented in Section 4.4.8.

4.4.4 Manning's n 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 4.4-4. Recommended values for vegetative linings should be determined using Figure 4.4-1, 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 4.4-6). Figure 4.4-1 is used iteratively as described in Section 4.4.6. Recommended Manning's values for natural channels that are either excavated or dredged and natural are given in Table 4.4-5. For natural channels, Manning's n values should be estimated using experienced judgement 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.

Table 4.4-2 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, 1991

Table 4.4-3 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 Mixtures 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, 1996

4.4.5 Uniform Flow Calculations

4.4.5.1 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 subsections 4.4.12, 4.4.13 and 4.4.14.

4.4.5.2 Manning's Equation

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

$$v = (1.49/n)\, R^{2/3} S^{1/2} \tag{4.4.1}$$

$$Q = (1.49/n)\, A\, R^{2/3} S^{1/2} \tag{4.4.2}$$

$$S = [Q_n/(1.49\, A\, R^{2/3})]^2 \tag{4.4.3}$$

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.

4.4.5.3 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 coefficients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter.

4.4.5.4 Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from equation 4.4.2. The slope can be calculated using equation 4.4.3 when the discharge, roughness coefficient, area, and hydraulic radius are known.

Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 4.4-2 and 4.4-3. Figure 4.4-2 provides a general solution for the velocity form of Manning's Equation, while Figure 4.4-3 provides a solution of Manning's Equation for trapezoidal channels.

Table 4.4-4 Manning's Roughness Coefficients (n) for Artificial Channels

Category	Lining Type	Depth Ranges 0-0.5 ft	0.5-2.0 ft	>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, 1988.

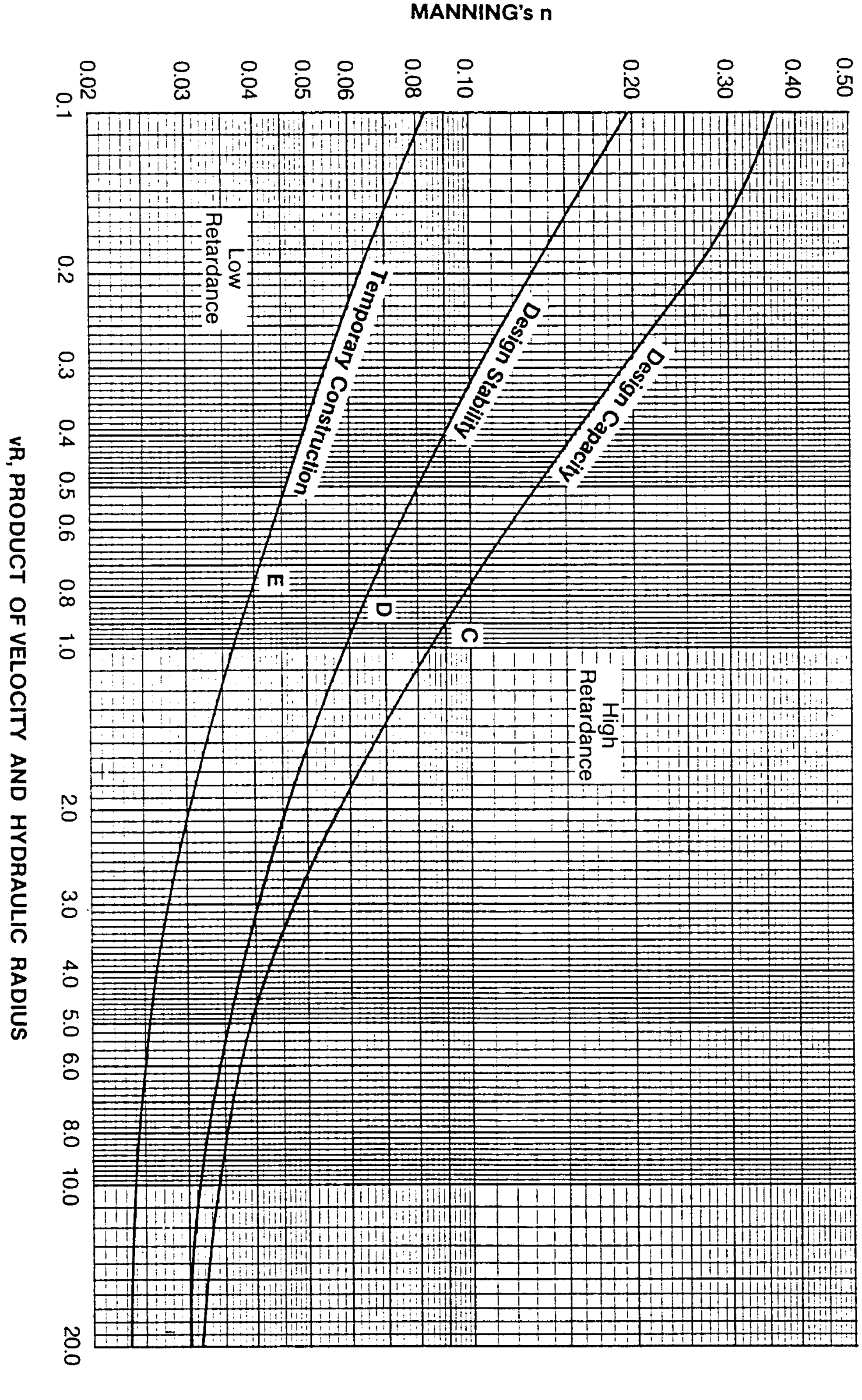


Figure 4.4-1 Manning's n Values for Vegetated Channels
(Source: USDA, TP-61, 1947)

Table 4.4-5 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 section, 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. Streams 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

Table 4.4-5 Uniform Flow Values of Roughness Coefficient n (continued)

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 offer 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, 1988

Table 4.4-6 Classification of Vegetal Covers as to Degrees of Retardance

Retardance	Cover	Condition
A	Weeping Lovegrass	Excellent stand, tall (average 30")
	Yellow Bluestem Ischaemum	Excellent stand, tall (average 36")
B	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (average 12")
	Native grass mixture 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	Fair stand, uncut (10 - 48")
	Bermuda grass	Good stand, mowed (average 6")
	Common lespedeza	Good stand, uncut (average 11")
	Grass-legume mixture: summer (orchard grass redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6 - 8")
	Centipede grass	Very dense cover (average 6")
	Kentucky bluegrass	Good stand, headed (6 - 12")
D	Bermuda grass	Good stand, cut to 2.5"
	Common lespedeza	Excellent stand, uncut (average 4.5")
	Buffalo grass	Good stand, uncut (3 - 6")
	Grass-legume mixture: 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	Good stand, cut to 1.5"
	Bermuda grass	Burned stubble

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

Source: HEC-15, 1988

General Solution Nomograph

The following steps are used for the general solution nomograph in Figure 4.4-2:

(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 4.4-3 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.

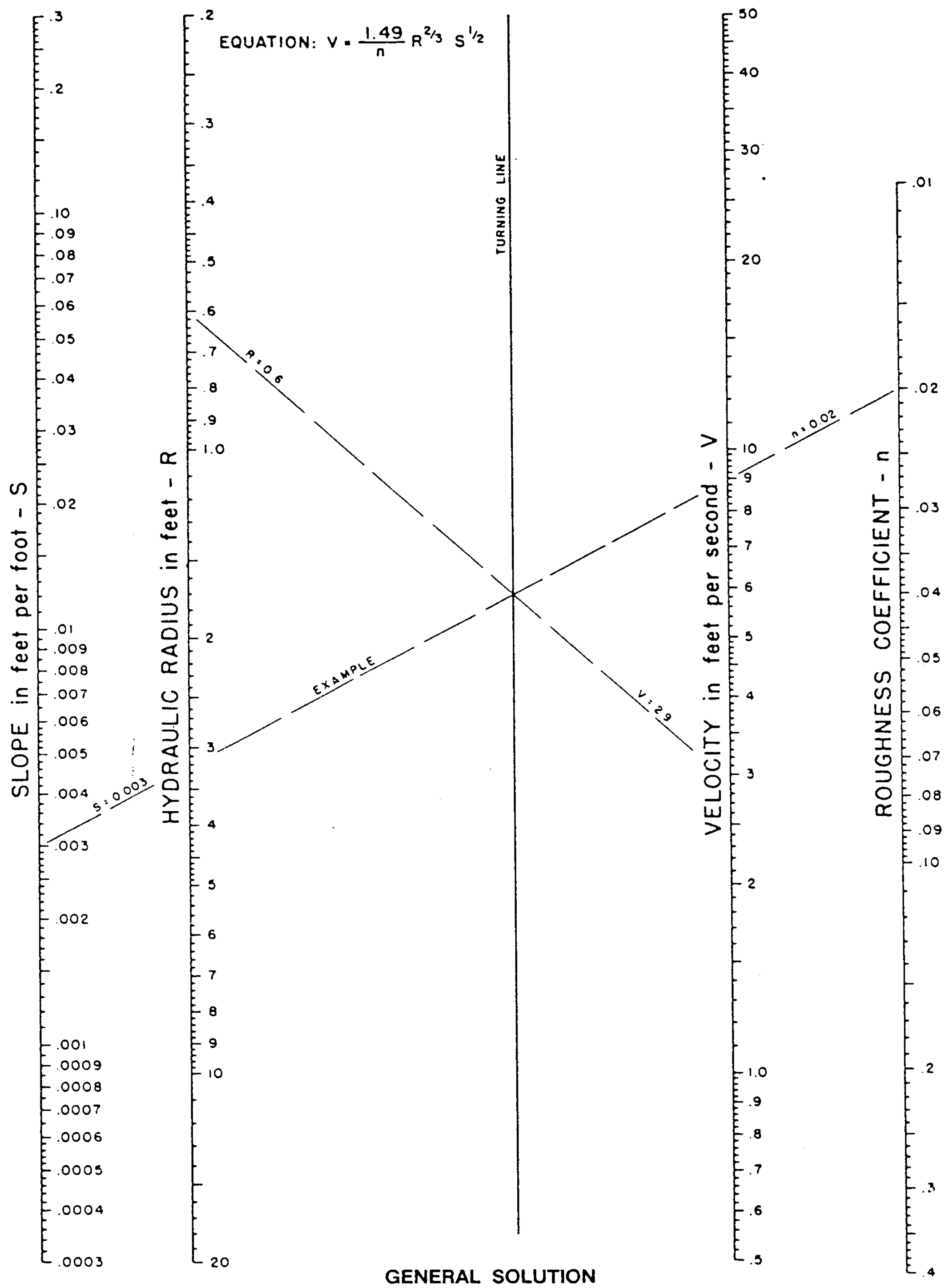
(2) Given d, find Q

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.

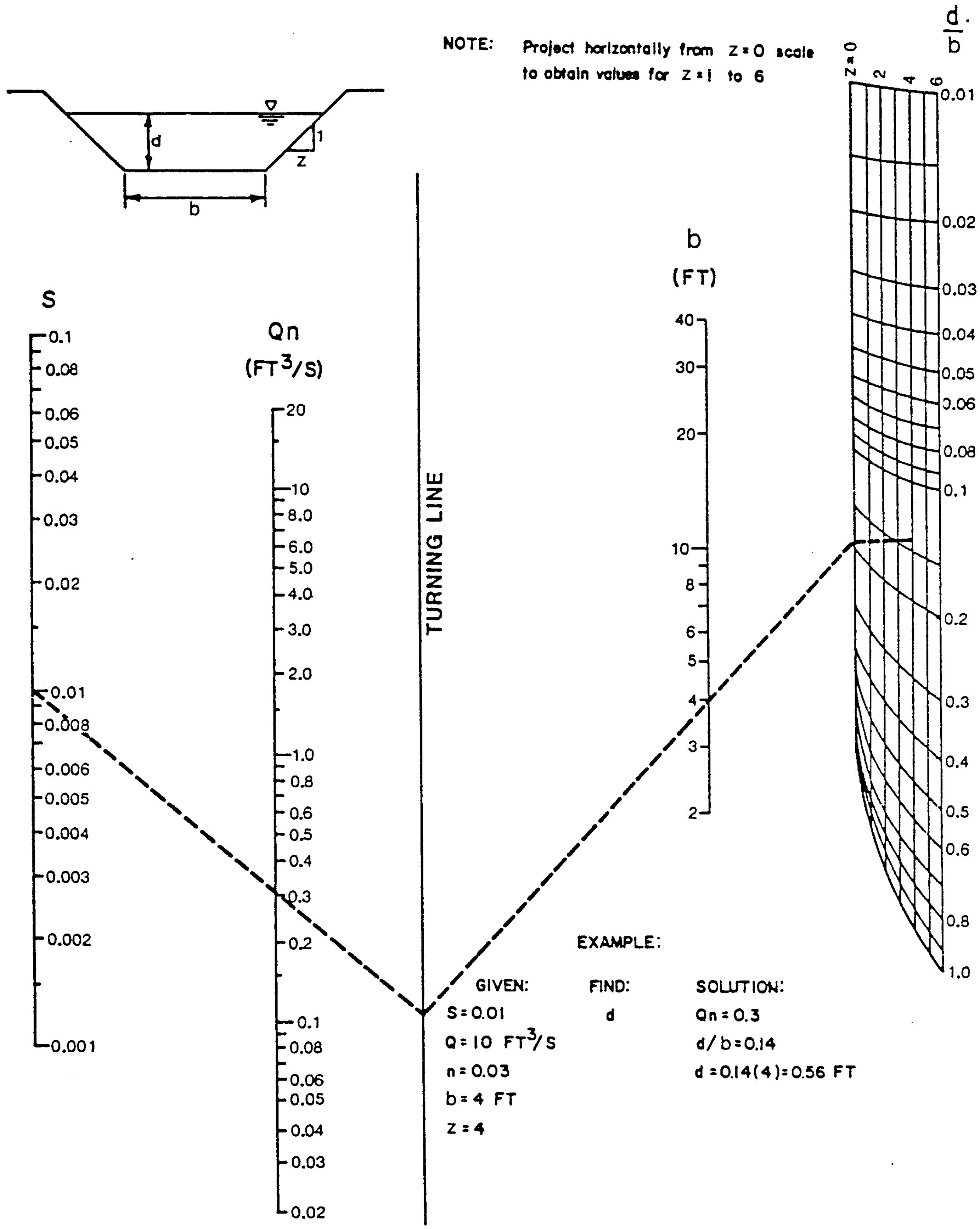
c. Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Q_n scale.

d. Divide the value of Qn obtained in Step 3c by the n value to find the design discharge, Q.



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

Figure 4.4-2 Nomograph for the Solution of Manning's Equation



Reference: USDOT, FHWA, HEC-15 (1986).

Figure 4.4-3 Solution of Manning's Equation for Trapezoidal Channels

4.4.5.5 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:

$$AR^{2/3} = (Qn)/(1.49\ S^{1/2}) \tag{4.4.4}$$

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 4.4.4 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 4.4-4 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:

$$K_T = (Qn)/(b^{8/3}S^{1/2}) \tag{4.4.5}$$

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 4.4-4 with the value of K_T 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 4.4.11

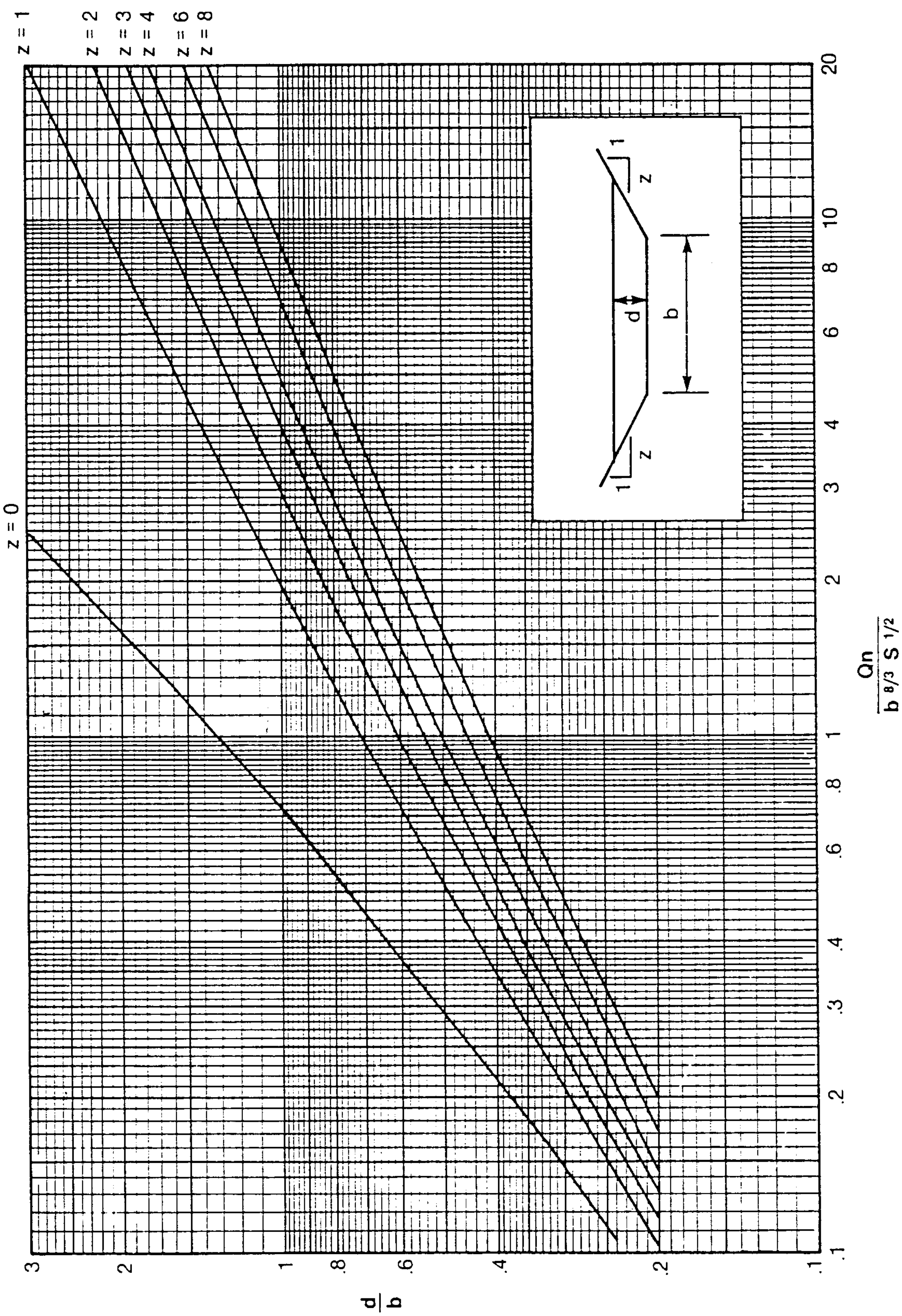


Figure 4.4-4 Trapezoidal Channel Capacity Chart
(Source: Nashville Storm Water Management Manual, 1988)

4.4.6 Critical Flow Calculations

4.4.6.1 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:

$$Q^2/g = A^3/T \qquad \textbf{(4.4.6)}$$

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 4.4-6.

4.4.6.2 Semi-Empirical Equations

Semi-empirical equations (as presented in Table 4.4-7) or section factors (as presented in Figure 4.4-5) 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:

$$Z = Q/(g^{0.5}) \qquad \textbf{(4.4.7)}$$

Where: Z = critical flow section factor
Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.3 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:

$$Fr = v/(gA/T)^{0.5} \tag{4.4.8}$$

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)

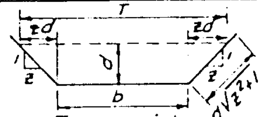
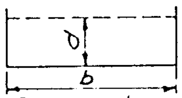
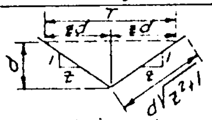
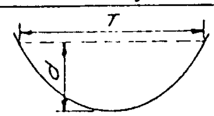
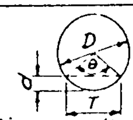
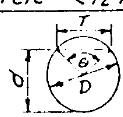
If Fr is greater than 1.0, flow is supercritical; if it is under 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

Table 4.4-7 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]	$d_c = 0.81[Q^2/(gz^{0.75}b^{1.25})]^{0.27} - b/30z$	$0.1 < 0.5522\ Q/b^{2.5} < 0.4$ For $0.5522\ Q/b^{2.5} < 0.1$, use rectangular channel equation
3. Triangular[3]	$d_c = [(2Q^2)/(gz^2)]^{1/5}$	N/A
4. Circular[4]	$d_c = 0.325(Q/D)^{2/3} + 0.083D$	$0.3 < d_c/D < 0.9$
5. General[5]	$(A^3/T) = (Q^2/g)$	N/A

Where: d_c = critical depth (ft)
Q = design discharge (cfs)
G = acceleration due to gravity (32.3 ft/s^2)
b = bottom width of channel (ft)
z = side slopes of a channel (horizontal to vertical)
D = diameter of circular conduit (ft)
A = cross-sectional area of flow (ft^2)
T = top width of water surface (ft)

[1]See Figure 4.4-5 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 θ 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 4.4-5 Open Channel Geometric Relationships for Various Cross Sections

4.4.7 Vegetative Design

4.4.7.1 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 4.4-6. 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 4.4-6. 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 not presented below are required. References include HEC-15 (USDOT, FHWA, 1986) and HEC-14 (USDOT, FHWA, 1983).

4.4.7.2 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 4.4-3 to assign a maximum velocity, v_m 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 4.4-1. Use retardance Class D for permanent vegetation and E for temporary construction.

(Step 4) Calculate the hydraulic radius using the equation:

$$R = (vR)/v_m \quad \textbf{(4.4.9)}$$

Where: R = hydraulic radius of flow (ft)
vR = value obtained from Figure 4.4-1 in Step 3
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:

$$vR = (1.49\ R^{5/3}\ S^{1/2})/n \quad \textbf{(4.4.10)}$$

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 4.4-1 for the assumed n value in Step 3. If the values are not reasonably 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 4.4-3 or 4.4-4, as described in Section 4.4.4.4. The depth of flow for other channel shapes can be evaluated using the trial and error procedure described in Section 4.4.4.5.

(Step 8) If bends are considered, calculate the length of downstream protection, L_p, for the bend, using Figure 4.4-6. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, L_p.

4.4.7.3 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 4.4-5 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 4.4-1 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 4.4.1) or Figure 4.4-2 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.

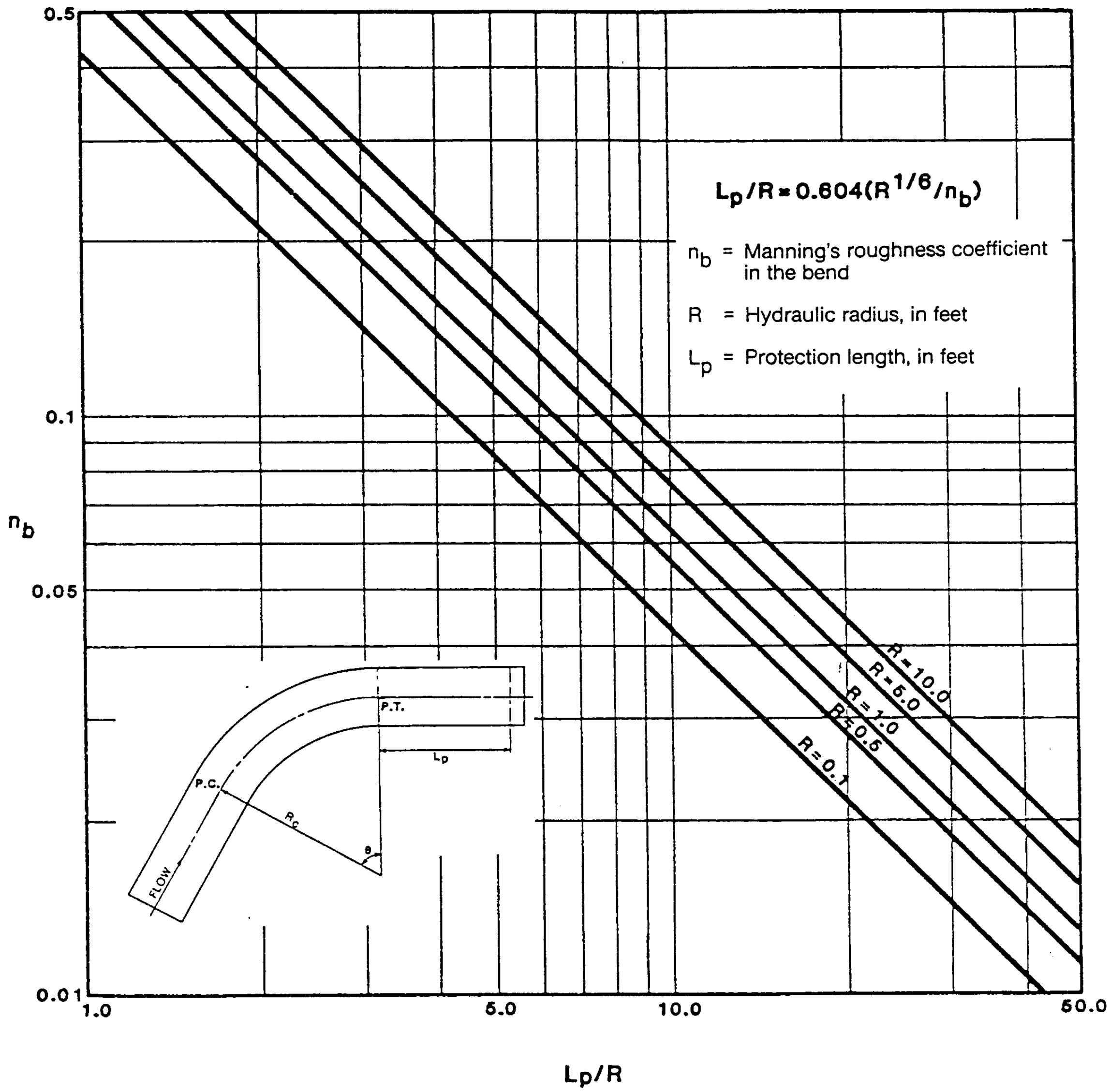
(Step 7) Add an appropriate freeboard to the final depth from Step 6. Generally, 20% is adequate.

(Step 8) If bends are considered, calculate superelevation of the water surface profile at the bend using the equation:

$$\Delta d = (v^2 T)/(g R_c) \tag{4.4.11}$$

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 4.4-6 Protection Length, Lp, Downstream of Channel Bend

4.4.8 Riprap Design

4.4.8.1 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 placement in both natural and prismatic channels and has the following assumptions 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.

4.4.8.2 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:

$$n = 0.0395\,(d_{50})^{1/6} \qquad (4.4.12)$$

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)

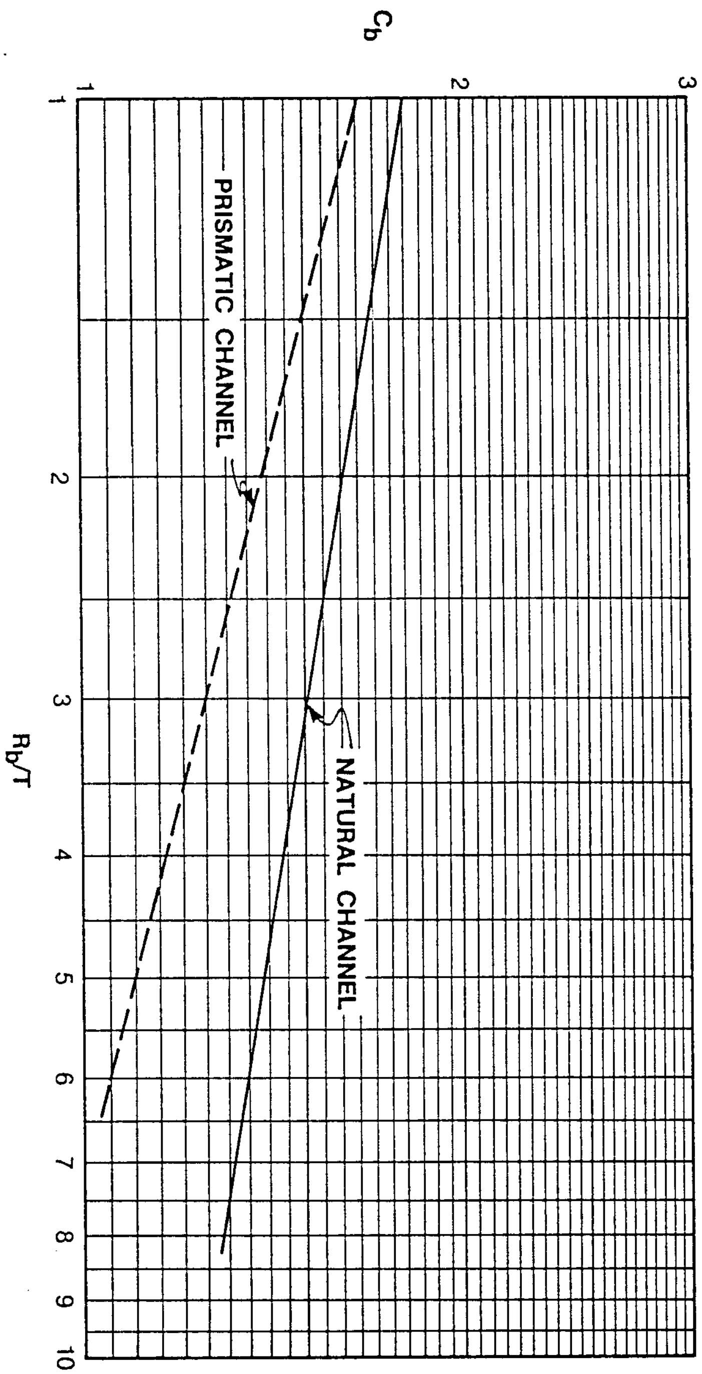
(Step 2) If rock is to be placed at the outside of a bend, multiply the velocity determined in Step 1 by the bend correction coefficient, C_b, given in Figure 4.4-7 for either a natural or prismatic channel. This requires determining the channel top width, T, just upstream from the bend and the centerline 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 4.4-8.

(Step 4) Determine the required minimum d_{30} value from Figure 4.4-9, or from the equation:

$$d_{30}/D = 0.193\,Fr^{2.5} \qquad (4.4.13)$$

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 4.4.8), dimensionless
v = mean velocity above the stone (ft/s)
g = acceleration of gravity (32.2 ft/sec)



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 4.4-7 Riprap Lining Bend Correction Coefficient

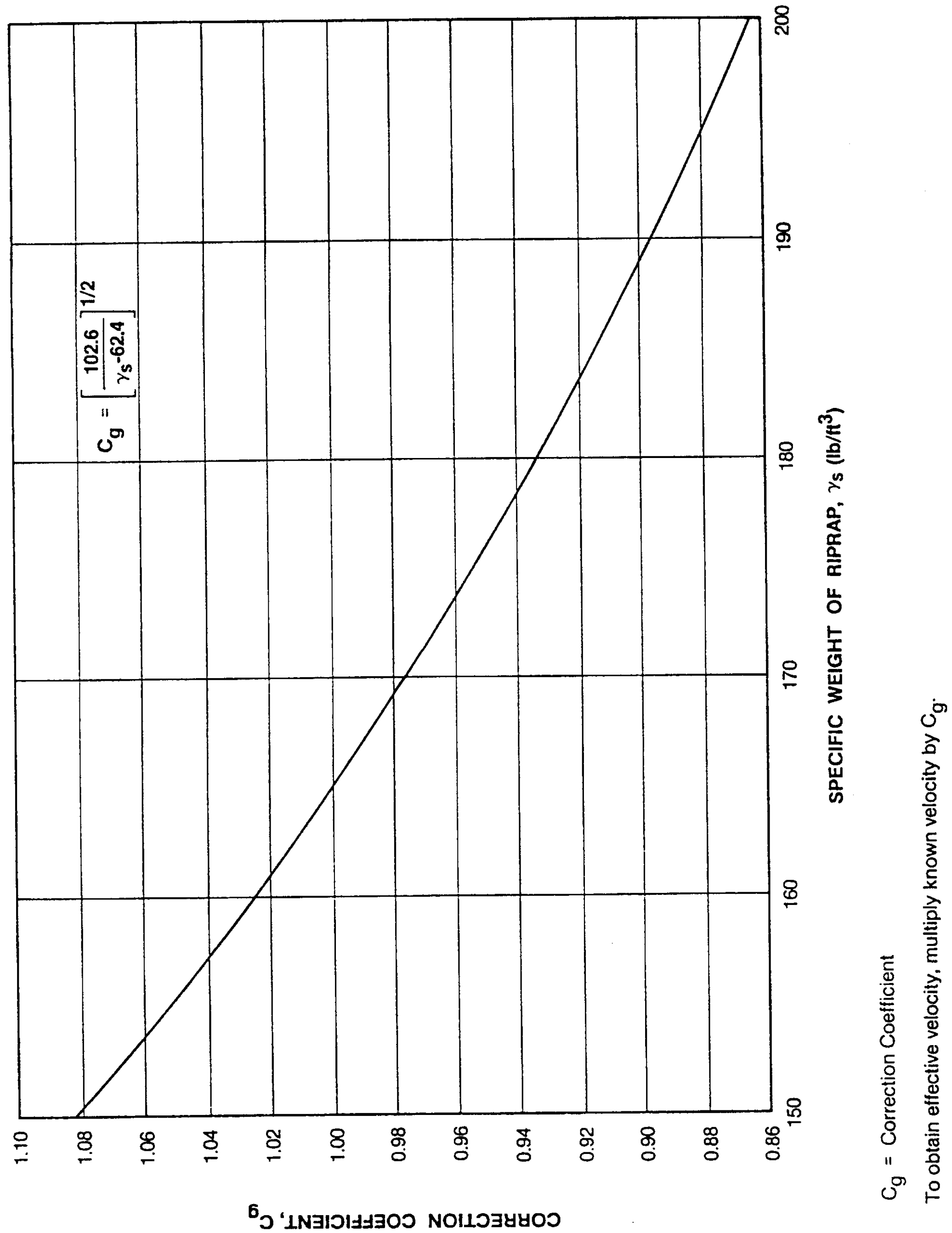
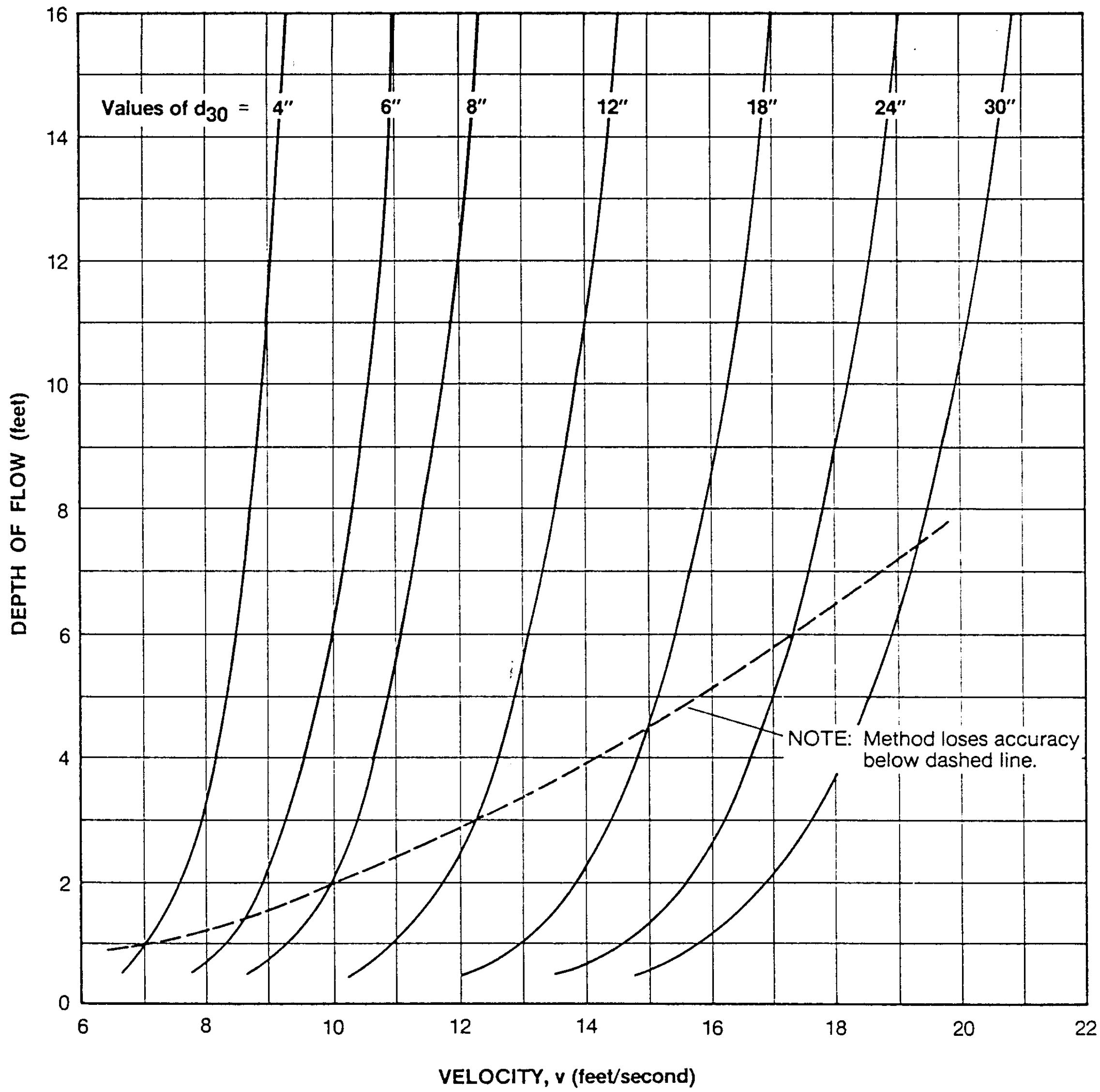


Figure 4.4-8 Riprap Lining Specific Weight Correction Coefficient
(Source: Nashville Storm Water Management Manual, 1988)



Reference: Reese (1988).

Figure 4.4-9 Riprap Lining d_{30} Stone Size – Function of Mean Velocity and Depth

(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:

$$W = 0.5236\ SW_s\ d^3 \qquad (4.4.14)$$

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 4.4-10 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.

4.4.9 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 4.4-2:

(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 4.4-3, the maximum velocity, v_m, 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 4.4-1, vR = 5.4

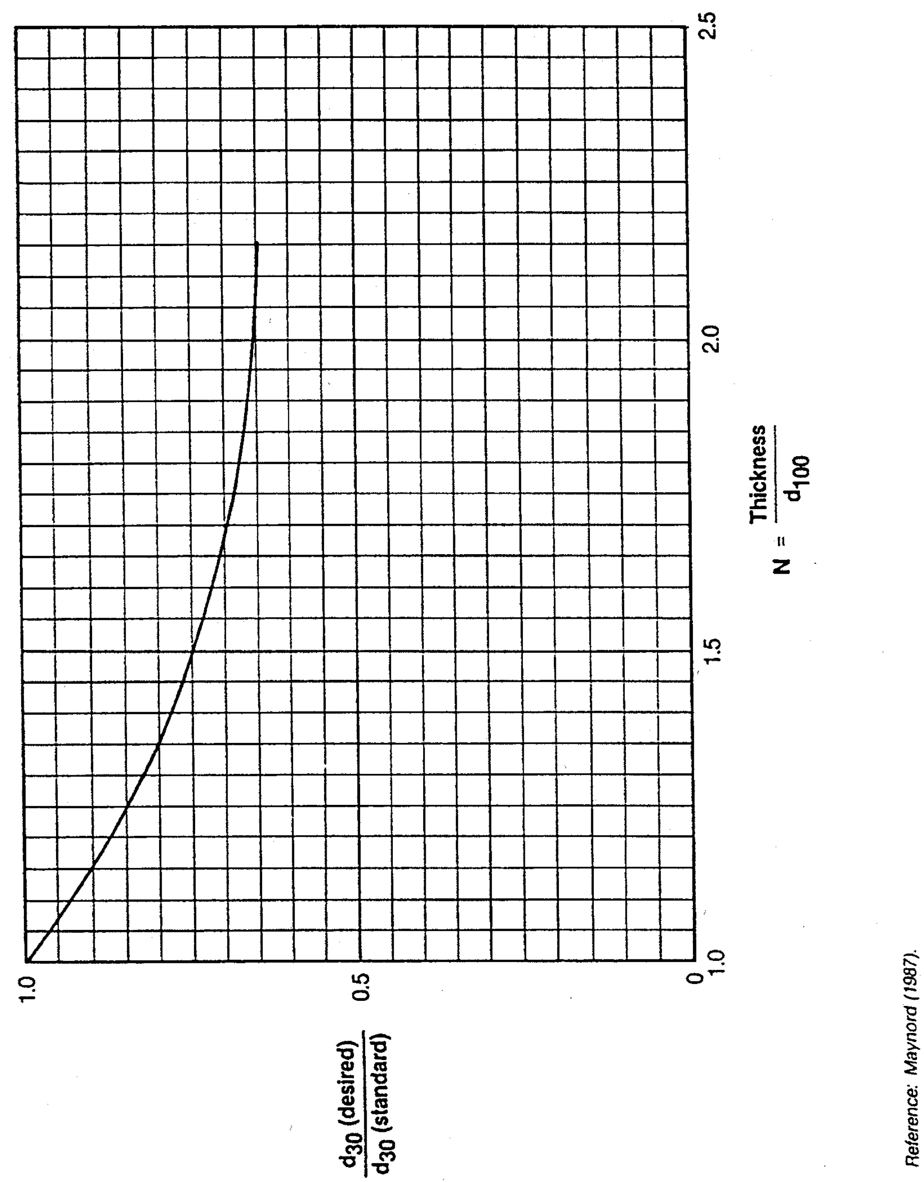


Figure 4.4-10 Riprap Lining Thickness Adjustment for d_{85}/d_{15} = 1.0 to 2.3

(Source: Maynord, 1987)

(3) Use equation 4.4.9 to calculate the value of R: R = 5.4/4 = 1.35 ft

(4) Use equation 4.4.10 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 (Figure 4.4-1)**	**R (eq. 4.4.9)**	**vR (eq. 4.4.10)**
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 4.4-3 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 4.4.1)
Fr = 0.76 (equation 4.4.8)
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 4.4-5):
A = (b + zd) d = [10 + (3) (1)] (1) = 13.0 square 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 4.4-1 for retardance Class C (n = 0.047)

(5) Use Figure 4.4-2 or equation 4.4.1 to find the velocity for S = 0.015, R = 0.796, and n = 0.047: <u>v = 3.34 ft/s</u>

(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^2)	R (ft)	Velocity Q/A (ft/sec)	vR	Manning's n (Fig. 4.4-3)	Velocity (Eq. 4.4.11)
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 4.4.8)
n (capacity) = 0.048, d = 1.1 ft, v = 3.45 ft/s, Froude number = 0.64 (equation 4.4.8)

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.

(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 4.4-7, 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 4.4-8 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 ft/s.

(4) Determine minimum d_{30} from Figure 4.4-9 or equation 4.4.13 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 4.4-10, 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 4.4-6.

4.4.10 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.

4.4.11 Rectangular, Triangular, and Trapezoidal Open Channel Design Figures

4.4.11.1 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.

4.4.11.2 Description of Figures

Figures given in subsections 4.4.12, 4.4.13 and 4.4.14 at the end of this section 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 (subsection 4.4.12) 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). 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 (subsection 4.4.13) 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.

4.4.11.3 Instructions for Rectangular and Trapezoidal Figures

Figures in subsection 4.4.12 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

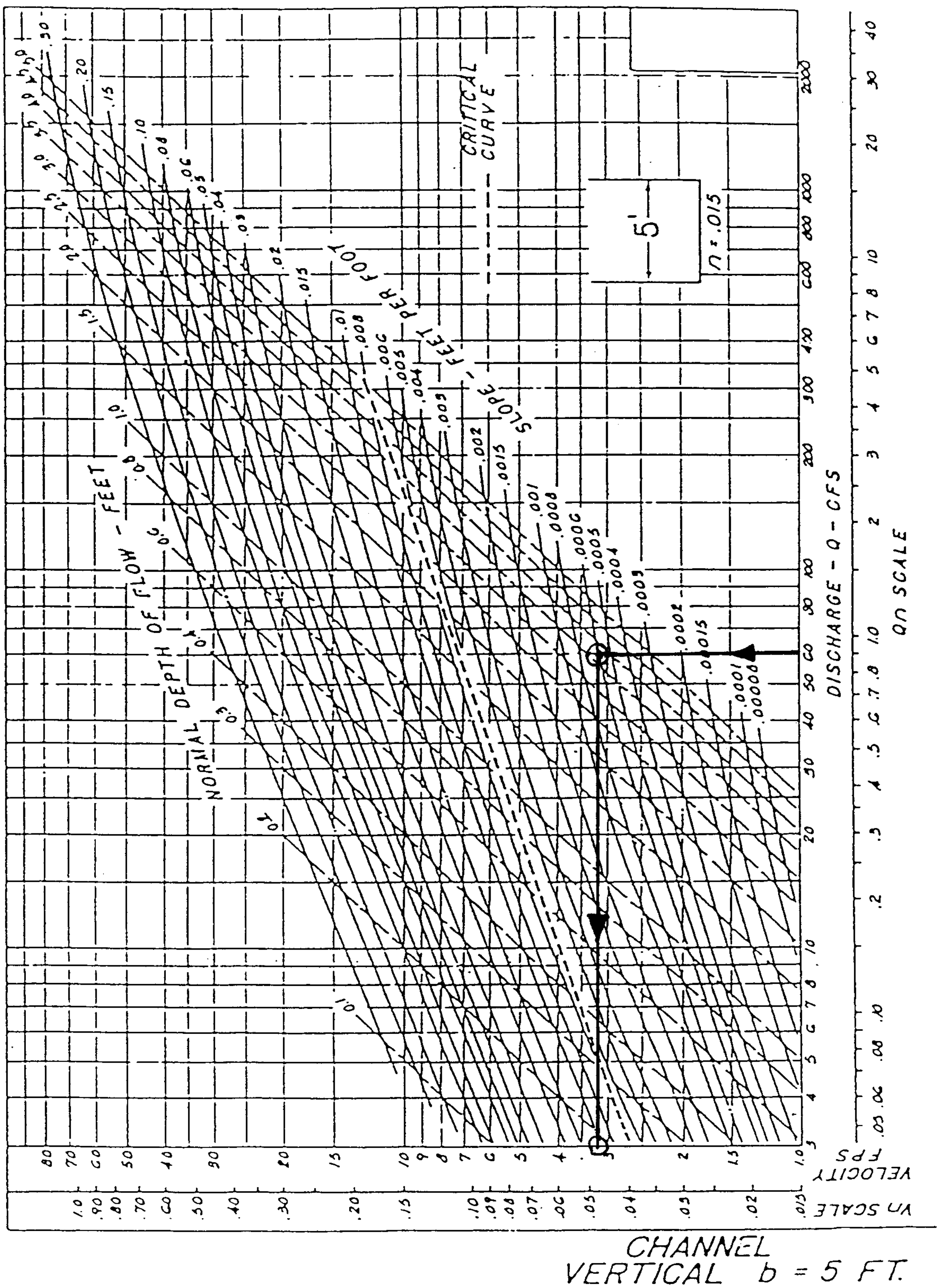
Procedure:

(1) From subsection 4.4.12 select the rectangular figure for a 5-ft width (Figure 4.4-11).

(2) From 60 cfs on the Q scale, move vertically to intersect the slope line S = .0006, and from the depth lines read d_n = 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.



Source: Federal Highway Administration

Figure 4.4-11 Example Nomograph #1

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 4.4-12).

(2) From 50 cfs on the Q scale, move vertically to intersect the slope line S = 0.002 and from the depth lines read d_n = 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 4.4-13).

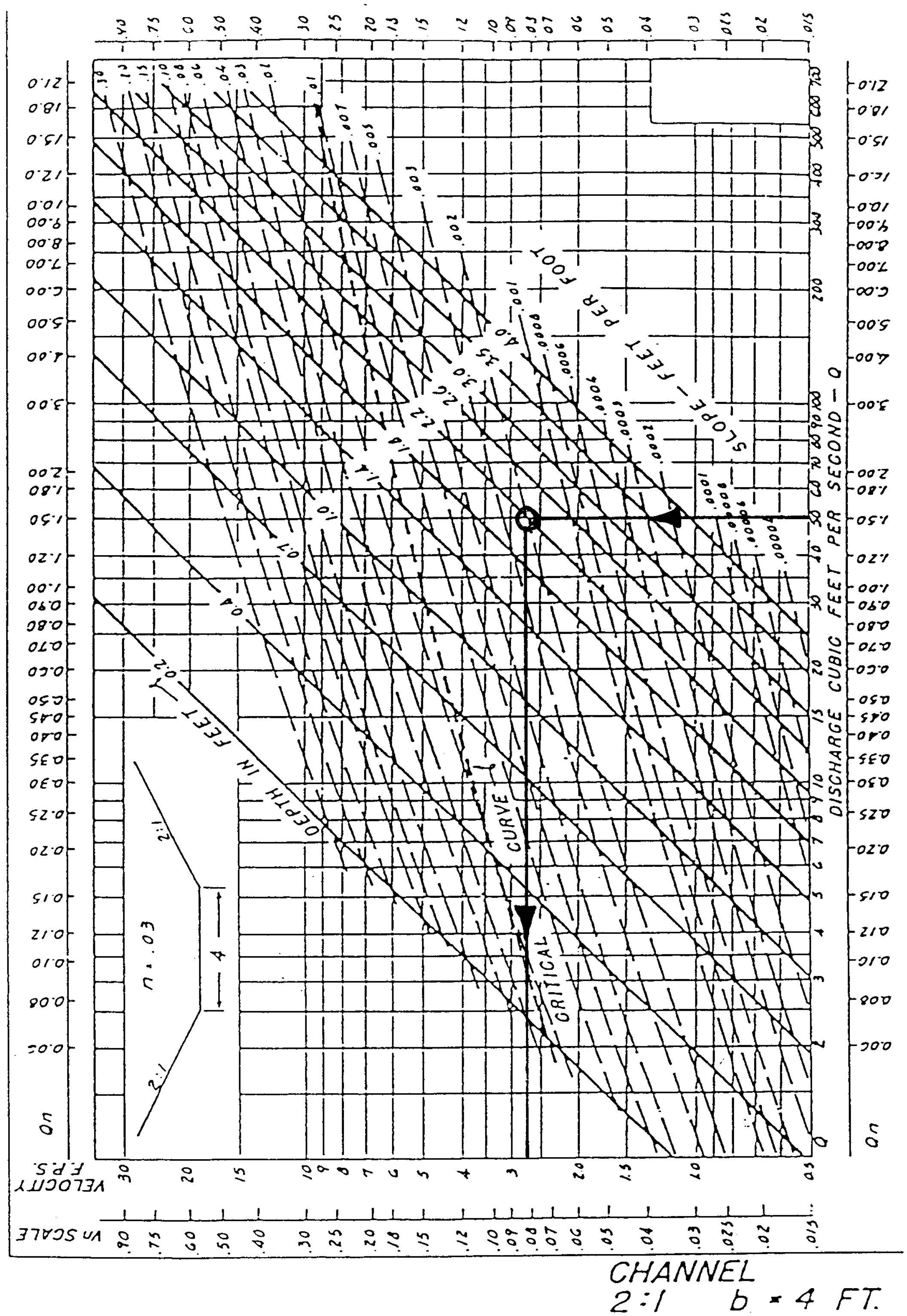
(2) Multiply Q by n to obtain Qn: 80 x 0.025 = 2.0.

(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.

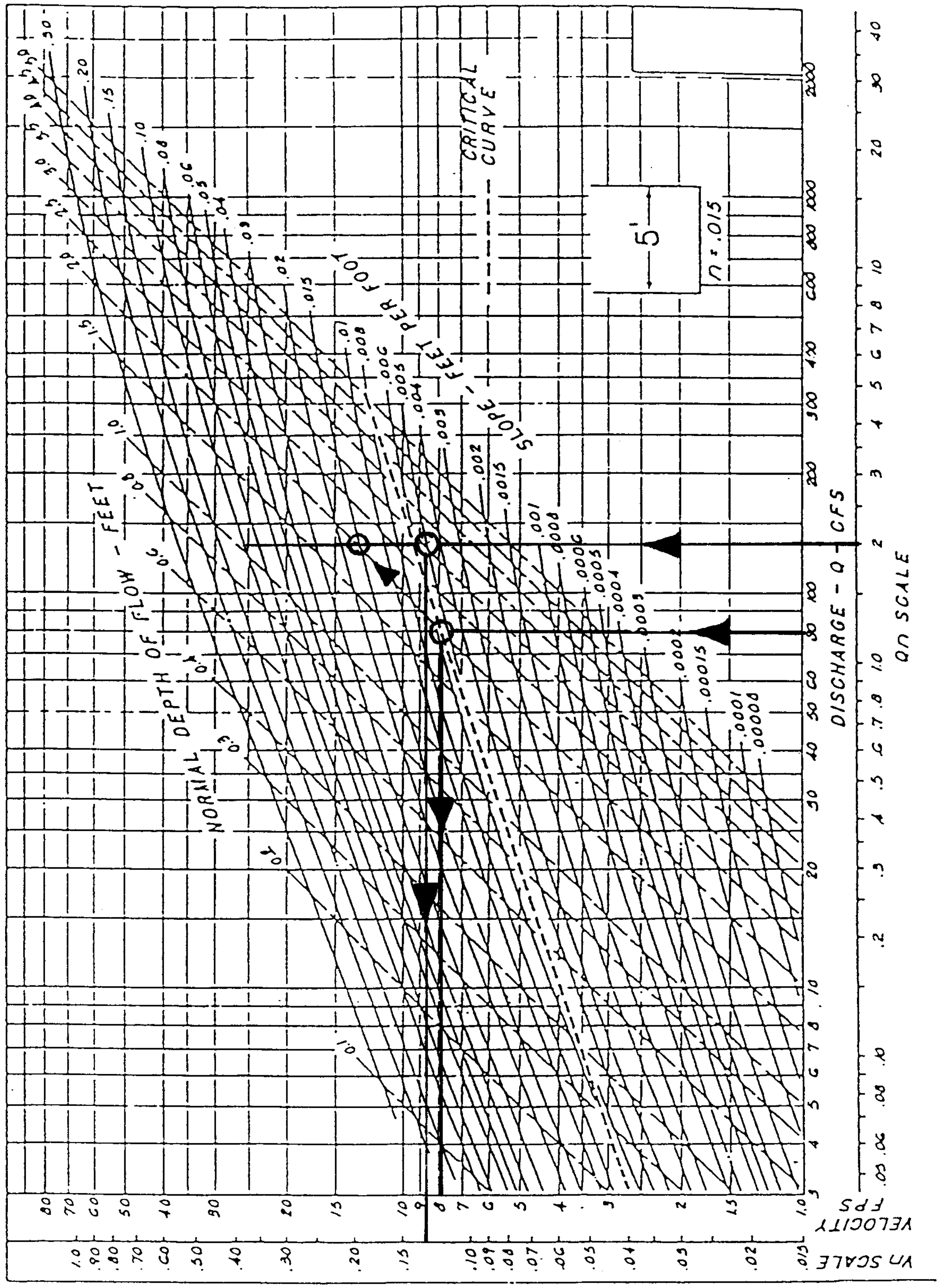
(5) Critical depth and critical velocity are independent of the value of n so their values can be read at the intersection of the critical curve with a vertical line through the discharge. For 80 cfs, on Figure 4.4-13, d_c = 2.0 ft and V_c = 7.9 ft/s. The normal velocity, 5.2 ft/s (from step 4), is less 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.



Source: Federal Highway Administration

Figure 4.4-12 Example Nomograph #2



CHANNEL
VERTICAL b = 5 FT.

Source: Federal Highway Administration

Figure 4.4-13 Example Nomograph #3

4.4.11.4 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 subsection 4.4.14. 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.

4.4.11.5 Description of Figures

The figures in subsection 4.4.14 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 4.4-6) and the product V times R is shown in Figure 4.4-1. As indicated in Table 4.4-6, 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 4.4-6 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.

4.4.11.6 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 and (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 4.4-2 and 4.4-3. 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 4.4-2 and 4.4-3), 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 subsection 4.4.13 select figure for 4:1 side slopes (see Figure 4.4-14).

(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 4.4-3.

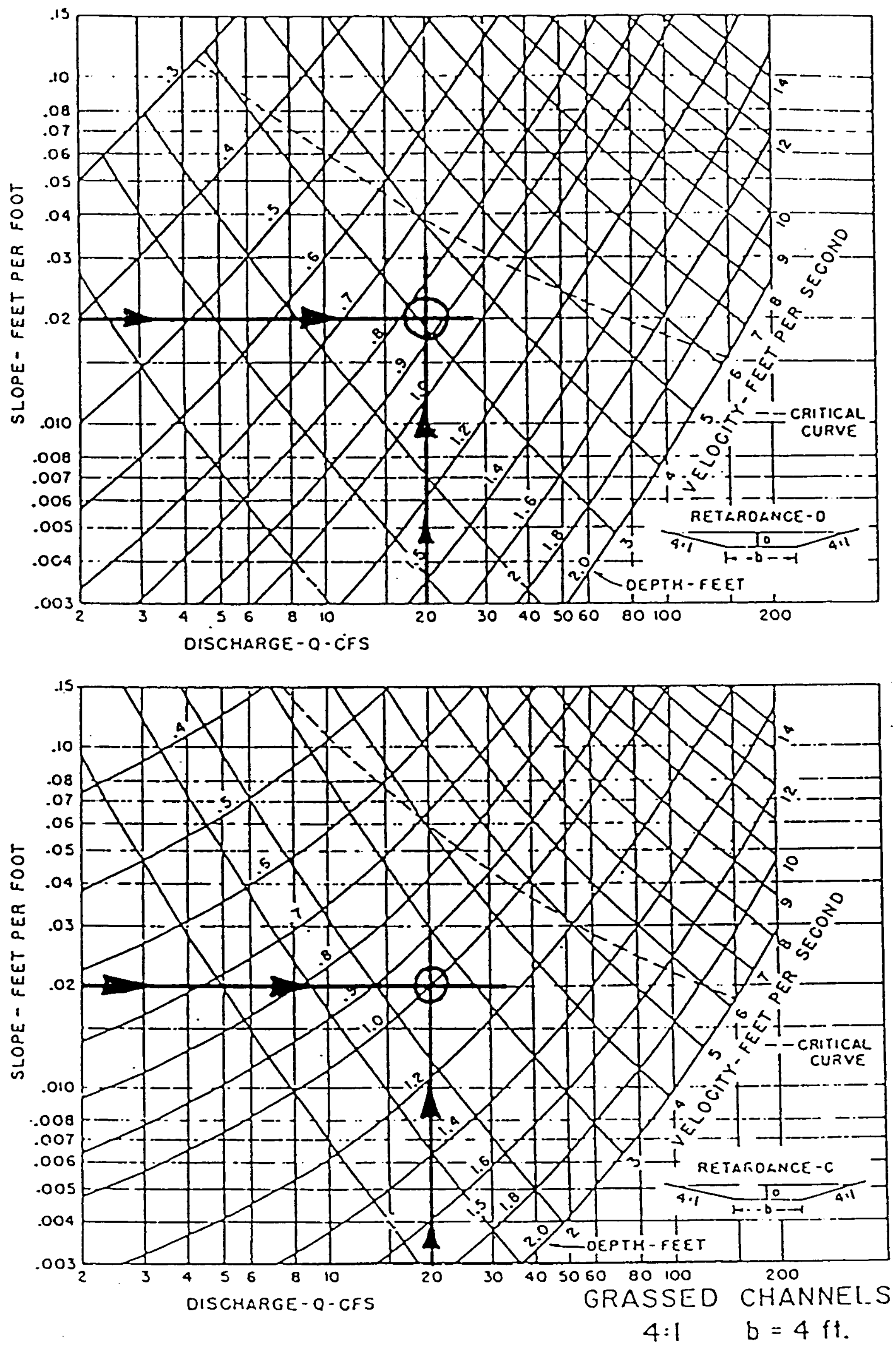
Example Design Problem 2

Given: The channel and discharge of Example 1.

Find: The maximum grade on which the 20 cfs could safely be carried

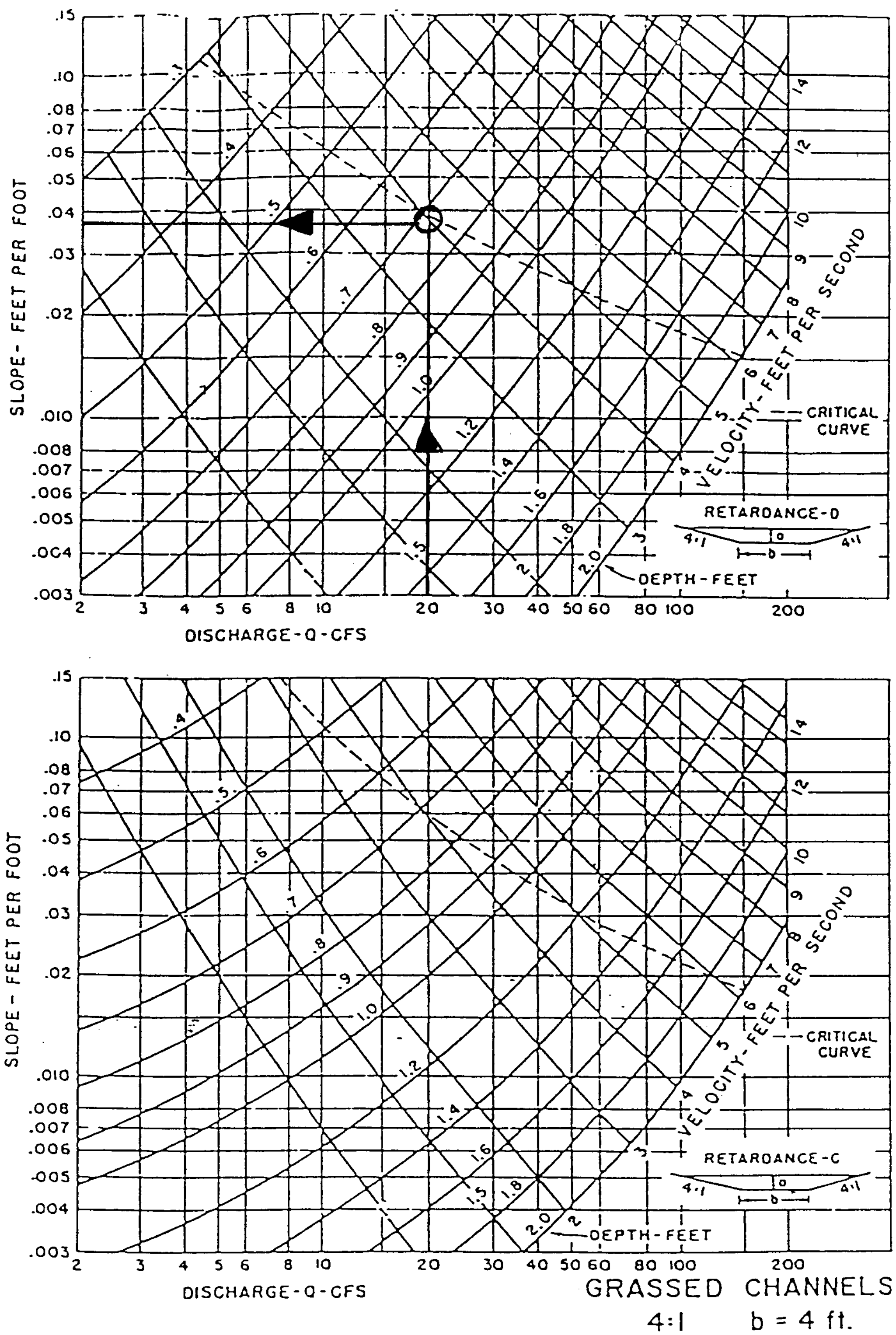
Procedure:

With an increase in slope (but still less than 5%), the allowable velocity is estimated to be 4 ft/s (see Table 4.4-3). On the upper graph of Figure 4.4-15 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.



Source: Federal Highway Administration

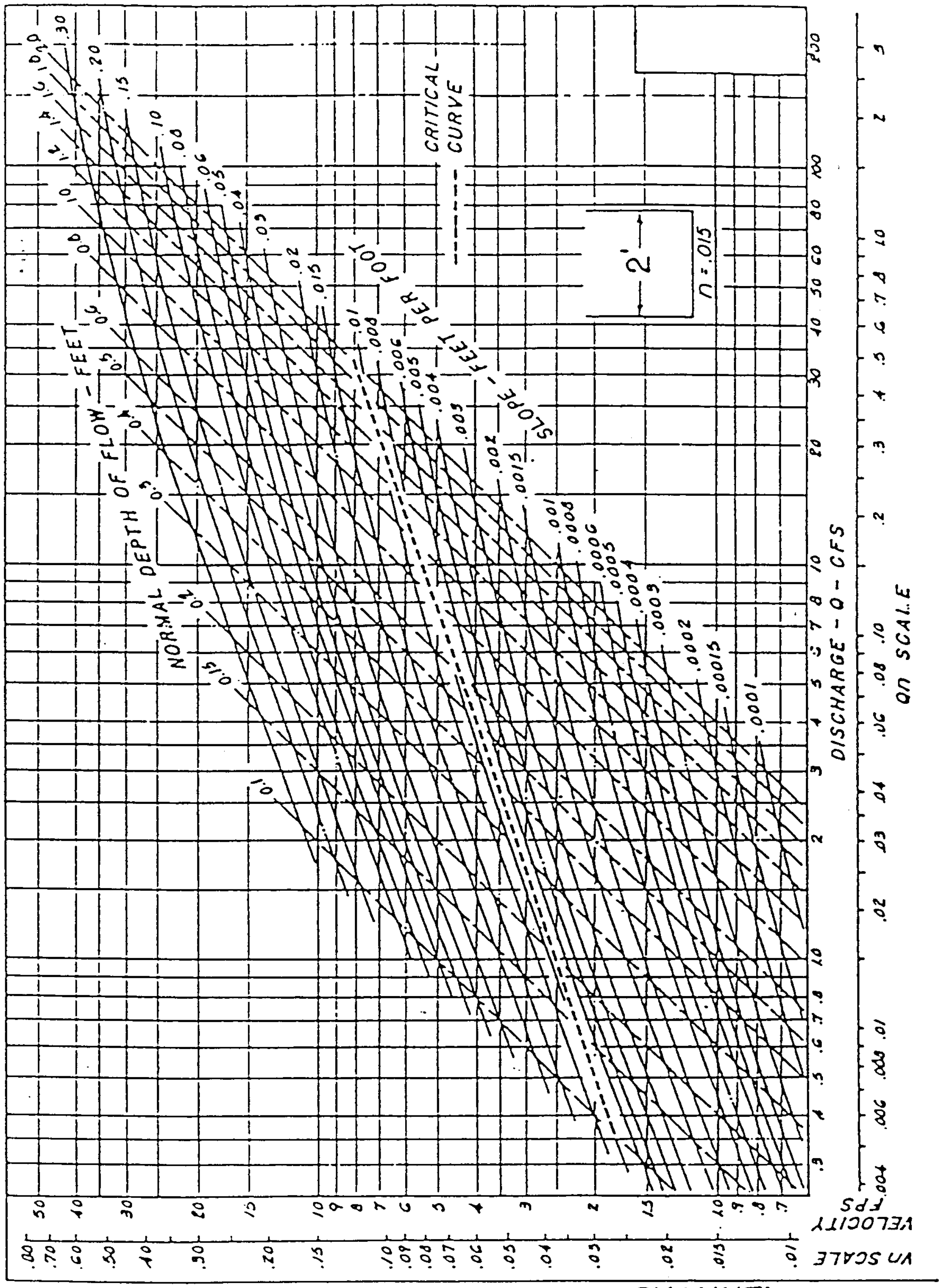
Figure 4.4-14 Example Nomograph #4



Source: Federal Highway Administration

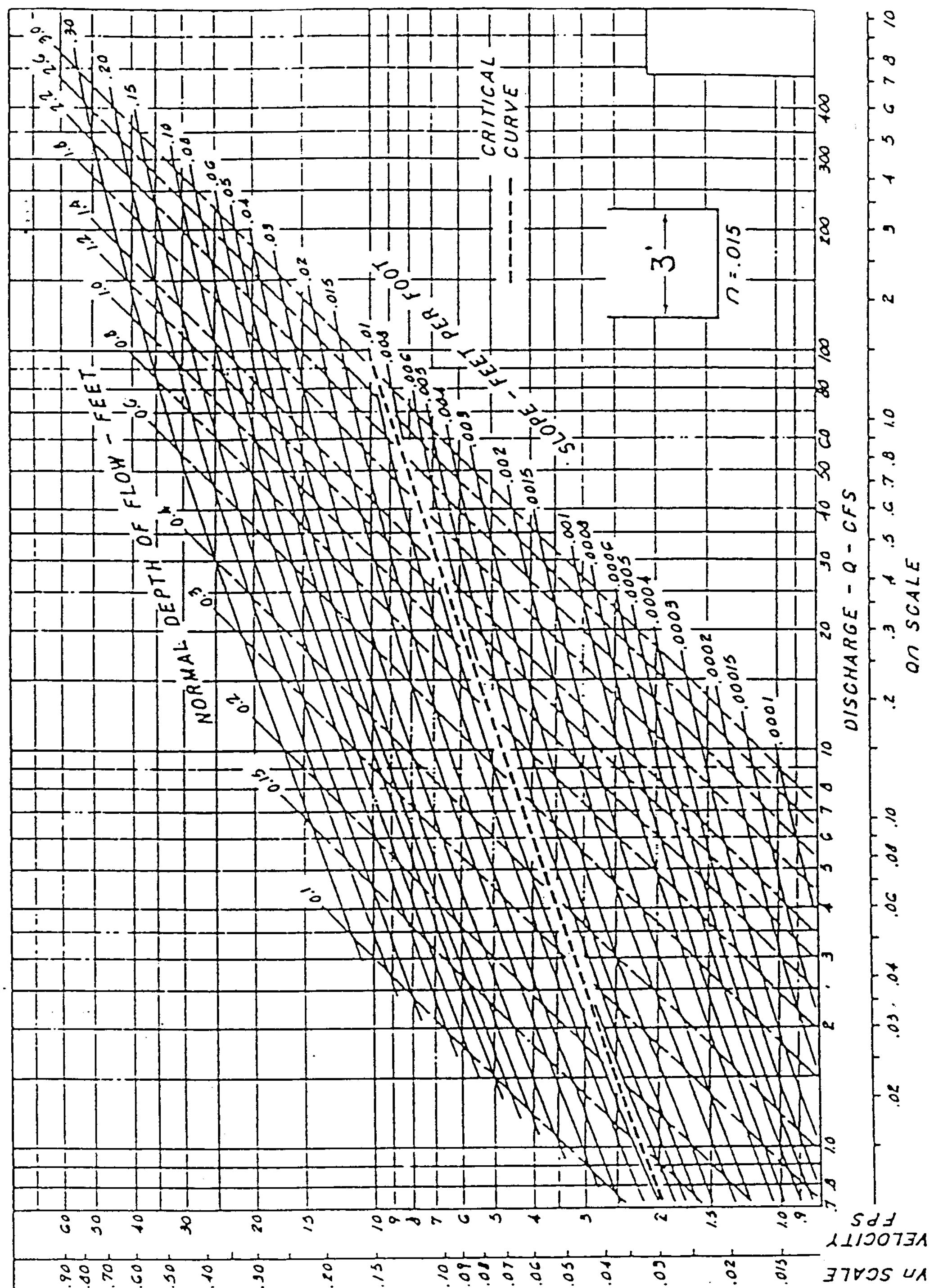
Figure 4.4-15 Example Nomograph #5

4.4.12 Open Channel Design Figures



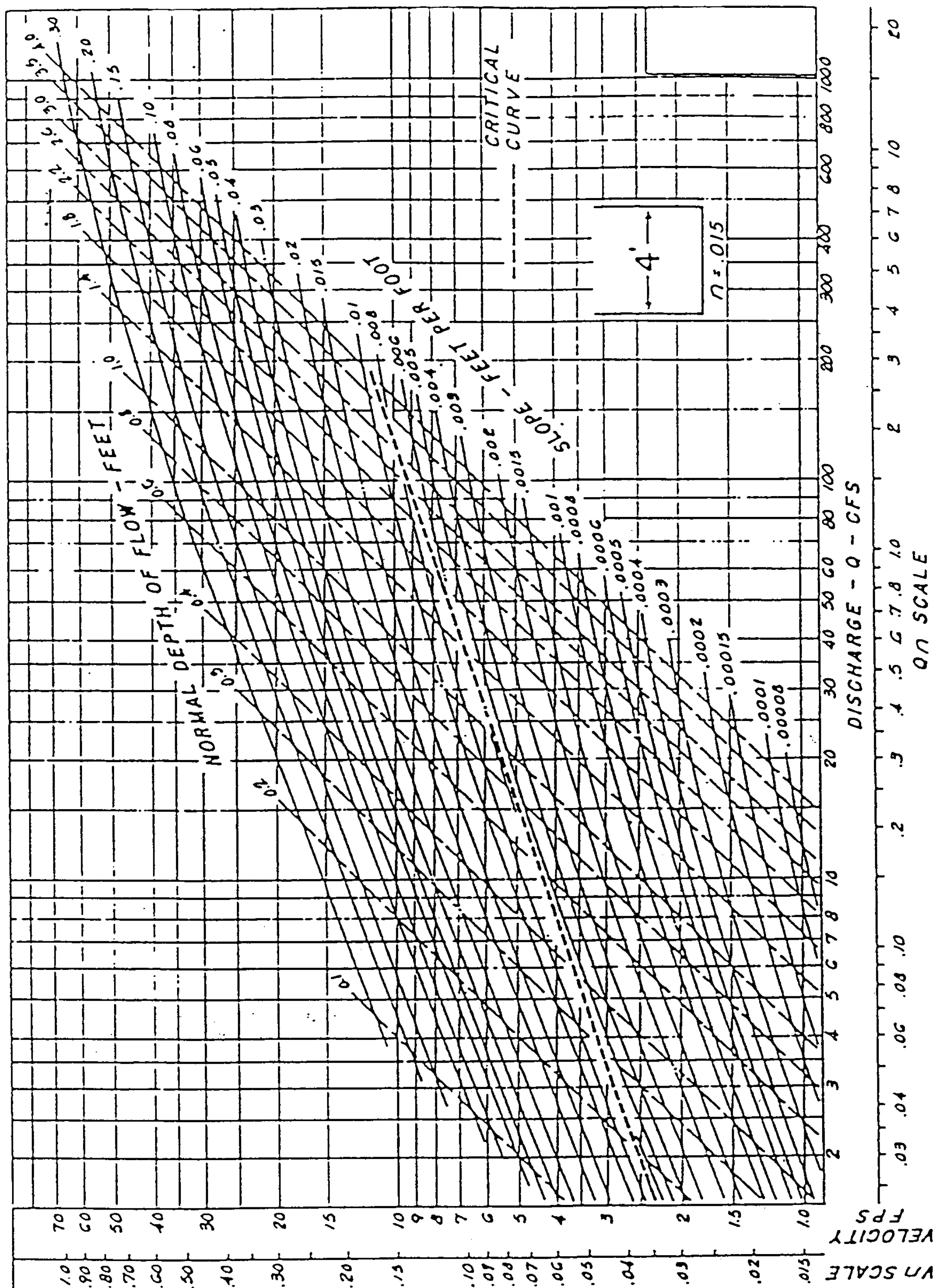
CHANNEL
VERTICAL b = 2 FT.

Source: Federal Highway Administration



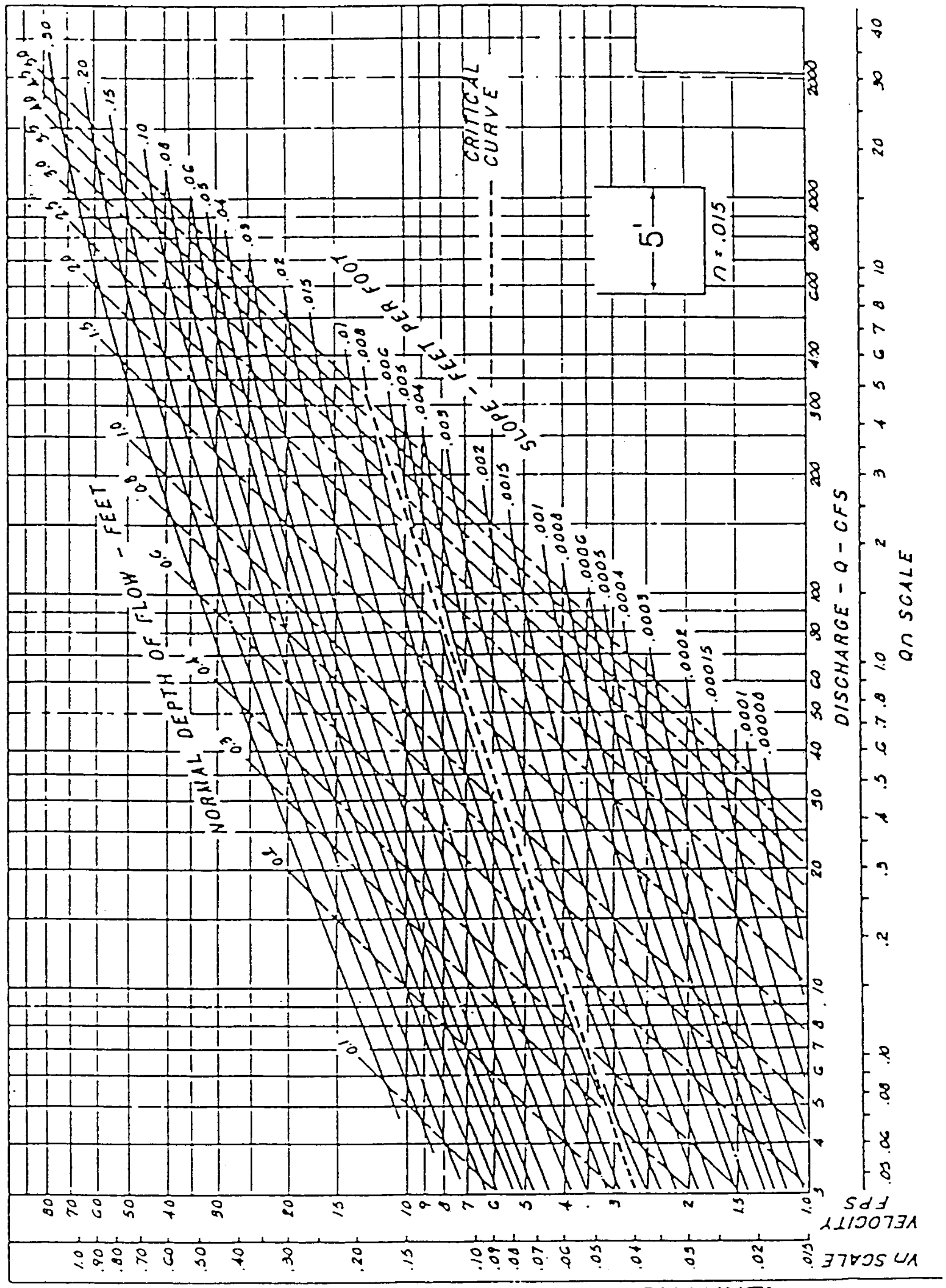
CHANNEL
VERTICAL b = 3 FT.

Source: Federal Highway Administration



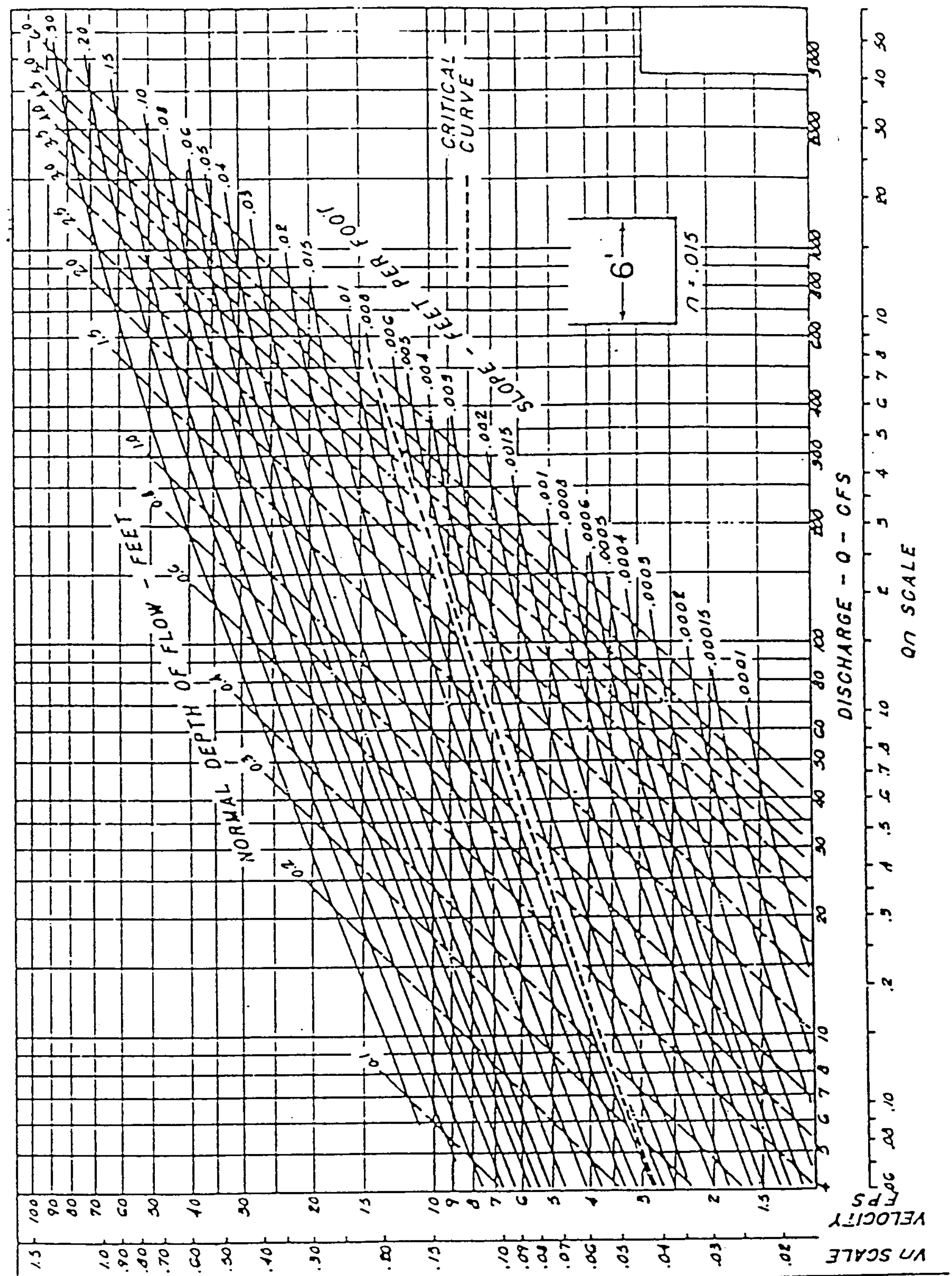
CHANNEL
VERTICAL b = 4 FT.

Source: Federal Highway Administration



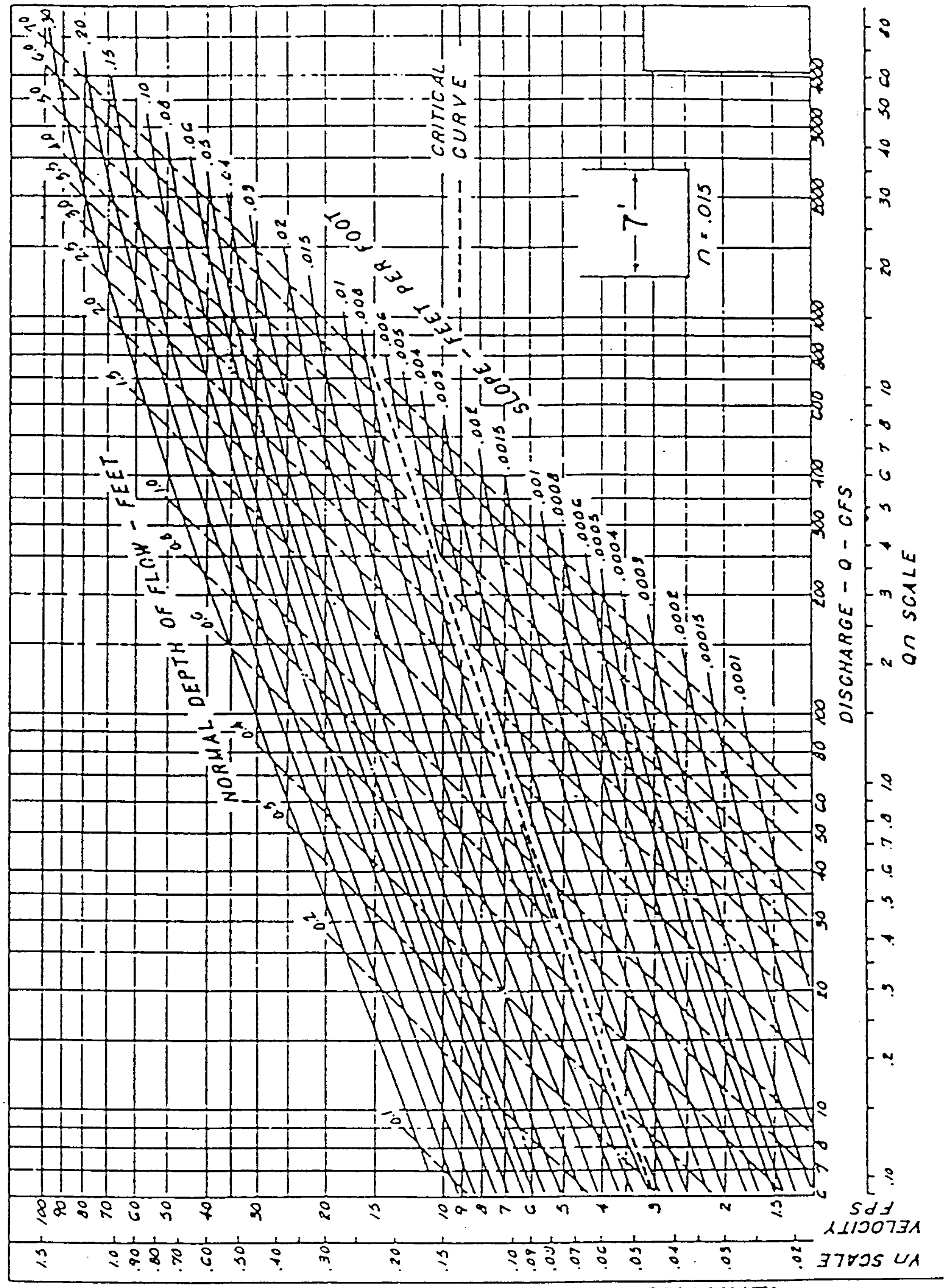
CHANNEL
VERTICAL b = 5 FT.

Source: Federal Highway Administration



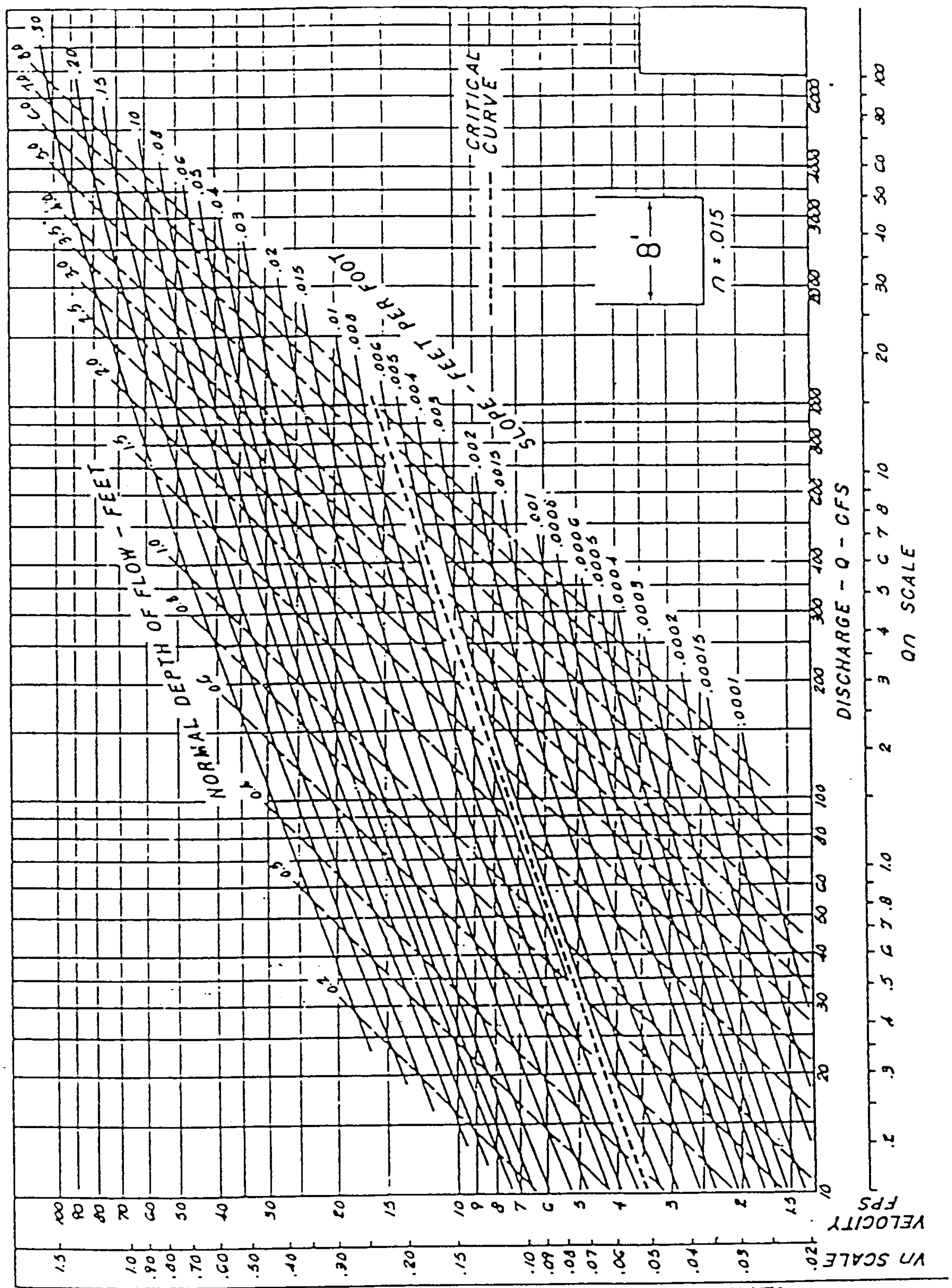
CHANNEL
VERTICAL b = 6 FT.

Source: Federal Highway Administration



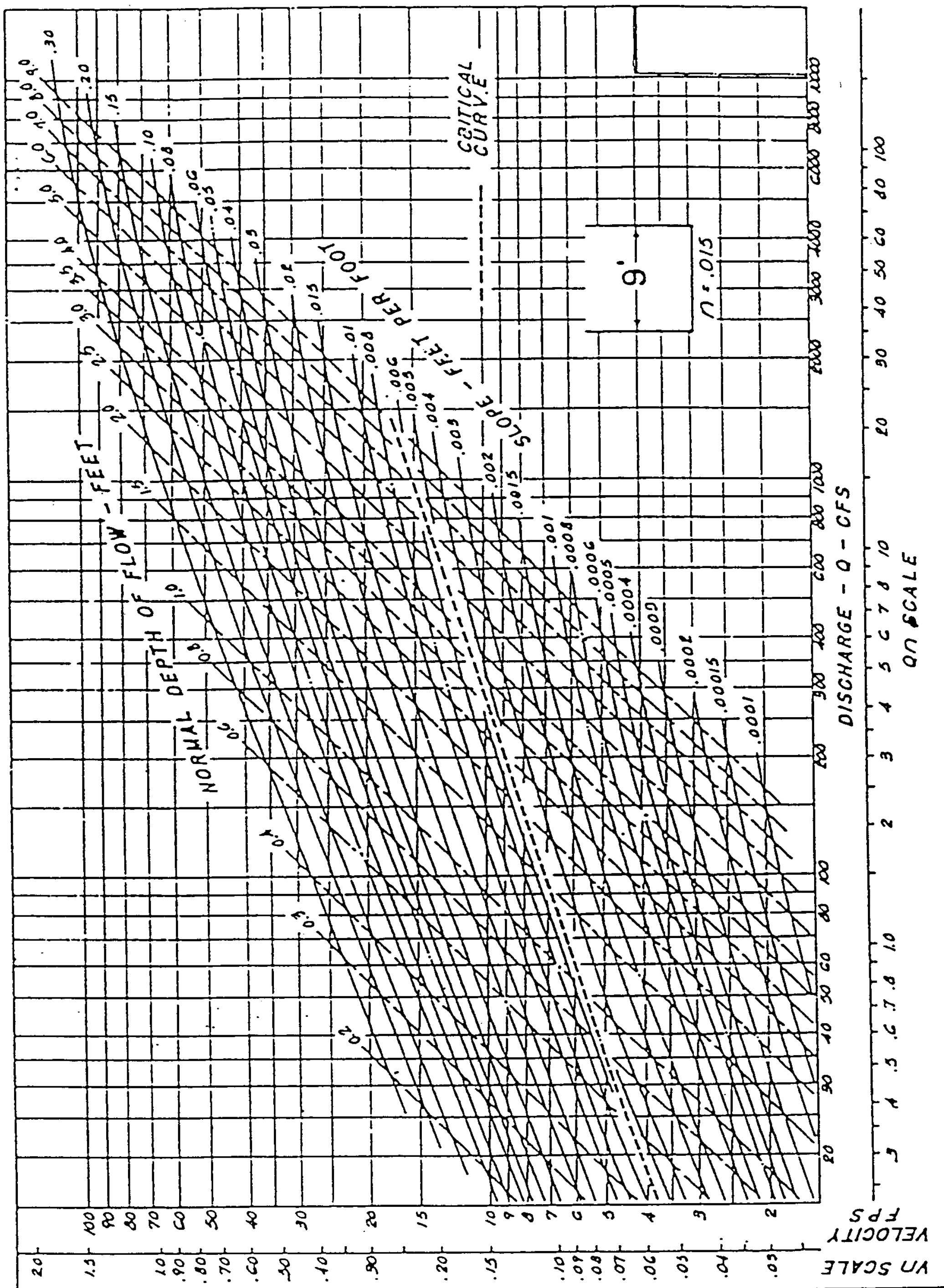
CHANNEL
VERTICAL b = 7 FT.

Source: Federal Highway Administration



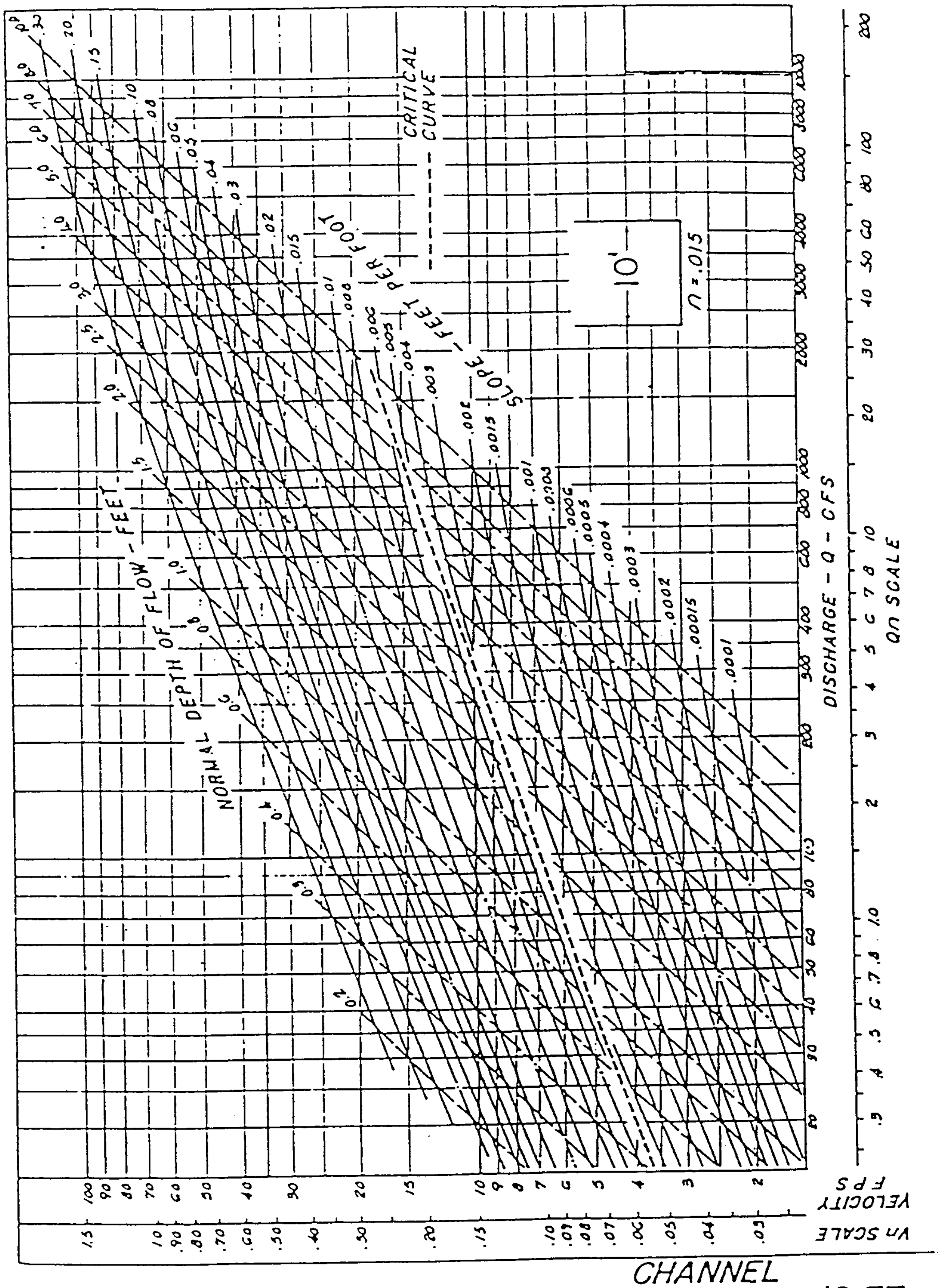
CHANNEL
VERTICAL b = 8 FT.

Source: Federal Highway Administration



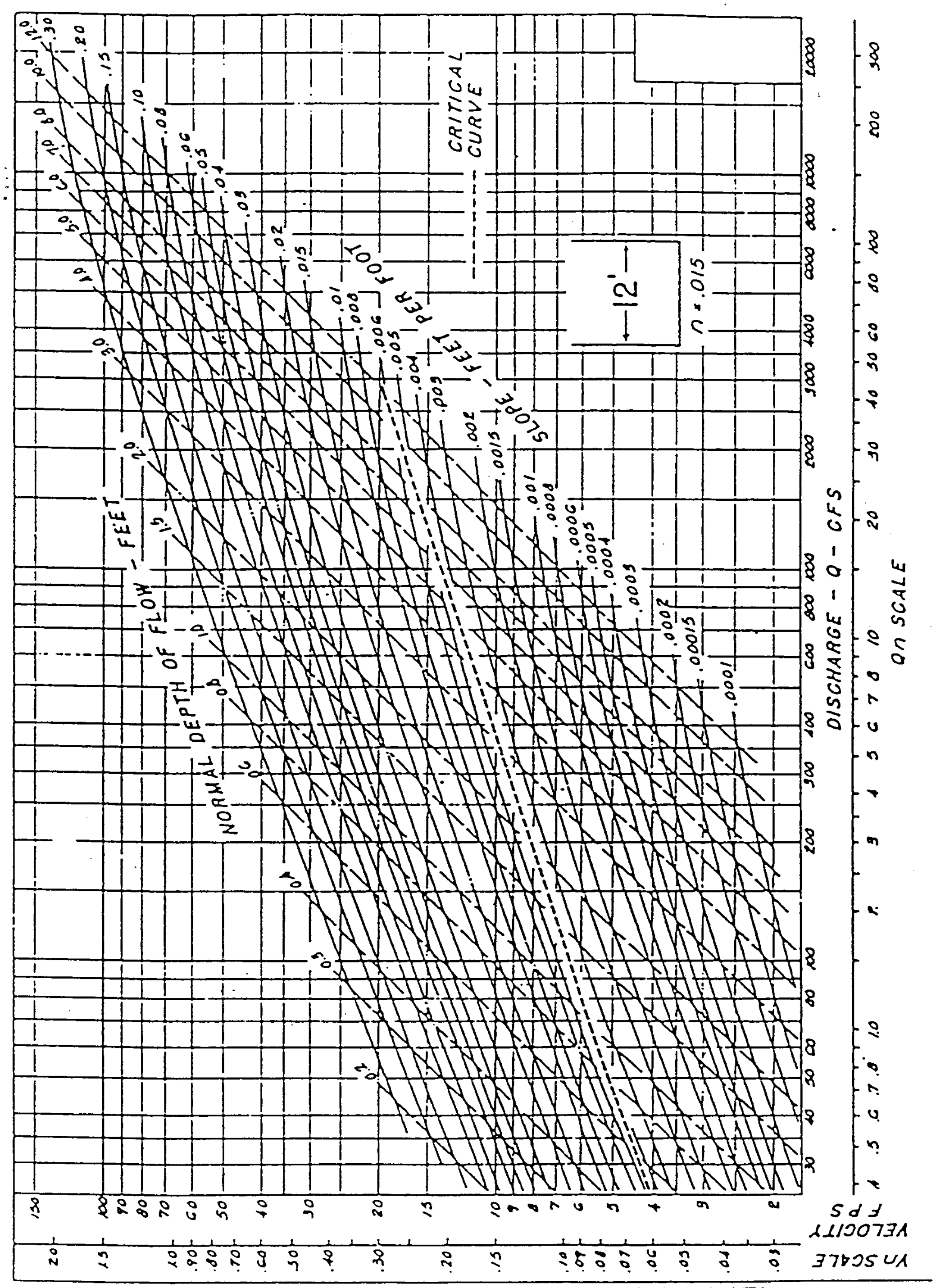
CHANNEL
VERTICAL b = 9 FT.

Source: Federal Highway Administration



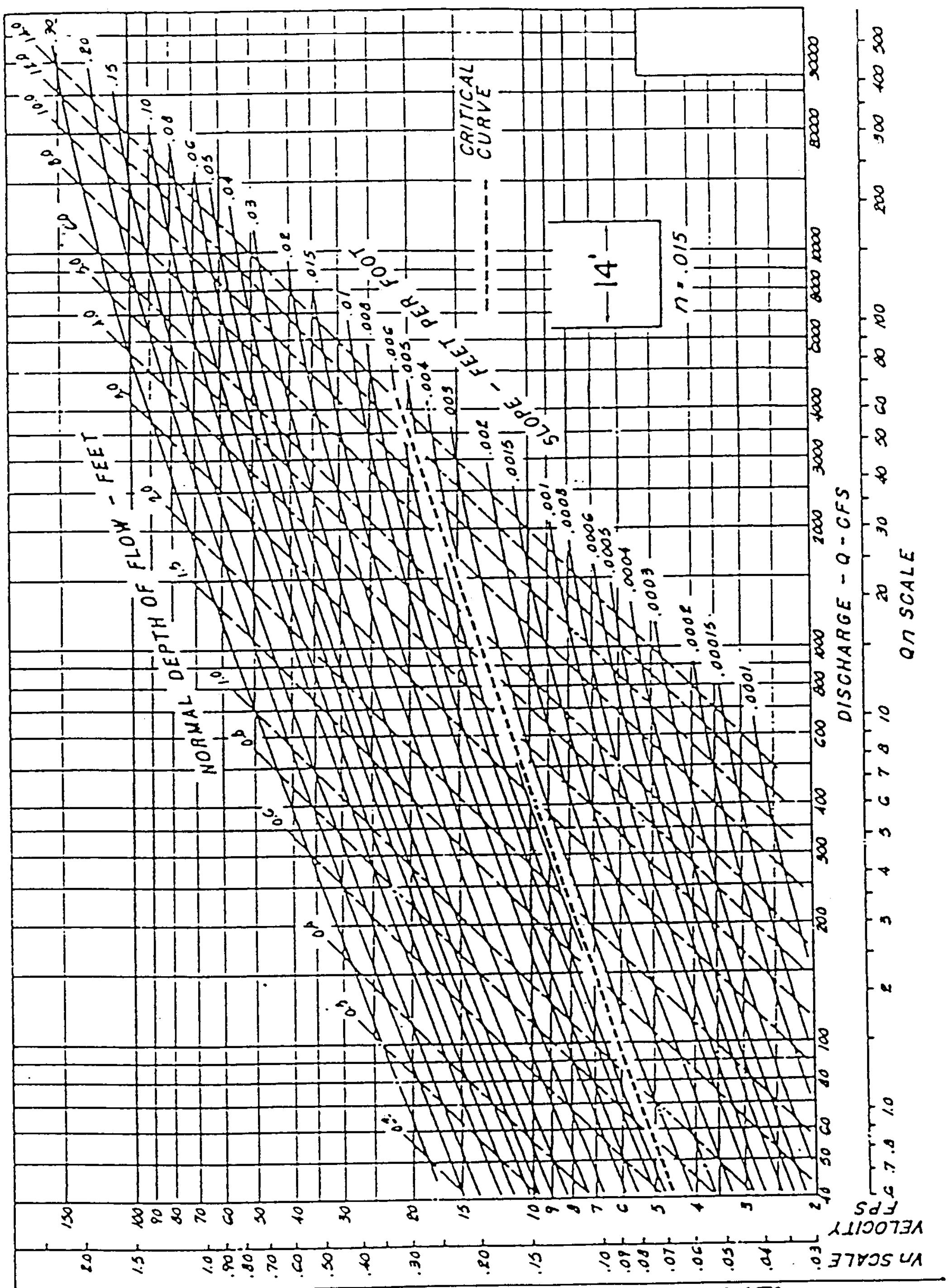
CHANNEL
VERTICAL b = 10 FT.

Source: Federal Highway Administration



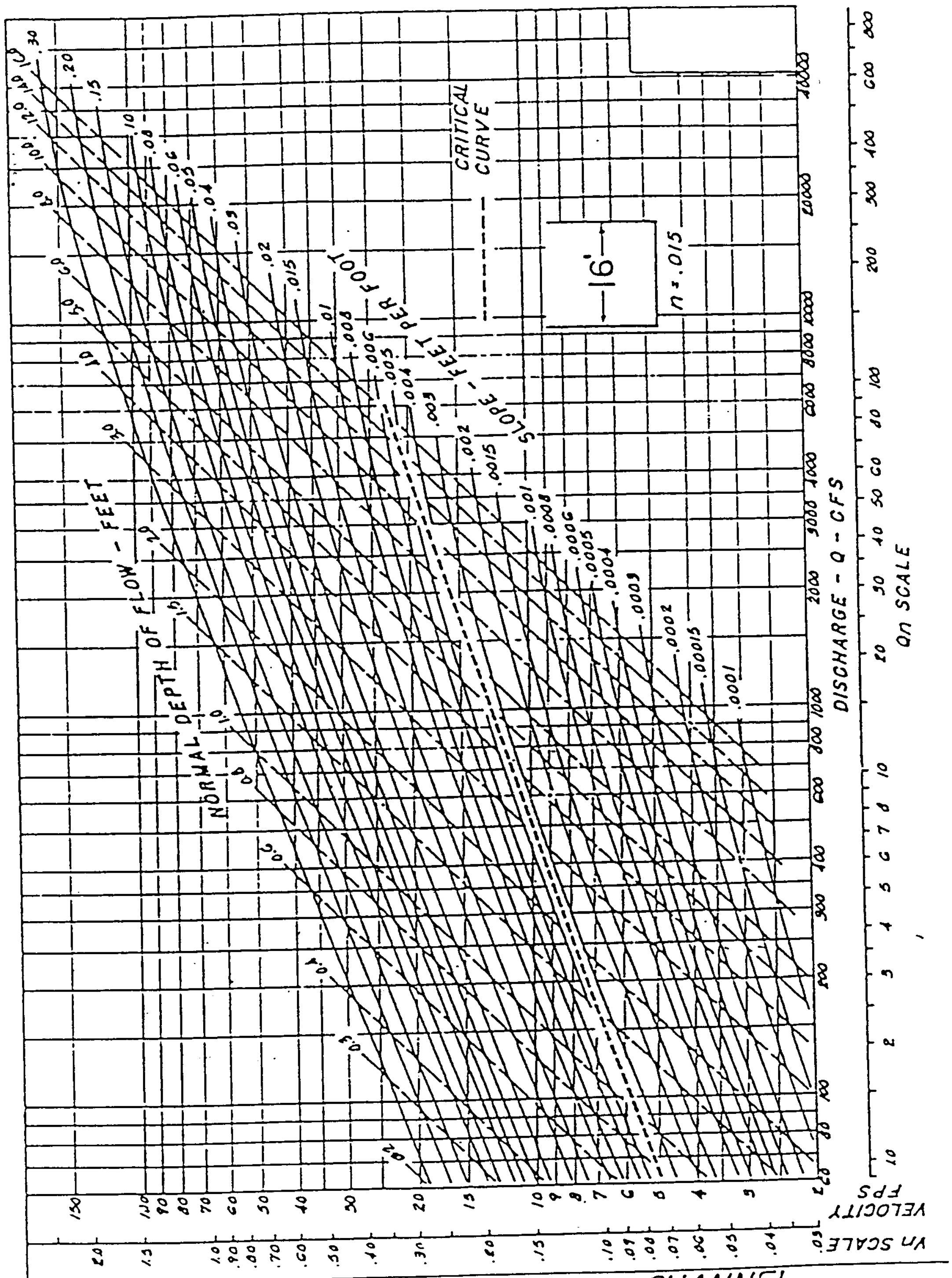
CHANNEL
VERTICAL b = 12 FT.

Source: Federal Highway Administration



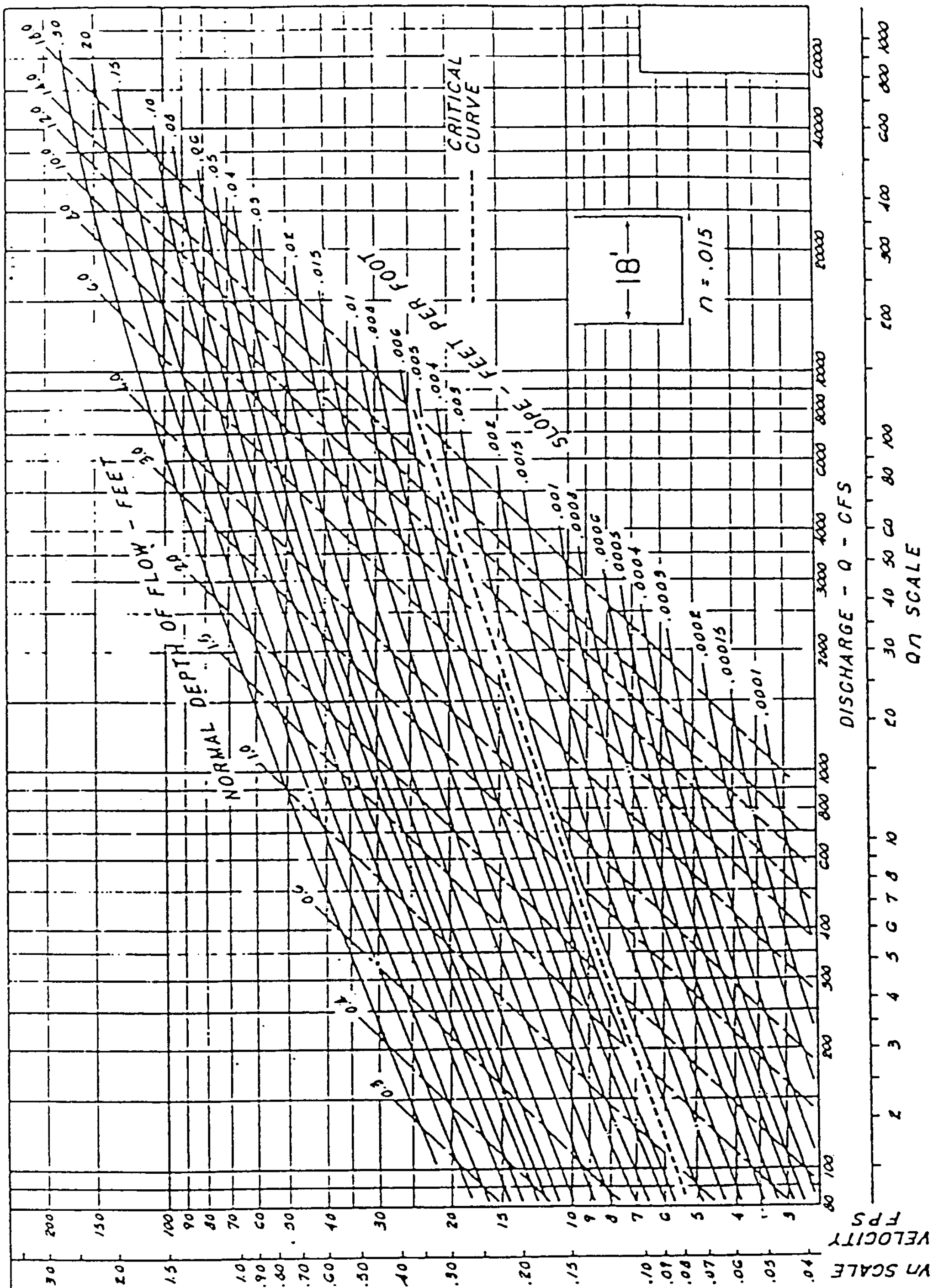
CHANNEL
VERTICAL b = 14 FT.

Source: Federal Highway Administration.



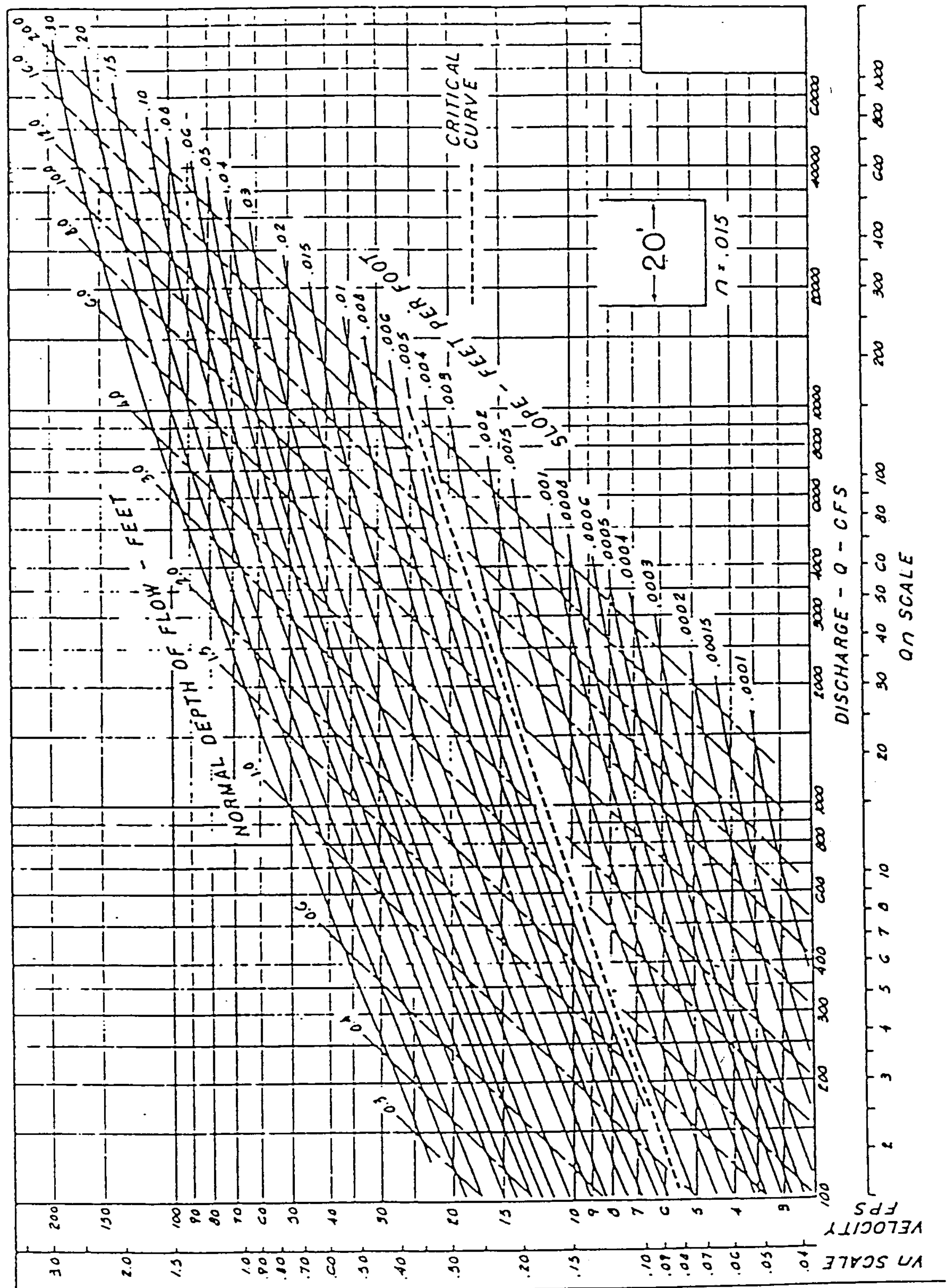
CHANNEL
VERTICAL b = 16 FT.

Source: Federal Highway Administration



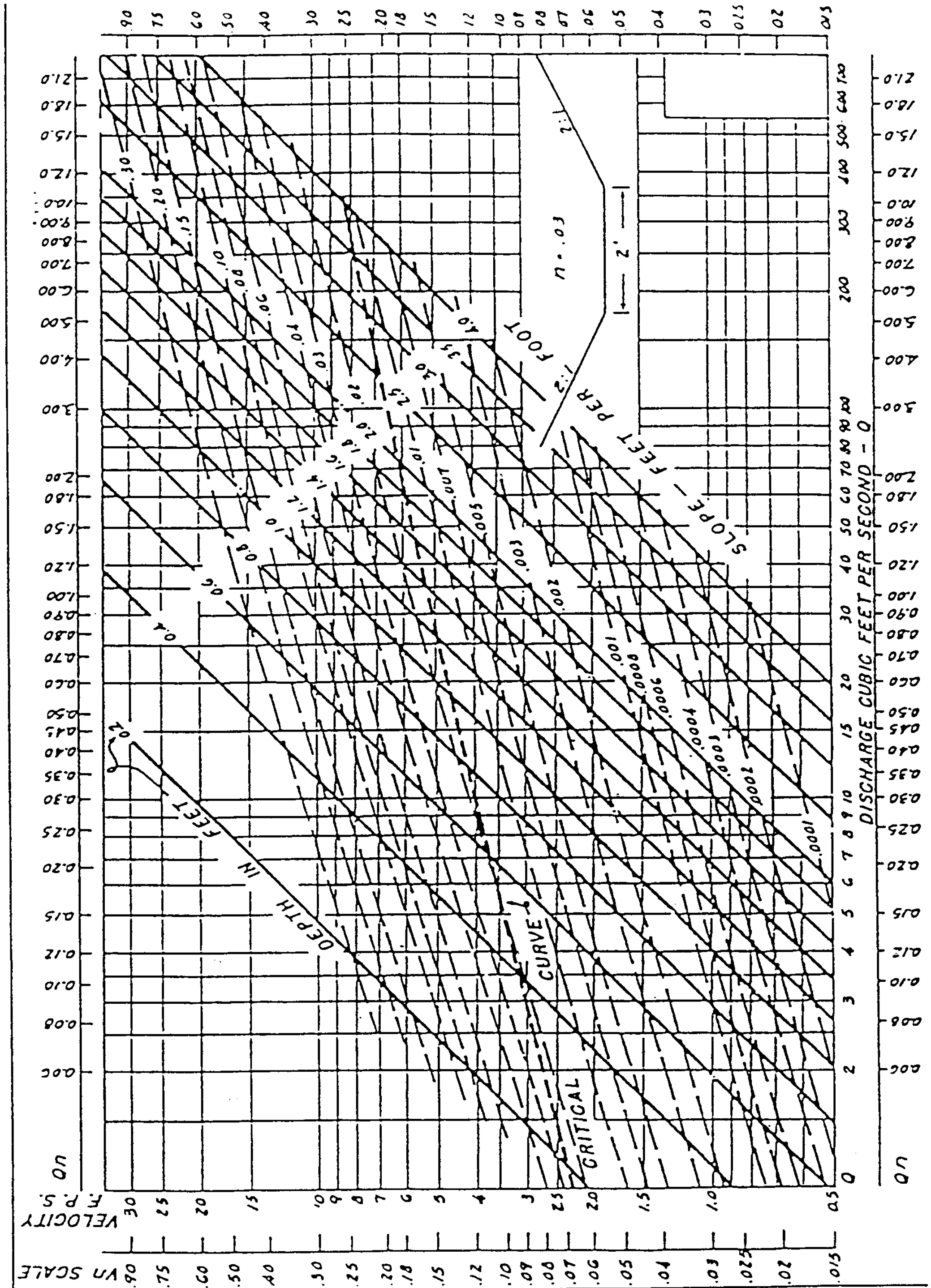
CHANNEL
VERTICAL b = 18 FT.

Source: Federal Highway Administration



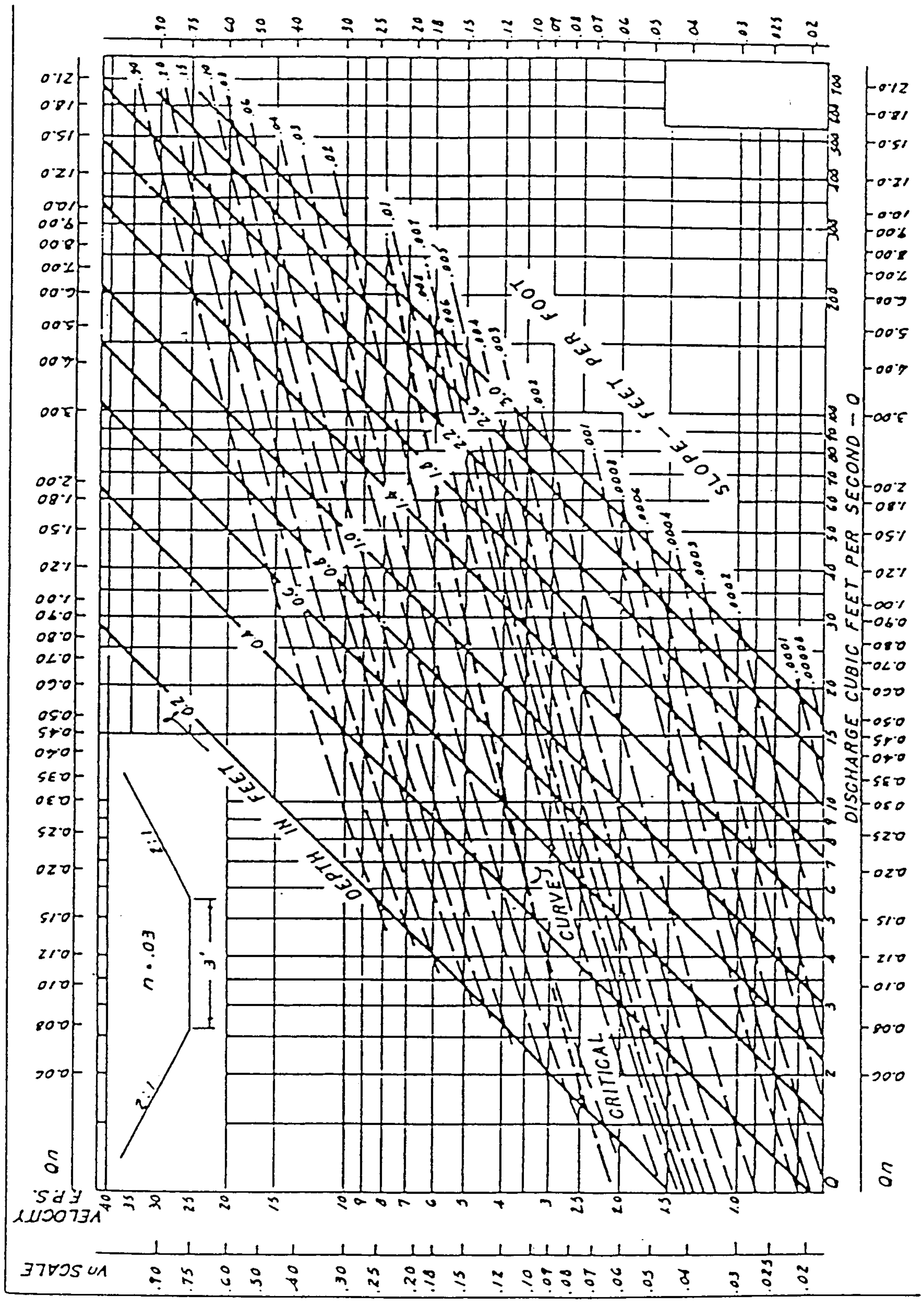
CHANNEL
VERTICAL b = 20 FT.

Source: Federal Highway Administration



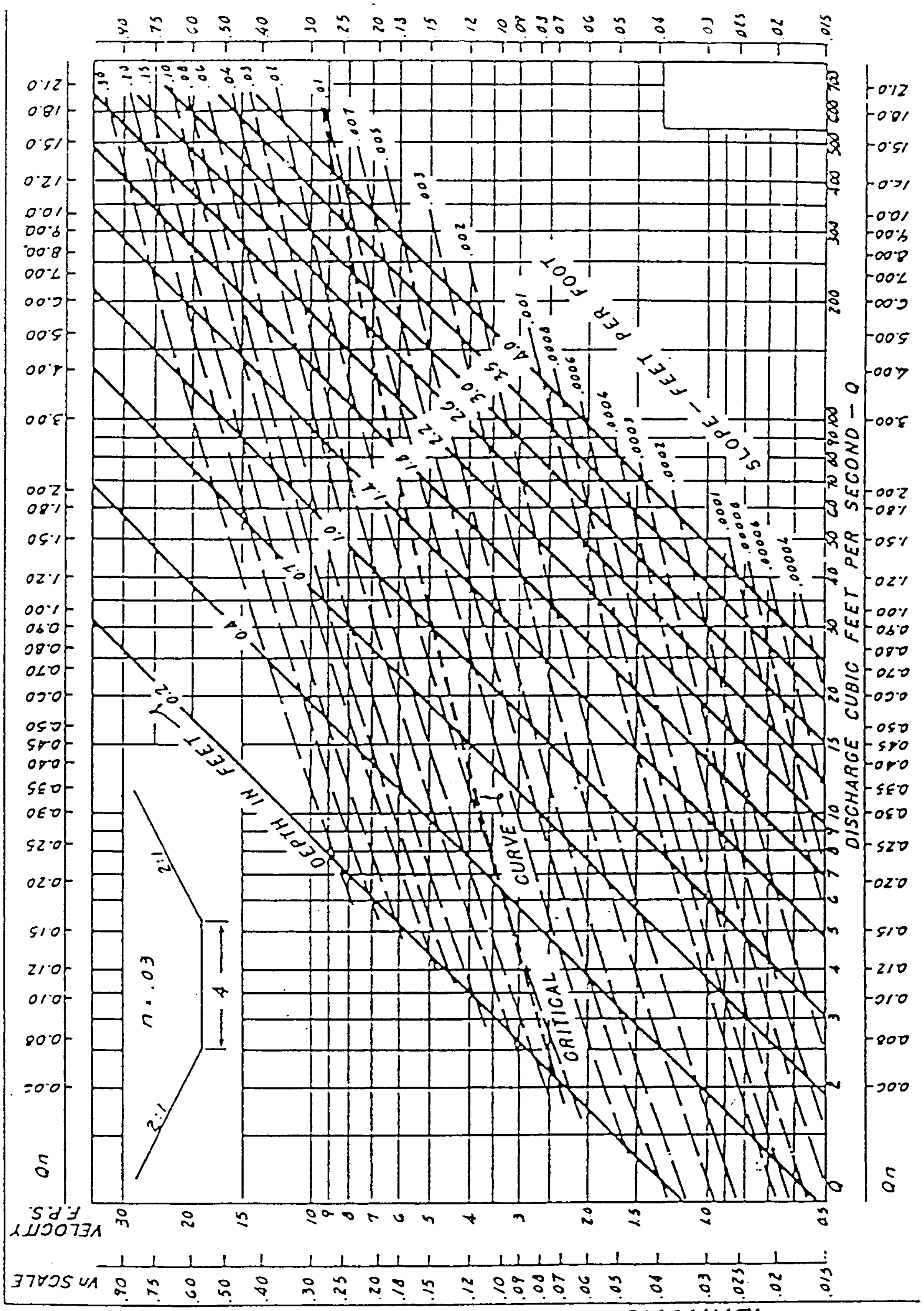
CHANNEL
2:1 b = 2 FT.

Source: Federal Highway Administration



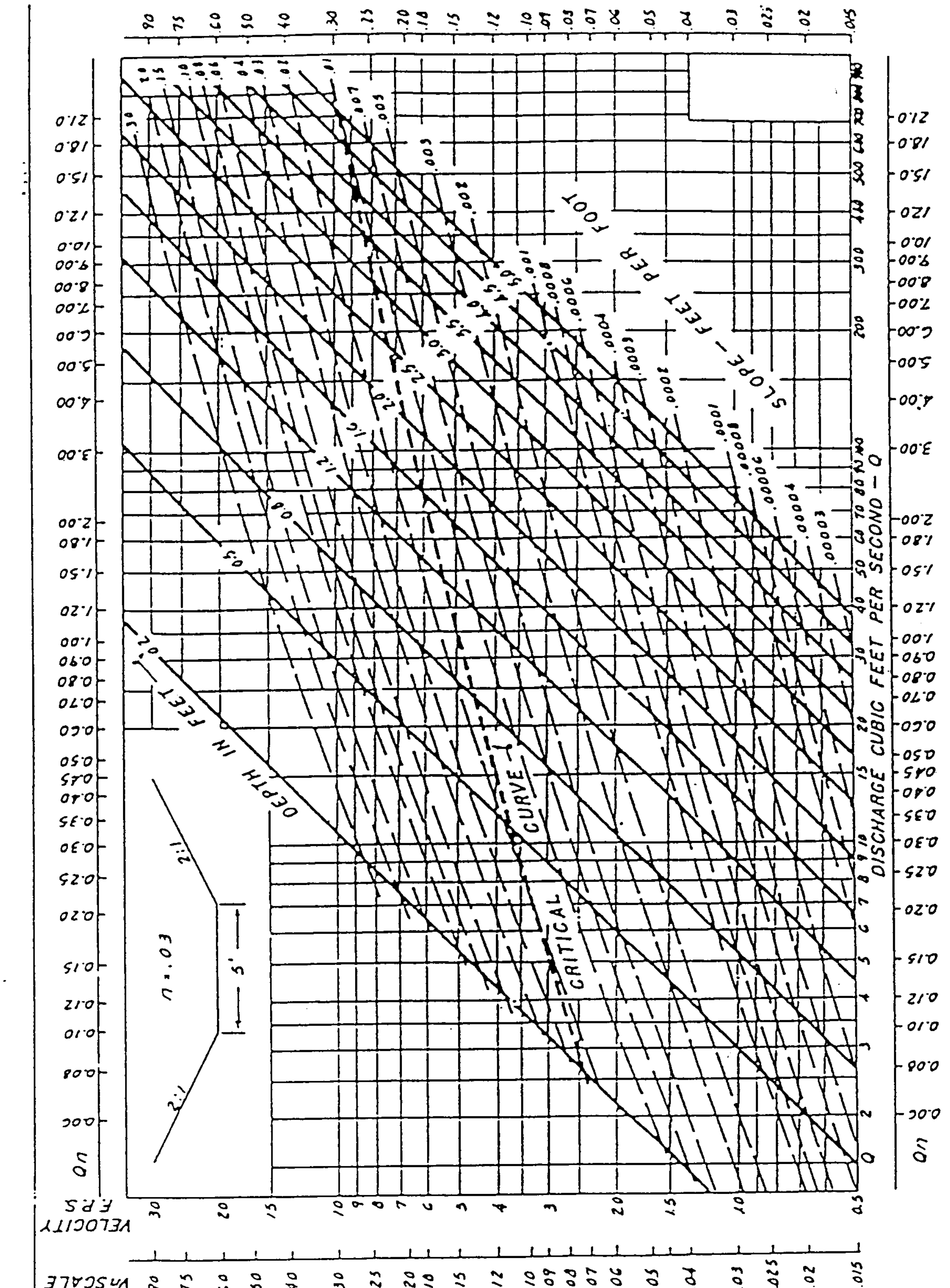
CHANNEL
2:1 b = 3 FT.

Source: Federal Highway Administration



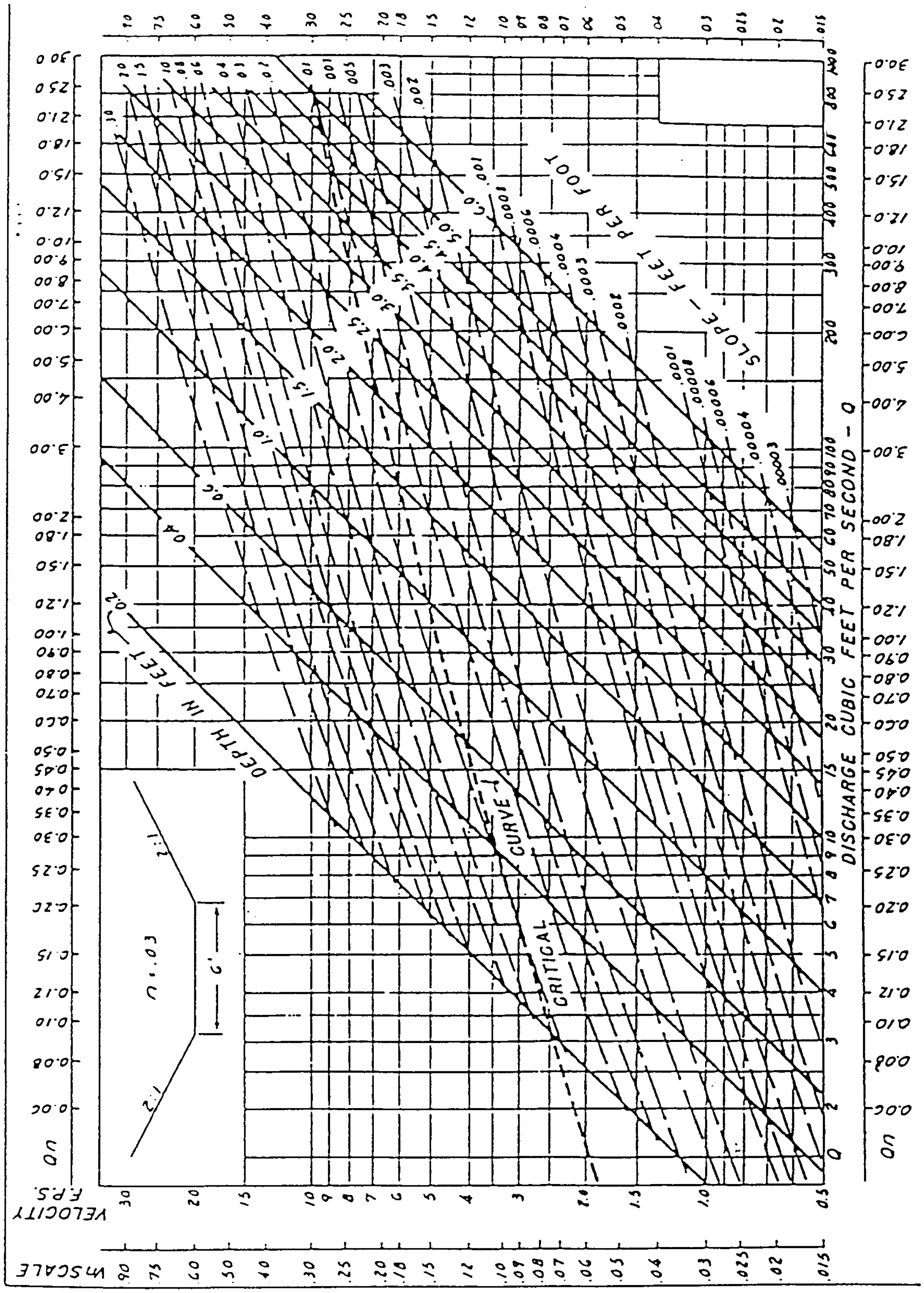
CHANNEL
2:1 b = 4 FT.

Source: Federal Highway Administration



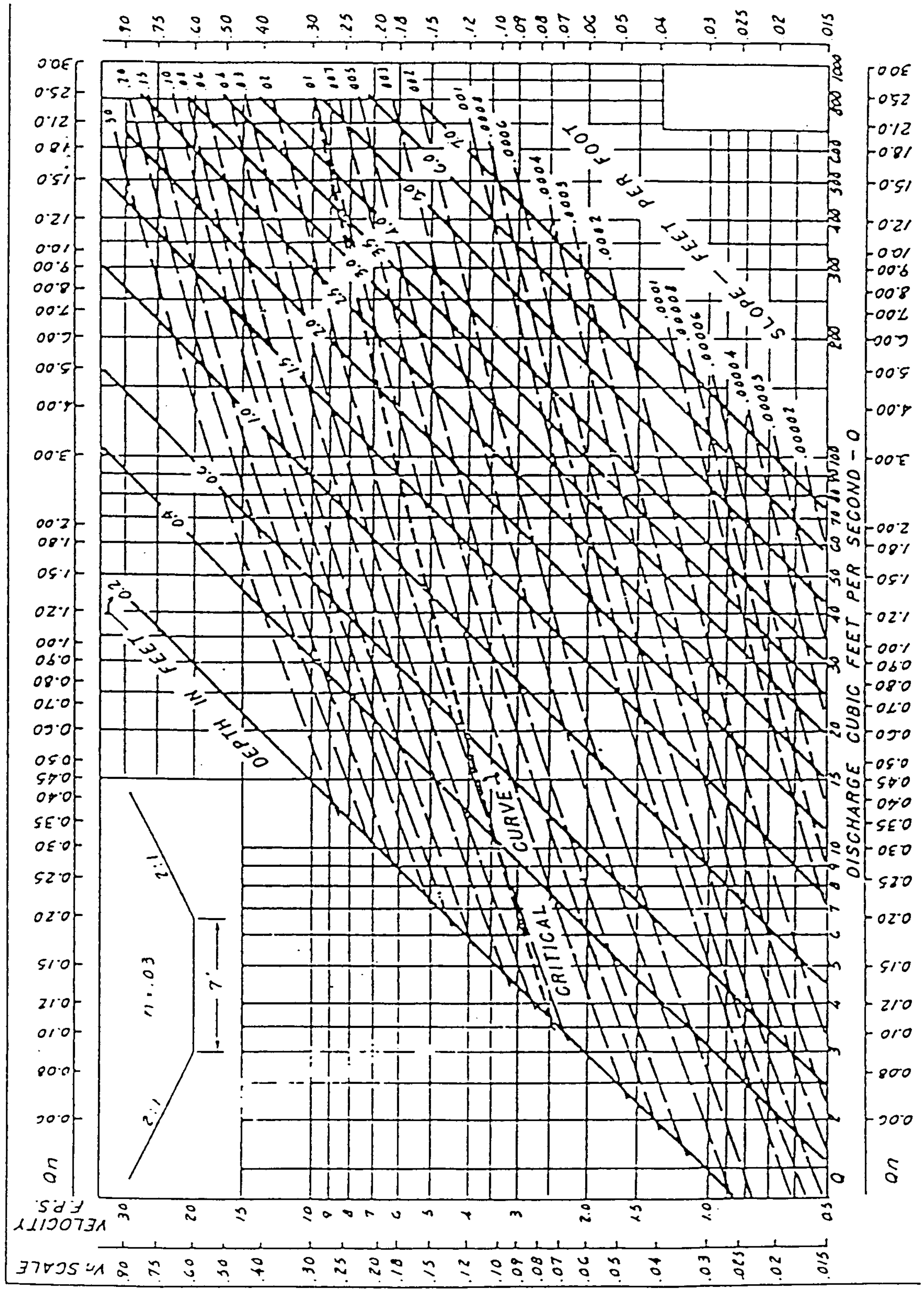
CHANNEL
2:1 b = 5 FT.

Source: Federal Highway Administration



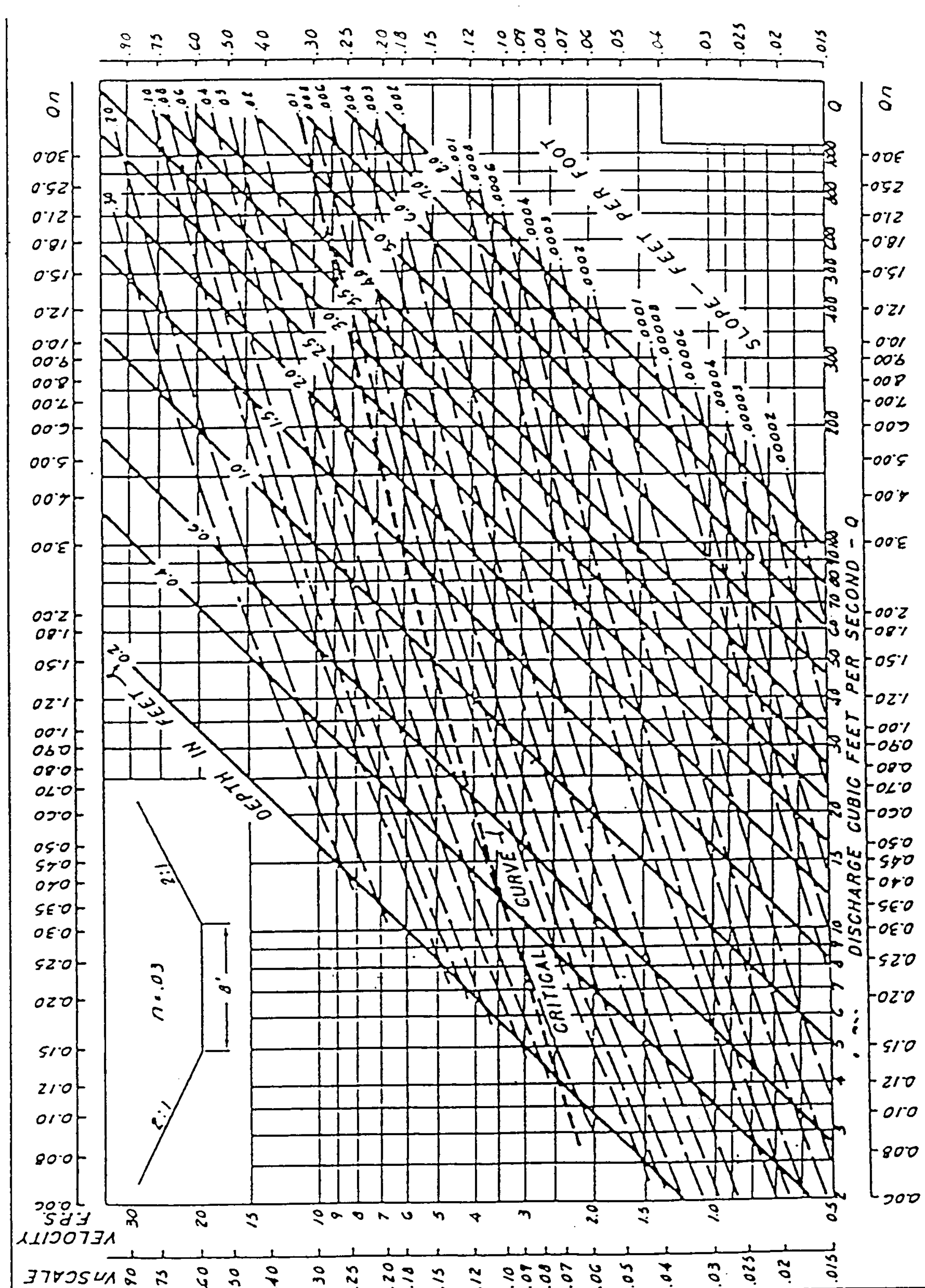
CHANNEL
2:1 b = 6 FT.

Source: Federal Highway Administration



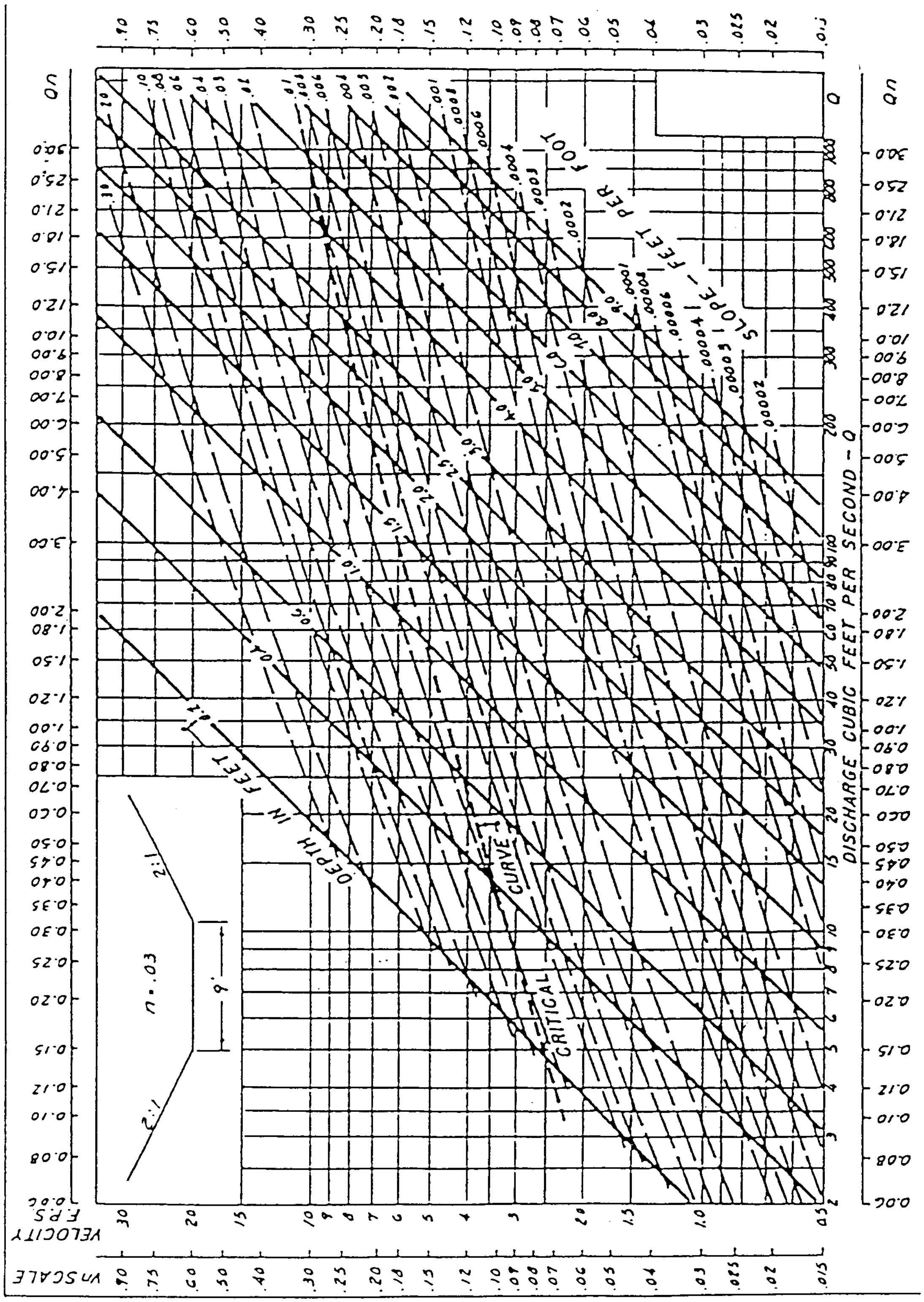
CHANNEL
2:1 b = 7 FT.

Source: Federal Highway Administration



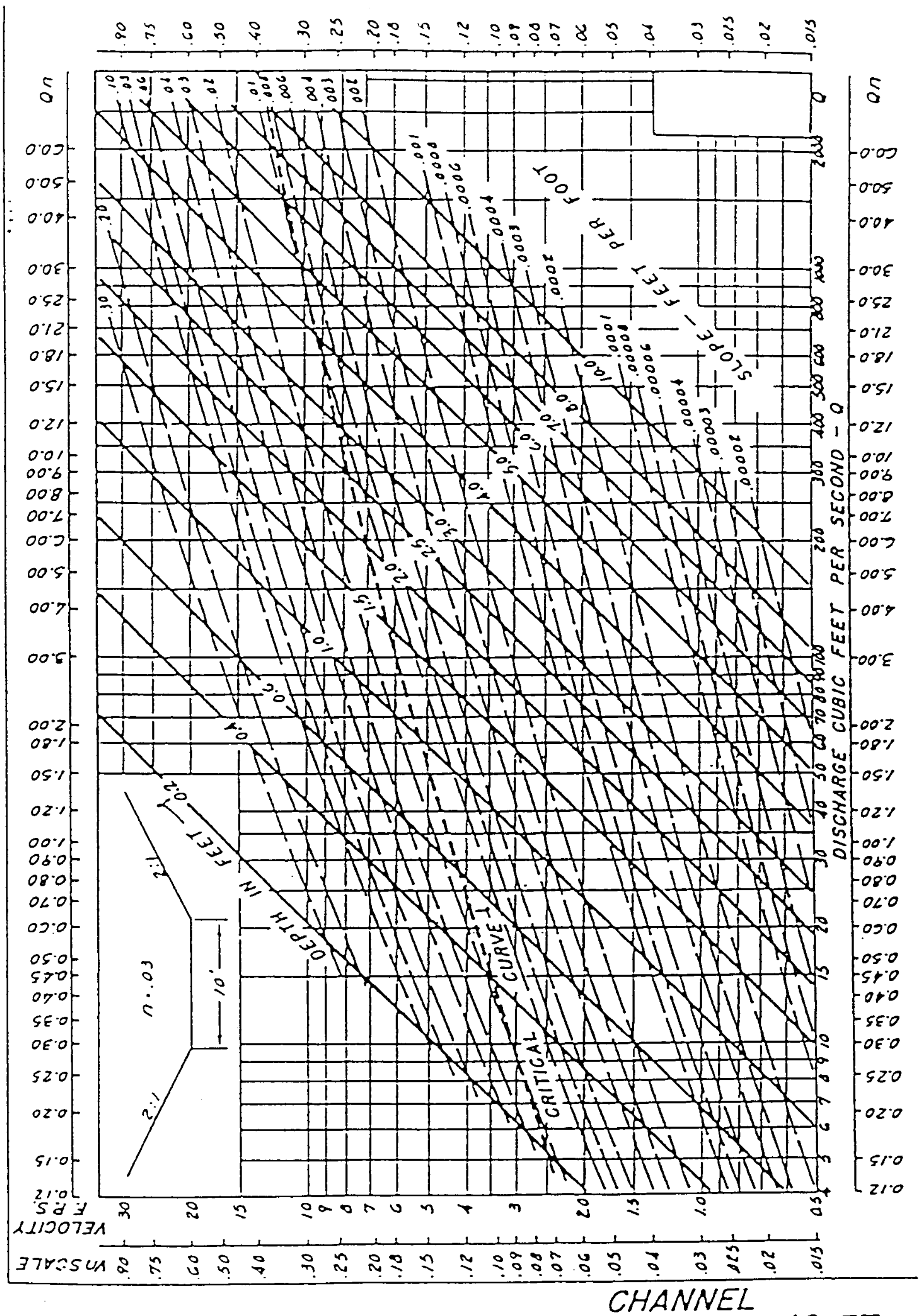
CHANNEL
2:1 b = 8 FT.

Source: Federal Highway Administration



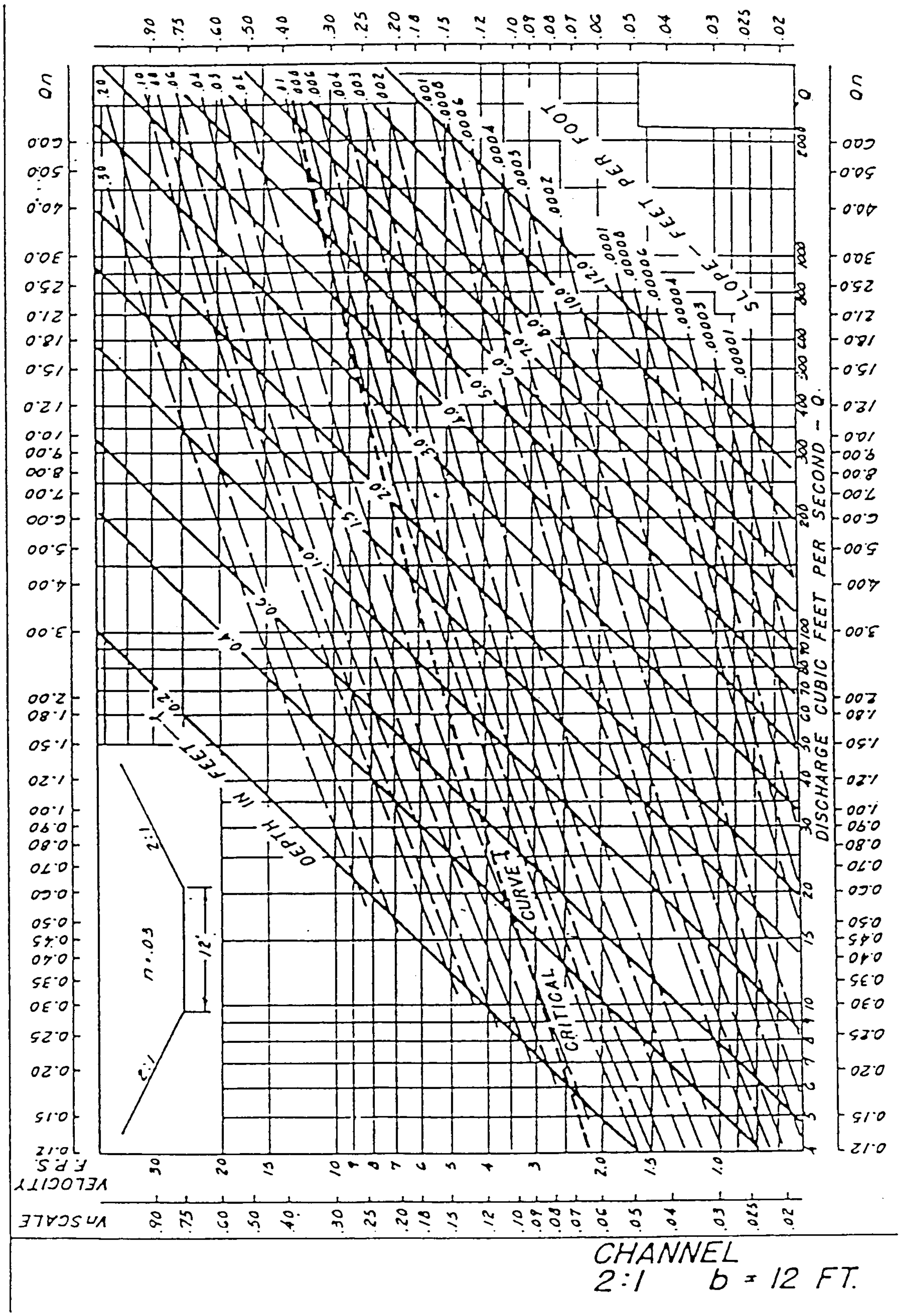
CHANNEL
2:1 b = 9 FT.

Source: Federal Highway Administration

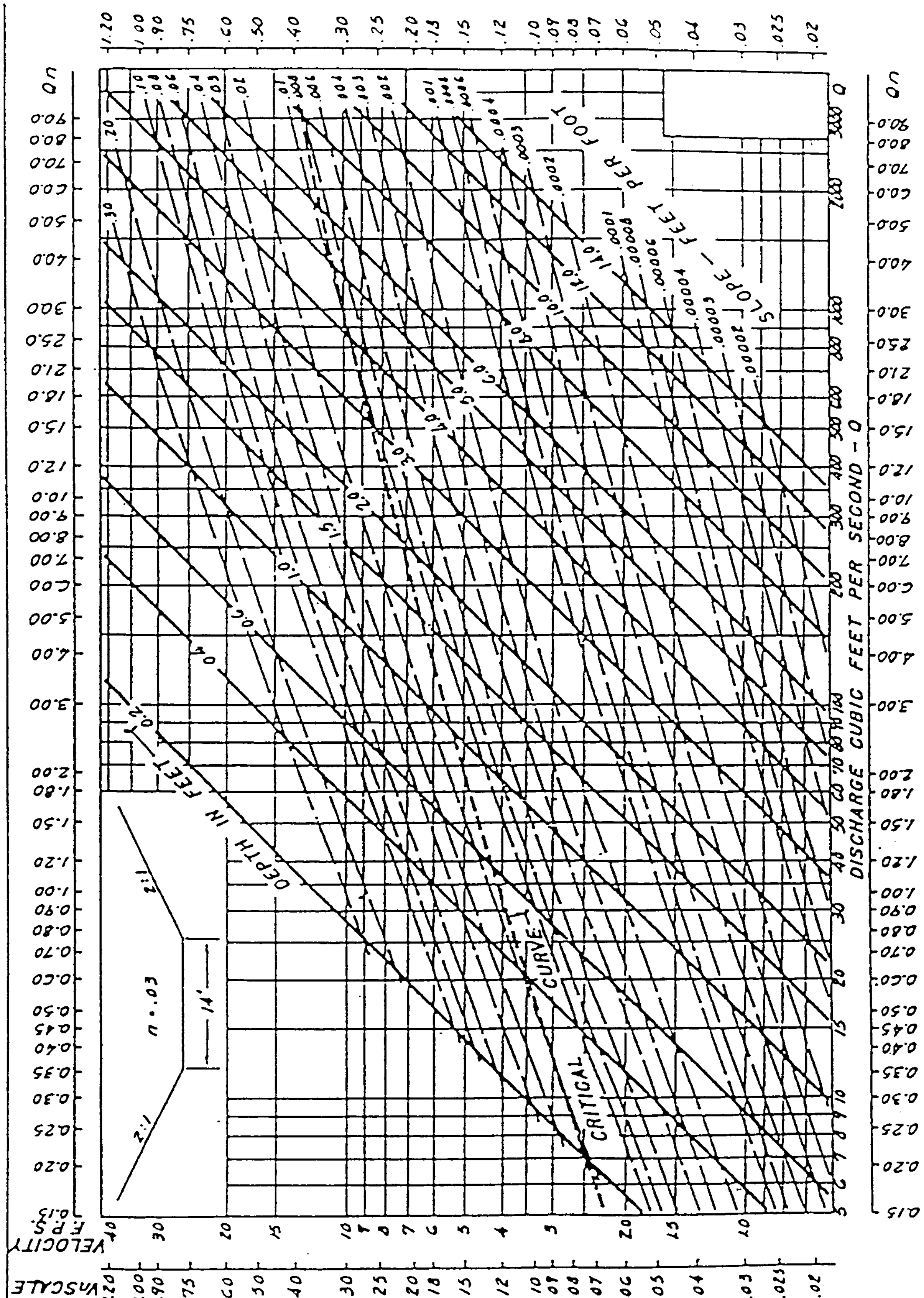


CHANNEL
2:1 b = 10 FT.

Source: Federal Highway Administration

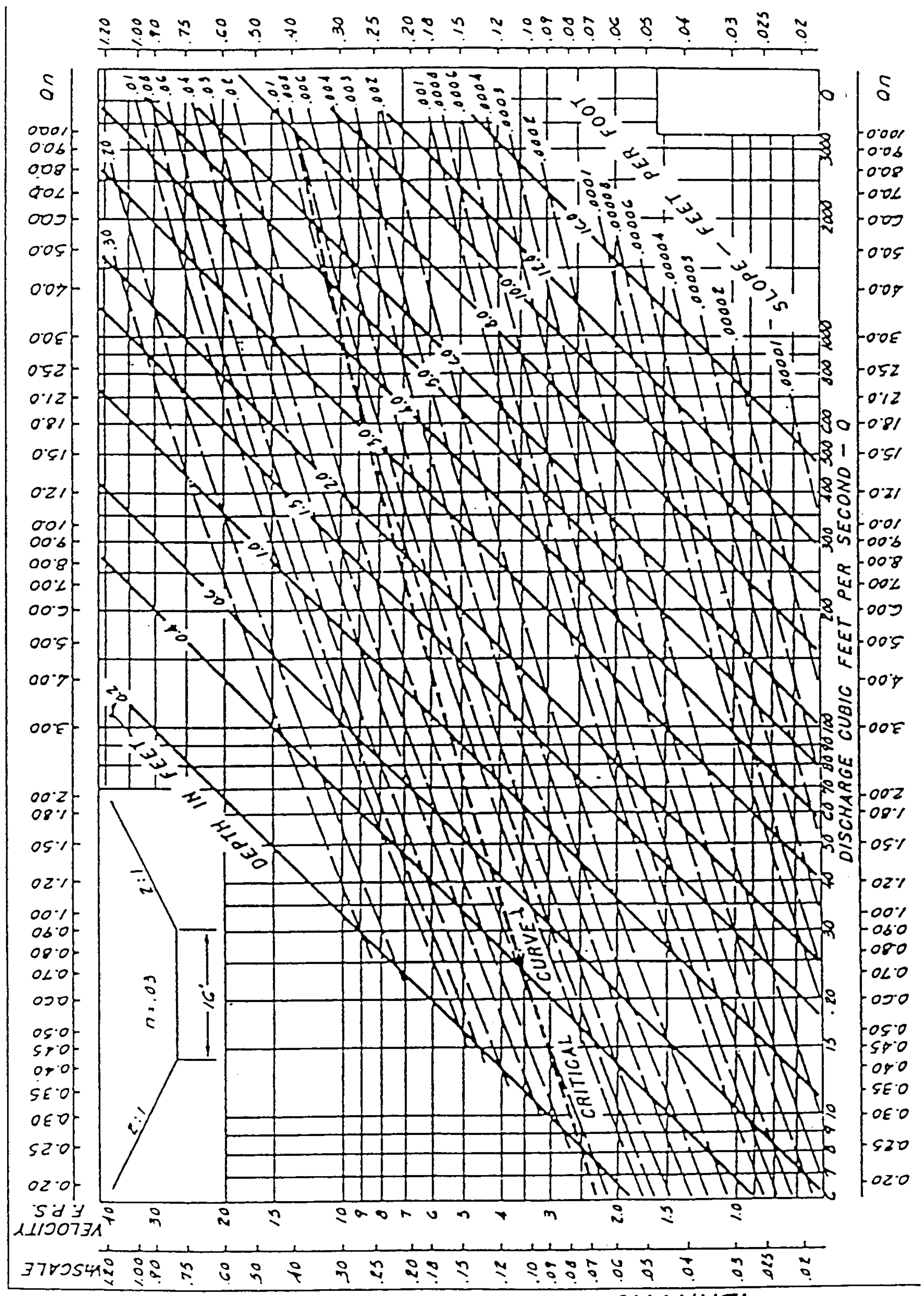


Source: Federal Highway Administration



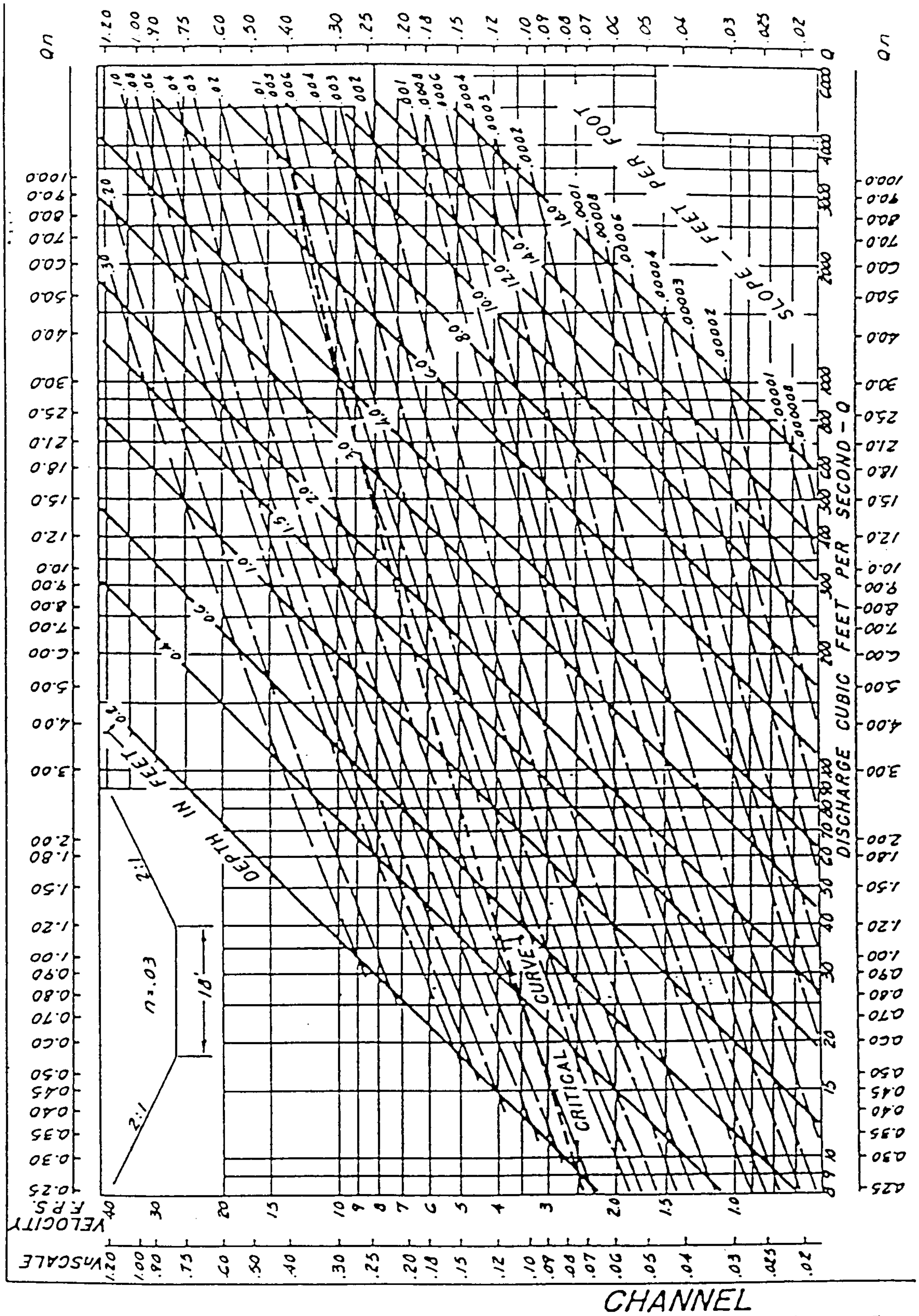
CHANNEL
2:1 b = 14 FT.

Source: Federal Highway Administration



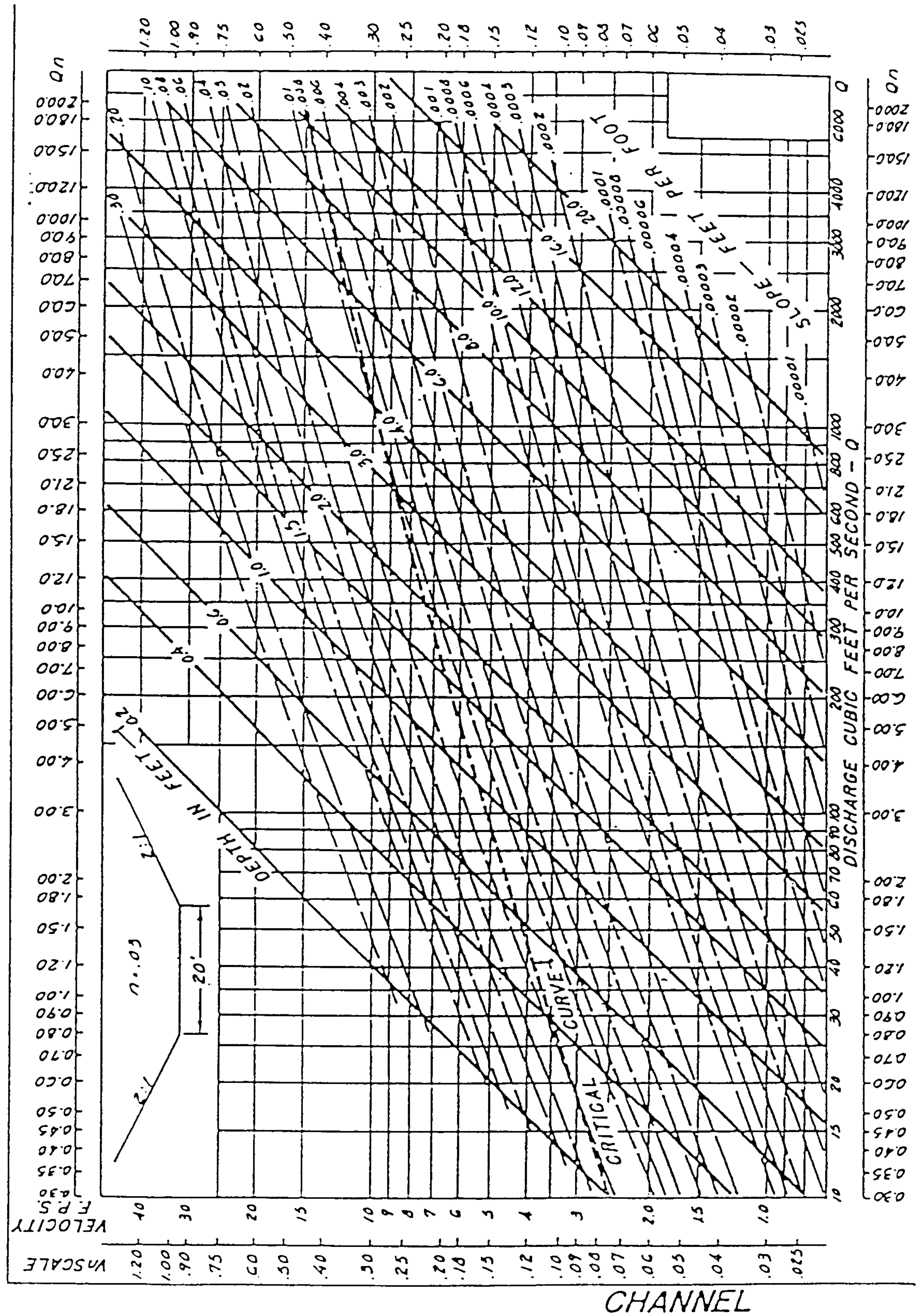
CHANNEL
2:1 b = 16 FT.

Source: Federal Highway Administration



CHANNEL
2:1 b = 18 FT.

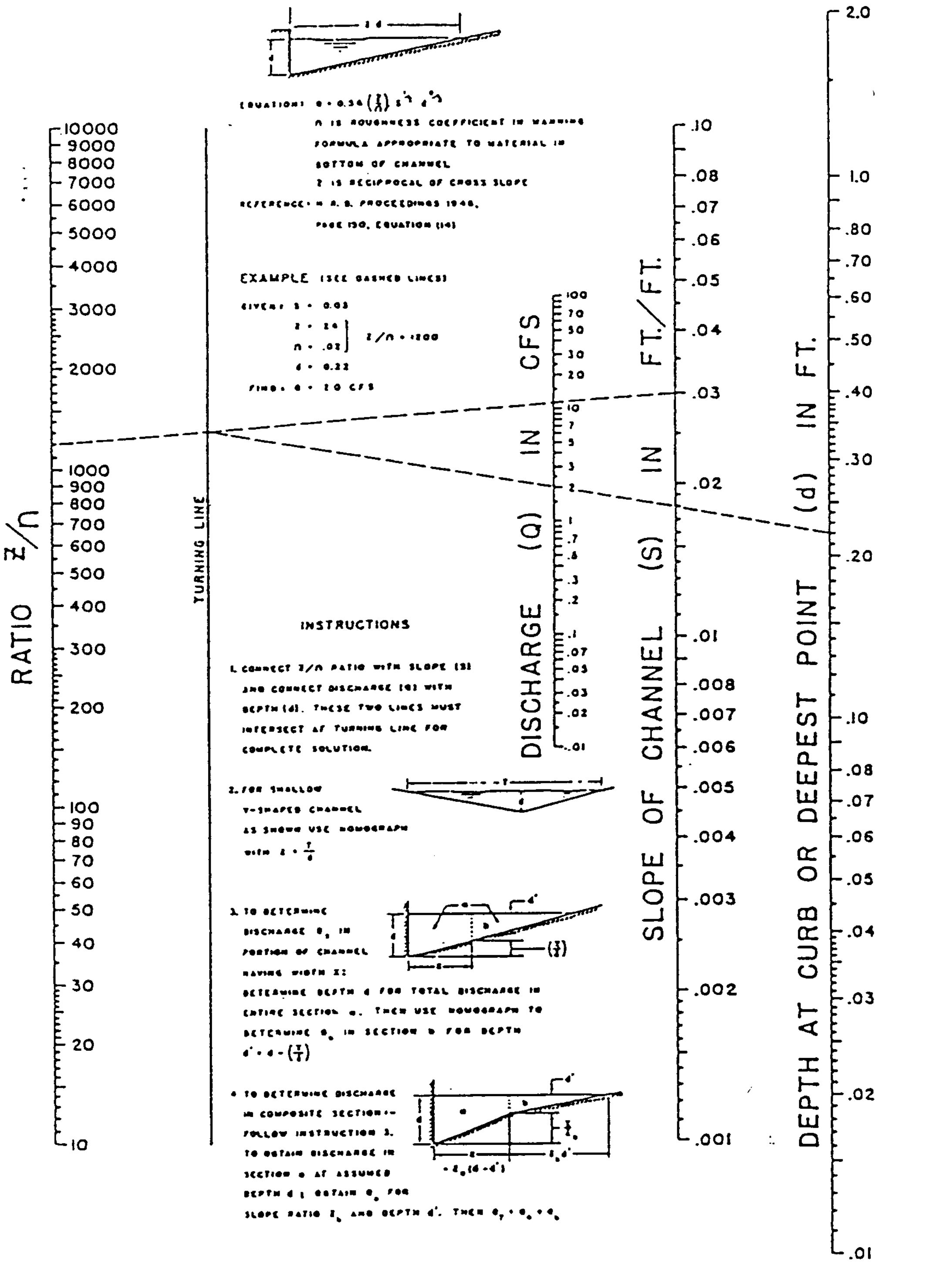
Source: Federal Highway Administration



CHANNEL
2:1 b = 20 FT.

Source: Federal Highway Administration

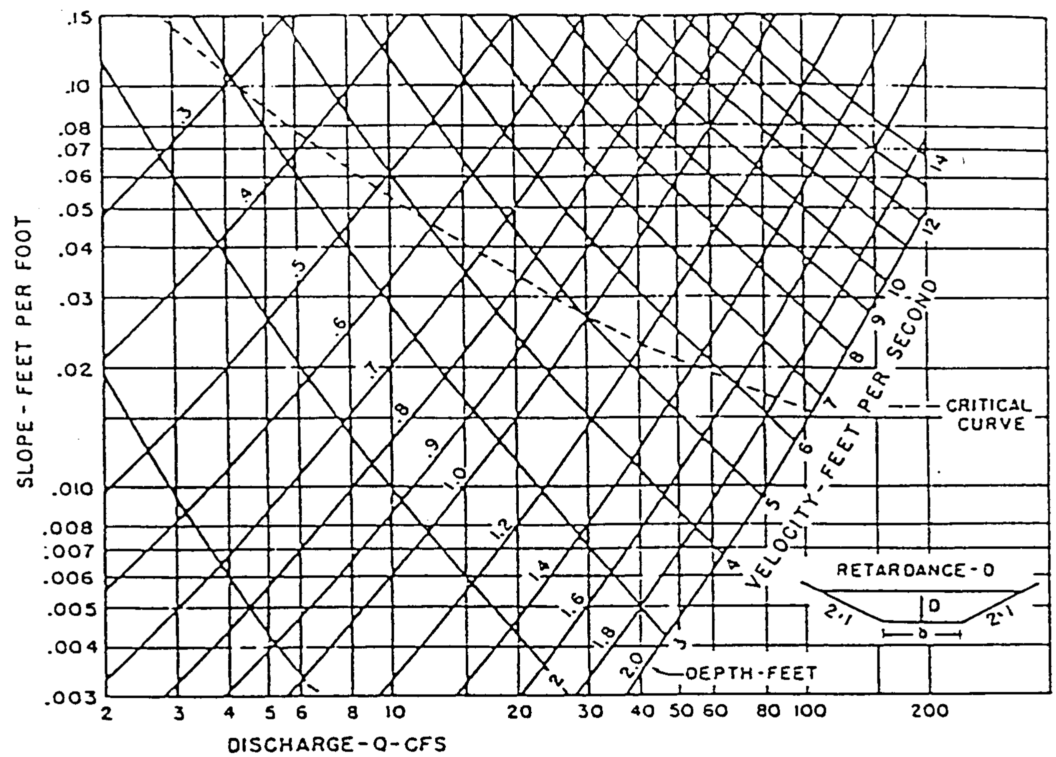
4.4.13 Triangular Channel Nomograph

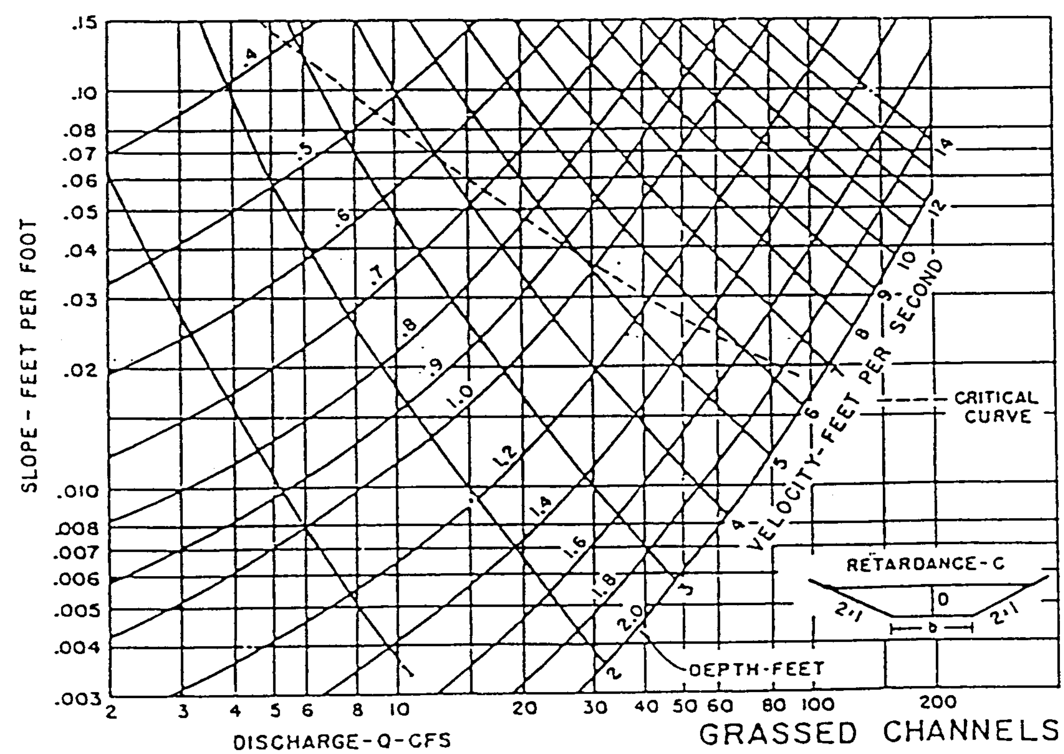


NOMOGRAPH FOR FLOW
IN TRIANGULAR CHANNELS

Source: Federal Highway Administration

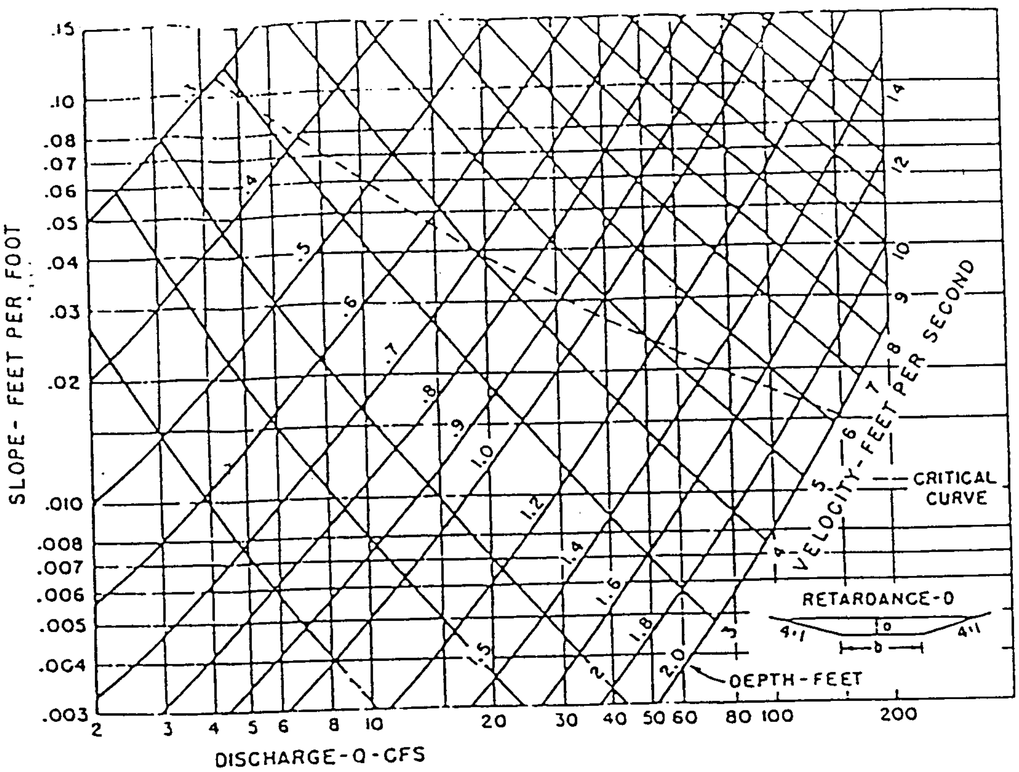
4.4.14 Grassed Channel Design Figures

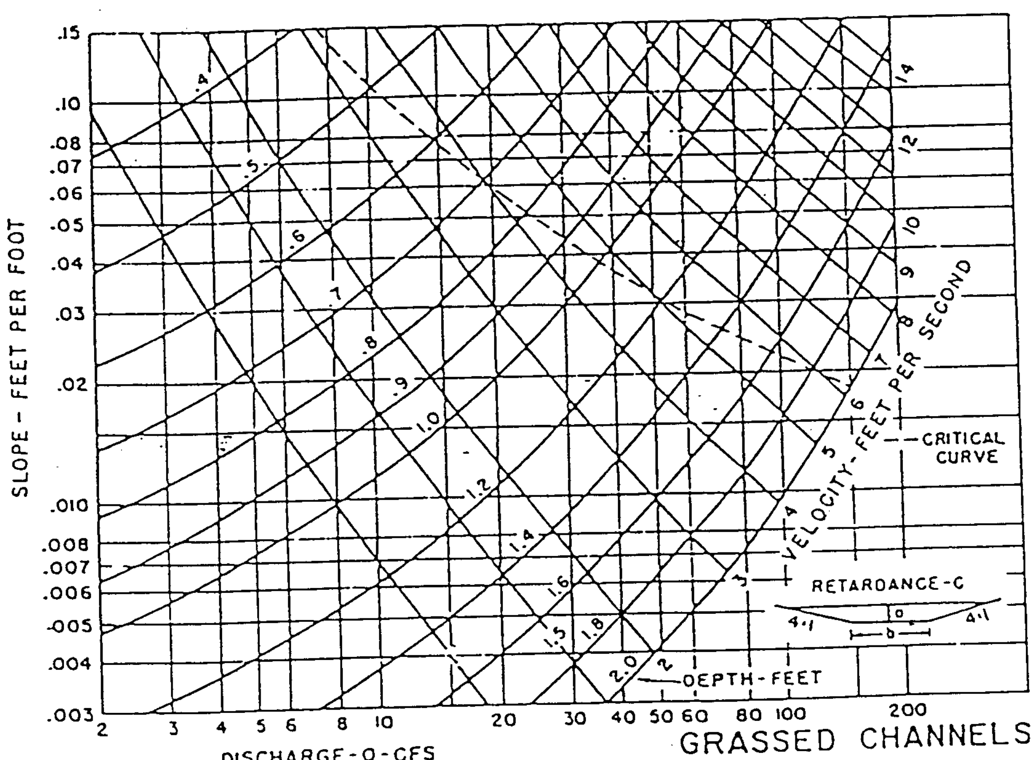




GRASSED CHANNELS
2:1 b = 4 ft.

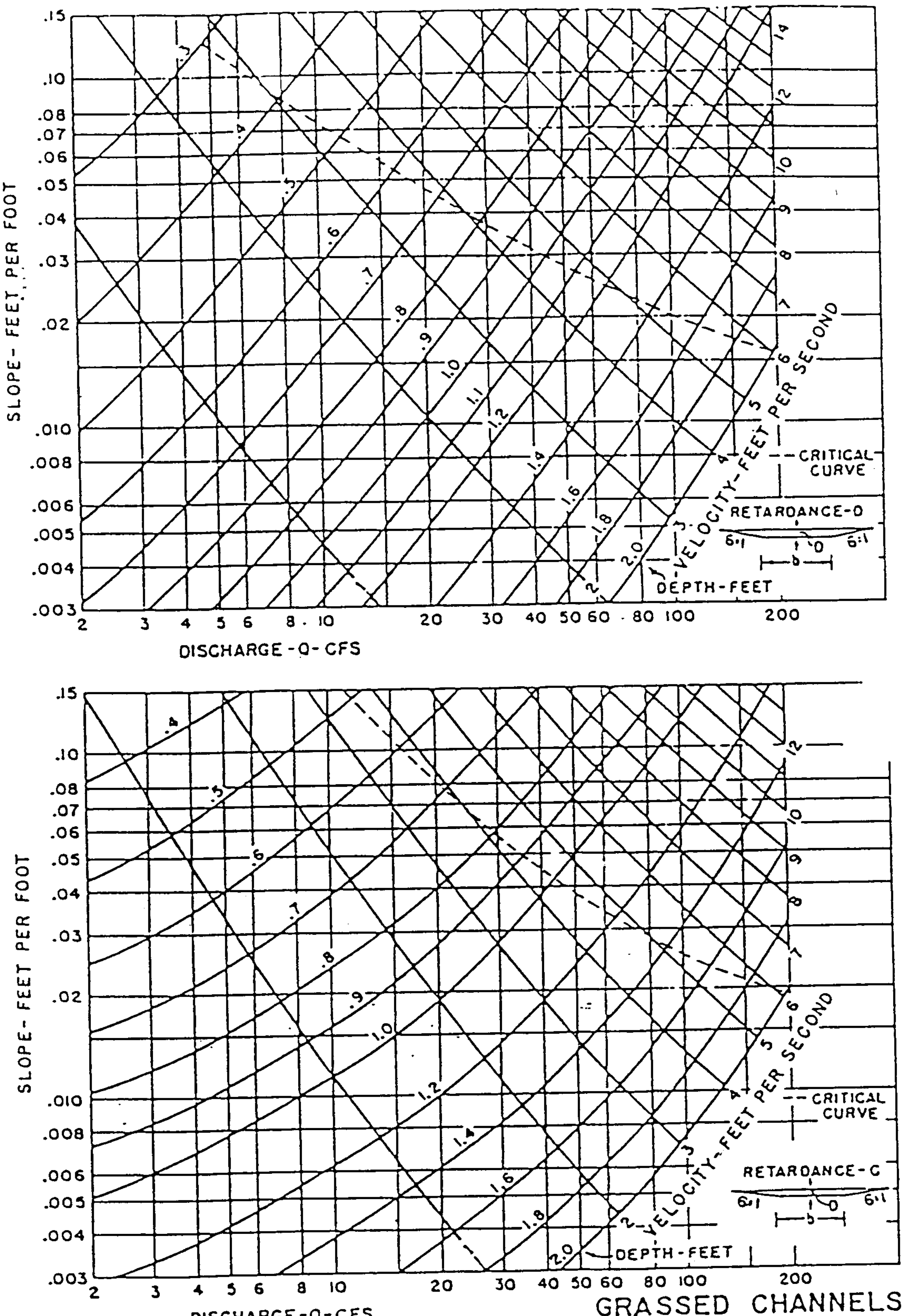
Source: Federal Highway Administration





GRASSED CHANNELS
4:1 b = 4 ft.

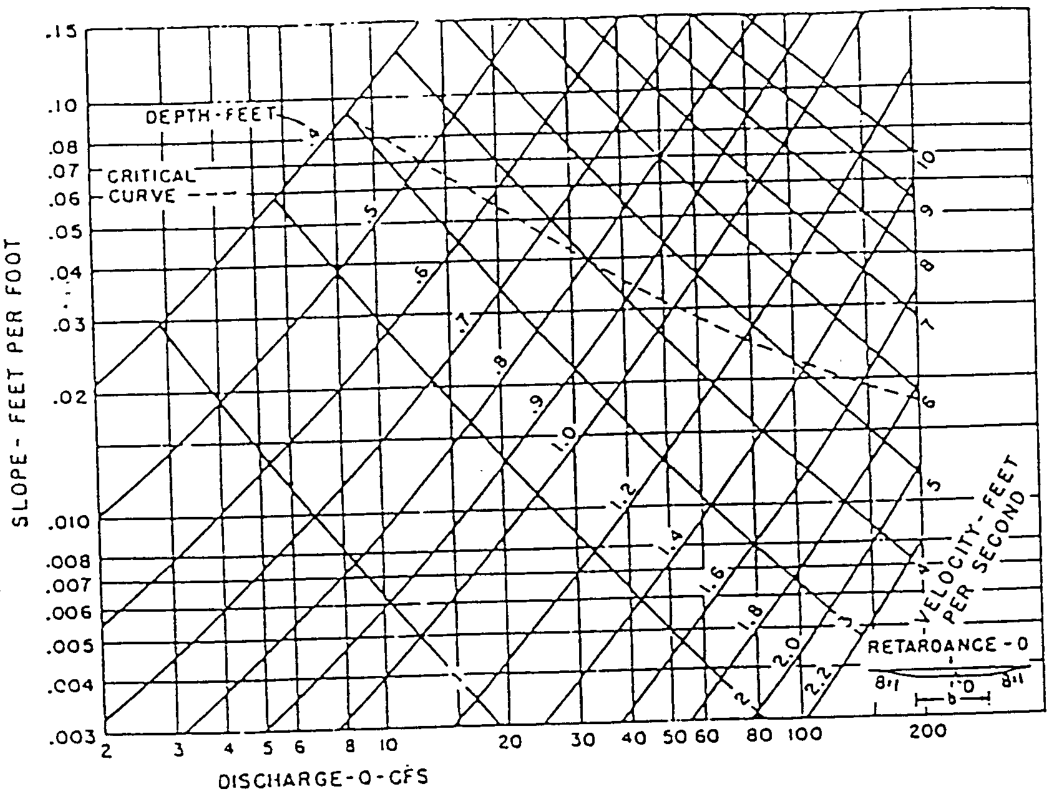
Source: Federal Highway Administration

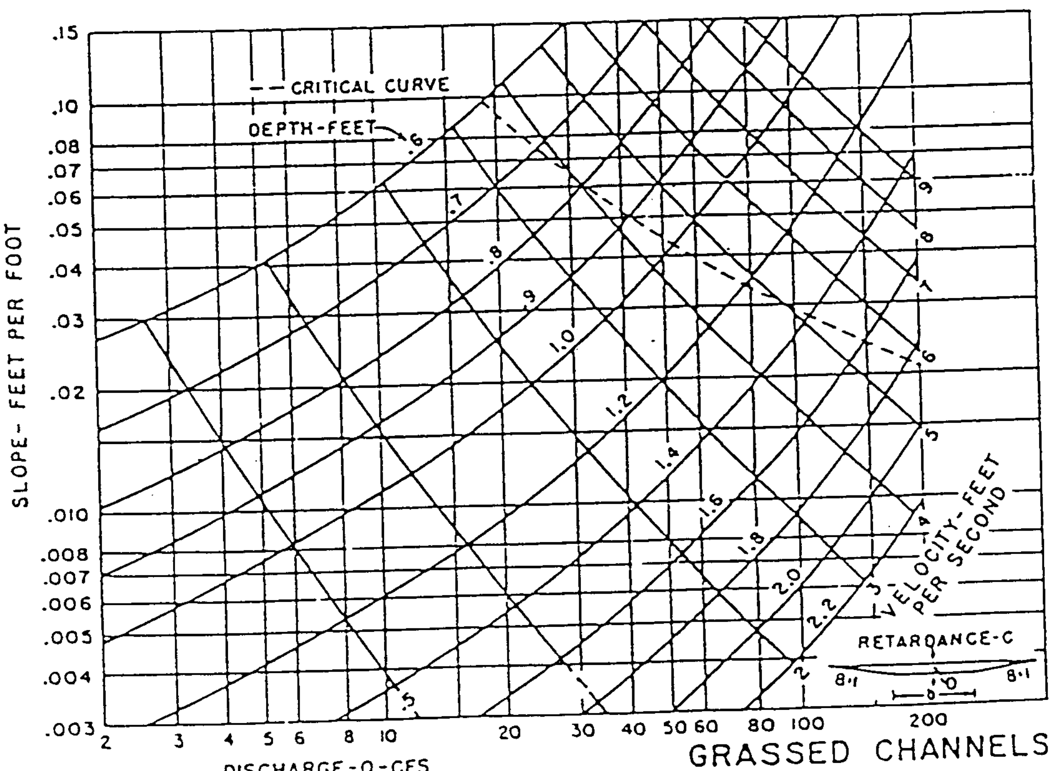


GRASSED CHANNELS

6:1 b=4 ft.

Source: Federal Highway Administration

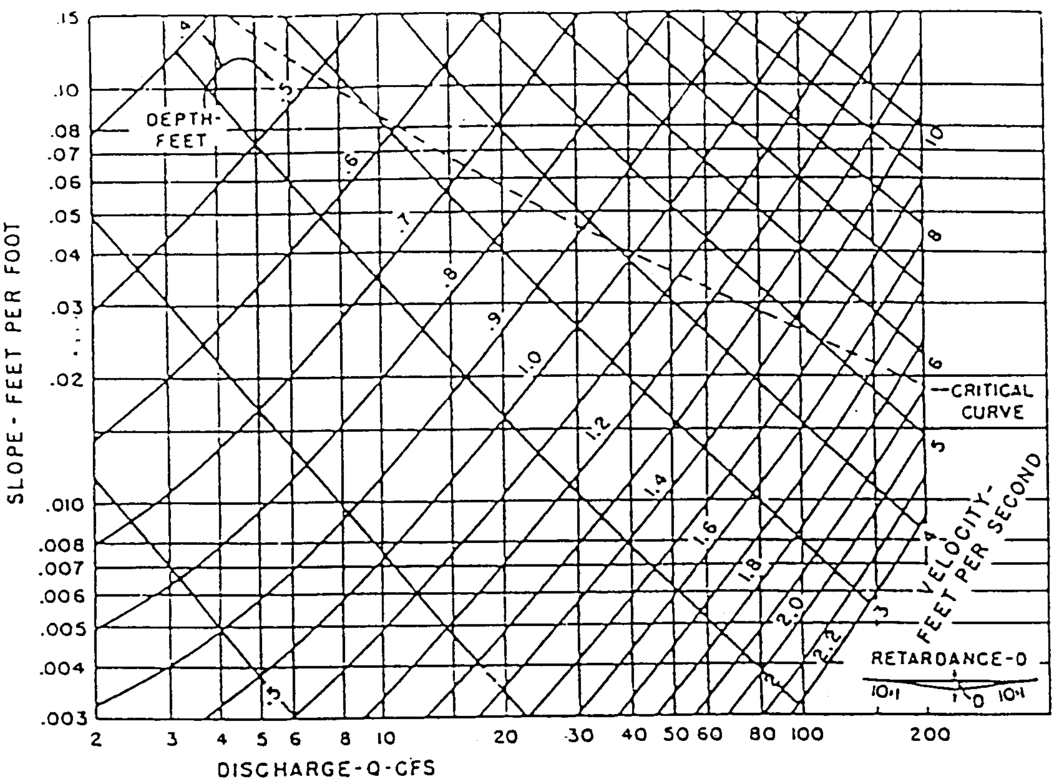


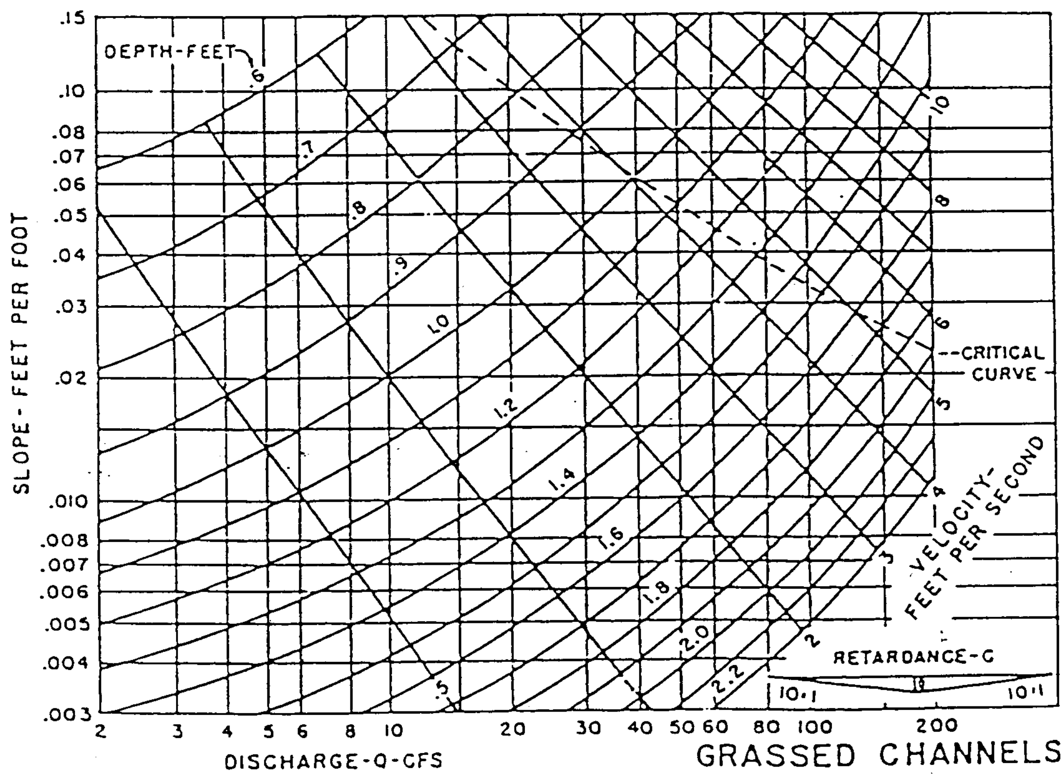


GRASSED CHANNELS

8:1 b = 4 ft.

Source: Federal Highway Administration





GRASSED CHANNELS
10:1

Source: Federal Highway Administration

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SECTION

4.5

ENERGY DISSIPATION DESIGN

4.5.1 Overview

4.5.1.1 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.

4.5.1.2 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.

4.5.1.3 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.

4.5.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.5-1 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.

Table 4.5-1 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^2
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

4.5.3 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:

a. 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.

b. 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.

c. 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 4.5-1 provides the riprap size recommended for use downstream of energy dissipators.

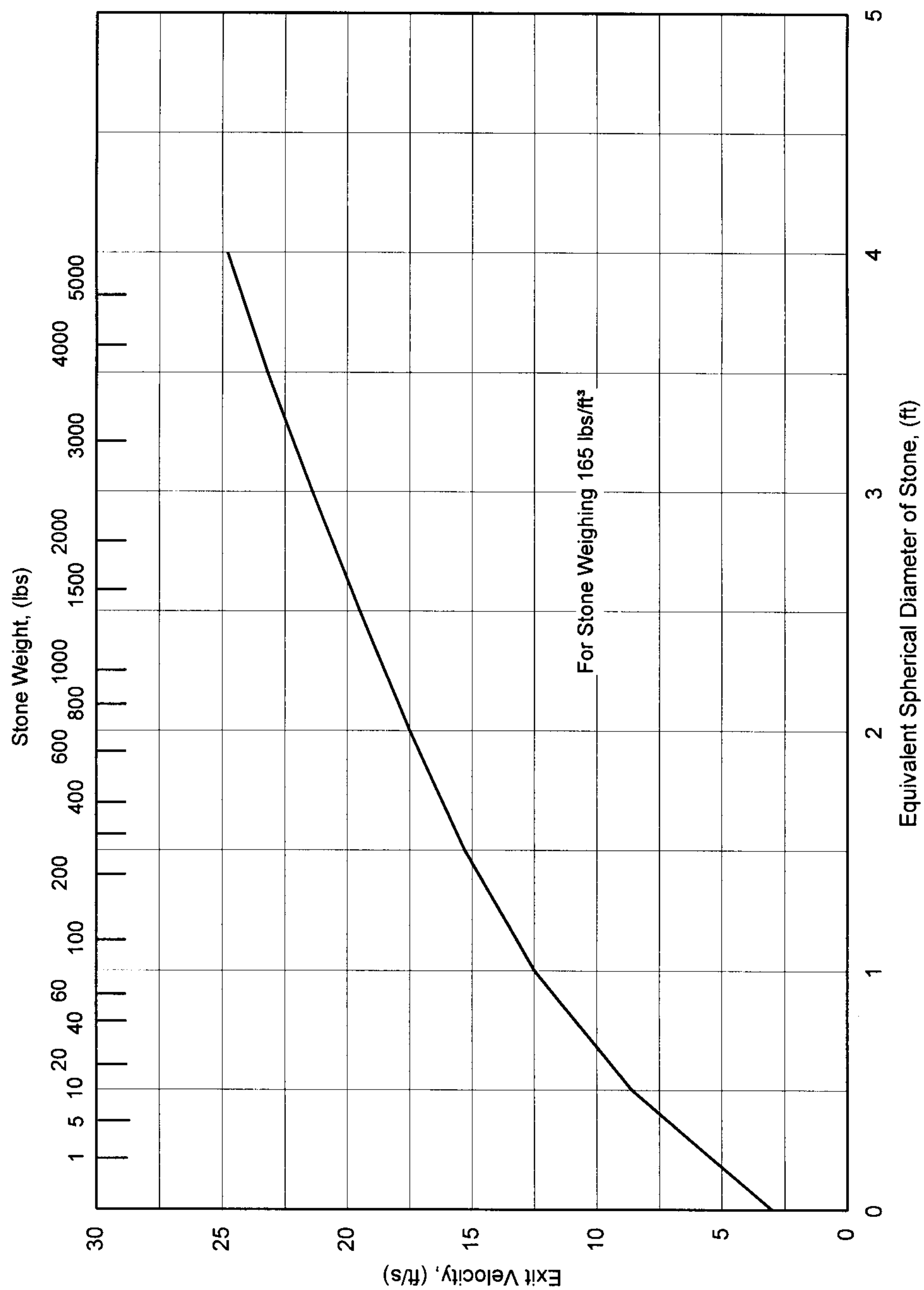


Figure 4.5-1 Riprap Size for Use Downstream of Energy Dissipator
(Source: Searcy, 1967)

4.5.4 Riprap Aprons

4.5.4.1 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.

4.5.4.2 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 4.5-2 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 4.5-3 should be used.

(Step 2) Determine the correct apron length and median riprap diameter, d_{50}, using the appropriate curves from Figures 4.5-2 and 4.5-3. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 4.5-4.

a. *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.

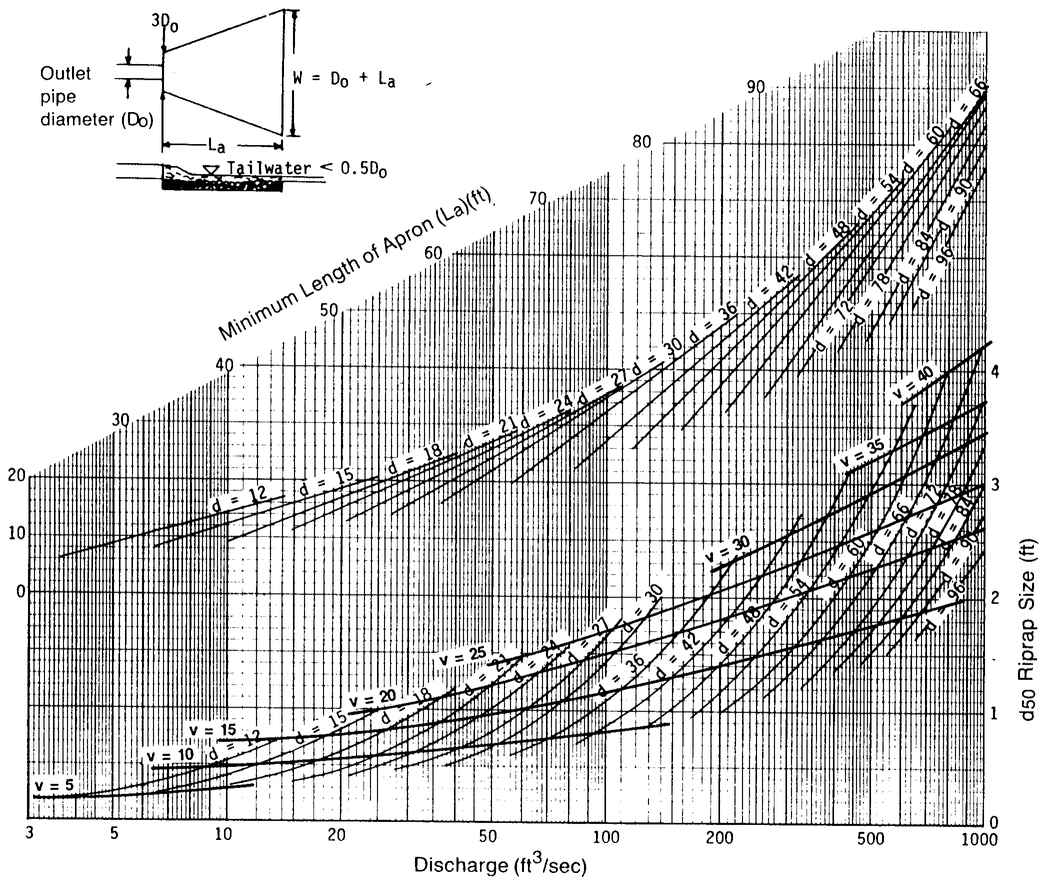
b. *For pipes flowing partially full:*

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.

c. *For box culverts:*

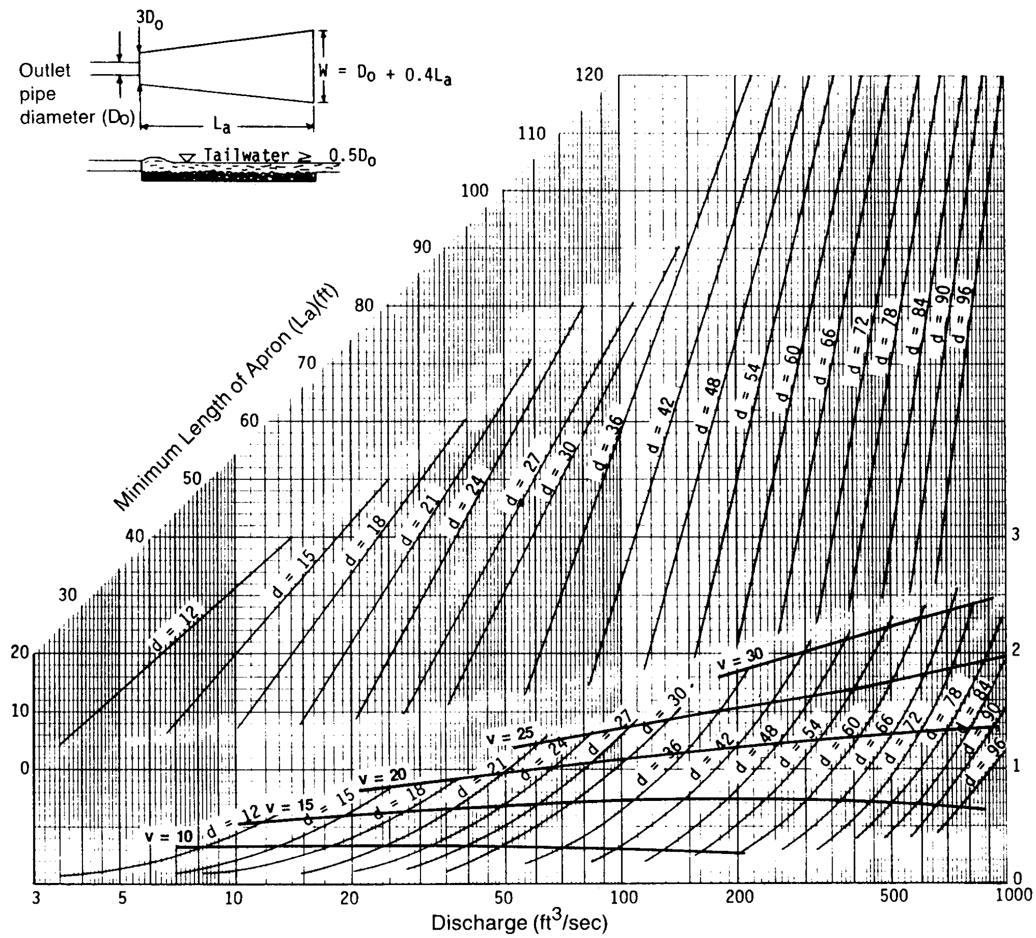
Use the depth of flow, d, in feet, and velocity, v, in feet/second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the 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 4.5-4. This will provide protection under either of the tailwater conditions.



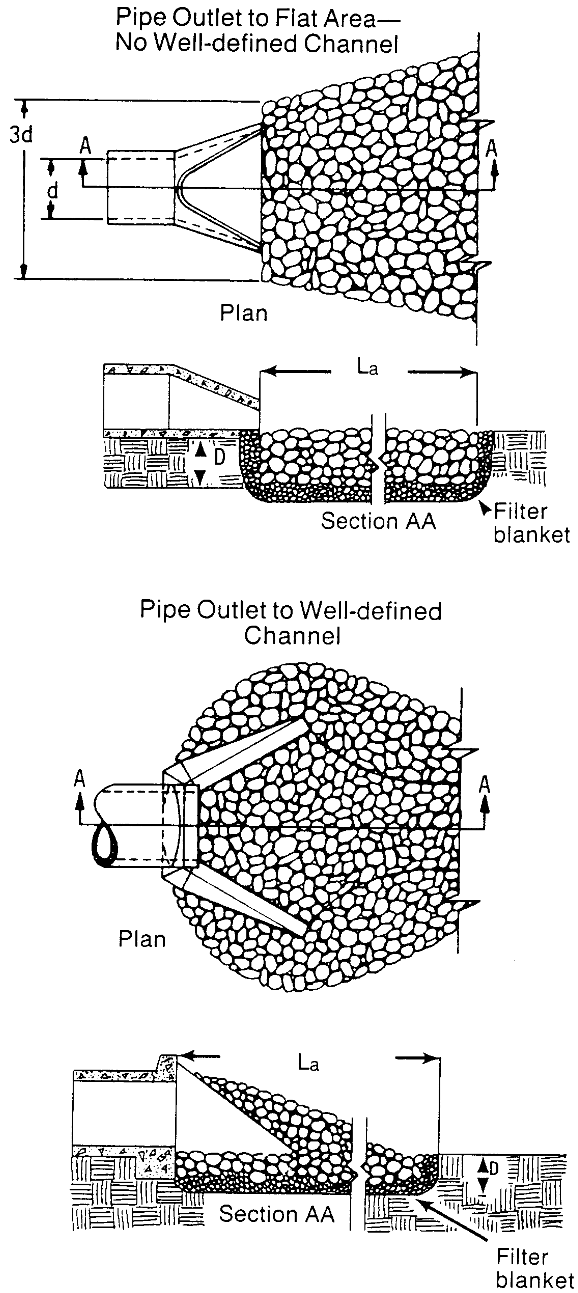
Curves may not be extrapolated.

Figure 4.5-2 Design of Riprap Apron under Minimum Tailwater Conditions
(Source: USDA, SCS, 1975)



Curves may not be extrapolated.

Figure 4.5-3 Design of Riprap Apron under Maximum Tailwater Conditions
(Source: USDA, SCS, 1975)



Notes

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 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.

Figure 4.5-4 Riprap Apron
(Source: Manual for Erosion and Sediment Control in Georgia, 1996)

4.5.4.3 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.

4.5.4.4 Example Designs

<u>Example 1</u> 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 4.5-2 for minimum tailwater conditions.

(3) By Figure 4.5-2, 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 4.5-3 for maximum tailwater conditions. v = Q/A = [600/(5) (10)] = 12 ft/s

(3) On Figure 4.5-3, at the intersection of the curve, d_o = 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 d_{max} = 1.5 (0.6) = 0.9 ft.

4.5.5 Riprap Basins

4.5.5.1 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.

4.5.5.2 Basin Features

General details of the basin recommended in this section are shown in Figure 4.5-5. 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 h_s 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.

4.5.5.3 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 4.5-6 or Figure 4.5-7 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 4.5-8, and check to see that $2 < h_s/d_{50} < 4$. Recycle computations if h_s/d_{50} falls out of this range.

(Step 4) Size basin as shown in Figure 4.5-5.

(Step 5) Where allowable dissipator exit velocity is specified:

a. Determine the average normal flow depth in the natural channel for the design discharge.

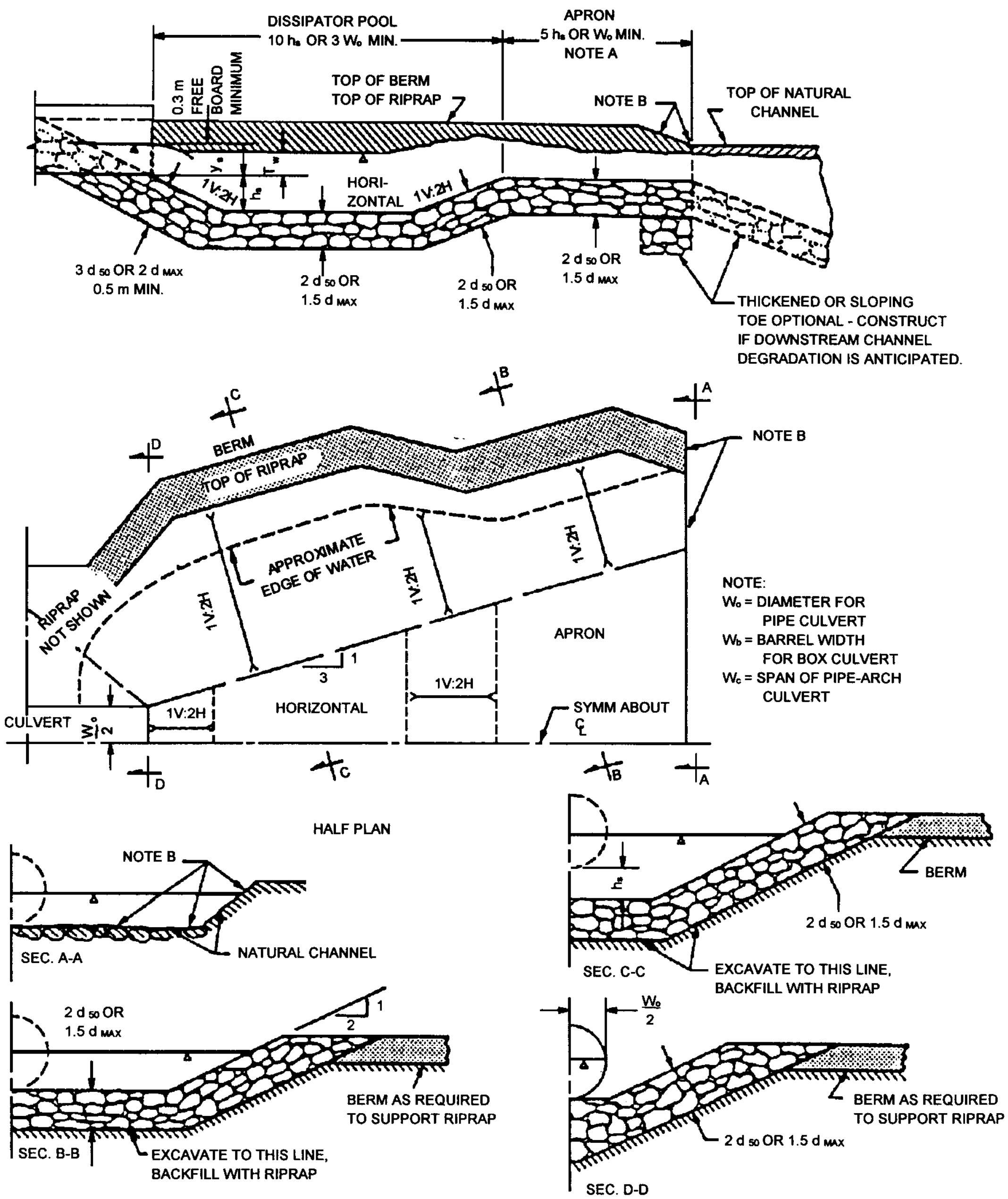
b. Extend the length of the energy basin (if necessary) so that the width of the energy basin at section A-A, Figure 4.5-5, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.

(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 4.5-9.
- Shape downstream channel and size riprap using Figure 4.5-1 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 4.5-5 Details of Riprap Outlet Basin
(Source: HEC-14, 1983)

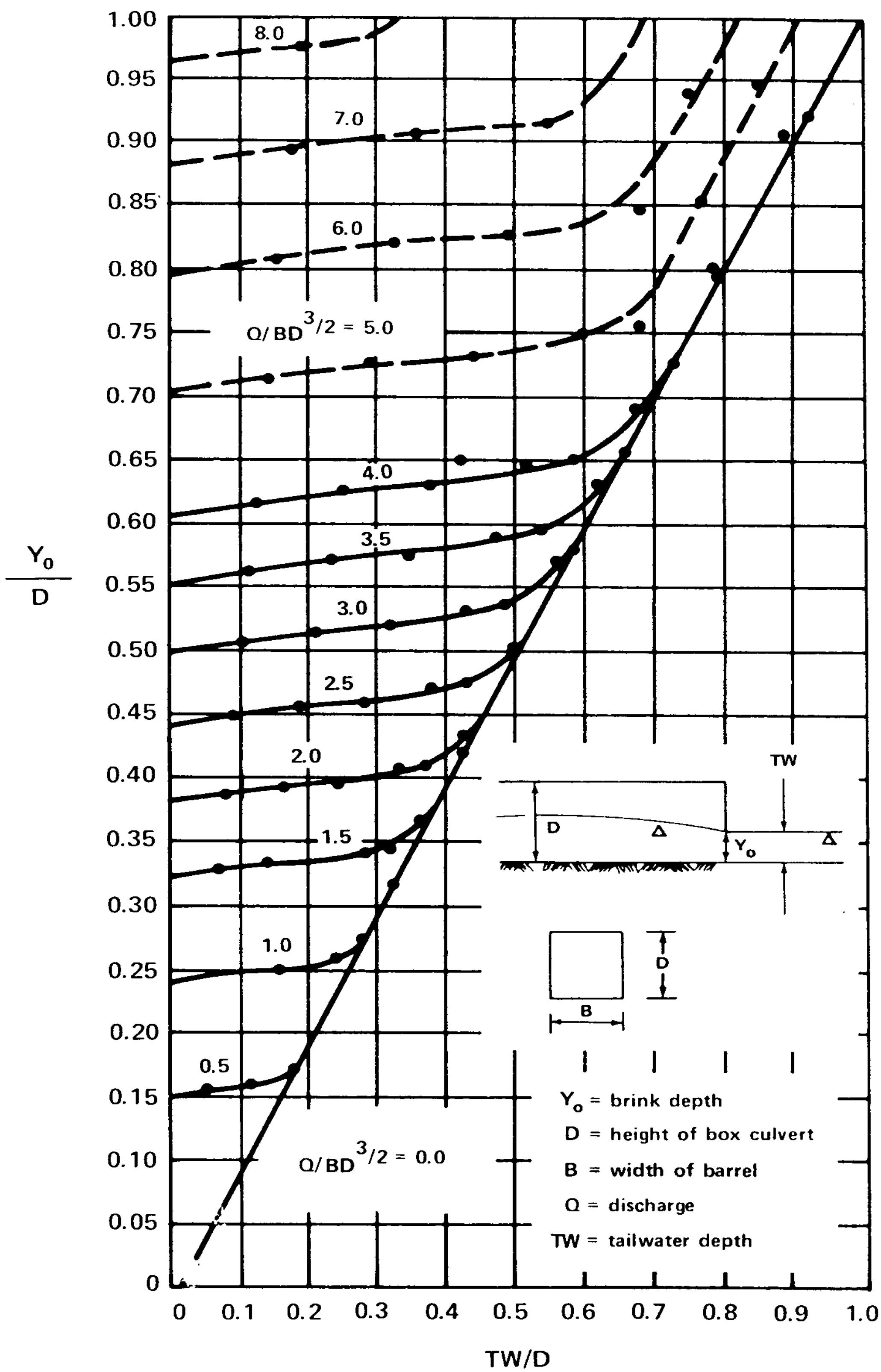


Figure 4.5-6 Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 1983)

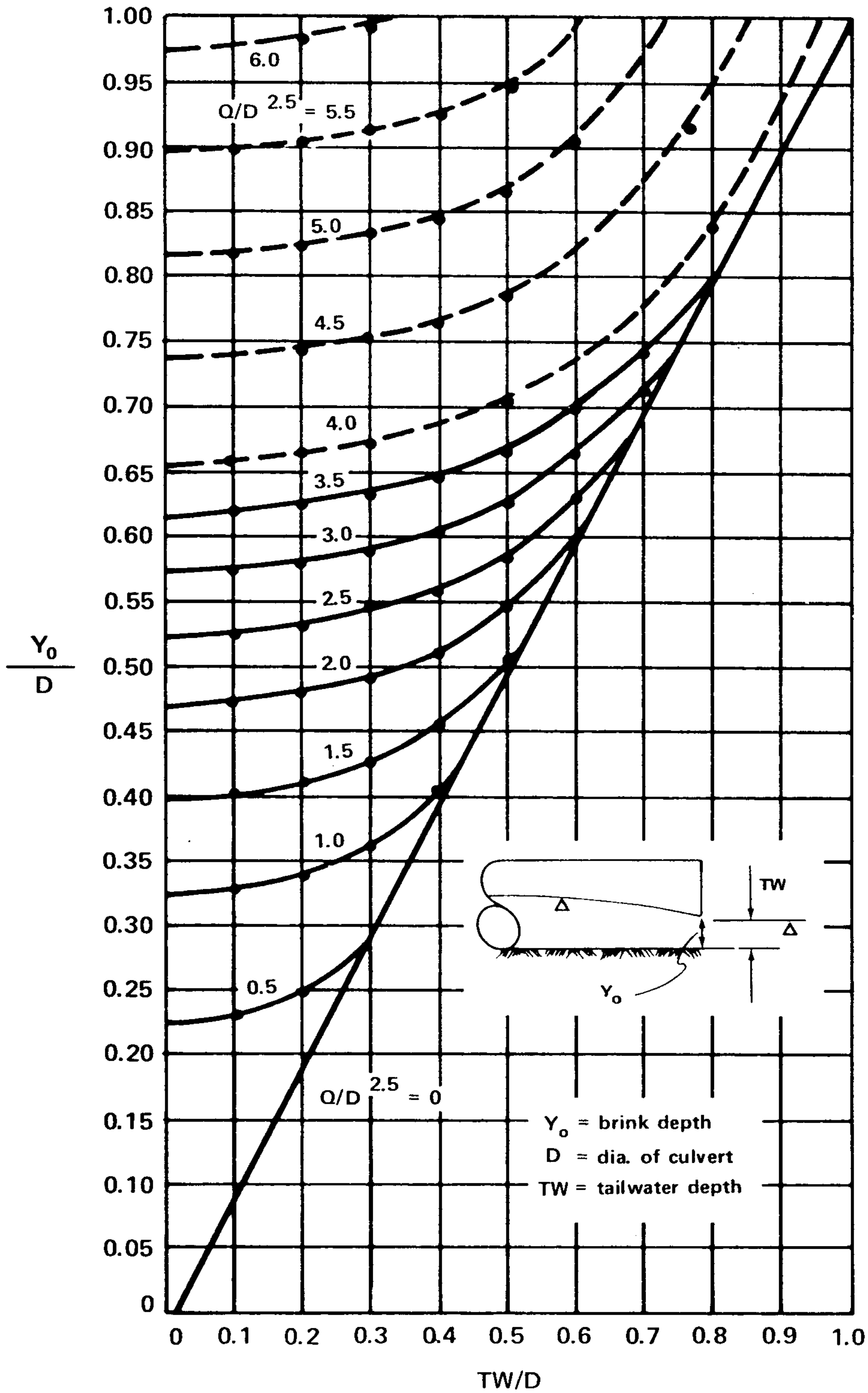


Figure 4.5-7 Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes

(Source: USDOT, FHWA, HEC-14, 1983)

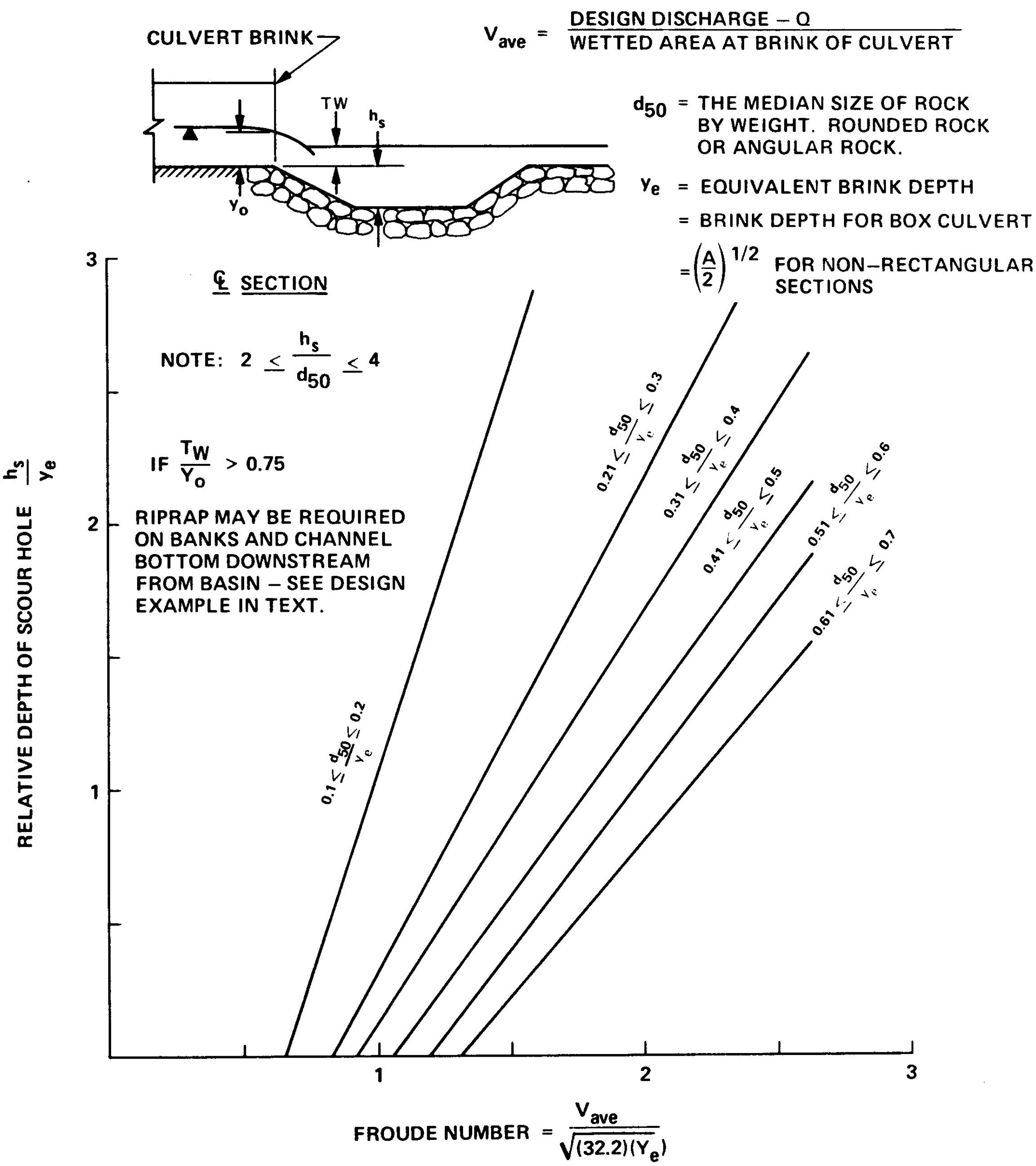


Figure 4.5-8 Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable
(Source: USDOT, FHWA, HEC-14, 1983)

4.5.5.4 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 4.4, *Open Channel Design.*

4.5.5.5 Example Designs

Following are some example problems to illustrate the design procedures outlined.

<u>Example 1</u>

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 4.5-5 and 4.5-8.

(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.

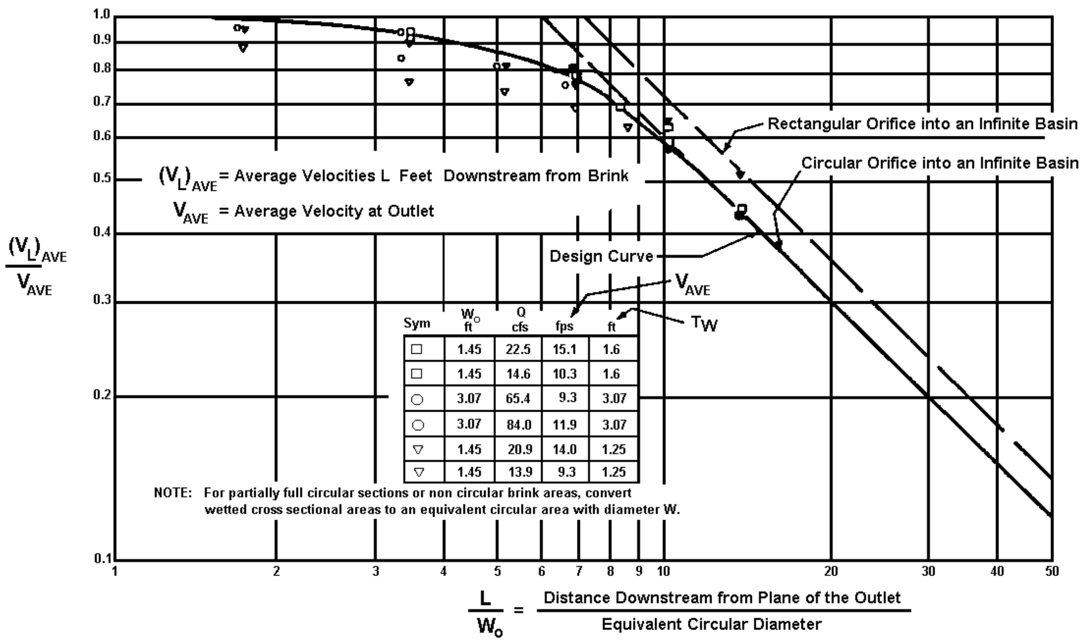


Figure 4.5-9 Distribution of Centerline Velocity for Flow from Submerged Outlets to Be Used for Predicting Channel Velocities Downstream from Culvert Outlet Where High Tailwater Prevails
(Source: USDOT, FHWA, HEC-14, 1983)

(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 4.5-8, $h_s/y_e = 1.6$, $h_s = 4 \times 1.6 = 6.4$ 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 4.5-5.

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 4.5-9 for estimating average velocity along the channel. Compute equivalent circular diameter D_e for brink area from:

$A = 3.14D_e^2/4 = y_o \times W_o = 4 \times 8 = 32 \text{ ft}^2$
$D_e = ((32 \times 4)/3.14)^{0.5} = 6.4$ ft
$V_o = 25$ ft/s (From Example 1)

(3) Set up the following table:

L/D_e	L (ft)	V_L/V_o	v_1 (ft/s)	Rock Size d_{50} (ft)
(Assume) $D_e = W_o$	(Compute)	(Fig. 4.5-9)		(Fig. 4.5-1)
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

*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 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 S_o = 0.004
Manning's n = 0.024
Normal depth in pipe for Q = 135 cfs is 4.5 ft
Normal velocity is 5.9 ft/s
Flow is subcritical
Tailwater depth TW = 2.0 ft

Find: Riprap basin dimensions for these conditions.

Solution:

(1) Determine y_o and V_o
From Figure 4.5-7, $y_o/D = 0.45$
$Q/D^{2.5} = 135/6^{2.5} = 1.53$
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 4.3)
For $y_o/D = d/D = 0.45$
$A/D^2 = 0.3428$; therefore, $A = 0.3428 \times 6^2 = 12.3\ ft^2$
$V_o = Q/A = 135/12.3 = 11.0$ ft/s

(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 4.5-8, $h_s/y_e = 0.75$; therefore, $h_s = 0.75 \times 2.48 = 1.86$ ft

Uniform Flow in Circular Sections Flowing Partly Full (From Section 4.3)
Check: $h_s/d_{50} = 1.86/0.62 = 3$, $2 < h_s/d_{50} < 4$ OK

(5) $L_s = 10 \times h_s = 10 \times 1.86 = 18.6$ ft or $L_s = 3 \times W_o = 3 \times 6 = 18$ ft;
therefore, use $L_s = 18.6$ ft

$L_B = 15 \times h_s = 15 \times 1.86 = 27.9$ 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 4.5-5. Figure 4.5-10 is provided as a convenient form to organize and present the results of riprap basin designs.

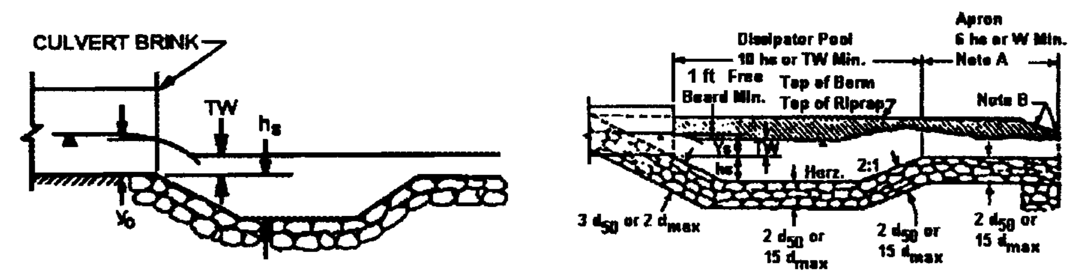
Note: When using the design procedure outlined in this section, it is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the $3 \times d_{50}$ thickness of riprap on the approach and the $2 \times d_{50}$ 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 4.5-10 Riprap Basin Design Form
(Source: USDOT, FHWA, HEC-14, 1983)

4.5.6 Baffled Outlets

4.5.6.1 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 4.5-11. 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.

4.5.6.2 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 4.5-11 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.

$$W/d = 2.88Fr^{0.566} \text{ or } W = 2.88dFr^{0.566} \qquad (4.5.2)$$

Where: W = minimum basin width (ft)
d = depth of incoming flow (ft)
Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

The limits of the W/d ratio are from 3 to 10, which corresponds to Froude numbers 1 and 9. If the basin is much wider than W, flow will pass under the baffle and energy dissipation will not be effective.

(Step 3) Calculate the other basin dimensions as shown in Figure 4.5-11, 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 downstream channel invert.

(Step 6) Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 ft/s flowing full.

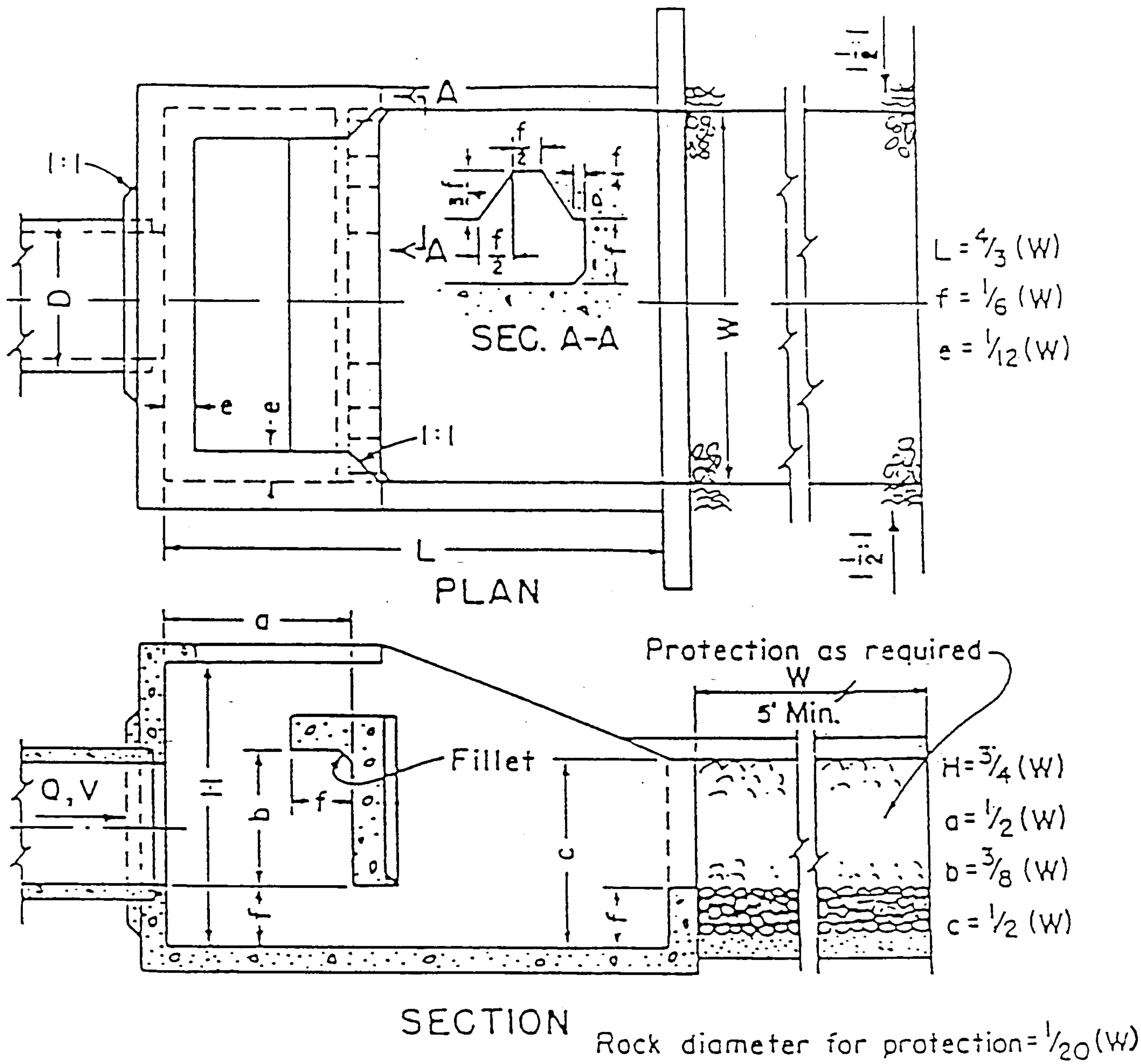


Figure 4.5-11 Schematic of Baffled Outlet
(Source: U.S. Dept. of the Interior, 1978)

(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.

4.5.6.3 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 4.5.2 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 4.5-11):

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 4.5-11.

(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.

References

Federal Highway Administration, 1983. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14.

Federal Highway Administration, 1967. Use of Riprap for Bank Protection. Hydraulic Engineering Circular No. 11.

Searcy, James K., 1967. Use of Riprap for Bank Protection. Federal Highway Administration.

U.S. Department of Interior, Bureau of Reclamation, 1978. Design of Small Canal Structures.

APPENDIX

A

RAINFALL TABLES FOR GEORGIA

Table of Contents

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Table A-1
Albany

Hours	Minutes	Return Period 1	2	5	10	25	50	100
	n	0.8129	0.8464	0.7754	0.7632	0.7315	0.7132	0.7162
	a	56.99	81.06	73.44	79.68	81.36	83.71	94.20
	b	11	15	15	16	16	16	17
Hours	Minutes	Rainfall Intensity						
0.08	5	5.98	6.42	7.20	7.80	8.77	9.55	10.29
	6	5.70	6.16	6.93	7.53	8.48	9.23	9.97
	7	5.44	5.92	6.68	7.28	8.21	8.95	9.67
	8	5.20	5.70	6.46	7.05	7.96	8.68	9.39
	9	4.99	5.50	6.25	6.83	7.72	8.43	9.13
	10	4.80	5.32	6.05	6.63	7.50	8.20	8.89
	11	4.62	5.14	5.87	6.44	7.30	7.98	8.66
	12	4.45	4.98	5.70	6.26	7.11	7.78	8.45
	13	4.30	4.83	5.54	6.10	6.93	7.58	8.24
	14	4.16	4.69	5.40	5.94	6.76	7.40	8.05
0.25	15	4.03	4.56	5.26	5.80	6.60	7.23	7.87
	16	3.91	4.43	5.12	5.66	6.45	7.07	7.70
	17	3.80	4.31	5.00	5.53	6.30	6.92	7.54
	18	3.69	4.20	4.88	5.40	6.17	6.77	7.38
	19	3.59	4.10	4.77	5.28	6.04	6.63	7.23
	20	3.49	4.00	4.66	5.17	5.91	6.50	7.09
	21	3.41	3.90	4.56	5.06	5.80	6.37	6.96
	22	3.32	3.82	4.47	4.96	5.69	6.25	6.83
	23	3.24	3.73	4.38	4.87	5.58	6.14	6.71
	24	3.17	3.65	4.29	4.77	5.48	6.03	6.59
	25	3.09	3.57	4.20	4.68	5.38	5.92	6.48
	26	3.03	3.50	4.12	4.60	5.28	5.82	6.37
	27	2.96	3.43	4.05	4.52	5.19	5.73	6.27
	28	2.90	3.36	3.98	4.44	5.11	5.63	6.17
	29	2.84	3.29	3.91	4.36	5.02	5.54	6.07
0.50	30	2.78	3.23	3.84	4.29	4.94	5.46	5.98
	31	2.73	3.17	3.77	4.22	4.87	5.37	5.89
	32	2.68	3.12	3.71	4.15	4.79	5.29	5.80
	33	2.63	3.06	3.65	4.09	4.72	5.22	5.72
	34	2.58	3.01	3.59	4.02	4.65	5.14	5.64
	35	2.54	2.96	3.54	3.96	4.58	5.07	5.56
	36	2.49	2.91	3.48	3.91	4.52	5.00	5.48
	37	2.45	2.86	3.43	3.85	4.46	4.93	5.41
	38	2.41	2.81	3.38	3.80	4.40	4.87	5.34
	39	2.37	2.77	3.33	3.74	4.34	4.80	5.27
	40	2.33	2.73	3.28	3.69	4.28	4.74	5.21
	41	2.30	2.69	3.24	3.64	4.23	4.68	5.14
	42	2.26	2.65	3.20	3.59	4.17	4.63	5.08
	43	2.23	2.61	3.15	3.55	4.12	4.57	5.02
	44	2.19	2.57	3.11	3.50	4.07	4.52	4.96
0.75	45	2.16	2.53	3.07	3.46	4.02	4.46	4.90
	46	2.13	2.50	3.03	3.42	3.97	4.41	4.85
	47	2.10	2.46	2.99	3.37	3.93	4.36	4.79
	48	2.07	2.43	2.96	3.33	3.88	4.31	4.74
	49	2.04	2.40	2.92	3.29	3.84	4.26	4.69
	50	2.02	2.37	2.89	3.26	3.80	4.22	4.64
	51	1.99	2.34	2.85	3.22	3.75	4.17	4.59
	52	1.96	2.31	2.82	3.18	3.71	4.13	4.54
	53	1.94	2.28	2.79	3.15	3.67	4.09	4.49
	54	1.91	2.25	2.76	3.11	3.64	4.05	4.45
	55	1.89	2.22	2.72	3.08	3.60	4.00	4.40
	56	1.87	2.20	2.69	3.05	3.56	3.96	4.36
	57	1.85	2.17	2.67	3.02	3.53	3.93	4.32
	58	1.82	2.15	2.64	2.98	3.49	3.89	4.28
	59	1.80	2.12	2.61	2.95	3.46	3.85	4.24
1	60	1.78	2.10	2.58	2.92	3.42	3.81	4.20
2	120	1.06	1.22	1.50	1.75	1.98	2.18	2.45
3	180	0.76	0.90	1.13	1.28	1.46	1.62	1.79
6	360	0.44	0.51	0.67	0.75	0.89	0.99	1.11
12	720	0.25	0.31	0.39	0.45	0.53	0.58	0.66
24	1440	0.15	0.18	0.23	0.26	0.31	0.35	0.38

Table A-2
Atlanta

		Return Period						
		1	2	5	10	25	50	100
	n	0.75129	0.8542	0.7846	0.7768	0.7475	0.7519	0.7378
	a	35.11	66.20	62.28	69.74	72.79	83.83	87.36
	b	7	12	12	13	13	14	14
Hours	Minutes	Rainfall Intensity						
0.08	5	5.43	5.89	6.74	7.38	8.39	9.16	9.95
	6	5.11	5.61	6.45	7.08	8.06	8.81	9.58
	7	4.84	5.35	6.18	6.80	7.75	8.50	9.24
	8	4.59	5.12	5.94	6.55	7.48	8.20	8.93
	9	4.37	4.91	5.71	6.32	7.22	7.93	8.64
	10	4.18	4.72	5.51	6.10	6.99	7.68	8.38
	11	4.00	4.55	5.32	5.91	6.77	7.45	8.13
	12	3.84	4.38	5.15	5.72	6.56	7.24	7.90
	13	3.70	4.23	4.98	5.55	6.37	7.03	7.68
	14	3.57	4.09	4.83	5.39	6.20	6.84	7.48
0.25	15	3.44	3.96	4.69	5.24	6.03	6.67	7.28
	16	3.33	3.84	4.56	5.10	5.87	6.50	7.10
	17	3.23	3.73	4.44	4.97	5.73	6.34	6.93
	18	3.13	3.62	4.32	4.84	5.59	6.19	6.77
	19	3.04	3.52	4.21	4.72	5.46	6.05	6.62
	20	2.95	3.43	4.11	4.61	5.33	5.91	6.48
	21	2.87	3.34	4.01	4.51	5.22	5.79	6.34
	22	2.80	3.26	3.92	4.41	5.10	5.67	6.21
	23	2.73	3.18	3.83	4.31	5.00	5.55	6.09
	24	2.66	3.10	3.74	4.22	4.90	5.44	5.97
	25	2.60	3.03	3.66	4.13	4.80	5.33	5.85
	26	2.54	2.96	3.59	4.05	4.71	5.23	5.75
	27	2.48	2.90	3.52	3.97	4.62	5.14	5.64
	28	2.43	2.83	3.45	3.90	4.53	5.05	5.54
	29	2.38	2.77	3.38	3.82	4.45	4.96	5.45
0.50	30	2.33	2.72	3.32	3.75	4.38	4.87	5.36
	31	2.28	2.66	3.26	3.69	4.30	4.79	5.27
	32	2.24	2.61	3.20	3.62	4.23	4.71	5.18
	33	2.20	2.56	3.14	3.56	4.16	4.64	5.10
	34	2.16	2.52	3.09	3.50	4.09	4.56	5.02
	35	2.12	2.47	3.04	3.45	4.03	4.49	4.95
	36	2.08	2.43	2.99	3.39	3.97	4.43	4.87
	37	2.05	2.38	2.94	3.34	3.91	4.36	4.80
	38	2.01	2.34	2.89	3.29	3.85	4.30	4.73
	39	1.98	2.30	2.85	3.24	3.80	4.24	4.67
	40	1.95	2.27	2.81	3.19	3.74	4.18	4.60
	41	1.92	2.23	2.76	3.15	3.69	4.12	4.54
	42	1.89	2.19	2.72	3.10	3.64	4.06	4.48
	43	1.86	2.16	2.68	3.06	3.59	4.01	4.42
	44	1.83	2.13	2.65	3.02	3.54	3.96	4.37
0.75	45	1.80	2.09	2.61	2.98	3.50	3.91	4.31
	46	1.78	2.06	2.58	2.94	3.45	3.86	4.26
	47	1.75	2.03	2.54	2.90	3.41	3.81	4.21
	48	1.73	2.00	2.51	2.86	3.37	3.76	4.16
	49	1.71	1.98	2.48	2.83	3.33	3.72	4.11
	50	1.68	1.95	2.44	2.79	3.29	3.68	4.06
	51	1.66	1.92	2.41	2.76	3.25	3.63	4.02
	52	1.64	1.90	2.38	2.72	3.21	3.59	3.97
	53	1.62	1.87	2.35	2.69	3.18	3.55	3.93
	54	1.60	1.85	2.33	2.66	3.14	3.51	3.88
	55	1.58	1.82	2.30	2.63	3.11	3.47	3.84
	56	1.56	1.80	2.27	2.60	3.07	3.44	3.80
	57	1.54	1.78	2.25	2.57	3.04	3.40	3.76
	58	1.53	1.76	2.22	2.54	3.01	3.36	3.72
	59	1.51	1.74	2.20	2.52	2.98	3.33	3.69
1	60	1.49	1.72	2.17	2.49	2.95	3.30	3.65
2	120	0.96	1.14	1.40	1.58	1.84	2.02	2.21
3	180	0.68	0.81	1.01	1.14	1.32	1.46	1.61
6	360	0.39	0.48	0.60	0.69	0.80	0.90	0.97
12	720	0.23	0.28	0.36	0.41	0.47	0.53	0.58
24	1440	0.14	0.17	0.20	0.23	0.27	0.30	0.33

Table A-3
Athens

		Return Period						
		1	2	5	10	25	50	100
	n	0.6166	0.8407	0.7642	0.7096	0.6796	0.6625	0.6485
	a	19.19	65.12	59.88	54.12	56.50	58.95	61.57
	b	4	13	13	12	12	12	12
Hours	Minutes	Rainfall Intensity						
0.08	5	5.19	5.75	6.60	7.26	8.25	9.04	9.82
	6	4.93	5.49	6.33	6.97	7.94	8.70	9.46
	7	4.69	5.25	6.08	6.70	7.65	8.39	9.13
	8	4.48	5.04	5.85	6.46	7.38	8.10	8.83
	9	4.29	4.84	5.64	6.24	7.14	7.84	8.55
	10	4.12	4.66	5.45	6.03	6.91	7.60	8.29
	11	3.97	4.49	5.27	5.84	6.70	7.37	8.04
	12	3.82	4.34	5.10	5.66	6.50	7.16	7.82
	13	3.69	4.19	4.94	5.50	6.32	6.97	7.61
	14	3.57	4.06	4.80	5.35	6.15	6.78	7.41
0.25	15	3.45	3.94	4.67	5.20	5.99	6.61	7.23
	16	3.35	3.82	4.54	5.06	5.84	6.45	7.05
	17	3.25	3.71	4.42	4.94	5.70	6.30	6.89
	18	3.16	3.61	4.31	4.82	5.57	6.15	6.74
	19	3.07	3.51	4.20	4.70	5.44	6.02	6.59
	20	2.99	3.42	4.10	4.59	5.32	5.89	6.45
	21	2.92	3.33	4.00	4.49	5.20	5.76	6.32
	22	2.85	3.25	3.91	4.39	5.10	5.65	6.19
	23	2.78	3.17	3.83	4.30	4.99	5.53	6.07
	24	2.71	3.10	3.75	4.21	4.90	5.43	5.96
	25	2.65	3.03	3.67	4.13	4.80	5.33	5.85
	26	2.60	2.96	3.60	4.05	4.71	5.23	5.74
	27	2.54	2.90	3.52	3.97	4.63	5.14	5.64
	28	2.49	2.84	3.46	3.90	4.54	5.05	5.55
	29	2.44	2.78	3.39	3.83	4.47	4.96	5.46
0.50	30	2.39	2.72	3.33	3.76	4.39	4.88	5.37
	31	2.35	2.67	3.27	3.70	4.32	4.80	5.28
	32	2.30	2.62	3.21	3.64	4.25	4.73	5.20
	33	2.26	2.57	3.16	3.58	4.18	4.65	5.12
	34	2.22	2.52	3.10	3.52	4.12	4.58	5.05
	35	2.18	2.48	3.05	3.46	4.05	4.51	4.97
	36	2.15	2.43	3.00	3.41	3.99	4.45	4.90
	37	2.11	2.39	2.96	3.36	3.94	4.39	4.83
	38	2.08	2.35	2.91	3.31	3.88	4.32	4.77
	39	2.04	2.31	2.87	3.26	3.83	4.27	4.70
	40	2.01	2.27	2.82	3.21	3.77	4.21	4.64
	41	1.98	2.24	2.78	3.17	3.72	4.15	4.58
	42	1.95	2.20	2.74	3.13	3.67	4.10	4.52
	43	1.92	2.17	2.71	3.08	3.63	4.05	4.47
	44	1.90	2.14	2.67	3.04	3.58	4.00	4.41
0.75	45	1.87	2.11	2.63	3.00	3.53	3.95	4.36
	46	1.84	2.07	2.60	2.96	3.49	3.90	4.31
	47	1.82	2.04	2.56	2.93	3.45	3.85	4.26
	48	1.80	2.02	2.53	2.89	3.41	3.81	4.21
	49	1.77	1.99	2.50	2.86	3.37	3.77	4.16
	50	1.75	1.96	2.47	2.82	3.33	3.72	4.12
	51	1.73	1.93	2.44	2.79	3.29	3.68	4.07
	52	1.71	1.91	2.41	2.76	3.25	3.64	4.03
	53	1.68	1.88	2.38	2.72	3.22	3.60	3.99
	54	1.66	1.86	2.35	2.69	3.18	3.57	3.94
	55	1.64	1.84	2.32	2.66	3.15	3.53	3.90
	56	1.63	1.81	2.30	2.63	3.12	3.49	3.87
	57	1.61	1.79	2.27	2.61	3.08	3.46	3.83
	58	1.59	1.77	2.24	2.58	3.05	3.42	3.79
	59	1.57	1.75	2.22	2.55	3.02	3.39	3.75
1	60	1.55	1.73	2.20	2.52	2.99	3.36	3.72
2	120	0.93	1.09	1.31	1.53	1.75	1.91	2.15
3	180	0.67	0.76	0.98	1.11	1.26	1.41	1.56
6	360	0.37	0.45	0.58	0.67	0.75	0.86	0.93
12	720	0.22	0.27	0.34	0.39	0.46	0.51	0.55
24	1440	0.13	0.15	0.20	0.22	0.26	0.30	0.32

Table A-4
Augusta

Hours	Minutes	1	2	5	10	25	50	100
		Return Period						
	n	0.9624	0.8336	0.8285	0.8154	0.8028	0.8169	0.8106
	a	101.38	68.96	80.74	85.12	92.94	110.05	117.15
	b	16	14	15	15	15	16	16
Hours	Minutes	Rainfall Intensity						
0.08	5	5.41	5.92	6.75	7.40	8.39	9.15	9.93
	6	5.18	5.68	6.48	7.11	8.07	8.81	9.56
	7	4.96	5.45	6.24	6.85	7.77	8.50	9.22
	8	4.76	5.24	6.01	6.60	7.50	8.21	8.91
	9	4.58	5.05	5.80	6.38	7.25	7.94	8.62
	10	4.41	4.88	5.61	6.17	7.01	7.69	8.35
	11	4.25	4.71	5.43	5.97	6.80	7.45	8.10
	12	4.10	4.56	5.26	5.79	6.59	7.23	7.86
	13	3.97	4.42	5.11	5.62	6.40	7.03	7.64
	14	3.84	4.29	4.96	5.46	6.23	6.84	7.44
0.25	15	3.72	4.16	4.82	5.32	6.06	6.66	7.24
	16	3.61	4.05	4.69	5.18	5.90	6.49	7.06
	17	3.50	3.94	4.57	5.04	5.75	6.33	6.88
	18	3.40	3.84	4.46	4.92	5.61	6.17	6.72
	19	3.31	3.74	4.35	4.80	5.48	6.03	6.56
	20	3.22	3.65	4.24	4.69	5.35	5.89	6.41
	21	3.14	3.56	4.15	4.58	5.23	5.76	6.27
	22	3.06	3.48	4.05	4.48	5.12	5.64	6.14
	23	2.98	3.40	3.96	4.38	5.01	5.52	6.01
	24	2.91	3.32	3.88	4.29	4.91	5.41	5.89
	25	2.84	3.25	3.80	4.20	4.81	5.30	5.77
	26	2.78	3.18	3.72	4.12	4.71	5.19	5.66
	27	2.72	3.12	3.65	4.04	4.62	5.10	5.55
	28	2.66	3.06	3.58	3.96	4.54	5.00	5.45
	29	2.60	3.00	3.51	3.89	4.46	4.91	5.35
0.50	30	2.55	2.94	3.45	3.82	4.38	4.82	5.26
	31	2.49	2.89	3.38	3.75	4.30	4.74	5.17
	32	2.44	2.83	3.32	3.69	4.23	4.66	5.08
	33	2.40	2.78	3.27	3.62	4.15	4.58	5.00
	34	2.35	2.74	3.21	3.56	4.09	4.51	4.92
	35	2.30	2.69	3.16	3.50	4.02	4.43	4.84
	36	2.26	2.64	3.11	3.45	3.96	4.36	4.76
	37	2.22	2.60	3.06	3.39	3.90	4.30	4.69
	38	2.18	2.56	3.01	3.34	3.84	4.23	4.62
	39	2.14	2.52	2.96	3.29	3.78	4.17	4.55
	40	2.11	2.48	2.92	3.24	3.72	4.11	4.48
	41	2.07	2.44	2.88	3.20	3.67	4.05	4.42
	42	2.04	2.41	2.83	3.15	3.62	3.99	4.36
	43	2.00	2.37	2.79	3.11	3.57	3.94	4.30
	44	1.97	2.34	2.75	3.06	3.52	3.88	4.24
0.75	45	1.94	2.30	2.72	3.02	3.47	3.83	4.18
	46	1.91	2.27	2.68	2.98	3.43	3.78	4.13
	47	1.88	2.24	2.64	2.94	3.38	3.73	4.08
	48	1.85	2.21	2.61	2.90	3.34	3.68	4.02
	49	1.82	2.18	2.57	2.87	3.30	3.64	3.97
	50	1.80	2.15	2.54	2.83	3.26	3.59	3.92
	51	1.77	2.12	2.51	2.79	3.22	3.55	3.88
	52	1.75	2.10	2.48	2.76	3.18	3.50	3.83
	53	1.72	2.07	2.45	2.73	3.14	3.46	3.79
	54	1.70	2.05	2.42	2.70	3.10	3.42	3.74
	55	1.68	2.02	2.39	2.66	3.07	3.38	3.70
	56	1.65	2.00	2.36	2.63	3.03	3.34	3.66
	57	1.63	1.97	2.33	2.60	3.00	3.31	3.62
	58	1.61	1.95	2.31	2.57	2.97	3.27	3.58
	59	1.59	1.93	2.28	2.55	2.93	3.23	3.54
1	60	1.57	1.91	2.26	2.52	2.90	3.20	3.50
2	120	0.93	1.10	1.39	1.59	1.81	1.99	2.19
3	180	0.68	0.81	1.00	1.16	1.33	1.47	1.60
6	360	0.39	0.48	0.61	0.70	0.81	0.89	0.97
12	720	0.23	0.28	0.36	0.40	0.48	0.52	0.58
24	1440	0.13	0.16	0.20	0.23	0.27	0.30	0.33

Table A-5
Bainbridge

		Return Period						
		1	2	5	10	25	50	100
	n	0.7298	0.9026	0.8229	0.7865	0.7516	0.7493	0.732
	a	41.32	111.46	96.76	93.56	94.05	104.51	106.76
	b	9	18	18	18	18	19	19
Hours	Minutes	Rainfall Intensity						
0.08	5	6.02	6.58	7.33	7.95	8.91	9.66	10.42
	6	5.73	6.33	7.08	7.68	8.63	9.37	10.12
	7	5.46	6.10	6.84	7.44	8.37	9.10	9.83
	8	5.23	5.89	6.63	7.22	8.12	8.84	9.56
	9	5.01	5.69	6.42	7.00	7.90	8.61	9.31
	10	4.82	5.51	6.24	6.81	7.68	8.38	9.08
	11	4.64	5.34	6.06	6.62	7.48	8.17	8.85
	12	4.48	5.17	5.89	6.45	7.30	7.97	8.64
	13	4.33	5.02	5.73	6.28	7.12	7.79	8.45
	14	4.19	4.88	5.59	6.13	6.95	7.61	8.26
0.25	15	4.06	4.75	5.45	5.98	6.79	7.44	8.08
	16	3.94	4.62	5.31	5.84	6.64	7.28	7.91
	17	3.83	4.50	5.19	5.71	6.50	7.13	7.75
	18	3.73	4.39	5.07	5.59	6.36	6.98	7.59
	19	3.63	4.28	4.96	5.47	6.23	6.85	7.45
	20	3.54	4.18	4.85	5.35	6.11	6.71	7.31
	21	3.45	4.08	4.75	5.24	5.99	6.59	7.17
	22	3.37	3.99	4.65	5.14	5.88	6.47	7.04
	23	3.29	3.90	4.56	5.04	5.77	6.35	6.92
	24	3.22	3.82	4.47	4.95	5.67	6.24	6.80
	25	3.15	3.74	4.38	4.86	5.57	6.13	6.69
	26	3.09	3.66	4.30	4.77	5.47	6.03	6.58
	27	3.02	3.59	4.22	4.69	5.38	5.93	6.47
	28	2.96	3.52	4.14	4.61	5.29	5.84	6.37
	29	2.91	3.45	4.07	4.53	5.21	5.75	6.28
0.50	30	2.85	3.39	4.00	4.45	5.12	5.66	6.18
	31	2.80	3.32	3.93	4.38	5.05	5.57	6.09
	32	2.75	3.26	3.87	4.31	4.97	5.49	6.00
	33	2.70	3.21	3.81	4.25	4.90	5.41	5.92
	34	2.66	3.15	3.75	4.18	4.83	5.34	5.84
	35	2.61	3.10	3.69	4.12	4.76	5.26	5.76
	36	2.57	3.04	3.63	4.06	4.69	5.19	5.68
	37	2.53	2.99	3.58	4.00	4.63	5.12	5.61
	38	2.49	2.95	3.52	3.95	4.56	5.05	5.53
	39	2.45	2.90	3.47	3.89	4.50	4.99	5.46
	40	2.41	2.85	3.42	3.84	4.45	4.92	5.40
	41	2.38	2.81	3.38	3.79	4.39	4.86	5.33
	42	2.34	2.77	3.33	3.74	4.33	4.80	5.27
	43	2.31	2.73	3.29	3.69	4.28	4.74	5.20
	44	2.28	2.69	3.24	3.64	4.23	4.69	5.14
0.75	45	2.25	2.65	3.20	3.60	4.18	4.63	5.08
	46	2.22	2.61	3.16	3.55	4.13	4.58	5.03
	47	2.19	2.58	3.12	3.51	4.08	4.53	4.97
	48	2.16	2.54	3.08	3.47	4.03	4.48	4.92
	49	2.13	2.51	3.04	3.43	3.99	4.43	4.86
	50	2.11	2.47	3.00	3.39	3.94	4.38	4.81
	51	2.08	2.44	2.97	3.35	3.90	4.33	4.76
	52	2.06	2.41	2.93	3.31	3.86	4.29	4.71
	53	2.03	2.38	2.90	3.27	3.82	4.24	4.66
	54	2.01	2.35	2.87	3.24	3.78	4.20	4.62
	55	1.99	2.32	2.83	3.20	3.74	4.16	4.57
	56	1.96	2.29	2.80	3.17	3.70	4.11	4.53
	57	1.94	2.26	2.77	3.14	3.66	4.07	4.48
	58	1.92	2.24	2.74	3.10	3.63	4.03	4.44
	59	1.90	2.21	2.71	3.07	3.59	3.99	4.40
1	60	1.88	2.18	2.68	3.04	3.56	3.96	4.36
2	120	1.10	1.30	1.63	1.88	2.12	2.31	2.59
3	180	0.81	0.97	1.18	1.34	1.58	1.74	1.91
6	360	0.48	0.58	0.73	0.83	0.98	1.09	1.20
12	720	0.28	0.35	0.44	0.51	0.59	0.68	0.74
24	1440	0.16	0.20	0.26	0.30	0.35	0.39	0.43

Table A-6
Brunswick

		Return Period						
		1	2	5	10	25	50	100
	n	1.3651	0.9392	0.9606	0.987	1.0276	1.0428	1.0599
	a	1026.50	145.04	194.58	250.55	359.87	433.52	522.14
	b	40	22	26	29	33	35	37
Hours	Minutes	Rainfall Intensity						
0.08	5	5.68	6.56	7.19	7.72	8.56	9.25	9.94
	6	5.51	6.34	6.97	7.50	8.34	9.02	9.69
	7	5.35	6.14	6.77	7.29	8.13	8.79	9.46
	8	5.20	5.95	6.58	7.10	7.92	8.58	9.24
	9	5.06	5.77	6.40	6.91	7.73	8.38	9.02
	10	4.92	5.60	6.23	6.74	7.54	8.18	8.82
	11	4.79	5.44	6.06	6.57	7.37	8.00	8.63
	12	4.66	5.29	5.91	6.41	7.20	7.82	8.44
	13	4.54	5.14	5.76	6.26	7.04	7.65	8.26
	14	4.43	5.01	5.63	6.12	6.88	7.49	8.09
0.25	15	4.32	4.88	5.49	5.98	6.74	7.33	7.93
	16	4.22	4.76	5.37	5.85	6.60	7.18	7.77
	17	4.11	4.65	5.25	5.73	6.46	7.04	7.61
	18	4.02	4.54	5.13	5.61	6.33	6.90	7.47
	19	3.93	4.43	5.02	5.49	6.20	6.77	7.33
	20	3.84	4.33	4.92	5.38	6.08	6.64	7.19
	21	3.75	4.24	4.82	5.27	5.97	6.52	7.06
	22	3.67	4.15	4.72	5.17	5.86	6.40	6.93
	23	3.59	4.06	4.63	5.07	5.75	6.28	6.81
	24	3.51	3.98	4.54	4.98	5.65	6.17	6.69
	25	3.44	3.90	4.46	4.89	5.55	6.06	6.58
	26	3.37	3.82	4.37	4.80	5.45	5.96	6.47
	27	3.30	3.75	4.29	4.71	5.36	5.86	6.36
	28	3.23	3.68	4.22	4.63	5.27	5.76	6.26
	29	3.17	3.61	4.14	4.55	5.18	5.67	6.16
0.50	30	3.11	3.55	4.07	4.48	5.09	5.58	6.06
	31	3.05	3.48	4.00	4.40	5.01	5.49	5.96
	32	2.99	3.42	3.94	4.33	4.93	5.40	5.87
	33	2.94	3.36	3.87	4.26	4.86	5.32	5.78
	34	2.88	3.31	3.81	4.20	4.78	5.24	5.70
	35	2.83	3.25	3.75	4.13	4.71	5.16	5.61
	36	2.78	3.20	3.69	4.07	4.64	5.09	5.53
	37	2.73	3.15	3.64	4.01	4.57	5.01	5.45
	38	2.68	3.10	3.58	3.95	4.51	4.94	5.38
	39	2.64	3.05	3.53	3.89	4.44	4.87	5.30
	40	2.59	3.01	3.48	3.84	4.38	4.80	5.23
	41	2.55	2.96	3.43	3.78	4.32	4.74	5.16
	42	2.50	2.92	3.38	3.73	4.26	4.67	5.09
	43	2.46	2.88	3.33	3.68	4.20	4.61	5.02
	44	2.42	2.84	3.29	3.63	4.15	4.55	4.95
0.75	45	2.38	2.80	3.24	3.58	4.09	4.49	4.89
	46	2.35	2.76	3.20	3.53	4.04	4.43	4.83
	47	2.31	2.72	3.16	3.49	3.99	4.38	4.77
	48	2.27	2.68	3.12	3.44	3.93	4.32	4.71
	49	2.24	2.65	3.08	3.40	3.89	4.27	4.65
	50	2.21	2.61	3.04	3.36	3.84	4.22	4.59
	51	2.17	2.58	3.00	3.32	3.79	4.17	4.54
	52	2.14	2.55	2.96	3.28	3.74	4.12	4.48
	53	2.11	2.51	2.93	3.24	3.70	4.07	4.43
	54	2.08	2.48	2.89	3.20	3.66	4.02	4.38
	55	2.05	2.45	2.86	3.16	3.61	3.97	4.33
	56	2.02	2.42	2.82	3.12	3.57	3.93	4.28
	57	1.99	2.39	2.79	3.09	3.53	3.88	4.23
	58	1.96	2.37	2.76	3.05	3.49	3.84	4.18
	59	1.94	2.34	2.73	3.02	3.45	3.80	4.14
1	60	1.91	2.31	2.70	2.98	3.41	3.75	4.09
2	120	1.15	1.33	1.65	1.85	2.18	2.35	2.60
3	180	0.83	0.96	1.22	1.37	1.59	1.77	2.00
6	360	0.49	0.58	0.74	0.85	1.00	1.08	1.18
12	720	0.29	0.35	0.45	0.52	0.59	0.67	0.74
24	1440	0.18	0.21	0.26	0.30	0.34	0.40	0.45

Table A-7
Columbus

Hours	Minutes	Return Period						
		1	2	5	10	25	50	100
	n	0.7101	0.8694	0.7965	0.7638	0.7534	0.7151	0.72
	a	33.80	80.37	73.21	72.53	82.30	77.31	87.89
	b	8	14	14	14	15	14	15
Hours	Minutes	Rainfall Intensity						
0.08	5	5.47	6.21	7.02	7.65	8.61	9.41	10.17
	6	5.19	5.94	6.74	7.36	8.30	9.08	9.82
	7	4.94	5.70	6.48	7.09	8.02	8.76	9.49
	8	4.72	5.47	6.24	6.84	7.75	8.48	9.19
	9	4.52	5.26	6.03	6.61	7.51	8.21	8.92
	10	4.34	5.07	5.82	6.40	7.28	7.97	8.66
	11	4.18	4.89	5.64	6.21	7.07	7.74	8.42
	12	4.03	4.73	5.47	6.02	6.87	7.52	8.19
	13	3.89	4.58	5.30	5.85	6.68	7.32	7.98
	14	3.76	4.44	5.15	5.69	6.51	7.13	7.78
0.25	15	3.65	4.30	5.01	5.54	6.35	6.96	7.59
	16	3.54	4.18	4.88	5.40	6.19	6.79	7.42
	17	3.44	4.06	4.75	5.26	6.04	6.63	7.25
	18	3.34	3.95	4.63	5.14	5.91	6.48	7.09
	19	3.25	3.85	4.52	5.02	5.77	6.34	6.94
	20	3.17	3.75	4.41	4.91	5.65	6.21	6.79
	21	3.09	3.65	4.31	4.80	5.53	6.08	6.66
	22	3.02	3.57	4.22	4.70	5.42	5.96	6.53
	23	2.95	3.48	4.13	4.60	5.31	5.85	6.40
	24	2.88	3.40	4.04	4.51	5.21	5.73	6.29
	25	2.82	3.33	3.96	4.42	5.11	5.63	6.17
	26	2.76	3.25	3.88	4.33	5.01	5.53	6.06
	27	2.71	3.18	3.80	4.25	4.92	5.43	5.96
	28	2.65	3.12	3.73	4.17	4.84	5.34	5.86
	29	2.60	3.05	3.66	4.10	4.75	5.25	5.76
0.50	30	2.55	2.99	3.59	4.03	4.67	5.16	5.67
	31	2.51	2.94	3.53	3.96	4.60	5.08	5.58
	32	2.46	2.88	3.47	3.89	4.52	5.00	5.50
	33	2.42	2.83	3.41	3.83	4.45	4.93	5.41
	34	2.38	2.78	3.35	3.77	4.38	4.85	5.33
	35	2.34	2.73	3.30	3.71	4.32	4.78	5.26
	36	2.30	2.68	3.25	3.65	4.25	4.71	5.18
	37	2.26	2.63	3.20	3.60	4.19	4.65	5.11
	38	2.23	2.59	3.15	3.55	4.13	4.58	5.04
	39	2.20	2.55	3.10	3.50	4.07	4.52	4.97
	40	2.16	2.51	3.05	3.45	4.02	4.46	4.91
	41	2.13	2.47	3.01	3.40	3.96	4.40	4.84
	42	2.10	2.43	2.97	3.35	3.91	4.35	4.78
	43	2.07	2.39	2.92	3.31	3.86	4.29	4.72
	44	2.04	2.36	2.88	3.26	3.81	4.24	4.67
0.75	45	2.02	2.32	2.85	3.22	3.76	4.19	4.61
	46	1.99	2.29	2.81	3.18	3.72	4.14	4.55
	47	1.96	2.25	2.77	3.14	3.67	4.09	4.50
	48	1.94	2.22	2.74	3.10	3.63	4.04	4.45
	49	1.91	2.19	2.70	3.06	3.59	3.99	4.40
	50	1.89	2.16	2.67	3.03	3.54	3.95	4.35
	51	1.87	2.13	2.63	2.99	3.50	3.91	4.30
	52	1.85	2.10	2.60	2.96	3.46	3.86	4.26
	53	1.82	2.08	2.57	2.92	3.43	3.82	4.21
	54	1.80	2.05	2.54	2.89	3.39	3.78	4.17
	55	1.78	2.02	2.51	2.86	3.35	3.74	4.12
	56	1.76	2.00	2.48	2.83	3.32	3.71	4.08
	57	1.74	1.98	2.46	2.80	3.28	3.67	4.04
	58	1.73	1.95	2.43	2.77	3.25	3.63	4.00
	59	1.71	1.93	2.40	2.74	3.21	3.60	3.96
1	60	1.69	1.91	2.38	2.71	3.18	3.56	3.93
2	120	1.03	1.18	1.46	1.69	1.92	2.12	2.37
3	180	0.72	0.85	1.09	1.23	1.41	1.55	1.73
6	360	0.43	0.50	0.65	0.74	0.83	0.96	1.06
12	720	0.25	0.30	0.38	0.44	0.51	0.57	0.63
24	1440	0.15	0.17	0.22	0.26	0.30	0.33	0.37

Table A-8
Macon

		Return Period						
		1	2	5	10	25	50	100
	n	0.8658	0.8379	0.7639	0.7309	0.6997	0.6818	0.6864
	a	63.20	69.07	62.98	62.45	64.40	66.67	75.72
	b	12	13	13	13	13	13	14
Hours	Minutes	Rainfall Intensity						
0.08	5	5.44	6.13	6.92	7.55	8.52	9.29	10.03
	6	5.18	5.86	6.64	7.26	8.21	8.96	9.69
	7	4.94	5.61	6.39	6.99	7.92	8.65	9.37
	8	4.72	5.39	6.15	6.75	7.65	8.37	9.07
	9	4.53	5.18	5.94	6.52	7.41	8.10	8.80
	10	4.35	4.99	5.74	6.31	7.18	7.86	8.55
	11	4.19	4.82	5.56	6.12	6.97	7.64	8.31
	12	4.03	4.66	5.39	5.94	6.77	7.43	8.09
	13	3.89	4.51	5.23	5.77	6.59	7.23	7.88
	14	3.76	4.36	5.08	5.62	6.42	7.05	7.69
0.25	15	3.64	4.23	4.94	5.47	6.26	6.88	7.51
	16	3.53	4.11	4.81	5.33	6.10	6.71	7.33
	17	3.42	4.00	4.69	5.20	5.96	6.56	7.17
	18	3.33	3.89	4.57	5.08	5.83	6.41	7.02
	19	3.23	3.79	4.46	4.96	5.70	6.28	6.87
	20	3.14	3.69	4.36	4.85	5.58	6.15	6.73
	21	3.06	3.60	4.26	4.74	5.46	6.02	6.60
	22	2.98	3.51	4.17	4.64	5.35	5.91	6.47
	23	2.91	3.43	4.08	4.55	5.25	5.79	6.35
	24	2.84	3.35	3.99	4.46	5.15	5.69	6.24
	25	2.77	3.28	3.91	4.37	5.05	5.58	6.13
	26	2.71	3.21	3.84	4.29	4.96	5.49	6.02
	27	2.65	3.14	3.76	4.21	4.87	5.39	5.92
	28	2.59	3.08	3.69	4.14	4.79	5.30	5.82
	29	2.54	3.01	3.62	4.07	4.71	5.22	5.73
0.50	30	2.48	2.96	3.56	4.00	4.63	5.13	5.64
	31	2.43	2.90	3.50	3.93	4.56	5.05	5.55
	32	2.39	2.85	3.44	3.87	4.49	4.98	5.47
	33	2.34	2.79	3.38	3.80	4.42	4.90	5.39
	34	2.30	2.74	3.33	3.74	4.35	4.83	5.31
	35	2.25	2.70	3.27	3.69	4.29	4.76	5.24
	36	2.21	2.65	3.22	3.63	4.23	4.69	5.17
	37	2.17	2.60	3.17	3.58	4.17	4.63	5.10
	38	2.14	2.56	3.12	3.53	4.11	4.57	5.03
	39	2.10	2.52	3.08	3.48	4.06	4.51	4.96
	40	2.07	2.48	3.03	3.43	4.00	4.45	4.90
	41	2.03	2.44	2.99	3.38	3.95	4.39	4.84
	42	2.00	2.40	2.95	3.34	3.90	4.34	4.78
	43	1.97	2.37	2.91	3.29	3.85	4.29	4.72
	44	1.94	2.33	2.87	3.25	3.80	4.24	4.66
0.75	45	1.91	2.30	2.83	3.21	3.76	4.19	4.61
	46	1.88	2.27	2.80	3.17	3.71	4.14	4.56
	47	1.85	2.24	2.76	3.13	3.67	4.09	4.51
	48	1.82	2.20	2.73	3.09	3.63	4.04	4.46
	49	1.80	2.18	2.69	3.06	3.59	4.00	4.41
	50	1.77	2.15	2.66	3.02	3.55	3.96	4.36
	51	1.75	2.12	2.63	2.99	3.51	3.91	4.31
	52	1.73	2.09	2.60	2.95	3.47	3.87	4.27
	53	1.70	2.06	2.57	2.92	3.43	3.83	4.23
	54	1.68	2.04	2.54	2.89	3.40	3.79	4.18
	55	1.66	2.01	2.51	2.86	3.36	3.76	4.14
	56	1.64	1.99	2.48	2.83	3.33	3.72	4.10
	57	1.62	1.96	2.45	2.80	3.29	3.68	4.06
	58	1.60	1.94	2.43	2.77	3.26	3.65	4.02
	59	1.58	1.92	2.40	2.74	3.23	3.61	3.98
1	60	1.56	1.90	2.38	2.71	3.20	3.58	3.95
2	120	0.99	1.16	1.43	1.64	1.90	2.09	2.29
3	180	0.71	0.84	1.03	1.17	1.33	1.48	1.61
6	360	0.41	0.50	0.61	0.70	0.82	0.91	0.99
12	720	0.24	0.29	0.36	0.41	0.48	0.54	0.59
24	1440	0.14	0.17	0.21	0.24	0.28	0.32	0.34

Table A-9 Metro Chattanooga								
		Return Period						
		1	2	5	10	25	50	100
	N	0.8326	0.8531	0.8275	0.8051	0.7844	0.7726	0.7852
	A	46.13	62.15	70.63	73.41	79.26	84.31	99.21
	B	10	12	13	13	13	13	14
Hours	Minutes	Rainfall Intensity						
0.08	5	4.84	5.54	6.46	7.16	8.21	9.04	9.83
	6	4.59	5.28	6.18	6.86	7.87	8.67	9.44
	7	4.36	5.04	5.92	6.58	7.56	8.33	9.09
	8	4.16	4.83	5.69	6.33	7.28	8.02	8.76
	9	3.97	4.63	5.47	6.09	7.02	7.74	8.46
	10	3.81	4.45	5.27	5.88	6.78	7.48	8.18
	11	3.66	4.28	5.09	5.68	6.55	7.24	7.92
	12	3.52	4.13	4.92	5.50	6.35	7.01	7.68
	13	3.39	3.99	4.77	5.33	6.15	6.80	7.46
	14	3.27	3.86	4.62	5.17	5.97	6.61	7.25
0.25	15	3.16	3.74	4.48	5.02	5.81	6.42	7.05
	16	3.06	3.62	4.35	4.88	5.65	6.25	6.87
	17	2.97	3.51	4.23	4.75	5.50	6.09	6.69
	18	2.88	3.41	4.12	4.62	5.36	5.94	6.53
	19	2.79	3.32	4.01	4.51	5.23	5.79	6.37
	20	2.72	3.23	3.91	4.40	5.10	5.66	6.22
	21	2.64	3.15	3.82	4.29	4.99	5.53	6.08
	22	2.57	3.07	3.73	4.19	4.87	5.41	5.95
	23	2.51	2.99	3.64	4.10	4.77	5.29	5.82
	24	2.45	2.92	3.56	4.01	4.67	5.18	5.70
	25	2.39	2.85	3.48	3.92	4.57	5.07	5.59
	26	2.33	2.79	3.41	3.84	4.48	4.97	5.48
	27	2.28	2.73	3.34	3.77	4.39	4.88	5.37
	28	2.23	2.67	3.27	3.69	4.31	4.78	5.27
	29	2.18	2.62	3.20	3.62	4.22	4.70	5.18
0.50	30	2.14	2.56	3.14	3.55	4.15	4.61	5.08
	31	2.09	2.51	3.08	3.49	4.07	4.53	4.99
	32	2.05	2.46	3.03	3.43	4.00	4.45	4.91
	33	2.01	2.42	2.97	3.37	3.93	4.38	4.83
	34	1.98	2.37	2.92	3.31	3.87	4.30	4.75
	35	1.94	2.33	2.87	3.25	3.80	4.24	4.67
	36	1.90	2.29	2.82	3.20	3.74	4.17	4.60
	37	1.87	2.25	2.77	3.15	3.68	4.10	4.53
	38	1.84	2.21	2.73	3.10	3.63	4.04	4.46
	39	1.81	2.17	2.69	3.05	3.57	3.98	4.39
	40	1.78	2.14	2.64	3.00	3.52	3.92	4.33
	41	1.75	2.10	2.60	2.96	3.47	3.87	4.27
	42	1.72	2.07	2.56	2.91	3.42	3.81	4.21
	43	1.69	2.04	2.53	2.87	3.37	3.76	4.15
	44	1.67	2.00	2.49	2.83	3.32	3.71	4.09
0.75	45	1.64	1.97	2.45	2.79	3.28	3.66	4.04
	46	1.62	1.95	2.42	2.75	3.24	3.61	3.98
	47	1.59	1.92	2.39	2.72	3.19	3.56	3.93
	48	1.57	1.89	2.35	2.68	3.15	3.52	3.88
	49	1.55	1.86	2.32	2.65	3.11	3.48	3.84
	50	1.53	1.84	2.29	2.61	3.07	3.43	3.79
	51	1.50	1.81	2.26	2.58	3.04	3.39	3.74
	52	1.48	1.79	2.23	2.55	3.00	3.35	3.70
	53	1.46	1.77	2.20	2.52	2.96	3.31	3.65
	54	1.45	1.74	2.18	2.49	2.93	3.27	3.61
	55	1.43	1.72	2.15	2.46	2.89	3.24	3.57
	56	1.41	1.70	2.13	2.43	2.86	3.20	3.53
	57	1.39	1.68	2.10	2.40	2.83	3.16	3.49
	58	1.37	1.66	2.08	2.37	2.80	3.13	3.45
	59	1.36	1.64	2.05	2.35	2.77	3.10	3.42
1	60	1.34	1.62	2.03	2.32	2.74	3.06	3.38
2	120	0.88	1.04	1.26	1.45	1.65	1.83	2.02
3	180	0.63	0.75	0.93	1.06	1.24	1.36	1.50
6	360	0.38	0.45	0.56	0.64	0.73	0.82	0.89
12	720	0.22	0.27	0.33	0.38	0.44	0.49	0.54
24	1440	0.13	0.15	0.19	0.22	0.25	0.28	0.30

Table A-10
Peachtree City

Hours	Minutes	Return Period 1	2	5	10	25	50	100
	n	0.7669	0.8184	0.7769	0.7471	0.7191	0.7243	0.7107
	a	38.81	57.93	61.46	61.89	64.80	74.70	77.93
	b	8	11	12	12	12	13	13
Hours	Minutes	Rainfall Intensity						
0.08	5	5.43	5.99	6.80	7.45	8.45	9.21	9.99
	6	5.13	5.70	6.51	7.14	8.11	8.85	9.61
	7	4.86	5.44	6.24	6.86	7.80	8.53	9.27
	8	4.63	5.20	6.00	6.60	7.52	8.23	8.95
	9	4.42	4.99	5.77	6.36	7.26	7.96	8.66
	10	4.23	4.79	5.57	6.15	7.02	7.71	8.39
	11	4.06	4.62	5.38	5.95	6.80	7.48	8.14
	12	3.90	4.45	5.20	5.76	6.59	7.26	7.91
	13	3.76	4.30	5.04	5.59	6.40	7.05	7.69
	14	3.63	4.16	4.89	5.43	6.22	6.86	7.49
0.25	15	3.50	4.03	4.75	5.28	6.06	6.69	7.30
	16	3.39	3.90	4.62	5.13	5.90	6.52	7.12
	17	3.29	3.79	4.49	5.00	5.75	6.36	6.95
	18	3.19	3.68	4.38	4.88	5.61	6.21	6.79
	19	3.10	3.58	4.27	4.76	5.48	6.07	6.64
	20	3.01	3.49	4.16	4.65	5.36	5.94	6.49
	21	2.93	3.40	4.06	4.54	5.24	5.81	6.36
	22	2.86	3.31	3.97	4.44	5.13	5.69	6.23
	23	2.79	3.23	3.88	4.35	5.03	5.57	6.10
	24	2.72	3.16	3.80	4.25	4.92	5.46	5.99
	25	2.66	3.08	3.72	4.17	4.83	5.36	5.87
	26	2.60	3.02	3.64	4.09	4.74	5.26	5.77
	27	2.54	2.95	3.57	4.01	4.65	5.16	5.66
	28	2.49	2.89	3.50	3.93	4.57	5.07	5.57
	29	2.43	2.83	3.43	3.86	4.49	4.98	5.47
0.50	30	2.38	2.77	3.37	3.79	4.41	4.90	5.38
	31	2.34	2.72	3.31	3.73	4.33	4.82	5.29
	32	2.29	2.67	3.25	3.66	4.26	4.74	5.21
	33	2.25	2.62	3.19	3.60	4.19	4.67	5.13
	34	2.21	2.57	3.14	3.54	4.13	4.59	5.05
	35	2.17	2.52	3.09	3.49	4.07	4.52	4.98
	36	2.13	2.48	3.04	3.43	4.00	4.46	4.90
	37	2.09	2.44	2.99	3.38	3.95	4.39	4.83
	38	2.06	2.40	2.94	3.33	3.89	4.33	4.77
	39	2.03	2.36	2.90	3.28	3.83	4.27	4.70
	40	1.99	2.32	2.85	3.23	3.78	4.21	4.64
	41	1.96	2.28	2.81	3.19	3.73	4.15	4.58
	42	1.93	2.25	2.77	3.14	3.68	4.10	4.52
	43	1.90	2.21	2.73	3.10	3.63	4.05	4.46
	44	1.87	2.18	2.69	3.06	3.58	4.00	4.40
0.75	45	1.85	2.15	2.66	3.02	3.54	3.95	4.35
	46	1.82	2.12	2.62	2.98	3.49	3.90	4.30
	47	1.80	2.09	2.59	2.94	3.45	3.85	4.25
	48	1.77	2.06	2.55	2.90	3.41	3.80	4.20
	49	1.75	2.03	2.52	2.87	3.37	3.76	4.15
	50	1.72	2.00	2.49	2.83	3.33	3.72	4.10
	51	1.70	1.98	2.46	2.80	3.29	3.67	4.06
	52	1.68	1.95	2.43	2.77	3.26	3.63	4.01
	53	1.66	1.93	2.40	2.74	3.22	3.59	3.97
	54	1.64	1.90	2.37	2.71	3.18	3.55	3.93
	55	1.62	1.88	2.34	2.68	3.15	3.52	3.88
	56	1.60	1.86	2.32	2.65	3.12	3.48	3.84
	57	1.58	1.83	2.29	2.62	3.08	3.44	3.81
	58	1.56	1.81	2.27	2.59	3.05	3.41	3.77
	59	1.54	1.79	2.24	2.56	3.02	3.37	3.73
1	60	1.53	1.77	2.22	2.54	2.99	3.34	3.69
2	120	0.97	1.19	1.44	1.60	1.85	2.07	2.24
3	180	0.69	0.82	1.02	1.16	1.33	1.47	1.62
6	360	0.40	0.49	0.61	0.70	0.81	0.91	0.98
12	720	0.24	0.29	0.36	0.42	0.48	0.53	0.58
24	1440	0.14	0.17	0.21	0.24	0.28	0.30	0.33

Table A-11
Rome

		Return Period						
		1	2	5	10	25	50	100
	N	0.7332	0.8496	0.8975	0.9424	0.9846	1.0341	1.0218
	A	30.17	63.48	102.01	146.36	214.67	308.98	323.71
	B	6	12	16	19	22	25	25
Hours	Minutes	Rainfall Intensity						
0.08	5	5.20	5.72	6.64	7.32	8.36	9.17	10.02
	6	4.88	5.45	6.36	7.05	8.07	8.87	9.69
	7	4.60	5.20	6.12	6.79	7.80	8.58	9.38
	8	4.36	4.98	5.89	6.55	7.54	8.31	9.09
	9	4.14	4.78	5.67	6.33	7.30	8.06	8.82
	10	3.95	4.59	5.48	6.13	7.08	7.82	8.56
	11	3.78	4.42	5.30	5.93	6.86	7.60	8.32
	12	3.63	4.27	5.13	5.75	6.67	7.38	8.09
	13	3.48	4.12	4.97	5.58	6.48	7.18	7.87
	14	3.36	3.99	4.82	5.42	6.30	6.99	7.66
0.25	15	3.24	3.86	4.68	5.27	6.13	6.81	7.47
	16	3.13	3.74	4.55	5.13	5.97	6.64	7.28
	17	3.03	3.63	4.42	5.00	5.82	6.48	7.10
	18	2.94	3.53	4.31	4.87	5.68	6.32	6.93
	19	2.85	3.43	4.20	4.75	5.54	6.17	6.77
	20	2.77	3.34	4.09	4.63	5.41	6.03	6.62
	21	2.69	3.25	3.99	4.52	5.29	5.90	6.47
	22	2.62	3.17	3.90	4.42	5.17	5.77	6.33
	23	2.56	3.10	3.81	4.32	5.06	5.64	6.20
	24	2.49	3.02	3.72	4.23	4.95	5.52	6.07
	25	2.43	2.95	3.64	4.14	4.85	5.41	5.94
	26	2.38	2.89	3.56	4.05	4.75	5.30	5.83
	27	2.32	2.82	3.49	3.97	4.65	5.19	5.71
	28	2.27	2.76	3.42	3.89	4.56	5.09	5.60
	29	2.23	2.71	3.35	3.81	4.47	4.99	5.49
0.50	30	2.18	2.65	3.28	3.74	4.39	4.90	5.39
	31	2.14	2.60	3.22	3.67	4.31	4.81	5.29
	32	2.10	2.55	3.16	3.60	4.23	4.72	5.20
	33	2.06	2.50	3.10	3.53	4.15	4.64	5.11
	34	2.02	2.45	3.05	3.47	4.08	4.56	5.02
	35	1.98	2.41	2.99	3.41	4.01	4.48	4.93
	36	1.95	2.37	2.94	3.35	3.94	4.40	4.85
	37	1.91	2.33	2.89	3.29	3.87	4.33	4.77
	38	1.88	2.29	2.84	3.24	3.81	4.26	4.69
	39	1.85	2.25	2.80	3.19	3.75	4.19	4.62
	40	1.82	2.21	2.75	3.14	3.69	4.12	4.55
	41	1.79	2.18	2.71	3.09	3.63	4.06	4.48
	42	1.77	2.14	2.67	3.04	3.58	4.00	4.41
	43	1.74	2.11	2.63	2.99	3.52	3.94	4.34
	44	1.71	2.08	2.59	2.95	3.47	3.88	4.28
0.75	45	1.69	2.05	2.55	2.91	3.42	3.82	4.21
	46	1.67	2.02	2.51	2.86	3.37	3.76	4.15
	47	1.64	1.99	2.48	2.82	3.32	3.71	4.10
	48	1.62	1.96	2.44	2.78	3.27	3.66	4.04
	49	1.60	1.93	2.41	2.74	3.23	3.61	3.98
	50	1.58	1.90	2.37	2.71	3.18	3.56	3.93
	51	1.56	1.88	2.34	2.67	3.14	3.51	3.88
	52	1.54	1.85	2.31	2.63	3.10	3.46	3.82
	53	1.52	1.83	2.28	2.60	3.06	3.42	3.77
	54	1.50	1.81	2.25	2.57	3.02	3.37	3.72
	55	1.48	1.78	2.22	2.53	2.98	3.33	3.68
	56	1.46	1.76	2.20	2.50	2.94	3.28	3.63
	57	1.45	1.74	2.17	2.47	2.91	3.24	3.59
	58	1.43	1.72	2.14	2.44	2.87	3.20	3.54
	59	1.41	1.70	2.12	2.41	2.84	3.16	3.50
1	60	1.40	1.68	2.09	2.38	2.80	3.12	3.46
2	120	0.93	1.10	1.32	1.51	1.74	1.89	2.12
3	180	0.67	0.80	0.97	1.10	1.27	1.40	1.54
6	360	0.39	0.46	0.58	0.66	0.76	0.85	0.92
12	720	0.23	0.27	0.35	0.40	0.46	0.51	0.56
24	1440	0.13	0.16	0.20	0.23	0.27	0.30	0.33

Table A-12
Roswell

Hours	Minutes	1	2	5	10	25	50	100
		Return Period						
	n	0.7706	0.8603	0.8003	0.7739	0.771	0.7564	0.7445
	a	36.65	69.23	67.01	68.34	80.18	84.20	88.39
	b	7	13	13	13	14	14	14
		Rainfall Intensity						
0.08	5	5.40	5.76	6.63	7.30	8.28	9.08	9.87
	6	5.08	5.50	6.35	7.00	7.96	8.73	9.50
	7	4.80	5.26	6.09	6.73	7.67	8.42	9.16
	8	4.55	5.04	5.86	6.48	7.40	8.13	8.85
	9	4.33	4.85	5.65	6.25	7.15	7.86	8.56
	10	4.13	4.66	5.45	6.04	6.92	7.61	8.29
	11	3.95	4.50	5.27	5.84	6.70	7.38	8.05
	12	3.79	4.34	5.10	5.66	6.50	7.16	7.81
	13	3.64	4.20	4.94	5.49	6.32	6.96	7.60
	14	3.51	4.06	4.79	5.33	6.14	6.77	7.40
0.25	15	3.39	3.94	4.66	5.18	5.98	6.59	7.20
	16	3.27	3.82	4.53	5.04	5.82	6.43	7.03
	17	3.17	3.71	4.41	4.91	5.68	6.27	6.86
	18	3.07	3.61	4.29	4.79	5.54	6.12	6.70
	19	2.98	3.51	4.18	4.67	5.41	5.98	6.54
	20	2.89	3.42	4.08	4.56	5.29	5.85	6.40
	21	2.81	3.33	3.99	4.46	5.17	5.72	6.26
	22	2.74	3.25	3.89	4.36	5.06	5.60	6.13
	23	2.67	3.17	3.81	4.27	4.95	5.48	6.01
	24	2.60	3.10	3.72	4.18	4.85	5.37	5.89
	25	2.54	3.03	3.65	4.09	4.76	5.27	5.78
	26	2.48	2.96	3.57	4.01	4.67	5.17	5.67
	27	2.42	2.90	3.50	3.93	4.58	5.07	5.57
	28	2.37	2.84	3.43	3.86	4.49	4.98	5.47
	29	2.32	2.78	3.37	3.79	4.41	4.89	5.37
0.50	30	2.27	2.72	3.30	3.72	4.33	4.81	5.28
	31	2.22	2.67	3.24	3.65	4.26	4.73	5.19
	32	2.18	2.62	3.18	3.59	4.19	4.65	5.11
	33	2.14	2.57	3.13	3.53	4.12	4.58	5.03
	34	2.10	2.52	3.08	3.47	4.05	4.50	4.95
	35	2.06	2.48	3.02	3.42	3.99	4.43	4.88
	36	2.02	2.43	2.97	3.36	3.93	4.37	4.80
	37	1.98	2.39	2.93	3.31	3.87	4.30	4.73
	38	1.95	2.35	2.88	3.26	3.81	4.24	4.66
	39	1.92	2.31	2.84	3.21	3.76	4.18	4.60
	40	1.89	2.27	2.79	3.16	3.70	4.12	4.54
	41	1.86	2.24	2.75	3.12	3.65	4.06	4.47
	42	1.83	2.20	2.71	3.07	3.60	4.01	4.41
	43	1.80	2.17	2.67	3.03	3.55	3.95	4.36
	44	1.77	2.14	2.64	2.99	3.50	3.90	4.30
0.75	45	1.74	2.10	2.60	2.95	3.46	3.85	4.25
	46	1.72	2.07	2.56	2.91	3.41	3.80	4.19
	47	1.69	2.04	2.53	2.87	3.37	3.76	4.14
	48	1.67	2.02	2.50	2.84	3.33	3.71	4.09
	49	1.65	1.99	2.46	2.80	3.29	3.67	4.04
	50	1.63	1.96	2.43	2.77	3.25	3.62	4.00
	51	1.60	1.93	2.40	2.73	3.21	3.58	3.95
	52	1.58	1.91	2.37	2.70	3.17	3.54	3.91
	53	1.56	1.88	2.34	2.67	3.13	3.50	3.86
	54	1.54	1.86	2.32	2.64	3.10	3.46	3.82
	55	1.52	1.84	2.29	2.61	3.06	3.42	3.78
	56	1.50	1.81	2.26	2.58	3.03	3.39	3.74
	57	1.49	1.79	2.24	2.55	3.00	3.35	3.70
	58	1.47	1.77	2.21	2.52	2.97	3.31	3.66
	59	1.45	1.75	2.19	2.50	2.93	3.28	3.62
1	60	1.44	1.73	2.16	2.47	2.90	3.25	3.59
2	120	0.95	1.13	1.38	1.56	1.81	1.98	2.17
3	180	0.68	0.81	0.99	1.11	1.31	1.45	1.59
6	360	0.38	0.47	0.59	0.68	0.79	0.89	0.96
12	720	0.24	0.28	0.36	0.41	0.47	0.52	0.57
24	1440	0.14	0.16	0.20	0.23	0.27	0.30	0.32

Table A-13
Savannah

		Return Period						
		1	2	5	10	25	50	100
	N	0.7585	0.9086	0.8696	0.8401	0.8597	0.8619	0.8671
	A	47.79	117.57	125.46	126.12	167.17	191.57	220.00
	B	12	20	23	24	28	30	32
Hours	Minutes	Rainfall Intensity						
0.08	5	5.57	6.31	6.92	7.45	8.27	8.94	9.61
	6	5.34	6.09	6.71	7.24	8.06	8.73	9.39
	7	5.12	5.88	6.52	7.04	7.86	8.52	9.18
	8	4.93	5.69	6.33	6.86	7.68	8.33	8.98
	9	4.75	5.51	6.16	6.68	7.50	8.15	8.79
	10	4.58	5.35	6.00	6.52	7.33	7.97	8.61
	11	4.43	5.19	5.85	6.36	7.17	7.80	8.43
	12	4.29	5.04	5.70	6.21	7.01	7.64	8.27
	13	4.16	4.90	5.56	6.07	6.86	7.49	8.11
	14	4.04	4.77	5.43	5.94	6.72	7.34	7.96
0.25	15	3.92	4.65	5.31	5.81	6.59	7.20	7.81
	16	3.82	4.53	5.19	5.69	6.46	7.07	7.67
	17	3.72	4.42	5.07	5.57	6.34	6.94	7.53
	18	3.62	4.31	4.97	5.46	6.22	6.81	7.40
	19	3.53	4.21	4.86	5.35	6.10	6.69	7.27
	20	3.45	4.12	4.77	5.25	5.99	6.58	7.15
	21	3.37	4.03	4.67	5.15	5.89	6.46	7.04
	22	3.29	3.94	4.58	5.06	5.79	6.36	6.92
	23	3.22	3.86	4.49	4.97	5.69	6.25	6.81
	24	3.15	3.78	4.41	4.88	5.60	6.15	6.71
	25	3.09	3.70	4.33	4.80	5.50	6.06	6.61
	26	3.03	3.63	4.25	4.71	5.42	5.96	6.51
	27	2.97	3.56	4.18	4.64	5.33	5.87	6.41
	28	2.91	3.49	4.11	4.56	5.25	5.79	6.32
	29	2.86	3.42	4.04	4.49	5.17	5.70	6.23
0.50	30	2.81	3.36	3.97	4.42	5.09	5.62	6.14
	31	2.76	3.30	3.91	4.35	5.02	5.54	6.06
	32	2.71	3.24	3.85	4.29	4.95	5.46	5.97
	33	2.66	3.19	3.79	4.22	4.88	5.39	5.89
	34	2.62	3.13	3.73	4.16	4.81	5.31	5.82
	35	2.58	3.08	3.67	4.10	4.74	5.24	5.74
	36	2.54	3.03	3.62	4.04	4.68	5.18	5.67
	37	2.50	2.98	3.57	3.99	4.62	5.11	5.60
	38	2.46	2.94	3.52	3.93	4.56	5.04	5.53
	39	2.42	2.89	3.47	3.88	4.50	4.98	5.46
	40	2.39	2.85	3.42	3.83	4.44	4.92	5.39
	41	2.35	2.81	3.37	3.78	4.39	4.86	5.33
	42	2.32	2.76	3.33	3.73	4.33	4.80	5.27
	43	2.29	2.72	3.28	3.69	4.28	4.75	5.21
	44	2.26	2.69	3.24	3.64	4.23	4.69	5.15
0.75	45	2.23	2.65	3.20	3.60	4.18	4.64	5.09
	46	2.20	2.61	3.16	3.55	4.13	4.58	5.03
	47	2.17	2.58	3.12	3.51	4.08	4.53	4.98
	48	2.14	2.54	3.08	3.47	4.04	4.48	4.92
	49	2.11	2.51	3.04	3.43	3.99	4.43	4.87
	50	2.09	2.48	3.01	3.39	3.95	4.39	4.82
	51	2.06	2.44	2.97	3.35	3.91	4.34	4.77
	52	2.04	2.41	2.94	3.32	3.86	4.29	4.72
	53	2.01	2.38	2.90	3.28	3.82	4.25	4.67
	54	1.99	2.35	2.87	3.24	3.78	4.20	4.62
	55	1.97	2.33	2.84	3.21	3.74	4.16	4.58
	56	1.95	2.30	2.81	3.18	3.70	4.12	4.53
	57	1.93	2.27	2.78	3.14	3.67	4.08	4.49
	58	1.90	2.24	2.75	3.11	3.63	4.04	4.45
	59	1.88	2.22	2.72	3.08	3.59	4.00	4.40
1	60	1.86	2.19	2.69	3.05	3.56	3.96	4.36
2	120	1.04	1.28	1.55	1.80	2.13	2.30	2.55
3	180	0.79	0.92	1.17	1.33	1.57	1.72	1.90
6	360	0.46	0.56	0.71	0.81	0.94	1.03	1.15
12	720	0.28	0.33	0.43	0.49	0.57	0.63	0.70
24	1440	0.15	0.20	0.25	0.28	0.33	0.37	0.41

Table A-14
Toccoa

Hours	Minutes	1	2	5	10	25	50	100
		Return Period						
	n	1.1702	0.7757	0.7029	0.6719	0.6636	0.647	0.6336
	a	249.80	47.77	45.46	46.17	53.60	56.24	59.02
	b	25	11	11	11	12	12	12
Hours	Minutes	Rainfall Intensity						
0.08	5	4.67	5.56	6.48	7.17	8.18	8.99	9.80
	6	4.49	5.30	6.21	6.88	7.87	8.67	9.46
	7	4.33	5.07	5.96	6.62	7.60	8.37	9.14
	8	4.17	4.87	5.74	6.39	7.34	8.10	8.84
	9	4.03	4.68	5.54	6.17	7.11	7.84	8.58
	10	3.90	4.50	5.35	5.97	6.89	7.61	8.33
	11	3.77	4.34	5.18	5.79	6.69	7.40	8.10
	12	3.65	4.20	5.02	5.62	6.51	7.19	7.88
	13	3.54	4.06	4.87	5.46	6.33	7.01	7.68
	14	3.43	3.93	4.73	5.31	6.17	6.83	7.49
0.25	15	3.33	3.81	4.60	5.17	6.02	6.67	7.31
	16	3.24	3.70	4.48	5.04	5.87	6.51	7.15
	17	3.15	3.60	4.37	4.92	5.74	6.37	6.99
	18	3.06	3.50	4.26	4.81	5.61	6.23	6.84
	19	2.98	3.41	4.16	4.70	5.49	6.10	6.70
	20	2.90	3.33	4.07	4.60	5.37	5.97	6.57
	21	2.83	3.25	3.98	4.50	5.27	5.85	6.44
	22	2.76	3.17	3.89	4.41	5.16	5.74	6.32
	23	2.69	3.10	3.81	4.32	5.06	5.64	6.20
	24	2.63	3.03	3.74	4.24	4.97	5.53	6.09
	25	2.57	2.96	3.66	4.16	4.88	5.44	5.99
	26	2.51	2.90	3.59	4.08	4.80	5.34	5.89
	27	2.45	2.84	3.53	4.01	4.71	5.25	5.79
	28	2.40	2.79	3.46	3.94	4.63	5.17	5.70
	29	2.35	2.73	3.40	3.87	4.56	5.09	5.61
0.50	30	2.30	2.68	3.34	3.81	4.49	5.01	5.53
	31	2.25	2.63	3.29	3.75	4.42	4.93	5.45
	32	2.20	2.58	3.23	3.69	4.35	4.86	5.37
	33	2.16	2.54	3.18	3.63	4.29	4.79	5.29
	34	2.11	2.49	3.13	3.58	4.22	4.72	5.22
	35	2.07	2.45	3.08	3.53	4.16	4.66	5.15
	36	2.03	2.41	3.04	3.47	4.11	4.59	5.08
	37	2.00	2.37	2.99	3.43	4.05	4.53	5.01
	38	1.96	2.33	2.95	3.38	4.00	4.47	4.95
	39	1.92	2.30	2.91	3.33	3.94	4.42	4.89
	40	1.89	2.26	2.87	3.29	3.89	4.36	4.83
	41	1.85	2.23	2.83	3.25	3.85	4.31	4.77
	42	1.82	2.20	2.79	3.21	3.80	4.26	4.71
	43	1.79	2.16	2.75	3.17	3.75	4.21	4.66
	44	1.76	2.13	2.72	3.13	3.71	4.16	4.61
0.75	45	1.73	2.10	2.68	3.09	3.66	4.11	4.56
	46	1.70	2.07	2.65	3.05	3.62	4.06	4.51
	47	1.68	2.05	2.62	3.02	3.58	4.02	4.46
	48	1.65	2.02	2.59	2.98	3.54	3.98	4.41
	49	1.62	1.99	2.56	2.95	3.50	3.93	4.36
	50	1.60	1.97	2.53	2.92	3.47	3.89	4.32
	51	1.57	1.94	2.50	2.88	3.43	3.85	4.28
	52	1.55	1.92	2.47	2.85	3.39	3.81	4.23
	53	1.53	1.90	2.44	2.82	3.36	3.78	4.19
	54	1.50	1.87	2.42	2.79	3.32	3.74	4.15
	55	1.48	1.85	2.39	2.77	3.29	3.70	4.11
	56	1.46	1.83	2.37	2.74	3.26	3.67	4.07
	57	1.44	1.81	2.34	2.71	3.23	3.63	4.04
	58	1.42	1.79	2.32	2.68	3.20	3.60	4.00
	59	1.40	1.77	2.29	2.66	3.17	3.57	3.96
1	60	1.38	1.75	2.27	2.63	3.14	3.53	3.93
2	120	0.93	1.10	1.39	1.58	1.85	2.04	2.23
3	180	0.72	0.83	1.03	1.15	1.35	1.48	1.63
6	360	0.42	0.51	0.64	0.72	0.85	0.94	1.03
12	720	0.25	0.31	0.40	0.46	0.54	0.61	0.67
24	1440	0.14	0.18	0.23	0.26	0.31	0.34	0.38

Table A-15
Valdosta

		Return Period						
		1	2	5	10	25	50	100
	N	0.8292	0.9691	0.9554	0.9484	0.9409	0.944	0.9492
	A	64.11	144.16	165.56	181.52	205.19	232.46	263.26
	B	12	19	21	22	23	24	25
Hours	Minutes	Rainfall Intensity						
0.08	5	6.12	6.63	7.36	7.97	8.92	9.68	10.43
	6	5.83	6.37	7.10	7.70	8.63	9.37	10.11
	7	5.58	6.13	6.86	7.45	8.36	9.09	9.81
	8	5.35	5.91	6.63	7.21	8.11	8.82	9.53
	9	5.13	5.71	6.42	6.99	7.87	8.57	9.26
	10	4.94	5.52	6.22	6.78	7.65	8.33	9.01
	11	4.76	5.34	6.04	6.59	7.43	8.10	8.77
	12	4.60	5.17	5.86	6.40	7.23	7.89	8.55
	13	4.44	5.02	5.70	6.23	7.04	7.69	8.33
	14	4.30	4.87	5.54	6.07	6.86	7.50	8.13
0.25	15	4.17	4.73	5.40	5.91	6.69	7.32	7.94
	16	4.04	4.60	5.26	5.76	6.53	7.14	7.75
	17	3.93	4.47	5.12	5.62	6.38	6.98	7.58
	18	3.82	4.36	5.00	5.49	6.23	6.82	7.41
	19	3.72	4.25	4.88	5.36	6.09	6.67	7.25
	20	3.62	4.14	4.77	5.24	5.96	6.53	7.10
	21	3.53	4.04	4.66	5.13	5.83	6.39	6.95
	22	3.44	3.94	4.55	5.02	5.71	6.26	6.81
	23	3.36	3.85	4.45	4.91	5.59	6.14	6.68
	24	3.28	3.77	4.36	4.81	5.48	6.02	6.55
	25	3.21	3.68	4.27	4.71	5.37	5.90	6.42
	26	3.14	3.60	4.18	4.62	5.27	5.79	6.30
	27	3.07	3.53	4.10	4.53	5.17	5.68	6.19
	28	3.01	3.46	4.02	4.44	5.08	5.58	6.08
	29	2.95	3.39	3.94	4.36	4.98	5.48	5.97
0.50	30	2.89	3.32	3.87	4.28	4.90	5.38	5.87
	31	2.83	3.25	3.80	4.20	4.81	5.29	5.77
	32	2.78	3.19	3.73	4.13	4.73	5.20	5.67
	33	2.73	3.13	3.66	4.06	4.65	5.11	5.58
	34	2.68	3.08	3.60	3.99	4.57	5.03	5.49
	35	2.63	3.02	3.54	3.92	4.50	4.95	5.40
	36	2.59	2.97	3.48	3.86	4.43	4.87	5.32
	37	2.54	2.92	3.42	3.80	4.36	4.80	5.24
	38	2.50	2.87	3.37	3.74	4.29	4.72	5.16
	39	2.46	2.82	3.31	3.68	4.22	4.65	5.08
	40	2.42	2.77	3.26	3.62	4.16	4.58	5.01
	41	2.38	2.73	3.21	3.57	4.10	4.52	4.94
	42	2.35	2.68	3.16	3.52	4.04	4.45	4.87
	43	2.31	2.64	3.11	3.46	3.98	4.39	4.80
	44	2.28	2.60	3.07	3.41	3.93	4.33	4.73
0.75	45	2.24	2.56	3.02	3.37	3.87	4.27	4.67
	46	2.21	2.52	2.98	3.32	3.82	4.21	4.60
	47	2.18	2.49	2.94	3.27	3.77	4.16	4.54
	48	2.15	2.45	2.90	3.23	3.72	4.10	4.48
	49	2.12	2.42	2.86	3.19	3.67	4.05	4.43
	50	2.09	2.38	2.82	3.14	3.62	4.00	4.37
	51	2.06	2.35	2.78	3.10	3.58	3.95	4.32
	52	2.04	2.32	2.75	3.06	3.53	3.90	4.26
	53	2.01	2.29	2.71	3.02	3.49	3.85	4.21
	54	1.99	2.26	2.68	2.99	3.44	3.80	4.16
	55	1.96	2.23	2.64	2.95	3.40	3.76	4.11
	56	1.94	2.20	2.61	2.91	3.36	3.71	4.06
	57	1.91	2.17	2.58	2.88	3.32	3.67	4.02
	58	1.89	2.14	2.55	2.84	3.28	3.63	3.97
	59	1.87	2.11	2.52	2.81	3.25	3.59	3.93
1	60	1.85	2.09	2.49	2.78	3.21	3.55	3.88
2	120	1.08	1.26	1.54	1.75	2.04	2.24	2.47
3	180	0.77	0.94	1.14	1.30	1.47	1.65	1.82
6	360	0.46	0.54	0.68	0.78	0.91	1.00	1.11
12	720	0.25	0.32	0.40	0.46	0.54	0.60	0.67
24	1440	0.15	0.18	0.23	0.27	0.32	0.35	0.38

Table A-16
Vidalia

		Return Period						
		1	2	5	10	25	50	100
	N	0.8456	0.8202	0.7515	0.7203	0.6904	0.6912	0.6764
	A	62.75	73.13	66.71	66.01	67.85	76.37	78.89
	B	11	15	15	15	15	16	16
Hours	Minutes	Rainfall Intensity						
0.08	5	6.02	6.27	7.02	7.63	8.58	9.31	10.06
	6	5.72	6.02	6.77	7.37	8.29	9.02	9.75
	7	5.45	5.79	6.54	7.12	8.03	8.74	9.46
	8	5.20	5.59	6.32	6.90	7.79	8.49	9.19
	9	4.98	5.40	6.12	6.69	7.56	8.25	8.94
	10	4.78	5.22	5.94	6.50	7.35	8.03	8.71
	11	4.60	5.05	5.76	6.32	7.16	7.83	8.49
	12	4.43	4.90	5.60	6.15	6.97	7.63	8.28
	13	4.27	4.75	5.45	5.99	6.80	7.45	8.09
	14	4.13	4.62	5.31	5.84	6.64	7.28	7.90
0.25	15	3.99	4.49	5.18	5.70	6.48	7.11	7.73
	16	3.87	4.37	5.05	5.56	6.34	6.96	7.57
	17	3.75	4.26	4.93	5.44	6.20	6.81	7.41
	18	3.64	4.16	4.82	5.32	6.07	6.67	7.26
	19	3.54	4.05	4.71	5.21	5.95	6.54	7.12
	20	3.44	3.96	4.61	5.10	5.83	6.42	6.99
	21	3.35	3.87	4.51	5.00	5.72	6.29	6.86
	22	3.26	3.78	4.42	4.90	5.61	6.18	6.74
	23	3.18	3.70	4.33	4.81	5.51	6.07	6.62
	24	3.10	3.62	4.25	4.72	5.41	5.96	6.51
	25	3.03	3.55	4.17	4.63	5.31	5.86	6.40
	26	2.96	3.48	4.09	4.55	5.23	5.77	6.30
	27	2.90	3.41	4.02	4.47	5.14	5.67	6.20
	28	2.83	3.34	3.95	4.40	5.06	5.58	6.10
	29	2.77	3.28	3.88	4.32	4.98	5.50	6.01
0.50	30	2.72	3.22	3.82	4.25	4.90	5.42	5.92
	31	2.66	3.16	3.75	4.19	4.83	5.34	5.83
	32	2.61	3.11	3.69	4.12	4.75	5.26	5.75
	33	2.56	3.06	3.64	4.06	4.69	5.18	5.67
	34	2.51	3.00	3.58	4.00	4.62	5.11	5.59
	35	2.46	2.96	3.53	3.94	4.56	5.04	5.52
	36	2.42	2.91	3.47	3.89	4.49	4.98	5.45
	37	2.38	2.86	3.42	3.83	4.43	4.91	5.38
	38	2.34	2.82	3.38	3.78	4.38	4.85	5.31
	39	2.30	2.77	3.33	3.73	4.32	4.79	5.25
	40	2.26	2.73	3.28	3.68	4.27	4.73	5.18
	41	2.22	2.69	3.24	3.63	4.21	4.67	5.12
	42	2.19	2.65	3.20	3.59	4.16	4.61	5.06
	43	2.15	2.62	3.15	3.54	4.11	4.56	5.00
	44	2.12	2.58	3.11	3.50	4.06	4.51	4.95
0.75	45	2.09	2.54	3.08	3.46	4.02	4.46	4.89
	46	2.05	2.51	3.04	3.42	3.97	4.41	4.84
	47	2.02	2.48	3.00	3.38	3.93	4.36	4.79
	48	2.00	2.44	2.96	3.34	3.88	4.31	4.73
	49	1.97	2.41	2.93	3.30	3.84	4.26	4.69
	50	1.94	2.38	2.90	3.26	3.80	4.22	4.64
	51	1.91	2.35	2.86	3.23	3.76	4.18	4.59
	52	1.89	2.32	2.83	3.19	3.72	4.13	4.54
	53	1.86	2.30	2.80	3.16	3.68	4.09	4.50
	54	1.84	2.27	2.77	3.13	3.65	4.05	4.46
	55	1.82	2.24	2.74	3.09	3.61	4.01	4.41
	56	1.79	2.22	2.71	3.06	3.58	3.97	4.37
	57	1.77	2.19	2.68	3.03	3.54	3.94	4.33
	58	1.75	2.17	2.65	3.00	3.51	3.90	4.29
	59	1.73	2.14	2.63	2.97	3.48	3.86	4.25
1	60	1.71	2.12	2.60	2.94	3.44	3.83	4.21
2	120	0.99	1.22	1.47	1.67	1.96	2.14	2.37
3	180	0.71	0.86	1.07	1.20	1.42	1.57	1.71
6	360	0.41	0.50	0.63	0.73	0.86	0.96	1.05
12	720	0.24	0.30	0.37	0.44	0.51	0.57	0.63
24	1440	0.14	0.17	0.21	0.25	0.29	0.33	0.36

APPENDIX

B

SOILS INFORMATION FOR GEORGIA

(Source: Manual for Erosion and Sediment Control in Georgia, Fifth Edition, 2000)

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APPENDIX B-1

SOILS INFORMATION

The soils information in Appendix B has been assembled to assist the plan preparer and reviewer in accomplishing responsibilities under Act 599.

Appendix B-1 contains charts of Georgia's soils series with *estimated* soil properties and soils limitations for urban uses. These charts may be used in conjunction with published soil survey information or other soils maps available through the Soil and Water Conservation Districts and the Natural Resources Conservation Service.

Appendix B-2 contains excerpts from NRCS Technical Release No. 51, "Procedure for Computing Sheet and Rill Erosion on Project Areas." Also included is a method for *estimating* soil erodibility or "K" values and sediment delivery ratio charts. This Appendix should be of assistance in planning for land-disturbing activities.

Explanation of Charts: Appendix B-1 Soil Series Interpretations

Column 1: Soil Series

This column lists alphabetically the name of all the soil series which are used in the State of Georgia.

Column 2: Permeability

Soil permeability is the quality that enables soil to transmit water and air. Accepted as a measure of this quality is the rate at which soil transmits water while saturated. That rate is the "saturated hydraulic conductivity" of soil physics. In line with conventional usage in the engineering profession and traditional usage in the published soil surveys, this rate of flow, principally downward, continues to be expressed as "permeability". The permeability of a soil is the *rate of flow* for the most restrictive layer in the profile.

Soil permeability is rated using the numerical ranges shown below:

Permeability class	Numerical range (inches per hour)
Very slow	Less than 0.06
Slow	0.06-0.2
Moderately slow	0.2-0.6
Moderate	0.6-2.0
Moderately rapid	2.0-6.0
Rapid	6.0-20
Very Rapid	More that 20

Column 3: Soil Reaction

The degree of acidity or alkalinity of a soil is expressed in pH values. A soil with a pH 7.0 is precisely neutral in reaction. The pH ranges given in this column are the high and low values for the soil profile. The surface layer may be higher due to the addition of lime.

Column 4: Shrink-Swell Potential

Shrink-swell behavior is the quality that determines soil's volume change with change in moisture content. Building foundations, roads and other structures may be severely damaged by the shrinking and swelling of the soil. The volume change of soil is influenced by the amount of moisture change and the amount and kind of clay in the soil profile.

The shrink-swell interpretations are relevant to structures, such as houses and other low buildings, streets and roads, and parking lots. Three classes have been developed to express shrink-swell behavior; *low, moderate and high.*

Column 5 and 6: Corrosivity

Various metals and other materials corrode when on or in the soil, and some metals and materials corrode more rapidly when in contact with specific soils than when in contact with others. To be meaningful, corrosivity must be rated in relation to specific structural material. In these columns the soil series are given ratings on potential for inducing corrosion of uncoated steel (column 5) and of concrete (column 6).

Soils are assigned to one of three classes of corrosivity: *low, moderate, or high.*

Columns 7 and 8: Depth to Watertable and Bedrock

The depth to the watertable is given in feet (to the nearest half-foot). The value given is an indication of how close to the soil surface the watertable will rise during the wet season.

Depth to bedrock is given in inches. Hardness of rock may range from "rippable", which can be excavated using a single tooth ripping attachment on a 200-300 horsepower tractor, to "hard", where excavation may require blasting. Rock hardness should be determined by on-site-investigation.

Both the depth to watertable and bedrock are *estimates*—actual depths may vary from site to site.

Column 9: Flood Frequency

Flood frequency is an indicator of how often if ever, floods occur. Ratings are as follows:

None: No reasonable possibility of flooding.

Rare: Flooding unlikely but possible under unusual weather conditions.

Occasional: Flooding is expected infrequently under usual weather conditions.

Frequent: Flooding is likely to occur often under usual weather conditions.

Column 10: Hydrologic Soil Group

The hydrologic soil group parameter, A, B, C, or D, is an indication of the minimum rate of infiltration obtained for a bare soil after prolonged wetting.

The hydrologic soil groups range from A, which are deep sands or gravels with low runoff, to D, which are soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material.

Some soil series may have a dual hydrologic soil group rating. The dual ratings are given for certain wet soils that can be adequately drained. The criteria used is making dual group ratings are as follows:

1. Soils are rated D in their natural condition.
2. Drainage is feasible and practical.
3. Drainage improves the hydrologic group by at least two classes (from D to A or B).

Columns 11-16: Limitation of Soils For Urban Uses

The soils are rated L for slight limitations, M for moderate limitations, or S for severe limitations.

A *slight* soil limitation (L) is the rating given soils that have properties favorable for the rated use. The degree of limitation is minor and can be overcome easily. Good performance and low maintenance can be expected.

A *moderate* soil limitation (M) is the rating given soils that have properties moderately favorable for the rated use. This degree of limitation can be overcome or modified by special planning, design, or maintenance. During some part of the year, the performance of the structure or other planned use is somewhat less desirable than for soils rated *slight*. Some soils rated *moderate* required treatment such as artificial drainage, runoff control to reduce erosion, extended sewage absorption fields, extra excavation, or some modification of certain features through manipulation of the soil. For these soils, modification is needed for those construction plans generally used for soils of slight limitation. Modification may include special foundations, extra reinforcement of structures, sump pumps, etc.

A *severe* soil limitation (S) is the rating given soils with one or more properties unfavorable for the rated use, such as steep slopes, bedrock near the surface, flooding hazard, high shrink-swell potential, a seasonal high watertable or low bearing strength. This degree of limitation generally requires major soil reclamation, special design, or intensive maintenance. Some of these soils, however, can be improved by reducing or removing the soil feature that limits use. In most situations, however, it is difficult and costly to alter the soil or to design a structure to compensate for a severe degree of limitation.

Following the limitation rating symbol will be a lower case symbol to indicate the reason for the particular rating. These symbols are s-slope, w-wetness, f-flooding, pk-slow percolation, cl-too clayey, ss-shrink-swell potential, b-low bearing strength, r-shallow depth to rock, p-seepage, st-stones, cc-cutbanks may cave, h-too much humas, pf-poor filter, d-dense layer.

A number followed by a % indicates percent slope; for example 2-6% reads two to six percent slope.

Some soil series may have a flood frequency listing of two rating (Example: None-Occasional). Such soils may consequently have a two rating limitation. For example, a listing of

None: M; w
Occ; S; f

means, if the soil is located where no flooding occurs, it is rated Moderate due to wetness and, if occasional flooding occurs, it is rated as Severe due to flooding.

Column 11: Septic Tank Absorption Fields

A septic tank absorption field is a soil absorption system for sewage disposal. It is a subsurface tile system laid in such a way that effluent from the septic tank is distributed with reasonable uniformity into the natural soil. Criteria used for rating soils *(slight, moderate, and severe)* for use as absorption fields are based on the limitations of the soil to absorb effluent.

Column 12: Sewage Lagoon Areas

A sewage lagoon (aerobic) is a shallow lake used to hold sewage for the time required for bacterial decomposition. Sewage lagoons require consideration of the soils for two functions, (1) as a vessel for the impounded area and (2) as soil material for the enclosing embankment. The requirements for this embankment are the same as for other embankments designed to impound water. Enough soil material that is suitable for the structure must be available, and, when the lagoon is properly constructed, it must be capable of holding water with minimum seepage. The material should be free of coarse fragments (over 10 inches in diameter) that interfere with compaction.

Column 13: Shallow Excavations

These excavations require excavating or trenching to a depth of 5 or 6 feet. Note that limitation ratings for shallow excavations alone, though highly relevant, are insufficient for interpretations for ultimate uses, such as for dwellings with basements, sanitary landfills, cemeteries, and underground utility lines (sewers, pipelines, and cables). Additional soil features must be considered in evaluating for those uses. For example, additional interpretation concerning shrink-swell potential and corrosivity are needed for giving ratings for the ultimate use of soils for pipelines.

Column 14: Dwellings

This column gives ratings for undisturbed soils on which single-family dwellings or other structures with similar foundation requirements can be built. Buildings of more than three stories and other buildings requiring a foundation load in excess of that of a three-story dwelling are not considered in the entries in this column.

In some cases, a rating may differ depending on whether the dwelling will or will not have a basement. In such cases, the rating is marked with an asterisk (*) for dwellings *with* basements and a pound sign (#) for ones *without* basements.

Column 15: Small Commercial Buildings

This column provides limitations for commercial buildings of 3 stories or less.

Column 16: Local Roads and Streets

The limitation ratings given in this column apply to use of soils for construction and maintenance of improved local roads and streets that have all-weather surfacing—commonly asphalt or concrete—and that are expected to carry automobile traffic all year. The roads and streets consist of (1) underlying local soil material, whether cut or fill, that is called "the subgrade"; (2) the base material of gravel, crushed rock, lime-stabilized soil, or soil-cement-stabilized soils; and (3) the actual road surface or street pavement that is either flexible (asphalt), rigid (concrete), or, in some rural areas, gravel with binder in it. These roads and streets also are graded to shed water and conventional drainage measures are provided. With probable exception of the hard surfaces, the roads and streets are built mainly from the soil at hand; cuts and fills generally are limited to less than 6 feet of thickness. Excluded from consideration in the ratings in this column are highways designed for fast moving heavy trucks.

Also, the ratings cannot substitute for basic soil data and for on site investigation.

KEY TO SYMBOLS—SOIL SURVEY INTERPRETATIONS

Limitations of Soils:

L - Slight Limitation
M - Moderate Limitation
S - Severe Limitation

Reasons for Limitations:

s - slope
w - wetness
f - flooding
pk - slow percolation
cl - too clayey
ss - shrink-swell potential
b - low bearing strength
r - depth to rock
p - seepage
st - stones
cc - cutbank may cave
pf - poor filter
h - too much humus
d - dense layer

Appendix B-2 Soil Loss Predictions

The first portion of Appendix B-2 is the SCS Technical Release No. 51, "Procedure for Computing Sheet and Rill Erosion on Project Areas." It explains the use of the Universal Soil-Loss Equation.

Also included in Appendix B-2 is a guide for developing the Soil Erodibility Factor (K).

Another section is devoted to applying sediment delivery ratio charts to the Universal Soil-Loss Equation for estimating sediment yields.

Additional information includes a textural classification chart and a chart for comparing different soil classification systems.

SOIL SERIES INTERPRETATIONS

SOIL SERIES	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	CORROSIVITY		DEPTH TO:		FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
				STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)								
AILEY	0.06-0.2	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None	B	0-15%:S;pk 15+%:S;S,pk	0-7%:S;p 7+%:S;s,cc	0-15%:S;cc 15+%:S;s,cc	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s
ALAPAHA	0.2-0.6	4.5-5.5	Low	High	High	0-1.0	>60	None Rare Occ	D	None, Rare:S;w,pk Occ:S,w,f,pk	None,Rare:S;w,p Occ:S;w,f,p	S;w,cc	None:Sw Rare,Occ:S;w,f	None:S;w Rare:Occ:S;w,f	None,Rare:S;w Occ:S;w,f
ALBANY	0.6-2.0	4.5-6.0	Low	High	High	1.0-2.5	>60	None Rare Occ	C	None, Rare:S;w Occ:S;w,f	None,Rare:S;w Occ:S;w,f	S;w,cc	None:S;w Rare,Occ:S;w,f	None:S;w Rare:Occ:S;w,f	None,M;w Rare;M;w,f Occ:S;f
ALBERTVILLE	0.2-0.6	4.5-5.5	Mod.	Mod.	High	>6.0	40-60	None	C	2-15%:S;pk 15+%:S,s,pk	2-7%:M;s,r 7+%:S;s	2-8%:M;cl 8-15%:M;s,cl 15+%:S;s	2-8%:M;ss 8-15%:M;ss,s 15+%:S;s	2-4%:M;ss 4-8%:M;s,ss 8+%:S;s	2-15%:S;b 15+%:S;s,b
ALCOVY	0.6-0.2	4.5-5.5	Low	Low	High	2.0-3.0	>60	None	B	S;w,pk	S;w	2-8%:M;w 8+%:M;w,s	2-8%:M;w 8+%:M;w,s	2-4%:M;w 4-8%:M;w,s 8+%:S;s	2-8%:M;w 8+%:M;s
ALLEN	0.6-2.0	4.5-5.5	Low	Low	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;s,pk 15+%:S;s	2-7%:M;s 7+%:S;s	2-8%:M;cl 8-15%:M;s,cl 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;s,b 15+%:S;s
ALTAVISTA	0.6-2.0	3.6-6.0	Low	Mod.	Mod.	1.5-2.5	>60	None Rare Occ Freq	C	None, Rare:S;w Occ,Freq:S;w,f	None:S;w Rare,Occ,Freq: S;w,f	S;w	None:M;w# None:S;w* Rare,Occ,Freq: S;w,f	None:M;w Rare,Occ,Freq: S; f	None:M;w,b Rare:M;w,f,b Occ,Freq:S;f
AMERICUS	2.0-6.0	4.5-5.5	Low	Low	High	>6.0	>60	None	A	0.8%:L 8-15%:M;s 15+%:S;s	0-15%:S;p 15+%:S;s,0	0-15%:S;cc 15+%:S;s,cc	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s
ANGELINA	0.06-0.2	4.5-5.5	Low	High	High	+2-0	>60	Occ Freq	D	S;f,pk,w	S;f,w	S;w	S;f	S;f	S;f
ANGIE	0.06-0.2	3.6-5.5	High	High	Mod.	3.0-5.0	>60	None	D	S;pk,w	0-7%:S;w 7+%:S;s,w	0-8%:M;cl,w 8-12%:M;s,cl	S;ss	0-8%:S;ss 8+%:S;s,ss	S;b,ss
APISON	0.6-2.0	4.5-5.5	Low	Mod.	Mod.	>6.0	20-40	None	B	S;r	2-7%:S;r 7+%:S;s,r	2-8%:M;r 8-15%:M;s,r	2-8%:L# 2-8%:M;r* 8-12%:M;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;s,b
APPLING	0.6-2.0	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None	B	0-8%:M;pk 8-15%:M,s,pk	0-2%:M;p 2-7%:M;s,p 7+%:S;s	0-8%:M;cl 8-15%:M;s,cl	0-8%:L 8-15%:M;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:M;b 8-15%:M;s,b
ARAGON	0.06-0.2	3.6-5.5	Mod.	High	High	>6.0	>60	None	C	2-15%:S;pk 15+%:S;s,pk	2-7%:M;s 7+%:S;s	2-8%:M;cl 8-15%:M;s,cl 15+%:S;s	2-8%:M;ss 8-15%:M;s,ss 15+%:S;s	2-4%:M;b 4-8%:M;s,ss 8+%:S;s	2-15%:M;ss 15+%:S;b,s
ARKAQUA	0.6-2.0	5.1-6.0	Low	High	Mod.	1.5-2.0	>60	Freq	C	S;f,w	S;f,w	S;w	S;f# S;f,w*	S;f	S;b,f
ARDILLA	0.2-0.6	4.5-6.0	Low	High	High	1.0-2.0	>60	None	C	S;w,pk	S;w	S;w	S;w	S;w	M;w
ARMUCHEE	0.2-0.6	4.5-5.5	Mod.	Mod.	Mod.	>6.0	20-36	None	C	5-15%:S;r 15+%:S;r	5-7%:S;r 7+%:S;s,r	5-8%:M;r,cl 8-15%:M;r,cl,s 15+%:S;s	5-8%:M;ss#,r* 8-15%:M;ss,s#,r* 15+%:S;s	5-8%:M;ss,s 8+%:S;s	5-15%:S;b 15%:S;b,s
ARUNDEL	>0.06	3.6-5.5	High	High	High	>6.0	20-40	None	C	2-15%:S;r,pk 15+%:S;r,pk,s	2-7%:S;r 7+%:S;r,s	2-8%:M;r,cl 8-15%:M;r,cl,s 15+%:S;s	2-15%:S;ss 15+%:S;ss,s	2-8%:S;ss 8+%:S;ss,s	2-15%:S;ss,b 15+%:S;ss,b,s

SOIL SERIES INTERPRETATIONS

SOIL SERIES	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	CORROSIVITY		DEPTH TO:		FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
				STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)								
ASHE	2.0-6.0	4.5-6.0	Low	Low	High	>6.0	20-40	None	B	2-15%:S;r 15+%:S;s,r	2-7%:S;r,p 7+%:S;r,s,p	2-8%:M;r 8-15%:M;r,s 15+%:S;s	2-8%:M;r # 8-15%:M;r,s # 2-15%:S;r* 15+%:S;s# r*	2-4%:M;r 4-8%:M;r,s 8+%:S;s	2-8%:M;r 8-15%:M;s,r 15+%:S;s
ASHLAR	2.0-6.0	4.5-5.5	Low	Low	High	>6.0	20-40	None	B	0-15%:S;r 15+%:S;s	0-7%:S;r,p 7+%:S;s,r,p	0-15%:S;r 15+%:S;s,r	0-15%:S;r* 0-15%:M;s,r# 15+%:S;s	0-4%:M;r 4-8%:M;r,s 8+%:S;s	0-8%:M;r 8-15%:M;s,r 15+%:S;s
AUGUSTA	0.6-2.0	4.5-6.0	Low	High	Mod.	1.0-2.0	>60	None Rare Occ Freq	C	None,Rare:S:w Occ,Freq:S;W,f	None,Rare:S;w Occ,Freq:S;w,f	S;w	None:S;w Rare,Occ,Freq:S; w,f	None:S;w Rare,Occ,Freq:S; w,f	None:M;w,b Rare:M;w,b,f Occ,Freq:S;f
BARFIELD	0.2-0.6	6.1-7.8	High	High	Low	>6.0	8-20	None	D	1-15%:S;r 15+%:S;r,s	1-7%:S;r 7+%:S;s,r,	1-15%:S;r 15+%:S;r,s	1-15%:S;r,ss 15+%:S;r,ss,s	1-8%:S;ss,r 8+%:S;ss,r,s	1-15%:S;r,b 15+%:S;r,b,s
BAYBORO	0.06-0.2	3.6-5.5	Mod.	High	High	0-0.5	>60	Occ Freq	D	S;w,pk,f	S;w,f	S;w	S;w,f	S;w,f	S;w,b,f
BIBB	0.6-2.0	4.5-5.5	Low	High	Mod.	0.5-1.5	>60	Occ Freq	C	S;f,w	S;f,w,p	S;f,w	S;w,cc	S;f,w	S;f,w
BIGBEE	6.0-20	4.5-6.0	Low	Low	Mod.	3.5-6.0	>60	Rare Occ Freq	A	Rare:S;w,pf Occ,Freq;S;f,w,pf	Rare:S;p Occ,Freq:S;p,f	S;cc	S;f	S;f	Rare:M;f Occ,Freq:S;f
BINNSVILLE	0.06-0.2	7.4-8.4	Mod.	Mod.	Low	>6.0	7-20	None	D	1-15%:S,r 15+%:S,r,s	1-7%:S:r 7+%:S;r,s	1-15%:S;r 15+%:S;r,s	1-8%:Mod;ss,r# 8-15%:Mod:ss,r,s# 1-15%:S;r* 15+%:S;S#r*	1-4%:M;ss,r 4-8%:M;ss,r,s 8+%:S;s	1-15%:S;b 15+%:S;b,s
BLADEN	0.06-0.2	3.6-5.5	Mod.	High	High	0-1.0	>60	Occ Freq	D	S;w,f	L	S;w	S;w,f	S;w,f	S;w,f,b
BLANTON	0.6-2.0	4.5-6.0	Low	High	High	4.0-6.0	>60	None	A	0-8%:M;w 8-15%:M;w,s 15+%:S;s	0-7%:S;p 7+%:S;p,s	0-15%:S;cc 15+%:S;s,cc	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8-12%:S;s	0-8%:L 8-15%:M;s 15+%:S;s
BODINE	2.0-6.0	3.6-5.5	Low	Low	High	>6.0	>60	None	B	5-8%:M;st 8-15%:M;s,st 15+%:S;s	S;p,s	5-8%:M;st 8-15%:M;s,st 15+%:S;s	5-8%:M;st 8-15%:M;s,st 15+%:S;s	8%:M;st 8+%:S;s	5-8%:M;st 8-15%:M;s,st 15+%:S;s
BOHICKET	<0.06	6.1-8.4	High	High	High	+3-0	>60	Freq	D	S;f,pk	S;f	S;f	S;f,ss	S;f,ss	S;f,ss,b
BONIFAY	0.2-0.6	4.5-6.5	Low	Low	High	4.0-5.0	>60	None	A	0-8%:M;w,pk 8-12%:M;w,pk,s	0-7%:S;p 7+%:S;p,s	S;cc	0-8%:L# 0-8%:M;w* 8-12%:M;s#,w*	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-12%:M;s
BONNEAU	0.6-2.0	4.5-5.5	Low	Low	High	3.5-5.0	>60	None	A	S;w	0-7%:S;p 7+%:S;p,s	S;cc	0-8%:L# 0-8%:M;w* 8-12%:M;s#,w*	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-12%:M;s
BOSWELL	<0.06	4.5-5.5	High	High	Mod.	>6.0	>60	None	D	S;pk	1-2%:L 2-7%:M;s 7+%:S;s	1-8%:M;cl 8-15%:M;cl,s 15+%:S;s	1-15%:S;ss 15+%:S;s,ss	1-8%:S;ss 8+%:S;s,ss	1-15%:S;ss,b 15+%:S;s,ss,b
BRADDOCK	0.6-2.0	3.6-5.5	Mod.	High	Mod.	>6.0	>60	None	B	0-8%:M;pk 8-15%:M;pk,s 15+%:S;s	0-7%:S;p 7+%:S;s,p	0-8%:M;cl 8-15%:M;cl,s 15+%:S;s	0-8%:M;ss 8-15%:M;ss,s 15+%:S;s	0-4%:M;ss 4-8%:M;ss,s 8+%:S;s	0-15%:S;b 15+%:S;s,b
BRADSON	0.6-2.0	4.5-6.0	Low	High	Mod.	>6.0	>60	None	B	2-8%:L 8-15%:M;s 15+%:S;s	2-7%:M;s 7+%:s;s	1-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:L 8-15%:M;s 8+%:S;s	2-4%:L 4-8%:M;s	2-15%:S;b 15+%:S;b,s

SOIL SERIES INTERPRETATIONS

	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
				CORROSIVITY		DEPTH TO:									
SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
BRASSTOWN	0.6-2.0	4.5-6.0	Low	Mod.	High	>6.0	40-60	None	B	2-8%:M;r,pk 8-15%:M;r,pk,s 15+%:S;s	2-7%:M;r,s 7+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;b,s 15+%:S;s
BREVARD	0.6-2.0	4.5-6.0	Low	Mod.	Mod.	>6.0	>60	None	B	2-8%:L 8-15%:M;s 15+%:S;s	2-7%:M;p,s 7+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
BROOKMAN	0.06-0.2	4.5-7.8	Mod.	Mod.	Mod.	0-1.0	>60	None Rare Occ Freq	D	None, Rare:S;w,pk Occ,Freq:S;f,w,pk	L	S;w	None:S;w Rare,Occ,Freq:S; f,w	None:S;w Rare,Occ,Freq:S; f,w	None,Rare:S;b,w Occ,Freq:S;f,w,b
BUNCOMBE	6.0-20	4.5-6.0	Low	Low	Mod.	>6.0	>60	Rare Occ Freq	A	Rare:S;pf Occ,Freq:S;f,pf	S;p,f	S;cc	S;f	S;f	Rare:M;f Occ,Freq:S;f
BURTON	0.6-6.0	3.6-6.0	Low	High	High	>6.0	20-40	None	B	5-%:S;r 15+%:S;r,s	5-7%:S;p,r 7+%:S;p,r,s	5-15%:S;r 15+%:S;r,s	5-15%:M;r# 5-15%:S;r* 15+%:S;s#,r*	5-8%:M;s,r 8+%:S;s	5-8%:M;r 8-15%:M;s,r 15+%:S;s
CAHABA	0.6-2.0	4.5-6.0	Low	Mod.	Mod.	>6.0	>60	None Rare Occ	B	None:L Rare:M;f Occ:S;f	None,Rare:S;P Occ:S;p,f	S;cc	None:L Rare,Occ:S;f	None:L Rare:Occ:S;f	None:L Rare:M;f Occ:S,f
CAINHOY	<0.06	6.6-8.4	High	High	High	+1-1.0	>60	Freq	A	S;pf	0-7%:S;p 7+%:S;p,s	S;cc	0-8%:L 8-10%:M;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-10%:M;s
CAPE FEAR	0.06-0.2	4.5-6.0	Mod.	High	High	0-1.5	>60	None Rare	D	S;w,pk	None:S;p,w Rare:S;p,w,f	S;w	None:S;w Rare:S;w,f	None:S;w Rare:S;w,f	S;b,w
CAPERS	<0.06	6.6-8.4	High	High	High	+1-1.0	>60	Freq	D	S;w,f,pk	S;f,w	S;f,w	S;f,w,ss	S;f,w,ss	S; f,b,w
CAPSHAW	0.06-0.2	5.1-7.8	Mod.	High	Mod.	3.5-5.0	48>60	None	C	S;pk	0-2%:M;r 2-7%:M;s,r 7+%:S;s	M;r,w,cl	0-8%:M;ss# 8-12%:M;ss,s# M;ss,w*r	0-4%:M;ss 4-8%:M;s,ss 8+%:S;s	S;b
CARNEGIE	0.2-0.6	4.5-5.5	Low	Low	Mod.	>6.0	>60	None	C	S;pk	2-7%:M;s 7+%:S;s	2-8%:M;cl,d 8-12%:M;s,cl,d	2-8%:M;s 8-12%:M;s	2-8%:M;s 8-12%:S;s	0-8%:M;b 8-12%:M;s,b
CARTECAY	2.0-6.0	5.1-6.5	Low	Low	Mod.	0.5-1.5	>60	Occ Freq	C	S;w,f	S;w,p,f	S;w,cc	S;f,w	S;f,w	S;f,w
CATASKA	2.0-20	4.5-5.5	Low	Low	Mod.	>6.0	20-40	None	D	10-15%:S;r 15+%:S;r,s	S;p,r,s	10-15%:S;r 15+%:S;r,s	10-15%:M;s,r# 10-15%:S;r* 15+%:S;s#r*	S;s	10-15%:M;r,s 15+%:S;s
CECIL	0.6-0.2	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None	B	0-8%:M;pk 8-15%:M;pk,s 15+%:S;s	0-2%:M;p 2-7%:M;p,s 7+%:S;s	0.8%:M;cl 8-15%:M;cl,s 15+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-18%:M;s 8+%:S;s	0-8%:M;b 0-15%:M;b,s 15+%:S;s
CEDARBLUFF	0.06-0.2	5.1-6.0	Mod.	High	Mod.	0.5-1.0	>60	Freq	C	S;f,pk,w	L	S;w	S;f,w	S;f,w	S;f,w,b
CENTENARY	2.0-0.6	4.5-6.0	Low	Mod.	High	3.5-5.0	>60	None	A	S;w,p,f	S;p	S;cc	M;w* L#	L	L
CHANDLER	2.0-0.6	4.5-6.0	Low	Low	High	>6.0	>60	None	B	S;s	S;s	S;s	S;s	S;s	S;s,b

SOIL SERIES INTERPRETATIONS

ESTIMATED SOIL PROPERTIES										LIMITATION OF SOILS FOR URBAN USES					
				CORROSIVITY		DEPTH TO:									
SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
CHASTAIN	0.06-0.2	4.5-6.0	Mod.	High	High	0-1.0	>60	Occ Freq	D	S;f,w,pk	S;f,p	S;w,cc	S;f,w	S;f,w	S;f,w,b
CHATUGE	0.6-2.0	4.5-6.0	Mod.	High	High	1.0-2.0	>60	Occ	D	S;w,f	S;w,f,p	S;w,f	S;w,f	S;w,f	S;w,f,
CHENNEBY	0.6-2.0	4.5-6.0	Low	High	Mod.	1.0-2.5	>60	Occ Freq	C	S;f,w	S;p,f,w	S;w	S;f,w	S;f,w	S;b,f
CHESTATEE	0.6-2.0	4.5-5.5	Low	High	High	>6.0	>60	None	B	2-8%:M;st 8-10%:M;s,st 15+%:S;s	2-7%:M;s 7+%:S;s	2-8%:M;st 8-15%:M;s,st 15+%:S;s	2-8%:M;st 8-15%:M;s,st 15+%:S;s	2-4%:M;st 4+%:S;s	2-8%:M;st 8-15%:M;s,st,b 15+%:S;s
CHEWACLA	0.6-2.0	4.5-7.8	Low	High	Mod.	0.5-1.5	>60	Rare Occ Freq	C	Rare:S;s Occ,Freq:S;f,w	Rare:S;w Occ,Freq:S;f,w	S;w	S;w,f	S;w,f	Rare:S;w,b Occ,Freq:S;w,f,b
CHIPLEY	6.0-2.0	4.5-6.0	Low	Low	High	2.0-3.0	>60	None	C	S;w,pf	S;p,w	S;cc,w	M;w# S;w*	0-4%:M;w 4-8%:M;s,w	M;w
CHIPOLA	2.0-6.0	4.5-5.5	Low	Low	High	>6.0	>60	None	A	L	0-7%:S;p 7+%:S;p,s	S;cc	L	0-4%:L 4-8%:M;s	L
CHISOLM	0.6-2.0	4.5-6.0	Low	Low	High	3.0-5.0	>60	None	A	0-8%:M;w 8-10%:M;w,s	0-7%:S;p,w 7+%:S;p,w,s	S;cc	0-8%:L# 0-8%:M;w* 8-10%:M;s#,w*	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-10%:M;s
CLARENDON	0.2-2.0	4.5-5.5	Low	Mod.	High	2.0-3.0	>60	None	C	S;pk,w	S;w	S;w	M;w# S;w*	M;w	M;w
CLIFTON	0.6-2.0	4.5-6.5	Low	Low	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;pk,s 15+%:S;s	2-7%:S;p 7+%:S;p,s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-15%:S;b 15+%:S;b,s
COLFAX	0.06-0.2	4.5-5.5	Mod.	High	High	0.5-1.5	>60	None	C	S;pk,w	0-7%:S;w 7+%:S;w,s	S;w	S;w	0-4%:S;w 4+%:S;s	M;w
COLVARD	2.0-6.0	5.1-7.8	Low	Low	Mod.	4.0-6.0	>60	Occ	B	S;f	S;p,f	S;cc	S;f	S;f	S;f
CONASAUGA	0.06-0.2	3.6-6.5	Mod.	High	High	>6.0	20-40	None	C	0-15%:S;pk,r 15+%:S;s,pk	0-7%:S;r,s 7+%:S;s,r	0-8%:M;r,cl 8-15%:M;s,r,cl 15+%:S;s	0-8%:M;ss 8-15%:M;s,ss 15+%:S;s	0-4%:M;ss 4-8%:M;s,ss 8+%:S;s	0-15%:S;b 15+%:S;s,b
CONGAREE	0.6-2.0	4.5-7.3	Low	Mod.	Mod.	2.5-4.0	>60	None Rare Occ Freq	B	None, Rare:S;w Occ,Freq:S;w	None:S;w Rare,Occ,Freq:S; w,f	None,Rare:M;w Occ,Freq:M;w,f	None:L# None:M;w* Occ,Freq:S;f	None:L Rare,Occ,Freq: S;f	None:M;b Rare:M;b,f Occ,Freq:S;f
COOSAW	0.6-2.0	4.5-5.5	Low	Mod.	High	2.0-3.0	>60	None	B	S;w,pf	S;w,p	S;w,cc	M;w# S;w*	M;w	M;w
COWARTS	0.06-0.6	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None	C	S;pk	2-7%:M;s 7+%:S;s	2-8%:L 8-15%:M;s	2-8%:L 8-15%:M;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s
COWEE	0.6-2.0	4.5-6.0	Low	Mod.	High	>6.0	20-40	None	B	8-15%:S;r 15+%:S;r,s	S;r,s	8-15%:M;r,s 15+%:S;s	8-15%:M;s# 8-15%:M;r,s* 15+%:S;s	S;s	8-15%:M;s 15+%:S;s

SOIL SERIES INTERPRETATIONS

SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	CORROSIVITY: STEEL	CORROSIVITY: CONCRETE	DEPTH TO: WATER TABLE (Ft.)	DEPTH TO: BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
COXVILLE	0.2-0.6	3.6-5.5	Mod.	High	High	0-1.5	>60	None	D	S;w,pk	S;w	S;w	S;w	S;w	S;w,b
CRAVEN	0.06-0.2	3.6-5.5	Mod.	High	High	2.0-3.0	>60	None	C	S;pk,w	0-2%:M;p 2-7%:M;p,s 7+%:S;s	S;w,cc	0-8%:M;w,ss,s# 8-12%:M;w,ss,s# S;w*	0-4%:M;w,ss 4-8%:M;w,ss,s 8+%:S;s	S;b
CUNNINGHAM	0.06-0.2	4.5-5.5	Mod.	High	High	>6.0	40-60	None	C	2-15%:S;pk 15+%:S;pk,s	2-7%:M;s,r 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:M;ss 8-15%:M;ss,s 15+%:S;s	2-8%:M;ss 4-8%:M;ss,s 8+%:S;s	2-15%:S;b 15+%:S;s,b
CUTHBERT	0.06-0.6	3.6-5.5	Mod.	High	High	>6.0	>60	None	C	5-15%:S;pk 15+%:S;pk,s	5-7%:M;s 7+%:S;s	5-8%:M;cl 8-15%:M;cl,s 15+%:S;s	5-8%:M;ss 8-15%:M;ss,s 15+%:S;s	5-8%:M;s,ss 8+%:S;s	5-15%:S;b 15+%:S;b,s
DASHER	2.0-6.0	3.6-4.4	Low	High	High	+3-0.5	>60	None	D	S;w,u	S;w,h	S;w,h	S;w,b	S;w,b	S;w
DAVIDSON	0.6-2.0	4.5-6.0	Low	High	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;pk,s 15+%:S;s	2-7%:M;p,s 7+%:S;s	2-8%:M;cl 8-15%:M;s,cl 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-15%:S;b 15+%:S;b,s
DECATUR	0.6-2.0	4.5-6.0	Mod.	High	Mod.	>6.0	>60	None	B	1-8%:L 8-15%:M;s 15+%:S;s	1-7%:M;p,s 7+%:S;s	1-8%:M;cl 8-15%:S;s 15+%:S;s	1-8%:M;ss 8-15%:M;s,ss 15+%:S;s	1-4%:M;ss 4-8%:M;s,ss 8+%:S;s	1-8%:M;b 8-15%:M;s,b 15+%:S;s
DEKALB	6.0-2.0	3.6-5.5	Low	Low	High	>6.0	20-40	None	C	0-15%:S;pf,r 15+%:S;pf,r,s	0-7%:S;r,p 7+%:S;r,s,p	0-15%:S;r 15+%:S;r,s	0-8%:M;r,st# 8-15%:M;r,st,s# 0-15%:S;,r* 15+%:S;s# 15+%:S;s;r*	0-4%:M;r,st 4-8%:M;r,st,s 8+%:S;s	0-8%:M;r,st 8-15%:M;r,st,s 15+%:S;s
DEWEY	0.6-2.0	4.5-5.5	Mod.	High	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;s,pk 15+%:S;s	2-7%:M;p,s 7+%:S;s	2-8%:M;cl 8-15%:M;s,cl 15+%:S;s	2-8%:M;ss 8-15%:M;s,ss 15+%:S;s	2-4%:M;ss 4-8%:M;s,ss 8+%:S;s	2-15%:S;b 15+%:S;s,b
DILLARD	0.2-0.6	4.5-5.5	Mod.	Mod.	High	2.0-3.0	>60	None Rare	C	S;w,pk	2-7%:S;w 7+%:S;s	S;w	None:M;w# None:S;w* Rare:S;f	None:M;w,s Rare:s;f	S;b
DOCENA	0.06-0.6	4.5-6.0	Mod.	Mod.	Mod.	1.5-3.0	>60	None Rare Occ	C	None, Rare:S;w,pk Occ:S;f,w,pk	None:S;w Rare,Occ:S;f,w	S;w	None:M;w# None:S;w* Rare,Occ:S;w,f	0-4%:None:M;w 4-6%None:M;w,s Rare,Occ:S;f	None:Rare:S;b Occ:S;b,f
DOGUE	0.2-0.6	3.6-5.5	Mod.	High	High	1.5-3.0	>60	None Rare	C	S;w,pk	0-7%:None:S;p,w 7+%:None:S;p,w,s 0-7%:Rare:S;p,f,w 7+%:Rare:S;p,f,s	S;cc,w	0-15%:None:M;ss,w# 0-15%:Rare:S;f# None:S;w* Rare:S;f,w#	8%None:M;ss,w,s# 8+%:None:S;s# 0-8%:Rare:S;f* 8+%:Rare:S;f,s*	S;b
DOTHAN	0.2-0.6	4.5-6.0	Low	Mod.	Mod.	3.0-5.0	>60	None	B	M;pk,w	0-2%:L 2-7%:M;s 7+%:S;s	0-8%:M;w 8-12%:M;w,s	0-8%:L# 8-12%:M;s# 0-8%:M;w* 8-10%:M;s*	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-10%:M;s
DOWELLTON	0.06-0.2	5.1-7.8	High	High	Mod.	0.5-1.0	40-60	None	D	S;w,pk	S;w	S;w,cl	S;w,ss	S;w,ss	S;w,ss,b
DUCKSTON	>20	3.6-8.4	Low	Low	Low	1.0-2.0	>60	Occ Freq	A/D	S;w,f,pf	S;w,f,p	S;w,cc	S;w,f	S;w,f	S;f
DUNBAR	0.2-0.6	4.5-5.5	Mod.	High	High	1.0-2.5	>60	None	D	S;w,pk	S;w	S;w	S;w	S;w	S;b
DUPLIN	0.2-0.6	4.5-5.5	Mod.	High	High	2.0-3.0	>60	None	C		S;w	M;w	M;w# S;w*	0-4%:M;w 4-8%:M;s,w	S;b

SOIL SERIES INTERPRETATIONS

SOIL SERIES	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	CORROSIVITY		DEPTH TO:		FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
				STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)								
DURHAM	0.6-2.0	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None	B	0-8%:L 8-10%:M;s	0-2%:M;p 2-7%:M;s 7+%:S;s	0-8%:L 8-10%:M;s	0-8%:L 8-10%:M;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-10%:M;s
DYKE	0.6-2.0	4.5-6.0	Mod.	High	Mod.	>6.0	>60	None	B	0-8%:M;pk 8-15%:M;pk,s 15+%:S;s	0-2%:M;p 2-7%:M;s,s 7+%:S;s	0-8%:M;cl 8-15%:M;cl,s 15+%:S;s	0-8%:M;ss 8-15%:M;s,ss 15+%:S;s	0-4%:M;ss 4-8%:M;ss,s 8+%:S;s	0-15%:S;b 15+%:S;s,b
ECHAW	2.0-20	4.5-6.0	Low	Low	High	2.5-5.0	>60	None	A	S;w,pf	S;p,w	M;cc	L# M;w*	L	L
EDGEMONT	0.6-6.0	3.6-5.5	Low	Low	High	>6.0	>40	None	B	0-8%:M;r,pk 8-15%:M;r,pk,s 15+%:S;s	0-7%:S;p 7+%:S;p,s	0-15%:S;cc 15+%:S;cc,s	0-8%:L# 0-8%:M;r* 8-15%:M;s#r* 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s
EDNEYTOWN	0.6-2.0	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None	B	2.8%:L 8-15%:M;S 15+%:S;s	2-7%:S;p 7+%:S;p,s	2-15%:S;cc 15+%:S;cc,s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
EDNEYVILLE	0.6-2.0	4.5-6.0	Low	Low	High	>6.0	>60	None	B	6-8%:L 8-15%:M;S 15+%:S;S	S;s	6-8%:L 8-15%:M;s 15+%:S;s	6-8%:L 8-15%:M;s 15+%:S;s	6-8%:M;s 8+%:S;s	6-8%:L 8-15%:M;s 15+%:S;s
ELLABELLE	0.6-2.0	4.5-5.5	Low	High	High	+1-0.5	>60	Freq	C	S;f,w	S;f,w	S;w,cc	S;f,w	S;f,w	S;f,w
EMORY	0.6-2.0	5.1-6.0	Low	Mod.	Mod.	5.0-6.0	>60	None Rare Occ	B	None:M;pk,w Rare:M;f,w,pk Occ:S;f	0-2%:None:M;p 2-5%:None:M;p,s Rare,Occ:S;f	None,Rare:L Occ:M;f	None:L# None:M;w* Rare,Occ:S;f	None:L Rare:Occ:S;f	None,Rare:S;b Occ:S;b,f
ENDERS	<0.06	3.6-5.5	High	High	High	>6.0	40-60	None	C	1-15%:S;pk 15+%:S;s,pk	1-7%:M;s,r 7+%:S;s	1-8%:M;cl 8-15%:M;cl,s 15+%:S;s	1-15%:S;ss 15+%:S;s,ss	1-8%:S;ss 8+%:S;s,ss	1-15%:S;b,ss 15+%:S;s,b,ss
ENNIS	2.0-6.0	4.5-6.0	Low	Low	.Mod.	>6.0	>60	Rare Occ	B	Rare:M;f Occ:S;f		Rare:L Occ:M;f	S;f	S;f	Rare:M;f Occ:S;f
ENON	0.06-0.2	5.1-7.8	High	High	Mod.	>6.0	>60	None	C	2-15%:S;pk 15+%:S;pk,s	2-7%:M;s 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-15%:S;ss 15+%:S;ss,s	2-8%:S;ss 8+%:S;ss,s	2-15%:S;b,ss 15+%:S;b,ss,s
ESTO	0.06-0.2	4.5-5.5	Mod.	High	High	>6.0	>60	None	B	2-15%:S;pk 15+%:S;pk,s	2-7%:M;s 7+%:S;s	0-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:M;ss 8-15%:M;ss,s 15+%:S;s	2-4%:M;ss 4-8%:M;ss,s 8+%:S;s	2-15%:S;b 15+%:S;b,s
ETOWAH	0.6-2.0	4.5-5.5	Low	Low	Mod.	>6.0	>60	None	B	0-8%:M;pk 8-15%:M;s;pk 15+%:S;s	0-2%:M;p 2-7%:M;s,p 7+%:S;s	2-8%:M;cl 8-15%:M;s,cl 15+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:M;b 8-15%:M;s,b 15+%:S;s
EUHARLEE	0.2-0.6	4.5-5.5	Low	Mod.	High	>6.0	>60	None	C	S;pk	2-7%:M;s 7+%:S;s	2-8%:M;cl 8-10%:M;s,cl	2-8%:L 8-10%:M;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:S;b 8-10%:S;s,b
EULONIA	0.2-0.6	4.5-6.5	Low	Mod.	High	1.5-3.5	>60	None	C	S;pk,w	L	S;w	M;w# S;w*	0-4%:M;w 4-6%:M;w,s	M;b,w
EUNOLA	0.6-2.0	4.5-5.5	Low	Low	High	1.5-2.5	>60	None Rare Occ	C	None,Rare:S;w Occ:S;w,f	None,Rare:S;w,p Occ:S;w,e,f	S;w,cc	None:M;w# None:S;w* Occ:S;f	None:M;w Rare,Occ:S;f	None:M;w Rare:M;wf Occ:S;f
EUSTIS	6.0-20	4.5-5.5	Low	Low	High	>6.0	>60	None	A	0-8%:L 8-15%:M;s 15+%:S;s	0-7%:S;p 7+%:S;p,s	0-15%:S;cc 15+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s

SOIL SERIES INTERPRETATIONS

	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
				CORROSIVITY		DEPTH TO:									
SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
EUTAW	<0.06	4.5-7.8	High	High	Mod.	0.5-1.5	>60	None	C	S;pk,w	L	S;w	M;w# S;w*	0-4%:M;w 4-6%:M;w,s	M;b,w
EVARD	0.6-2.0	4.5-6.0	Low	Mod.	High	>6.0	>60	None	B	2-8%:L 8-15%:M;s 15+%:S;s	2-7%:M;p,s 7+%:S;s	2-15%:S;cc 15%+:S;cc,s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
FACEVILLE	0.6-2.0	4.5-6.0	Low	Low	Mod.	>6.0	>60	None	B	0-8%:L 8-15%:M;s	0-2%:M;p 2-7%:M;s,p 7+%:S;s	0-8%:M;cl 8-15%:M;cl,s	0-8%:L 8-15%:M;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:M;b 8-15%:M;s,b
FANNIN	0.6-2.0	4.5-6.0	Low	Mod.	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;s,pk 15+%:S;s	2-7%:M;p,s 7+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-15%:S;b 15+%:S;b,s
FARRAGUT	0.2-0.6	4.5-5.5	Mod.	High	Mod.	>6.0	>48	None	C	2-15%:S;pk 15+%:S;s,pk	2-7%:M;s,s 7+%:S;s	2-8%:M;cl,s 8-15%:M;cl,s 15+%:S;s	2-8%:M;ss 8-15%:M;s,ss 8+%:S;s	2-4%:M;ss 4-8%:M;s,ss 8+%:S;s	2-15%:S;b 15+%:S;s,b
FLOMATION	6.0-2.0	4.5-6.0	Low	Low	Mod.	>6.0	>60	None	A	2-15%:S;pf 15+%:S;pf,s	2-7%:S;p 7+%:S;p,s	2-15%:S;cc 15+%:S;cc,s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
FOXWOTH	>20	4.5-6.0	Low	Low	High	3.5-6.0	>60	None	A	M;w	0-7%:S;s 7+%:S;p,s	S;cc	L# M;w*	0-4%:L 4-8%:M;s	L
FREEMANVILLE	0.2-0.6	5.1-6.0	Low	Mod.	Mod.	>6.0	>60	None	B	S;pk	0-2%:L 2-7%:M;s 7+%:S;s	0-8%:M;cl 8-12%:M;cl,s	0-8%:L 8-12%:M;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:M;b 8-12%:M;b,s
FRENCH	0.6-2.0	5.1-6.5	Low	Mod.	Mod.	1.0-2.5	>60	Occ Freq	C	S;f,w	S;f,w	S;cc,w	S;f,w	S;f,w	S;f
FRIPP	6.0-2.0	5.6-7.8	Low	Low	Low	>6.0	>60	None Rare	A	2-15%:S;pf 15+%:S;pf,s	2-7%:S;p 7+%:S;s,p	2-15%:S;cc 15+%:S;c,s	2-15%:None:M;s 15+%:None:S;s 2-15%:Rare:S;f 15+%:Rare:S;f,s	2-8%:None:M;s 8+%:None:S;s 2-8%:Rare:S;f 8+%:Rare:S;f,s	2-15%:None:M;s 2-15%:Rare:M;f,s
FULLERTON	0.6-2.0	4.5-5.5	Mod.	High	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:S;s 15+%:M;pk,s	2-7%:M;p,s 7+%:S:s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:M;ss 8-15%:None:M;s,ss 15+%:S;s	2-4%:M;ss 4-8%:M;s,ss 8+%:S;s	2-8%:M;ss,b 8-15%:M;ss,b,s 15+%:M;ss,b,s
FUQUAY	0.06-0.2	4.5-6.0	Low	Low	High	4.0-6.0	>60	None	B	0-8%:M;pk 8-10%:M;pk,s	0-2%:L 2-7%:M;s 7+%:S;s	0-8%:L 8-10%:M;s	0-8%:L# 8-10%:M;s# 0-8%:M;w* 8-10%:M;w,s*	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-10%:M;s
GALESTOWN	>6.0	3.6-5.5	Low	Low	High	>6.0	>60	None	A	0-15%:S;pf 15+%:S;pf,s	0-7%S;p 7+%:S;p,s	0-15%:S;cc 15+%:S;cc,s	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S:s
GAYLESVILLE	0.06-0.2	3.6-6.0	Mod.	High	High	0-1.5	>60	Freq	D	S;f,w,pk	L	S;w	S;w,f	S;w,f	S;w,f,b
GEORGEVILLE	0.6-2.0	4.5-5.5	Low	High	High	>6.0	>60	None	B	2-8%:M;pk 8-15:M;s,pk 15+%:S;s	2-7%:M;s,p 7+%:S;s	2-8%:M;cl 8-15%:M;s,cl 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4:L 4-8%:M;s 8+%:S;s	2-15%:S;b 15+%:S;s,b
GILEAD	0.06-0.6	4.5-5.5	Low	Mod.	High	1.5-2.5	>60	None	C	S;pk,w	0-2%:L 2-7%:M;s 7+%:S;s	S;w	0-8%:M;w# 8-15%:M;s,w# S;w*	0-4%:M;w 4-8%:M;s,w 8+%:S;s	S;b
GILPIN	0.6-2.0	3.6-5.5	Low	Low	High	>6.0	20-40	None	C	0-15%:S;r 15+%:S;r,s	0-7%:S;r 7+%:S;r,s	0-8%:M;r 8-15%:M;r,s 15+%:S;s	0-8%:L# 0-8%:M;r* 8-15%:M;s#,r* 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s

SOIL SERIES INTERPRETATIONS

SOIL SERIES	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	CORROSIVITY: STEEL	CORROSIVITY: CONCRETE	DEPTH TO: WATER TABLE (Ft.)	DEPTH TO: BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
GOLDSBORO	0.6-2.0	3.6-5.5	Low	Mod.	High	2.0-3.0	>60	None	B	S;w	S;w	S;w	M;w# S;w*	0-4%:M;w 4-8%:M;s,w	M;w
GOLDSTON	2.0-6.0	3.6-5.5	Low	Mod.	High	>6.0	10-20	None	C	2-15%:S;r 15+%:S;r,s	2-7%:S;r,p 7+%:S;r,s,p	2-15%:S;r 15+%:S;r,s	2-15%:M;r,s# 15+%:S;s S;r*	2-4%:M;r,s# 4-8%:M;s,r,st 8+%:S;s	2-8%:M;r,st 8-15%:S;s 15+%:S;s
GORGAS	2.0-6.0	4.5-5.5	Low	Low	Mod.	>6.0	10-20	None	D	2-15%:S;r 15+%:S;r,s	2-7%:S;p,k 7+%:S;p,r,s	2-15%:S;r 15+%:S;r,s	2-15%:S;r 15+%:S;r,s	2-8%:S;r 8+%:S;r,s	2-15%:s,r 15+%:S;r,s
GRADY	0.06-0.2	3.6-5.5	Mod.	High	High	+2-1.0	>60	None	D	S;pk,w	S;w	S;w	S;s	S;w	S;w
GREENVILLE	0.6-2.0	4.5-6.0	Low	Mod.	High	>6.0	>60	None	B	0-8%:M;pk 8-15%:M;s,pk	0-2%:M;p 2-7%:M;s,p 7+%:S;s	0-8%:M;cl 8-15%:M;s,cl	0-8%:L 8-15%:M;s	0-4%:L 4-8%:M;s 8-15%:S;s	0-8%:L 8-15%:M;s
GRITNEY	0.06-0.2	4.5-5.5	High	High	Mod.	>6.0	>60	None	C	S;pk	2-7%:M;s 7+%:S;s	M;cl	S;ss	2-8%:S;ss 8+%:S;ss,s	S;ss,b
GROVER	0.6-2.0	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None	B	2-8%:L 8-15%:M;s 15+%:S;s	2-7%:M;s,p 7+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;s,b 15+%:S;s
GUTHRIE	0.6-0.2	3.6-5.0	Low	High	High	0.5-1.0	>60	None Occ Freq	D	None, Rare:S;w,pk Occ,Freq:S;w,pk	L	S:w	None:S;w Rare,Occ,Freq:S;f,w	None:S;w Rare,Occ,Freq:S;f,w	None,Rare:S;b,w Occ,Freq:S;b,w,f
GWINNETT	0.6-2.0	5.1-6.6	Low	High	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;s,pk 15+%:S;s	2-7%:M;s,p 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;s,b 15+%:S;s
HAMBLEN	0.6-2.0	4.5-7.3	Low	Mod.	Mod.	2.0-3.0	>60	None Rare Occ Freq	C	None,Rare:S;w Occ,Freq:S;f,w	None,Rare:S;w Occ,Freq:S;f,w	None,Rare:M;w Occ,Freq:M;w,f	None:M;w# Rare,Occ,Freq:S;f# None:S;w* Rare,Occ,Freq:S;f,w*	None:M;w Rare,Occ,Freq:S;r	None,Rare:M;b,w Occ,Freq:S;f
HANCEVILLE	0.6-2.0	4.5-5.5	Mod.	High	High	>6.0	>60	None	B	0-8%:M;pk 8-15%:M;pk,s 15+%:S;s	0-2%:M;p 2-7%:M;p,s 7+%:S;s	0-8%:M;cl 8-15%:M;cl,s 15+%:S;s	0-8%:M;ss 8-15%:M;ss,s 15+%:S;s	0-4%:M;ss 4-8%:M;ss,s 8+%:S;s	0-15%:S;b 15+%:S;b,s
HARTSELLS	0.6-2.0	3.6-5.5	Low	Mod.	High	>6.0	20-40	None	B	2-15%:S;r 15+%:S;r,s	2-7%:S;r 7+%:S;r,s	2-15%:S;r 15+%:S;r,s	2-15%:M;r,s# 15+%:S;s S;r	2-4%:M;r 4-8%:M;s,r 8+%:S;s	2-8%:M;r 8-15%:M;s,r 15+%:S;s
HATBORO	0.6-2.0	4.5-7.3	Low	High	Mod.	0-0.5	>60	Occ Freq	D	S;f,w	S;f,w,p	S;w	S;f,w	S;f,w	S;f,w
HAYESVILLE	0.6-2.0	4.5-6.0	Low	Mod.	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;pk,s 15+%:S;s	2-7%:M;s,p 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;b,s 15-25%:S;s
HAYWOOD	6.0-2.0	5.1-6.5	Low	Low	Mod.	>6.0	>60	None	B	2-8%:S;pf 8-15%:M;s,pf 15+%:S;s,pf	2-7%:S;p 7+%:S;s,p	2-8%:L 8-15%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
HAZLEHURST	0.06-0.2	4.5-5.5	Low	High	High	0.5-2.0	>60	None	C	S;w,pk	0-2%:L 2-3%:M;s	S;w	S;w	S;w	S;w
HECTOR	2.0-6.0	4.5-5.5	Low	Low	Mod.	>6.0	10-20	None	D	2-15%:S;r 15+%:S;s,r	2-7%:S;r,p 7+%:S;s,r,p	2-15%:S;r 15+%:S;s,r	2-15%:S;r 2-8%:S;r* 15+%:S;r	2-8%:S;r 8+%:S;s,r	2-15%:S;r 15+%:S;s,r

SOIL SERIES INTERPRETATIONS

	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
				CORROSIVITY		DEPTH TO:									
SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
HELENA	0.06-0.2	4.5-5.5	High	High	High	1.5-2.5	>60	None	C	S;pk,w	0-2%:L 2-7%:M;s 7+%:S;s	S;w	S;ss# S;ss,w*	0-8%:S;ss 8+%:S;s,ss	S;ss,b
HENDERSON	0.06-0.2	4.5-5.5	Low	High	Mod.	>6.0	>60	None	B	2-15%:S;pk 15+%:S;pk,s	2-7%:M;s,p 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-15%:S;b 15+%:S;s,b
HERNDON	0.6-2.0	3.6-5.5	Low	High	High	>6.0	>60	None	B	2-8%:M;cl 8-15%:M;s,cl	2-7%:M;p,s 7+%:S;s	2-8%:M;cl 8-15%:M;s,cl	2-8%:L 8-15%:M;s	2-4%:L 4-8%:M;s 8+%:S;s	S;b
HEROD	0.6-2.0	5.1-7.3	Low	High	Mod.	0.5-1.5	>60	Freq	D	S;f,w	S;f,w	S;w	S;f,w	S;f,w	S;f,w
HIWASSEE	0.6-2.0	4.5-6.5	Mod.	Mod.	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%M;pk,s 15+%:S;s	0-7%:M;p,s 7+%:S;s	2-8%:M;cl 8-15%:M;s,cl 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-15%:S;b 15+%:S;s,b
HOLSTON	0.6-2.0	4.5-5.5	Low	Mod.	High	>6.0	>60	None	B	0-8%:M;pk 8-15%:M;s,pk 15+%:S;s	0-2%:M;p 2-7%:M;s,p 7+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s
HORNSVILLE	.02-.06	4.5-6.0	Low	High	High	2.5-3.5	>60	None Rare	C	S;w,pk	0-7%:None:S;w 7+%:None:S;w,s Rare:S;w	M:cl,w	None:L# None:M;w* Rare:S;f	0-4%:None:L 4-8%:None:M;s Rare:S;f	None:M;b Rare:M;b,f
HULETT	0.6-2.0	4.5-5.5	Low	High	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;s,pk	2-7%:M;s,p 7+%:S;s	2-8%:M:cl 8-15%:M;s,cl	2-8%:M;cl 8-15%:M:s	2-8%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:S;s,b
IREDELL	0.06-0.2	5.5-7.8	High	High	Low	1.0-2.0	>60	None	D	S;w,pk	0-7%:S;w 7+%:S;s,w	S;w	S;ss,w	0-8%:S;w,ss 8+%:S;w,ss,s	S;ss,b
IRVINGTON	0.06-0.2	4.5-5.5	Low	Mod.	Mod.	1.5-3.5	>60	None	C	S;w,pk	0-2%:L 2-5%:M;s	S;w	M;w# S;w*	M;w	M;w
IUKA	0.6-2.0	4.5-5.5	Low	Mod.	High	1.0-3.0	>60	Rare Occ Freq	C	Rare:S;w Occ, Freq:S; w,f	S;f,w	S;w	S;w,f	S;w,f	Rare:M;w,f Occ,Freq:S;f
IZAGORA	0.06-0.2	3.6-5.5	Mod.	Mod.	High	2.0-3.0	>60	None Rare Occ	C	None,Rare:S;W,pk Occ:S;f,w,pk	0-7%:None:S;w 7+%:None:S;w,s 0-7%:Rare,Occ:S;f,w 7+%:Rare,Occ:S;f,w,s	S;w	None:M;w# None:S;w* Rare,Occ:S;f,w	None:M;w Rare,Occ:S;f,w	None,Rare:S;b Occ:S;b,f
JOHNS	0.6-2.0	4.5-5.5	Low	Mod.	High	1.5-3.0	>60	None Rare	C	S;w	None:S;w,p Rare:S;w,f,p	S;w	None:M;w# None:S;w* Rare:S;w,f	None:M;w Rare:S;f	None:M;w Rare:M;w,f
JOHNSTON	2.0-6.0	4.5-5.5	Low	High	High	+1-1.5	>60	Freq	D	S;f,w	S;f,w,p	S;w	S;f,w	S;f,w	S;f,w
JUNALUSKA	0.6-2.0	4.5-6.0	Low	Mod.	High	>6.0	20-40	None	B	8-15%:S;r 15+%:S;r,s	S;r,s	S;f,w	8-15%:M;s# 8-15%:M;s,r* 15+%:S;s	S;s	8-15%:M;b,s 15+%:S;s
KALMIA	0.6-2.0	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None Rare	B	L	None:S;s Rare:S;f,p	8-15%:M;r,s 15+%:S;s	None:L Rare:S;f	0-4%:None:L 4-6%:None:M;s Rare:S;f	None:L Rare:M;f
KERSHAW	>20	4.5-6.0	Low	Low	High	>6.0	>60	None	A	S;pf	2-7%:S;s 7+%:S;s,p	S;cc	2-8%:L 8-15%:M;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s

SOIL SERIES INTERPRETATIONS

	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
				CORROSIVITY		DEPTH TO:									
SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
KINGSLAND	6.0-20	4.5-6.5	Low	Mod.	Mod.	0-0.5	>60	Occ Freq	A/D	S;f,w	S;f,p,h	S;w,h	S;f,w	S;f,w	S;f,w
KINSTON	0.6-2.0	4.5-5.5	Low	High	High	0-1.0	>60	Occ Freq	B/D	S;f,w	S;f,w	S;f,w	S;w	S;f,w	S;f,w,b
KOLOMOKI	0.6-2.0	4.5-6.0	Low	Mod.	Mod.	>6.0	>60	None Rare	B	None:L Rare:M;f	None:S;p Rare:S;p,f	S;cc	None:L Rare:S;f	0-4%:L 4-8%:M;s 8+%:S;f	None:M;b Rare:M;b,f
KUREB	6.0-20	4.5-7.3	Low	Low	Low	>6.0	>60	None	A	0-15%:S;pf 15+%:S; p,f,s	0-15%:S;p 15+%:S;p,s	0-15%:S;cc 15+%:S;s,cc	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s
LAKELAND	6.0-20	4.5-6.0	Low	Low	Mod.	>6.0	>60	None	A	0-8%:L 8-15%:M;s 15+%:S;s	0-7%:S;p 7+%:S;s,p	0-15%:S;cc 15+%:S;s,cc	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s
LEADVALE	0.06-0.6	4.5-5.5	Low	Mod.	Mod.	2.0-3.0	>48	None Rare	C	S;w,pk	0-7%:None:S;w 7+%:None:S;s,w 0-7%:Rare:S;f,w 7+%:Rare:S;f,w,s	S;w	None:M;w# None:S;w* Rare:S;f	0.8%:None:M;w 8+%:None:S;s Rare:S;f	0-8%:M;b,w 8-15%:M;sw,b
LEAF	0.06-0.2	3.6-5.5	High	High	Mod.	0.5-1.5	>60	None Rare Occ Freq	D	None,Rare:S;w,pk Occ,Freq:S;f,w,pk	None,Rare:L Occ,Freq:S;f	S;w	None:S;w,ss Rare,Occ,Freq:S;f, w,ss	None:S;w,ss Rare,Occ,Freq:S;f, w,ss	S;w,ss,b
LEEFIELD	.02-.06	4.5-5.5	Low	Mod.	High	1.5-2.5	>60	None	C	S;w,pk	M;w,p	S;w	M;w# S;w*	M;w	M;w
LEESBURG	0.6-2.0	4.5-5.5	Mod.	Low	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;pk,s 15+%:S;s	2-7%:M;p,s 7+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
LEON	0.6-6.0	3.6-5.5	Low	High	High	0-1.0	>60	None	B/D	S;w,pf	S;w,p	S;w,cc	S;w	S;w	S;w
LEVY	0.06-0.2	3.6-5.5	High	High	High	+2-+1	>60	Freq	D	S;f,pk	S;f	S;f	S;f,ss	S;f,ss	S;f,f
LILY	2.0-6.0	3.6-5.5	Low	Mod.	High	>6.0	20-40	None	B	0-15%:S;r 15+%:S;r,s	0-7%:S;p,r 7+%:S;p,r,s	0-15%:S;r 15+%:S;r,s	0-15%:M;r,s# 0-15%:S;r* 15+%:S;s#,r*	0-4%:M;r 4-8%:M;r,s 8+%:S;s	0-8%:M;r 8-15%:M;r,s 15+%:S;s
LINKER	0.6-2.0	3.6-5.5	Low	Low	High	>6.0	20-40	None	B	1-15%:S;r 15+%:S;s,r	1-7%:S;r 7+%:S;s,r	1-15%:S;r 15+%:S;s,r	1-8%:M;r# 8-15%:M;r,s# 1-15%:S;r* 15+%:S;s	1-4%:S;r 4-8%:M;r,s 8+%:S;s	1-8%:M;r 8-15%:M;r,s 15+%:S;s
LOCKHART	2.0-6.0	5.1-6.5	Low	Low	Mod.	>6.0	>60	None	B	2-8%:L 8-15%:M;S 15+%:S;s	2-7%:S;p 7+%:S;s,p	2-8%:L 8-15%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
LOCUST	0.06-0.2	4.5-5.5	Low	Mod.	High	1.5-2.0	>60	None	C	S;pk,w	0-7%:S;w 7+%:S;s,w	S;w	0-8%:M;w# 8-10%:M;s# S;w*	0-4%:M;w 4-8%:M;s,w 8+%:S;s	0-8%:M;w 8-10%:M;s,w
LOUISA	2.0-6.0	4.5-6.0	Low	Low	Mod.	>6.0	>60	None	B	6-8%:L 8-15%:M;s 15+%:S;s	S;p,s	6-8%:L 8-15%:M;s 15+%:S;s	6-8%:L 8-15%:M;s 15+%:S;s	6-8%:M;s 8+%:S;s	6-8%:L 8-15%:M;s 15+%:S;s
LOUISBURG	6.0-20	4.5-6.0	Low	Low	Mod.	>6.0	>40	None	B	2-8%:M;r 8-15%:M;r,s 15+%:S;s	2-7%:S;p 7+%:S;p,s	2-8%:M;r 8-15%:M;r,s 15+%:S;s	2-8%:M;r* 2-8%:L# 8-15%:M;s# 8-15%:M;r,s*	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s

SOIL SERIES INTERPRETATIONS

SOIL SERIES	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	CORROSIVITY: STEEL	CORROSIVITY: CONCRETE	DEPTH TO: WATER TABLE (Ft.)	DEPTH TO: BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
LOWNDES	0.6-2.0	4.5-6.0	Low	Mod.	High	>6.0	>60	None	B	5-8%:M;pk 8-15%:M;s,pk 15+%:S;s	5-7%:S;p 7+%:S;s,p	5-15%:S;cc 15+%:S;s,cc	15+%:S;s 5-8%:L 8-15%:M;s 15+%:S;s	5-8%:M;s 8+%:S;s	5-8%:L 8-15%:M;s 15+%:S;s
LUCY	0.6-2.0	4.5-5.5	Low	Low	High	>6.0	>60	None	A	0-8%:L 8-15%:M;s 15+%:S;s	0-7%:S;p 7+%:S;p,s	0-8%:M;cc 8-15%:M;s,cc 15+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s
LUMBEE	0.6-2.0	4.5-5.5	Low	High	High	0-1.5	>60	Rare Occ Freq	B/D	Rare:S;w Occ,Freq:S;w,f	Rare:S;p,w Occ,Freq:S;p,w,f	S;w,cc	S;w,f	S;w,f	Rare:S;w Occ,Freq:S;w,f
LYERLY	<0.06	4.5-7.3	High	High	Mod.	>6.0	20-40	None	D	S;pk,r	1-7%:S;r 7+%:S;s,r	S;r	S;ss S;r,ss	1-8%:S;ss 8+%:S;s,ss	S;ss,b
LYNCHBURG	0.6-2.0	3.6-5.5	Low	High	High	0.5-1.5	>60	None	C	S;w	S;w	S;w	S;w	S;w	S;w
LYNN HAVEN	0.6-6.0	3.6-5.5	Low	High	High	0-1.0	>60	None	B/D	S;w,pf	S;p,w	S;cc,w	S;w	S;w	S;w
MADISON	0.6-2.0	4.5-6.0	Low	High	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;pk,s 15+%:S;s	2-7%:M;p 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	0-15%:S;b 15+%:S;s;b
MANDARIN	0.6-2.0	3.6-7.3	Low	Mod.	High	1.5-3.5	>60	None	C	S;w,pf	S;pw	S;cc,w	M;w# S;w*	M;w	M;w
MARLBORO	0.6-2.0	4.5-6.0	Low	High	High	>6.0	>60	None	B	0-8%:M;pk 8-15%:M;pk,s	0-2%:M;p 2-7%:M;s,p 7+%:S;s	0-8%:M;cl 8-15%:M;cl,s	0-8%:L 8-15%:M;s	0-4%:L 4-8%:M;s 8-15%:S;s	0-8%:M;b 8-15%:M;b,s
MASADA	0.6-2.0	4.5-5.5	Mod.	Mod.	High	>6.0	>60	None	C	0-8%:M;pk 8-15%:M;pk,s 15+%:S;s	0-2%:M;p 2-7%:M;p,s 7+%:S;s	0-8%:M;cl 8-15%:M;cl,s 15+%:S;s	0-8%:M;ss 8-15%:M;ss,s 15+%:S;s	0-4%:M;ss 4-8%:M;s,ss 8+%:S;s	0-15%:S;b 15+%:S;b,s
MASCOTTE	0.6-2.0	3.6-5.5	Low	High	High	0-1.0	>60	None	B/D	S;w,pf	S;w,p	S;w,cc	S;w	S;w	S;w
MAXTON	0.6-2.0	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None Rare	B	L	S;p	S;cc	None:L Rare:S;f	0-4%:None:L 4-8%:None:M;s Rare:S;f	None:L Rare:M;f
MECKLENBURG	0.06-0.2	5.6-7.3	Mod.	High	Mod.	>6.0	>48	None	C	2-15%:S;pk 15+%:S;s,pk	2-7%:M;s,r,p 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:S;ss 8-15%:M;ss,s 15+%:S;s	2-4%:M;ss 4-8%:M;ss,s 8+%:S;s	2-15%:S;b 15+%:S;s,b
MEGGETT	0.06-0.2	5.1-8.4	High	High	Mod.	0-1.0	>60	None Rare Occ Freq	D	None,Rare:S;pk,w Occ,Freq:S;pk,w,f	None,Rare:S;w Occ,Freq:S;w,f	S;w	None:S;w,ss Occ,Freq:S;f,w,ss	None:S;w,ss Occ,Freq:S;f,w,ss	None:Rare:S;w,ss Occ,Freq:S;f,w,ss
MINVALE	0.6-2.0	4.5-5.5	Low	Mod.	Low	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;s,pk 15+%:S;s	2-7%:M;s,p 7+%:S;s	2-8%:L 8-15%:S;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;b,s 15+%:S;s
MOLENA	6.0-2.0	4.5-6.0	Low	Low	High	>6.0	>60	None Rare	A	0-15%:S;pf 15+%:S;pf,s	0-7%:S;p 7+%:S;s	0-15%:S;cc 15+%:S;cc,s	0-8%:None:L 8-15%:None:M;s 15+%:None:S;s 0-15%:Rare:S;f 15+%:Rare:S;f,s	0-4%:L 4-8%:M;s 8-15%:S;s Rare:S;f	0-8%:L 8-15%:M;s 15+%:S;s Rare:M;f
MONTEVALLO	0.6-2.0	4.5-6.0	Low	Mod.	Mod.	>6.0	10-20	None	D	2-15%:S;r 15+%:S;s,r	2-7%:S;r 7+%:S;s,r	2-15%:S;r 15+%:S;s,r	2-8%:M;r# 8-15%:S;s,r# 15+%:S;s 2-15%:S;r*	2-4%:M;r 4-8%:M;r,s 8+%:S;s	2-8%:M;r 8-15%:M;s,r 15+%:S;s

SOIL SERIES INTERPRETATIONS

SOIL SERIES	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
				CORROSIVITY		DEPTH TO:									
	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
MOUNTAINBURG	2.0-6.0	4.5-5.5	Low	Low	High	>6.0	12-20	None	D	1-15%:S;r 15+%:S;s,r	1-7%:S;r,p 7+%:S;s,r,p	1-15%:S;r 15+%:S;s,r	1-15%:S;r 15+%:S;r,s	1-8%:S;r 8+%:S;r,s	1-15%:S;r 15+%:S;r,s
MOUNTAINVIEW	0.6-2.0	4.5-5.5	Mod.	Mod.	Mod.	>6.0	>60	None	B	2-8%:L 8-15%:M;s	2-7%:M;s,p 7-15%:S;s	2-8%:L 8-15%:M;s	2-8%:L 8-15%:M;s	2-4%:L 4-8%:M;s 8-15%:S;s	2-8%:L 8-15%:M;s
MUCKALEE	0.6-2.0	5.1-8.4	Low	High	Mod.	0.5-1.5	>60	Freq	D	S;f,w	S;f,w	S;w,cc	S;f,w	S;f,w	S;f,w
MUSELLA	0.6-2.0	5.1-6.5	Low	Mod.	Mod.	>6.0	14-20	None	B	6-15%:S;r 15+%:S;r,s	S;r,s	6-15%:S;r 15+%:S;r,s	6-15%:M;r# 15+%:S;s# 6-15%:S;r* 15+%:S;r,s*	6-8%:M;s,r 8+%:S;s	6-15%:M;r,s 15+%:S;s
MYATT	0.2-2.0	3.5-5.5	Low	High	High	0-1.0	>60	None Rare Occ Freq	D	None, Rare:S;w,pk Occ, Freq:S;w,f,pk	None,Rare:S;w Occ,Freq:S;w,f	S;w	None:S;w Rare,Occ,Freq:S;w, f	None:S;w Rare,Occ,Freq:S;w, f	None,Rare:S;w Occ,Freq:S;w,f
NANKIN	0.2-0.6	4.5-5.5	Low	High	High	>6.0	>60	None	C	0-15%:S;pk 15+%:S;pk,s	0-2%:M;p 2-7%:M;p,s 7+%:S;s	0-8%:M;cl 8-15%:M;s,cl 15+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
NANTAHALA	0.6-2.0	4.5-6.0	Mod.	High	Mod.	>6.0	40-60	None	B	2-8%:M;r,pk 8-15%:M;r,pk,s 15+%:S;s	2-7%:M;r,s 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:M;ss 8-15%:M;ss,s 15+%:S;s	2-4%:M;ss 4-8%:M;ss,s 8+%:S;s	2-15%:S;b 15+%:S;b,s
NASON	0.6-2.0	4.5-5.5	Mod.	Mod.	High	>6.0	40-60	None	C	0-8%:M;pk,r 8-15%:M;s,r,pk 15+%:S;s	0-2%:M;p,r 2-7%:M;pk,r,s 7+%:S;s	0-8%:M;c 8-15%:S;s,cl 15+%:S;s	0-8%:M:ss 8-15%:M;s,ss 15+%:S;s	0-4%:M;ss 4-8%:M;s,ss 8+%:S;s	0-15%:S;b 15+%:S;s,b
NAUVOO	0.6-2.0	4.5-6.0	Low	Low	High	>6.0	40-60	None	B	2-8%:M;r,pk 8-15%:M;r,pk,s 15+%:S;s	2-7%:M;p,r,s 7+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;b,s 15+%:S;s
NELLA	0.6-2.0	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M,s,pk 15+%:S;s	2-7%:M;s,p,st 7+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
NORFOLK	0.6-2.0	4.5-5.5	Low	Mod.	High	4.0-6.0	>60	None	B	0-8%:M;w 8-10%:M;s,w	0-7%:M;p 7+%:S;s	0-8%:M;w 8+%:M;s,w	0-8%:L# 8-10%:M;s# 0-8%:M;w* 8-10%:M;w,s*	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-10%:M;s
OCHLOCKONEE	2.0-6.0	4.5-5.5	Low	Low	High	3.0-5.0	>60	Rare Occ	B	Rare:S;w Occ, Freq:S;w,f	S;f,p,w	M;w	S;f	S;f	Rare:M;f Occ,Freq:S;f
OCILLA	0.6-2.0	4.5-5.5	Low	High	Mod.	1.0-2.5	>60	None	C	S;w	S;w,p	S;w,cc	None:M;w# None:S;w* Rare:S;w,f	None:S;w Rare:S;f,w	None:M;w Rare:M;w,f
OGEECHEE	0.6-2.0	4.5-6.0	Low	High	High	0-1.0	>60	None Rare Occ Freq	B/D	None, Rare:S;w Occ, Freq:S;w,f	None,Rare:S;w Occ,Freq:S;w,f	S;w	None:S;w Rare,Occ,Freq:S;w,f	None:S;w Rare,Occ,Freq:S;w,f	None:Rare:S;w Occ,Freq:S;w,f
OKTIBBEHA	>0.06	4.5-8.4	High	High	High	>6.0	>60	None	D	1-15%:S;pk 15+%:S;pk,s	1-2%:L 2-7%:M;s 7+%:S;s	1-8%:M;cl 8-15%:M;cl,s 15+%:S;s	1-15%:S;ss 15+%:S;ss,s	1-8%:S;ss 15+%:S;ss,s	1-15%:S;ss,b 15+%:S;ss,b,s
OLUSTEE	0.6-2.0	4.5-5.5	Mod.	High	High	0-1.0	>60	None	B/D	S;w	S;w,p	S;w,cc	S;w	S;w	S;w
ORANGEBURG	0.6-2.0	4.5-6.0	Low	Mod.	Mod.	>6.0	>60	None	B	0-8%:L 8-15%:M;s 15+%:S;s	0-2%:M;p 2-7%:M;s,p 7+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s	0-8%L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s

SOIL SERIES INTERPRETATIONS

	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
				CORROSIVITY		DEPTH TO:									
SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
OSIER	6.0-2.0	3.6-6.0	Low	High	High	0-1.0	>60	None Rare Occ Freq	A/D	None, Rare:S;w,pf Occ, Freq:S;w,f,pf	None,Rare:S;w,p Occ,Freq:S;w,f,p	S;w,cc	None:S;w Rare,Occ,Freq:S;w,f	None:S;w Rare,Occ,Freq:S;w,f	None:Rare:S;w Occ,Freq:S;w,f
OUSLEY	6.0-20	4.5-5.5	Low	Low	High	1.5-3.0	>60	Occ Freq	C	S;f,w,pf	S;f,p,w	S;cc,w	S;f# S;f,w*	S;f	S;f
PACOLET	0.6-2.0	4.5-6.0	Low	High	High	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;s,pk 15+%:S;s	2-7%:M;s,p 7+%:S;s	2-8%:M;cl 8-15%:M;s,cl 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;s,b 15+%:S;s
PASQUATANK	0.6-2.0	4.5-5.5	Low	High	Mod.	1.0-2.0	>60	None	B/D	S;w	S;w,p	S;w	S;w	S;w	M;w
PELHAM	0.6-2.0	3.6-5.5	Low	High	High	0-1.0	>60	None Rare Occ Freq	B/D	None, Rare:S;w Occ, Freq:S;w,f	None,Rare:S;w,p Occ,Freq:S;w,f,p	S;w,cc	None:S;w Rare,Occ,Freq:S;w,f	None:S;w Rare,Occ,Freq:S;w,f	None:Rare:S;w Occ,Freq:S;f,w
PELION	.06-0.6	3.6-5.5	Low	High	High	1.0-2.5	>60	None	B/D	S;pk,w	0-7%:S;w 7+%:S;s	S;w	S;w	0-8%:S;w 8+%:S;s,w	0-8%:M;w 8-15%:M;w,s
PERSANTI	0.06-0.2	3.6-5.5	Mod.	High	High	2.0-3.5	>60	None	C	S;w,pk	0-2%:L 2-6%:M;s	S;w	M;ss,w# S;w*	0-4%:M;w,ss 4-6%:M;w,ss,s	M;b,ss,w
PICKNEY	6.0-20	3.6-6.0	Low	High	High	+1-1.0	>60	None	A/D	S;w,pf	S;w,p	S;w,cc	S;w	S;w	S;w
PLUMMER	0.6-2.0	3.6-5.5	Low	Mod.	High	0-1.0	>60	None Rare Occ Freq	B/D	None, Rare:S;w,pf Occ, Freq:S;w,f,pf	None,Rare:S;w,p Occ,Freq:S;w,f.p	S;w,cc	None:S;w Rare,Occ,Freq:S;w,f	None:S;w Rare,Occ,Freq:S;w,f	None:Rare:S;w Occ,Freq:S;w,f
POINDEXTER	0.6-2.0	5.1-7.3	Low	Mod.	Mod.	>6.0	20-40	None	B	2-15%:S;r 15+%:S;r,s	2-7%:S;p,r 7+%:S;p,r,s	2-8%:M;r 8-15%:M;r,s 15+%:S;s	2-8%:L#r* 8-15%:M;s# 8-15%:M;s,r* 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
PONZER	0.06-2.0	3.6-7.8	Low	High	High	0-1.0	>60	None Rare Occ Freq	D	Rare:S;w,pk Occ, Freq:S;f,w,pk	S;f,w,h	S;h,w	S;f,w,b	S;w,f,b	Rare:S;w,b Occ,Freq:S;f,w,b
POOLER	0.06-0.2	3.6-5.5	Mod.	High	High	0-1.0	>60	None	D	S;pk,w	L	S;w	S;w	S;w	S;w,b
PORTERS	2.0-6.0	4.5-6.0	Low	Low	High	>6.0	40-60	None	B	8-15%:M;r,s 6-8%:M;r 15+%:S;s	S;s,p	6-8%:M;r 8-15%:M;r,s 15+%:S;s	6-8%:L# 8-15%:M;s# 6-15%:M;r,s* 15+%:S;s	6-8%:M;s 8+%:S;s	6-8%:L 8-15%:M;s
PORTSMOUTH	0.6-2.0	3.6-6.0	Low	High	High	0-1.0	>60	None	B/D	S;w,pf	S; w,p	S;w,cc	None:S;w Rare:S;w,f	None:S;w Rare:S;w,f	S;w
POTTSBURG	0.6-2.0	3.6-6.0	Low	High	High	0-1.0	>60	None Rare Occ	B/D	None,Rare:S;w,pf Occ:S;f,w,pf	None,Rare:S;w Occ:S;p,f,w,	S;w,cc	None:S;w Rare,Occ:S;f,w	None:S;w Rare,Occ:S;f,w	None,Rare:S;w Occ:S;w,f
RABUN	0.6-2.0	5.1-6.5	Low	High	Mod.	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;s,pk 15+%:S;s	2-7%:M;s,p 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:L 8-15%:M;s 8+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;s,b 15+%:S;s
RAINS	0.6-2.0	3.6-5.5	Low	High	High	0-1.0	>60	None Rare Occ Freq	B/D	None,Rare:S;w Occ.Freq:S;f,w	None,Rare:S;w Occ,Freq:S;w,f	S;w	None:S;w Rare,Occ,Freq:S;w,f	None:S;w Rare,Occ,Freq:S;w,f	None,Rare:S;w Occ,Freq:S;w,f

SOIL SERIES INTERPRETATIONS

SOIL SERIES	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	CORROSIVITY		DEPTH TO:		FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
				STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)								
RAMSEY	6.-20	4.5-5.5	Low	Low	Mod.	>6.0	10-20	None	D	3-15%:S;r, 15+%:S;r,s	3-7%:S;p,r 7+%:S;p,r,s	3-15%:S;r 15+%:S;r,s	3-15%:S;r 15+%:S;r,s	3-8%:S;r 8+%:S;r,s	3-15%:S;r 15+%:S;r,s
RED BAY	0.6-2.0	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None	B	0-8%:L 8-15%:M;s	2-7%:M;p 2-7%:M;p,s 7+%:S;s	0-8%:L 8-15%:M;s	0-8%:L 8-15%:M;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s
REMBERT	0.06-0.2	4.5-5.5	Low	High	High	0-1.0	>60	None Rare	D	S;w,pk	M;p,w	S;w,cc	S;w	S;w	S;w
RICEBORO	0.06-0.2	4.5-5.5	Mod.	High	High	0.5-1.0	>60	Occ Freq	B/D	S;f,w,pk	S;f,w	S;cc,w	S;f,w	S;f,w	S;f,w
RIDGELAND	0.6-6.0	3.6-6.5	Low	Mod.	High	1.5-2.5	>60	None Rare	B/D	S;w,pf	None:S;p,w Rare:S;p,w,f	S;cc,w	None:M;w# None:S;w* Rare:S;f	None:M;w Rare:S;f	None:M;w Rare:M;w,f
RIGDON	0.6-2.0	4.5-5.5	Low	High	High	1.5-2.5	>60	None	C	S;w	S;w	S;cc,w	M;w# S;w*	M;w	M;w
RION	0.6-2.0	4.5-6.5	Low	Mod.	High	>6.0	>60	None	B	2-8%:L 8-15%:M;s 15+%:S;s	0-7%:S;p 7+%:S;p,s	2-15%:S;cc 15+%:S;cc,s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
RIVERVIEW	0.6-2.0	4.5-5.5	Low	Low	Mod.	3.0-5.0	>60	Occ Freq	B	S;f,w	S;f,p,w	S;cc	S;f	S;f	S;f
ROANOKE	0.06-0.2	3.6-6.5	Mod.	High	High	0-1.0	>60	None Rare Occ Freq	D	None, Rare:S;w,pk Occ,Freq:S;w,f,pk	None,Rare:S;w,p Occ,Freq:S;w,f,p	S;w	None:S;w Rare,Occ,Freq:S;w,f	None:S;w Rare,Occ,Freq:S;w,f	None,Rare:S;w,b Occ,Freq:S;w,b,f
ROBERTSDALE	0.06-0.2	4.5-5.5	Low	High	Mod.	1.0-2.5	>60	None	C	S;w,pk	L	S;w	S;w	S;w	M;w
ROME	0.6-2.0	4.5-5.5	Low	Low	High	>6.0	>60	None Rare Occ	B	None:L Rare:M;f Occ:S;f	0-2%:Rare:M;p 2-6%:None,Rare:M;p,s Occ:S;f	None,Rare:L Occ:M;f	None:L Rare,Occ:S;f	0-4%:None:L 4-6%:None:M;s Rare,Occ:S;f	None:M;b Rare:M;b,f Occ:S;f
RUTLEDGE	6.0-20	3.6-5.5	Low	High	High	0-10	>60	Occ Freq	B/D	S;w,f,pf	S;w,f,p	S;w,cc	S;w,f	S;w,f	S;w,f
SALUDA	0.6-2.0	4.5-5.5	Low	Mod.	High	>6.0	10-20	None	C	10-15%:S;r 15+%:S;s,r	S;s,r	10-15%:S;r 15+%:S;s,r	10-15%:M;s,r# 15+%:S;s# 10-15%:S;r* 15+%:S;r,s*	S;s	10-15%:M;s,r 15+%:S;s
SAPELO	0.6-2.0	3.6-5.5	Low	High	High	0.5-1.5	>60	None	D	S;w,pf	S;w,p	S;w,cc	S;w	S;w	S;w
SATILLA	2.0-6.0	4.5-6.0	Mod.	High	High	0-1.5	>60	None Rare Occ Freq	D	None,Rare:S;w Occ,Freq;S;f,w	None,Rare:S;p,h,w Occ,Freq:S;p,f,h	S;h,w	None:S;w,b Rare,Occ,Freq:S;f,w,b	None:S;w,b Rare,Ccc,Freq:S;f,w,b	None,Rare:S;w Occ,Freq:S;f,w
SAWYER	0.06-0.2	4.5-5.5	High	High	High	2.0-3.0	>60	None	C	S;pk,w	1-7%:S;w 7+%:S;w,s	S;w	M;w,ss# S;ss,w*	1-4%:M;w,ss 4-8%:M;w,ss,s	S;b
SCRANTON	6.0-20	4.5-6.0	Low	Low	High	0.5-1.5	>60	None Rare	A/D	S;w,pf	S;p,w	S;cc,w	None:S;w Rare:S;f,w	None:S;w Rare:S;f,w	S;w

SOIL SERIES INTERPRETATIONS

ESTIMATED SOIL PROPERTIES										LIMITATION OF SOILS FOR URBAN USES					
				CORROSIVITY		DEPTH TO:									
SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
SEQUATCHIE	0.6-2.0	4.5-5.5	Low	Low	Mod.	>6.0	>60	None Rare Occ Freq	B	None:L Rare:M;f Occ,Freq:S;f	0-7%:None:M;p 7+%:None:S;s 0-7%:Rare,Occ,Freq:S;f 7%:Rare,Occ,Freq:S;s,f	0-8%:None,Rare:L 8-12%:None,Rare:M;s 0-8%:Occ,Freq:M;f 8-12%:Occ:Freq:M;s	0-8%:None:L 8-12%:None:M;s Occ,Freq:S;f	0-4%:None:L 4-8%:None:M;s 8+%:None:S;s Rare,Occ,Freq:S;f	None:L Rare:M;f Occ,Freq:S;f
SHACK	0.2-0.6	4.5-5.5	Low	Mod.	High	2.0-4.0	>60	None	B	2-15%:S;w,pk 15+%:S;w,pk,s	2-7%:S;w 7+%:S;s,w	2-15%:M;w 15+%:S;s	2-15%:M;w,s# 15+%:S;s# S;w*	2-4%:M;w 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;b,s 15+%:S;s
SHELLBLUFF	0.6-2.0	4.5-6.5	Low	Mod.	Mod.	3.0-5.0	>60	Occ Freq	B	S;f,w	S;f,w	M;w,f	S;f	S;f	S;f,b
SOCO	2.0-6.0	3.6-5.5	Low	Mod.	High	>6.0	20-40	None	B	8-15%:S;r 15+%:S;r,s	S;p,r,s	8-15%:M;r,s 15+%:S;s	8-15%:M;s# 8-15%:M;r,s* 15+%:S;s	S;s	8-15%:M;s 15+%:S;s
STARR	0.6-2.0	5.1-6.5	Mod.	Mod.	Mod.	>6.0	>60	Rare Occ	C	Rare:M;f Occ:S;f	S;f	Rare:L Occ:M;f	S;f	S;f	Rare:M;ss,b,f Occ:S;f
STASER	0.6-6.0	5.6-7.3	Low	Low	Low	3.0-4.0	>60	Rare Occ	B	Rare:S;w Occ:S;w,f	S;f,w,p	Rare:M;w Occ:M;w,f	S;f	S;f	Rare:M;f Occ:S;f
STATE	0.6-2.0	4.5-6.5	Low	Mod.	High	4.0-6.0	>60	None Rare Occ	B	None:M;w,pk Rare:M;f,w,pk Occ:S;f	None,Rare:S;p Occ:S;p,f	S;cc	None:L# None:M;w* Rare,Occ:S;f	0-4%:None:L 4-6%:None:M;s Rare,Occ:S;f	None:M;b Rare:M;b,f Occ:S;f
STEEKEE	2.0-6.0	4.5-5.5	Low	Low	Mod.	>6.0	20-40	None	C	10-15%:S;r 15+%:S;r,s	S; r,s,p	10-15%:S;r 15+%:S;r,s	10-15%:M;s,r# 15+%:S;s# 10-15%:S;r 15+%:S;r,s	S;s	10-15%:M;r,s 15+%:S;s
STILSON	0.6-2.0	4.5-5.5	Low	Mod.	High	2.5-3.0	>60	None	B	S;w	S;w	S;cc	L# M;w*	0-4%:L 4-8%:M;s	L
SUBLIGNA	6.0-20	4.5-5.5	Low	Low	High	>6.0	>60	Rare Occ	B	Rare:S;pf Occ:S;f,pf	S;p,f	Rare:L Occ:M;f	S;f	S;f	Rare:M;f Occ:S;f
SUCHES	0.6-2.0	4.5-6.0	Low	High	Mod.	2.5-4.0	>60	Occ Freq	B	S;w,f	S;w,f	M;w,f	S;f	S;f	S;f
SUFFOLK	0.6-2.0	3.6-6.0	Low	Mod.	High	>6.0	>60	None	B	0-8%:L 8-15%:M;s 15+%:S;s	0-7%:S;p 7+%:S;p,s	0-15%:S;cc 15+%:S;cc,s	0-8%:L 8-15%:M;S; 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s
SULLIVAN	0.6-2.0	5.1-7.3	Low	Low	Low	4.0-6.0	>60	Occ Freq	B	S;f	S;f	M;f,w	S;f	S;f	S;f
SUMTER	0.6-2.0	7.4-8.4	High	Mod.	Low	>6.0	20-40	None	C	1-15%:S;r,pk 15+%:S;r,pk,s	1-7%:S;r 7+%:S;r,s	1-15%:M;r,cl 8-15%:M;r,cl,s 15+%:S;s	1-15%:S;ss 15+%:S;ss,s	1-8%:S;ss 8+%:S;ss,s	1-15%:S;ss,b 15+%:S;ss,b,s
SUNLIGHT	0.6-2.0	4.5-5.5	Low	Low	High	>6.0	10-20	None	D	8-15%:S;r 15+%:S;r,s	S;p,r,s	8-15%:S;r 15+%:S;r,s	8-15%:M;s,r# 15+%:S;s# 8-15%:S;r* 15+%:S;r,s	S;s	8-15%:M;r,s 15+%:S;s
SUNSWEET	0.2-0.6	4.5-5.5	Low	Mod.	Mod.	>6.0	>60	None	C	2-15%:S;pk 15+%:S;pk,s	2-7%:M;s 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;s,b 15+%:S;s
SURRENCY	0.6-2.0	3.6-5.5	Low	High	High	0-0.5	>60	Occ Freq	D	S;w,f	S;w,f,p	S;w,cc	S;w,f	S;w,f	S;w,f

SOIL SERIES INTERPRETATIONS

ESTIMATED SOIL PROPERTIES										LIMITATION OF SOILS FOR URBAN USES					
				CORROSIVITY		DEPTH TO:									
SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
SUSQUEHANNA	<0.06	4.5-5.5	High	High	High	>6.0	>60	None	D	1-15%:S;pk 15+%:S;pk,s	1-2%:L 2-7%:M;s 7+%:S;s	1-8%:M;cl 8-15%:M;cl,s 15+%:S;s	1-15%:S;ss 15+%:S;ss,s	1-8%:S;ss 8+%:S;ss,s	1-15%:S;b,ss 15+%:S;b,ss,s
SWAFFORD	0.2-0.6	4.5-5.5	Low	Mod.	Mod.	2.0-3.0	>60	None Rare	C	S;w,pk	S;w	S;w	None:M;w # None:S;w* Rare:S;f# Rare:S;f,w,*	0-4%:None:M;w 4-5%:None:M;w,s Rare:S;f	None:M;w,b Rare:M;w,b,f
SWEETAPPLE	0.2-0.6	4.0-5.5	Low	Mod.	Mod.	>6.0	>60	None	C	2-15%:S;pk 15+%:S;pk,s	2-7%:M;s 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;b,s 15+%:S;s
TAFT	0.06-0.2	4.0-5.5	High	High	High	1.0-2.0	>60	None	C	S;p,w	S;w	S;w	S;w	S;w	S;b
TALBOTT	0.2-0.6	5.1-7.8	Mod.	High	Mod.	>6.0	20-40	None	C	0-15%:S;pk,r 15+%:S;s,pk,r	0-7%:S;r 7+%:S;s,r	0-15%:S;r 15+%:S;s,r	0-15%:M;ss,r,s# 15+%:S;s# S;r	0-4%:M;ss,r 4-8%:M;s,ss,r 8+%:S;s	0-15%:S;b 15+%:S;s,b
TALLADEGA	0.6-2.0	4.5-5.5	Low	Mod.	High	>6.0	20-40	None	C	6-15%:S;r 15+%:S;r,s	S;r,s	6-8%:M;r 8-15%:M;r,s 15+%:S;s	6-15%:M;r# 6-15%:M;r,s* 15+%:S;s	6-8%:M;s 8+%:S;s	6-8%:L 8-15%:M;r 15+%:S;s
TALLAPOOSA	0.6-2.0	4.5-5.0	Low	Low	High	>6.0	10-20	None	C	5-15%:S;r 15+%:S;s,r	5-7%:S;r 7+%:S;s,r	8-15%:S;r 15+%:S;s,r	5-15%:M;r,s# 15+%:S;s# S;r*	5-8%:M;r,s 8+%:S;s	5-8%:M;r 8-15%:M;s,r 15+%:S;s
TANYARD	0.2-2.0	4.5-7.8	Low	Mod.	Mod.	1.5-2.5	>60	Occ Freq	C	S;f,w,pk	S;f,w	S;w	S;f	S;f	S;f,b
TATE	0.6-2.0	5.1-6.0	Low	Mod.	Mod.	>6.0	>60	None	B	2-8%:L 8-15%:M;s 15+%:S;s	2-7%:S;p 7+%:S;s,p	2-8%:L 8-15%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
TATUM	0.6-2.0	4.5-5.5	Mod.	High	High	>6.0	40-60	None	B	0-8%:M;pk,r 8-15%:M;s,r,pk 15+%:S;s	0-2%:M;p,r 2-7%:M;p,s,r 7+%:S;s	0-8%:M;cl 8-15%:M;s,cl 15+%:S;s	0-8%:M;ss 8-15%:M;s,ss 15+%:S;ss	0-4%:M;ss 4-8%:M;s.ss 8+%:S;s	0-15%:S;b 15+%:S;b,s
TAWCAW	0.06-0.2	4.5-6.5	Mod.	High	High	1.5-2.5	>60	Occ Freq	C	S;f,w,pk	S;f,w	S;w	S;f	S;f	S;f,b
THURMONT	0.6-2.0	4.5-5.5	Low	Mod.	High	4.0-6.0	>60	None	B	2-8%:M;w,pk 8-15%:M;w,pk,s 15+%:S;s	2-7%:M;p,s,w 7+%:S;s	2-8%:M;w 8-15%:M;w,s 15+%:S;s	2-8%:L# 2-8%:M;w* 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
TIDINGS	0.6-2.0	4.5-5.5	Low	Low	High	>6.0	40-60	None	B	2-8%:M;r,pk 8-15%:M;r,pk,s 15+%:S;s	2-7%:M;r,p,s 7+%:S;s	2-8%:M;r 8-15%:M;r,s 15+%:S;s	2-8%:L#M;r,s* 2-15%:M;s#,r* 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:M;b 8-15%:M;s,b 15+%:S;s
TIFTON	0.2-0.6	4.5-5.5	Low	Low	Mod.	3.5-6.0	>60	None	B	M;pk,w	0-2%:M;p 2-7%:M;s,p 7+%:S;s	M;w	L#	0-4%:L 4-8%:M;s	L
TOCCOA	2.0-6.0	5.1-6.5	Low	Low	Mod.	2.5-5.0	>60	None Rare Occ Freq	B	None,Rare:S;w Occ,Freq:S;w,f	None,Rare:S;w,p Occ,Freq:S;w,f,p	None,Rare:M;w Occ:Freq.:M;w,f	None:M;w Rare,Occ,Freq:S;f	None:M;w Rare,Occ,Freq:S;f	None:M;w Rare:M;w,f Occ,Freq:S;f
TOWNLEY	0.06-0.2	3.6-5.5	Mod.	Mod.	High	>6.0	20-40	None	C	2-15%:S;r 15+%:S;s,r	2-7%:S;r 7+%:S;s,r	2-8%:M;r,cl 8-15%:M;s,r,cl 15+%:S;s	2-8%:M;r,ss*ss# 8-15%:M;s,r,ss* 8-15%:M:ss,s# 15+%:S;s	2-4%:M;ss 4-8%:M;s,ss 8+%:S;s	2-15%:M;b 15+%:S;s,b
TOXAWAY	0.6-2.0	5.1-6.0	Low	High	Mod.	0-1.0	>60	Freq	B	S;w,f	S;w,f,p	S;w	S;w,f	S;w,f	S;w,f,b

SOIL SERIES INTERPRETATIONS

	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
				CORROSIVITY		DEPTH TO:									
SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
TRANSYLVANIA	0.6-2.0	4.5-6.0	Low	High	High	2.5-3.5	>60	Freq	B	S;f,w	S;f,w	S;f,w	S;f	S;f	S;f,b
TROUP	0.6-2.0	4.5-5.5	Low	Low	Mod.	>6.0	>60	None	A	0-8%:L 8-15%:M;s 15+%:S;s	0-7%:S;p 7+%:S;s,p	0-15%:S;cc 15+%:S;cc,s	0-8%:L 8-15%:M;s 15+%:S;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s 15+%:S;s
TSALI	0.6-2.0	4.0-6.0	Low	Mod.	High	>6.0	10-20	None	C	8-15%:S;r 15+%:S;r,s	S;r,s	8-15%:S;r 15+%:S;r,s	8-15%:M;r,s# 15+%:S;s# 8-15%:S;r* 15+%:S;r,s*	S;s	8-15%:M;r,b,s 15+%:S;s
TUPELO	0.06-0.2	4.5-8.4	High	High	Mod.	1.0-2.0	40-70	None Rare Occ Freq	D	None, Rare:S;w,pk Occ,Freq:S;w,pk,s	None,Rare:S;w Occ,Freq:S;f,w	S;w	None:S;ss,w Rare,Occ,Freq:S;f,w,ss	None:S;ss,w Rare,Occ,Freq:S;f,w,ss	None,Rare:S;ss,b Occ,Freq:S;ss,b,f
TUSQUITTEE	0.6-2.0	4.5-6.0	Low	Mod.	Mod.	>6.0	>60	None	B	2-8%:L 8-15%:M;s 15+%:S;s	2-7%:S;p 7+%:S;s,p	2-8%:L 8-15%:M;s 15+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 8-15%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
VALDOSTA	6.0-2.0	4.5-6.0	Low	Mod.	High	>6.0	>60	None	A	S;pf	S;p	S;cc	L	L	L
VANCE	0.06-0.2	4.5-5.5	Mod.	High	High	>6.0	>60	None	C	2-15%:S;pk 15+%:S;pk,s	2-7%:M;s 7+%:S;s	2-8%:M;cl 8-15%:M;cl,s 15+%:S;s	2-8%:M;ss 8-15%:M;ss,s 15+%:S;s	2-4%:M;ss 4-8%:M;ss,s 8+%:S;s	2-15%:S;b 15+%:S;b,s
VARINA	0.06-0.2	4.5-5.5	Low	Mod.	High	4.0-5.0	>60	None	C	S;pk	0-2%:L 2-7%:M;s 7+%:S;s	2-8%:M;cl,w 8-10%:M;s,cl,w	0-8%:L# 8-10%:M;s# 0-8%:M;w* 8-10%:M;w,s*	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:M;b 8-10%:M;s,b
VAUCLUSE	0.06-0.6	3.6-5.5	Low	Low	High	>6.0	>60	None	C	2-15%:S;pk 15+%:S;pk,s	2-7%:S;p 7+%:S;s,p	2-15%:S;cc 2-8%:S;cc 15+%:S;s,cc	2-8%:L 8-15%:M;s 15+%:S;s	2-4%:L 4-8%:M;s 8+%:S;s	2-8%:L 8-15%:M;s 15+%:S;s
WAGRAM	0.6-2.0	4.5-6.0	Low	Low	High	>6.0	>60	None	A	0-8%:L 8-15%:M;s	0-7%:S;p 7+%:S;s	0-8%:L 8-15%:M;S;	0-8%:L 8-15%:M;s	0-4%:L 4-8%:M;s 8+%:S;s	0-8%:L 8-15%:M;s
WAHEE	0.06-0.2	3.6-5.5	Mod.	High	High	0.5-1.5	>60	None Rare Occ Freq	D	None,Rare:S;w,pk Occ,Freq:S;w,f,pk	0-2%:None,Rare:L 2-4%:None,Rare:M;s Occ,Freq:S;f	S;w	None:S;w Rare,Occ,Freq:S;w,f	None:S;w Rare,Occ,Freq:S;w,f	None,Rare:S;b,w Occ,Freq:S;f,b,w
WATAUGA	0.6-2.0	4.5-6.0	Low	Mod.	Mod.	>6.0	>60	None	B	5-8%:L 8-15%:M;s 15+%:S;s	6-7%:M;r,s,p 7+%:S;s	5-8%:L 8-15%:M;s 15+%:S;s	5-8%:S; 8-15%:M;s 15+%:S;s	5-8%:M;s 8+%:S;s	5-15%:S;b 15+%:S;s,b
WAX	0.06-2.0	4.5-5.5	Low	Mod.	Mod.	1.5-3.0	>60	Occ Freq	C	S;pk,f	S;w,f	S;w	S;f,w	S;f	S;f
WAYNESBORO	0.6-2.0	4.5-5.5	Mod.	High	High	>6.0	>60	None	B	2-8%:M;pk 8-15%:M;s,pk 15+%:S;s	2-7%:M;s,p 7+%:S;s	2-8%:M;cl 8-15%:M;s,cl 15+%:S;s	2-8%:M;ss 8-15%:M;s,ss 15+%:S;s	2-4%:M;ss 4-8%:M;s,ss 8+%:S;s	2-8%:M;b,ss 8-15%:M;s,b,ss 15+%:S;s
WEDOWEE	0.6-2.0	4.5-5.5	Mod.	Mod.	High	>6.0	>60	None	B	0-2%:M;pk 8-15%:M;s,pk 15+%:S;s	0-2%:M;b 2-7%:M;s,p 7+%:S;s	0-8%:M;cl 8-15%:M;s,cl 15+%:S;s	0-8%:M;ss 8-15%:M;s,ss 8+%:S;s	0-4%:M;ss 4-8%:M;s,ss 8+%:S;s	0-8%:M;b,ss 8-15%:M;s,b,ss 8+%:S;s
WEHADKEE	0.6-2.0	4.5-6.5	Low	High	Mod.	0-0.5	>60	Occ Freq	D	S;f,w	S;f,w	S;w	S;f,w	S;f,w	S;f,w
WESTON	0.06-6.0	4.5.5.0	Low	High	High	0-1.0	>60	None	D	S;w,pk	S;p,w	S;cc,w	S;w	S;w	S;w

SOIL SERIES INTERPRETATIONS

	ESTIMATED SOIL PROPERTIES									LIMITATION OF SOILS FOR URBAN USES					
				CORROSIVITY		DEPTH TO:									
SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)	FLOOD FRE-QUENCY	HYDRO-GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
WHITWELL	0.6-2.0	4.5-5.5	Low	Mod.	Mod.	2.0-3.0	>60	None Rare Occ	C	None,Rare:S;w Occ:S;w,f	None,Rare:S;w Occ:S;f,w	S;w	None:M;w# None:S;w* Occ:S;f,w	0-4%:None:M;w 4-6%:None:M;w,s Rare,Occ:S;f	None:M;w Rare:M;w,f Occ:S;f
WICKHAM	0.6-2.0	4.5-6.0	Low	Mod.	High	>6.0	>60	None Rare	B	0-8%:L 8-15%:M;s Rare:M;f	0-7%:M;p,s 7+%:S;s Rare:S;f	0-8%:L 8-15%:M;s	0-8%:M;ss 8-15%:M;s,ss Rare:S;f	0-4%:L 4-8%:M;s 8+%:S;s Rare:S;s,f	0-8%:L 8-15%:M;s Rare:M;f
WICKSBURG	0.06-0.2	4.5-5.5	Mod.	High	High	>6.0	>60	None	B	S;pk,pf	0-7%:S;p 7+%:S;s,p	0-8%:M;cc 8-12%:M;s,cc	0-8%:M;ss 8-12%:M;s,ss	0-4%:L 4-8%:M;s 8-12%:S;s	0-8%:L 8-12%:M;s
WILKES	0.2-0.6	5.1-7.8	Mod.	Mod.	Mod.	>6.0	10-20	None	C	2-15%:S;r 15+%:S;s,r	2-7%:S;r 7+%:S;s,r	2-15%:S;r 15+%:S;s,r	2-15%:M;ss,r# 15+%:S;s# 2-15%:S;r* 15+%:S;r,s*	2-8%:M;r,ss 8+%:S;s	2-8%:M;r,ss 8-15%:M;s,r,ss 15+%:S;s
WOLFTEVER	0.2-0.6	4.5-5.5	Mod.	High	Mod.	2.5-3.5	>60	None Rare Occ	C	None, Rare:S;w,pk Occ:S;f,w,pk	0-7%:None:S;w 7+%:None:S;w,s 0-7%Rare,Occ:S;w,f 7+%Rare,Occ:S;l,w,s	None,Rare:M;cl,w Occ:M;cl,w,f	0-8%:None:M;w,ss 8-15%:None:M;w,ss,s Rare,Occ:S;f	0-8%:None:M;ss 8+%:None:S;s 0-8%:Rare,Occ:S;f 8+%:Rare:,Occ:S;f,s	None,Rare:S;b Occ:S;b,f
WORSHAM	<0.06	4.5-5.5	Mod.	High	Mod.	0-1.0	>60	None	D	S;pk,w	0-2%:L 2-7%:M;s 7+%:S;s	S;w	S;w	S;w	S;w,b
ZION	0.06-0.6	4.5-7.3	High	High	Mod.	>6.0	20-40	None	C	2-15%:S;r,pk 15+%:S;r,pk,s	2-7%:S;r 7+%:S;r,s	2-15%:S;r 15+%:S;r,s	2-15%:S;ss# 15+%:S;ss,s# 2-15%:S;r,ss* 15+%:S,r,ss,s*	2-8%:S;ss 8+%:S;s,ss	2-15%:S;b,ss 15+%:S;b,ss,s

SOIL SERIES INTERPRETATIONS

ESTIMATED SOIL PROPERTIES										LIMITATIONS OF SOILS FOR URBAN USES					
SOIL SERIES	PERMEABILITY (In./Hrs.)	SOIL REACTION (pH)	SHRINK-SWELL POTENTIAL	CORROSIVITY		DEPTH TO		FLOOD FREQUENCY	HYDRO GROUP	SEPTIC TANK ABSORPTION FIELDS	SEWAGE LAGOON AREAS	SHALLOW EXCAVATIONS	DWELLINGS *w/basement #w/o basement	SMALL COMMERCIAL BUILDINGS	LOCAL ROADS AND STREETS
				STEEL	CONCRETE	WATER TABLE (Ft.)	BEDROCK (In.)								
Allanton	2.00-6.00	3.6-5.5	Low	High	High	-2.0-0.0	>60	None	D	S;p	S;p	S;cc,p	S;p	S;p	S;p
Arkabutla	0.60-2.00	4.5-5.5	Low	High	High	1.0-1.5	>60	Occasional	C	S;f,w	S;f,w	S;w	S;f,w	S;f,w	S;b,f
Badin	0.60-2.00	3.5-6.5	Low	High	High	>6.0	20-40	None	B	2-10%:S;r 10+%:S;r,s	2-10%:S;r 10+%:S;r,s	M;r,cl	M;r,ss* M;ss#	M;ss,s	S;b
Bethera	0.06-0.60	3.6-6.0	Moderate	High	High	0.0-1.5	>60	Occasional	D	S;f,w,pk	S;f,w	S;w	S;f,w	S;f,w	S;b,w,f
Blaney	0.20-0.60	4.5-6.0	Low	Moderate	High	>6.0	>60	None	B	S;pk,pf	S;p	S;cc	L	M;s	L
Buckhead	0.60-2.00	4.5-6.5	Low	Moderate	High	>6.0	>60	None	B	M;s	S;p,s	S;cc	M;s	S;s	M;s
Cheoah	2.00-6.00	3.5-5.5	Low	Low	High	>6.0	40-60	None	B	5-15%:M;r,s, 15+%:S;s	S;p,s	M;s	M;s	S;s	M;s
Chestnut	2.00-6.00	3.6-6.0	Low	Low	High	>6.0	20-40	None	B	S;r,s	S;p,r,s	S;s	S;s	S;s	S;s
Craigsville	2.00-20.00	4.5-5.5	Low	Low	Moderate	>6.0	>60	Frequent	B	S;f,pf,st	S;p,f,st	S;cc,st	S;f,st	S;f,st	S;f,st
Fork	0.60-2.00	4.5-7.3	Low	High	High	1.0-2.0	>60	Occasional	C	S;f,w	S;f,w	S;w	S;f,w	S;f,w	S;f
Jefferson	2.00-6.00	4.5-5.5	Low	Moderate	High	>6.0	>60	None	B	S;s	S;p,s	S;s	S;s	S;s	S;s
Ketona	0.06-0.20	6.1-8.4	High	High	Moderate	0.5-1.0	40-72	Frequent	D	S;f,w,pk	S;f	S;w	S;f,w,ss	S;f,w,ss	S;ss,b,w
Kingsferry	0.6-2.00	3.6-5.5	Low	High	High	0.0-0.5	>60	None	B/D	S;w,pf	S;p,w	S;cc,w	S;w	S;w	S;w
Lloyd	0.60-2.00	4.5-6.5	Low	Moderate	Moderate	>6.0	>60	None	B	2-15%:M;pk,s 15+%:S;s	2-6%:M;p,s 6+%:S;s	M;cl,s	L	M;s	M;b
Pageland	0.20-0.60	3.6-6.0	Low	Moderate	Moderate	1.5-3.0	20-40	None	C	S;r,w,pk	S;r,w	S;w	S;w* MS;w#	M;w	S;b
Panama	0.60-2.00	4.5-6.0	Low	Moderate	High	3.5-5.0	>80	None	B	S;w,pk,s	S;s	S;s	S;s	S;s	S;s
Pigeonroost	0.60-2.00	3.5-6.0	Low	Moderate	High	>6.0	20-40	None	B	S;r	S;r,s	M;r,s	M;r,s* M;s#	S;s	M;s
Rawlings	0.60-6.00	4.5-7.3	Low	Moderate	High	>6.0	20-40	None	B	S;r	S;p,r,s	S;r	S;r* M;s,r#	S;s	M;r,s
Saunook	0.60-2.00	3.6-6.5	Low	Low	High	>6.0	>60	None	B	S;s	S;p,s	S;s	S;s	S;s	S;s
Sedgefield	0.06-0.20	4.5-6.5	Moderate	High	Moderate	1.0-1.5	>60	None	C	S;w,pk	S;s	S;w	S;w,ss	S;w,ss,s	S;ss,b
Shelocta	0.60-2.00	4.5-5.5	Low	Low	High	>6.0	>40	None	B	M;pk,s	S;p,s	M;s	M;s	S;s	M;s
Sipsey	0.60-2.00	4.5-6.0	Low	Moderate	High	>6.0	20-40	None	B	S;r,s	S;p,r,s	M;r,s	M;r,s* M;s#	S;s	M;s
Wake	6.00-20.00	4.5-6.0	Low	Moderate	Moderate	>6.0	8-20	None	D	2-10%:S;r 10+%:S;r,s	S;p,r	S;r	S;r	S;r	S;r
Wateree	2.00-6.00	4.5-6.0	Low	Low	High	>6.0	20-40	None	B	S;r	S;p,r	S;cc	M;r* L#	M;s	L
Wynott	0.06-0.20	4.5-6.5	Low	High	Moderate	>6.0	20-40	None	C	S;r,pk,s	S;p,r,s	S;s	S;ss,s	S;ss,s	S;ss,b,s

MISCELLANEOUS SPECIFICATIONS

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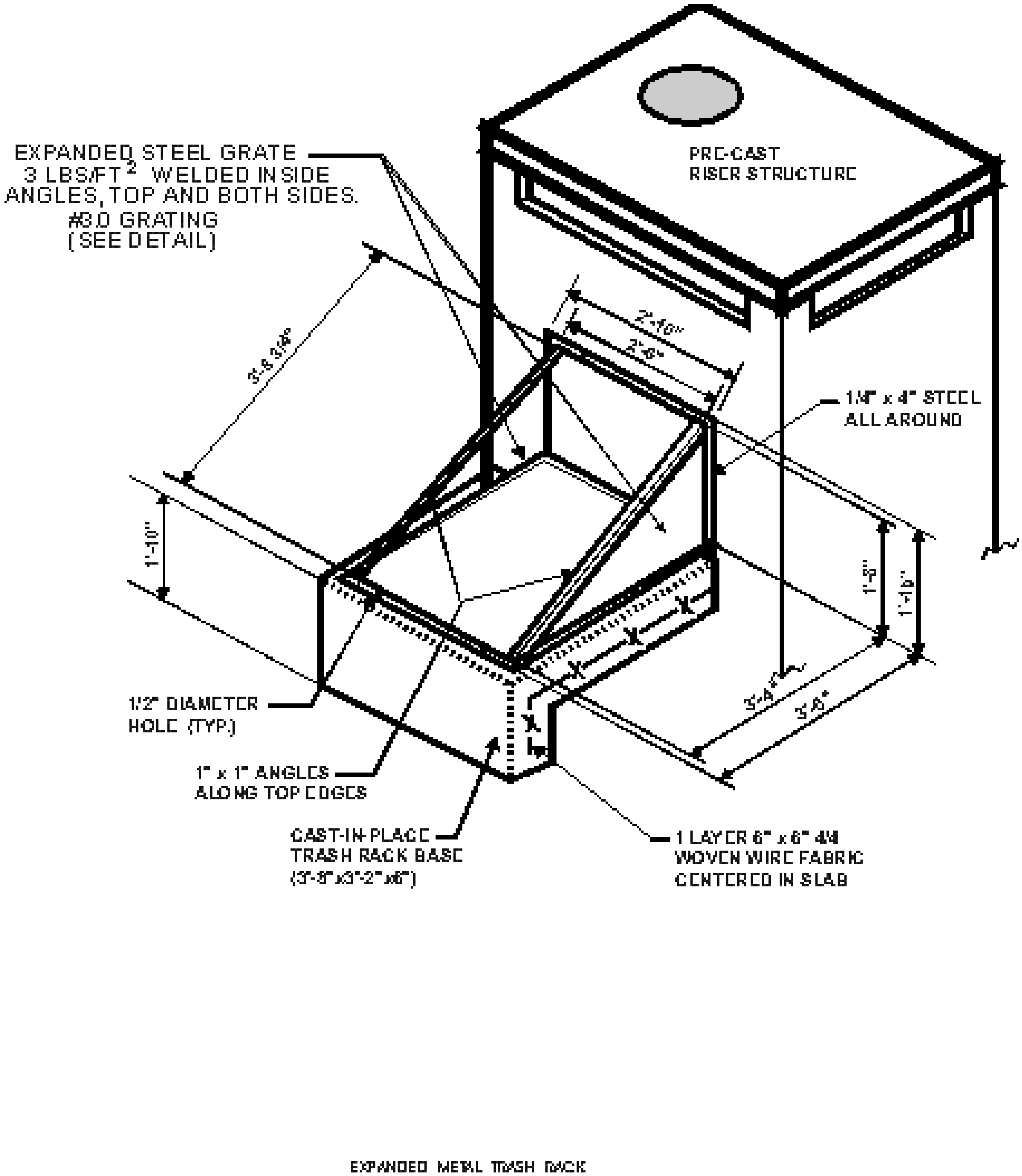
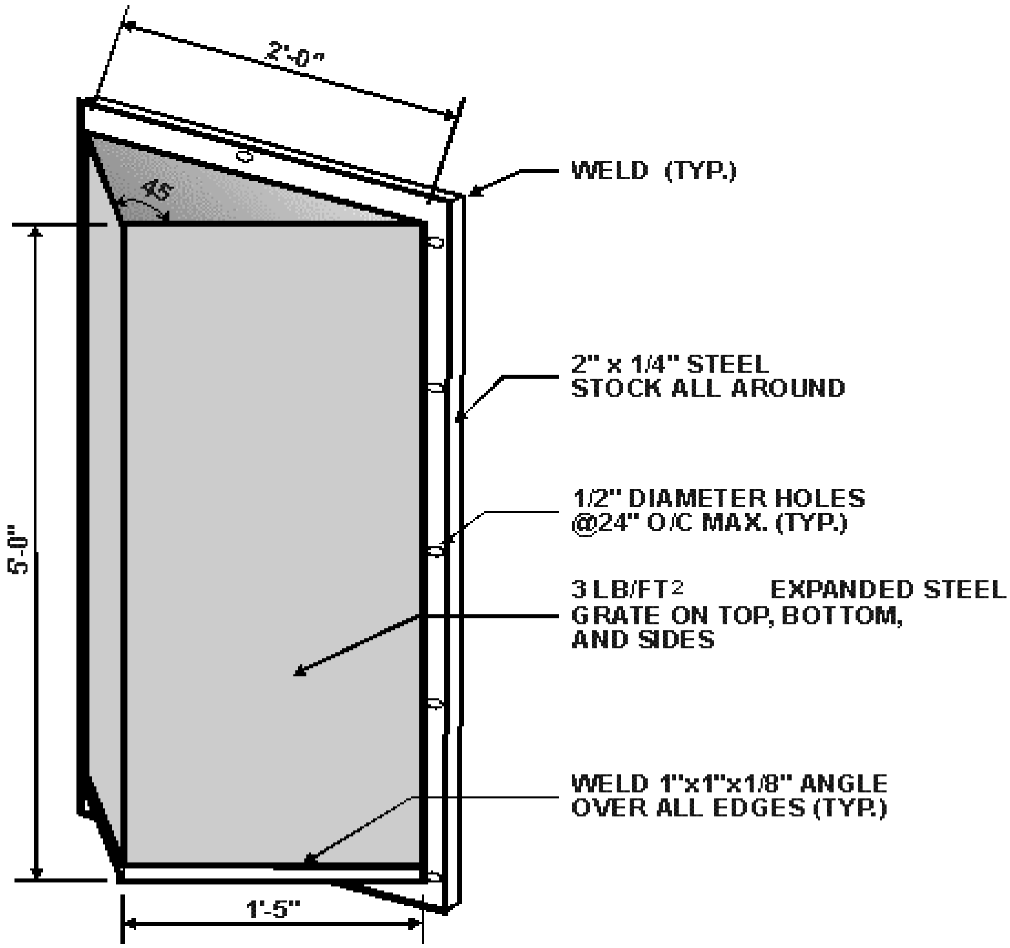


Figure C-1 Expanded Trash Rack



NOTES FOR TRASH RACK

1. TRASH RACK TO BE CENTERED OVER OPENING.
2. STEEL TO CONFORM TO ASTM A-36.
3. ALL SURFACES TO BE COATED WITH ZRC COLD GALVANIZING COMPOUND AFTER WELDING.
4. TRASH RACK TO BE FASTENED TO THE WALL WITH 1/2" MASONRY ANCHORS. TRASH RACK TO BE REMOVABLE.

TRASH RACK DETAIL

Figure C-2 Trash Rack Detail

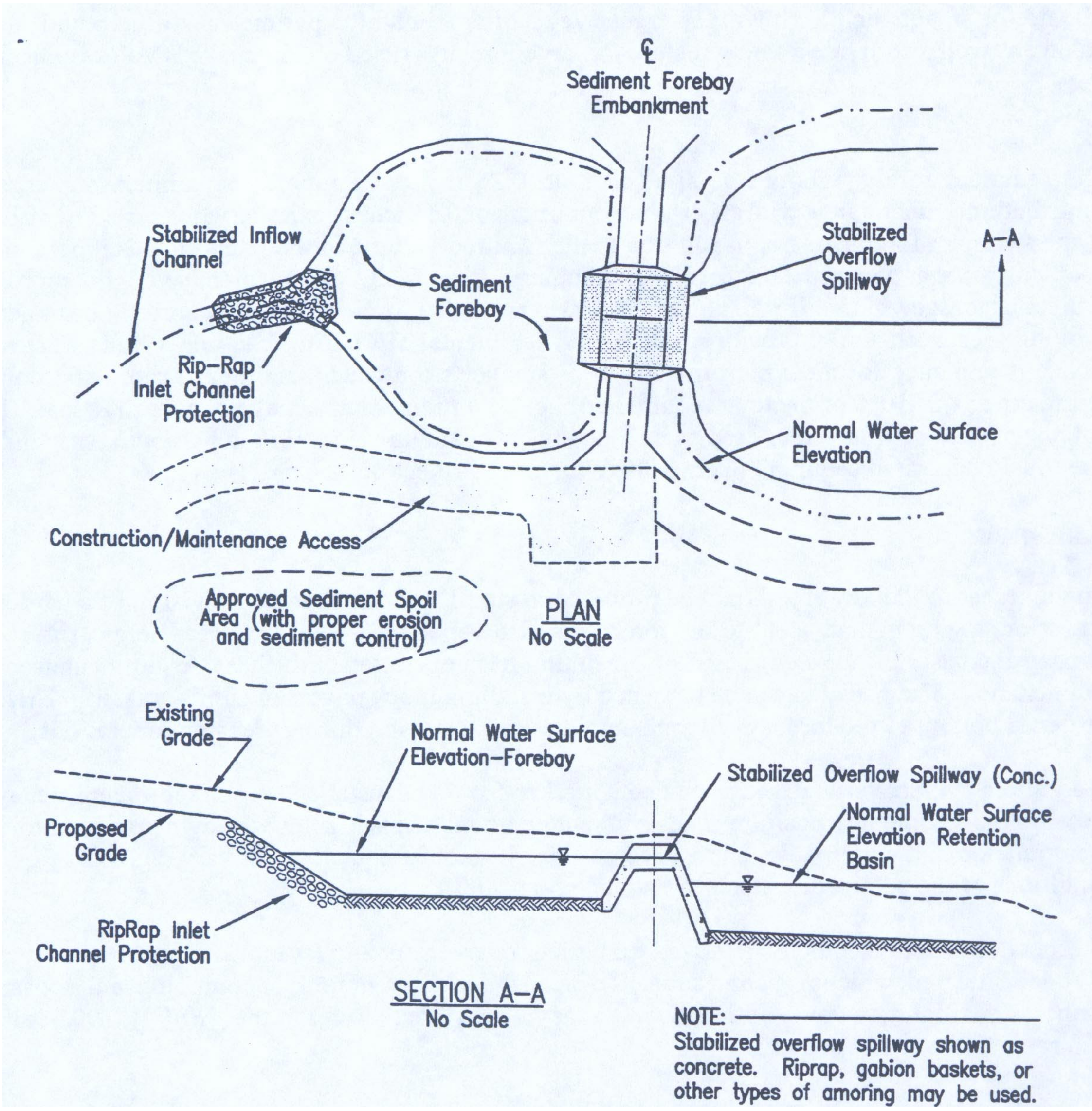


Figure C-3 Typical Sediment Forebay Plan and Section

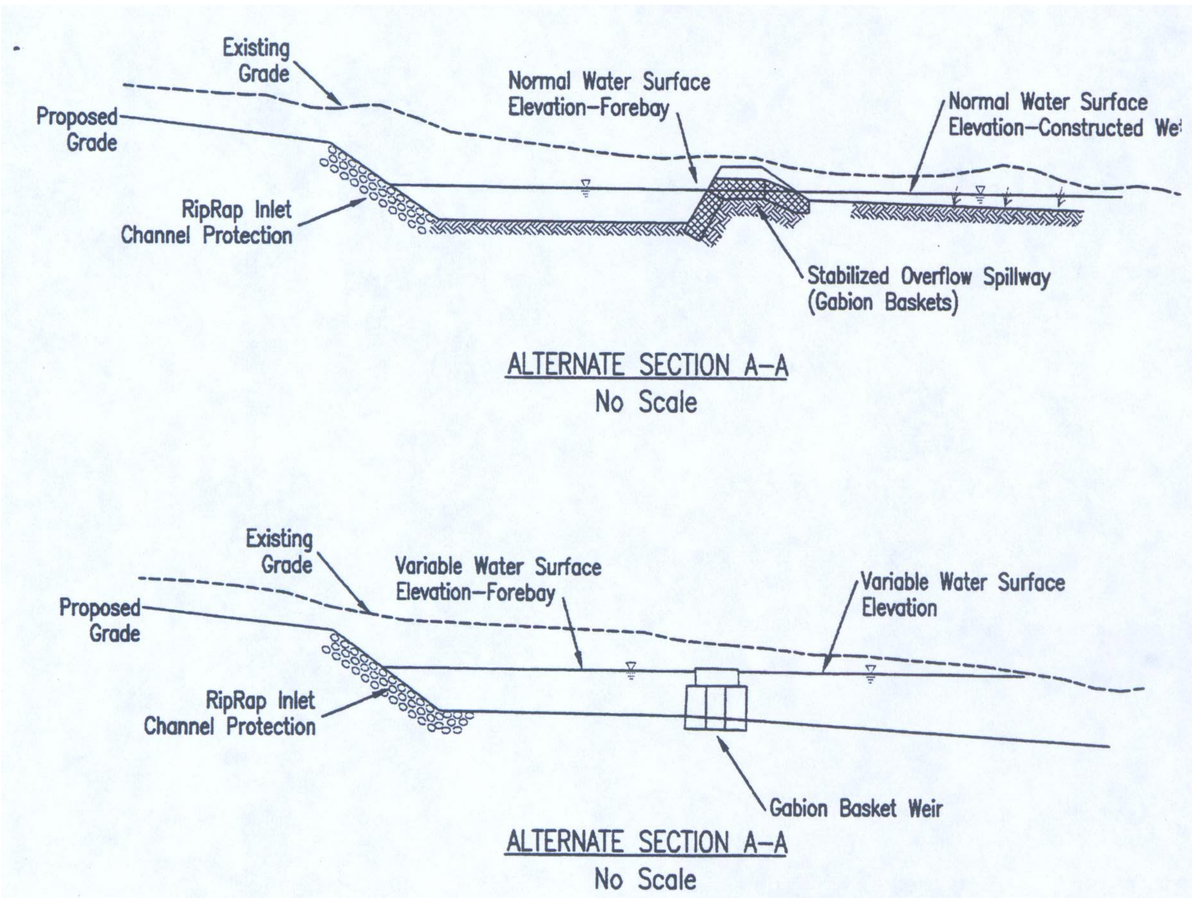


Figure C-4 Typical Sediment Forebay Alternate Sections

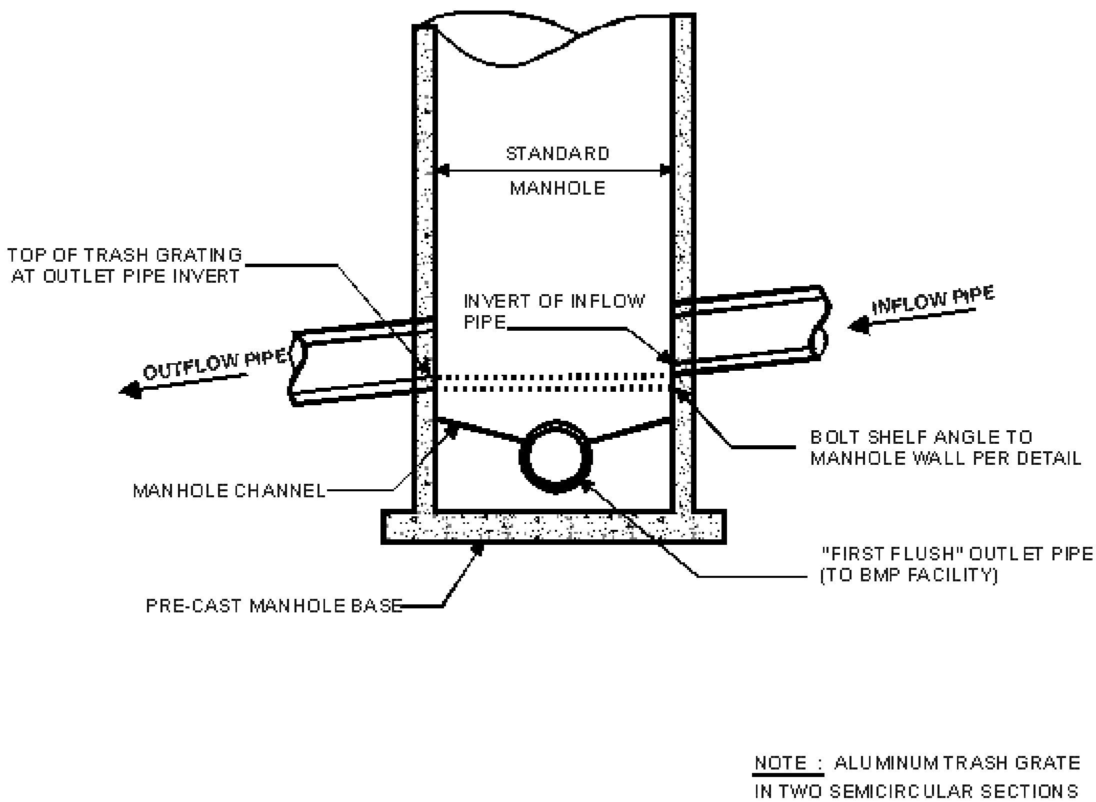
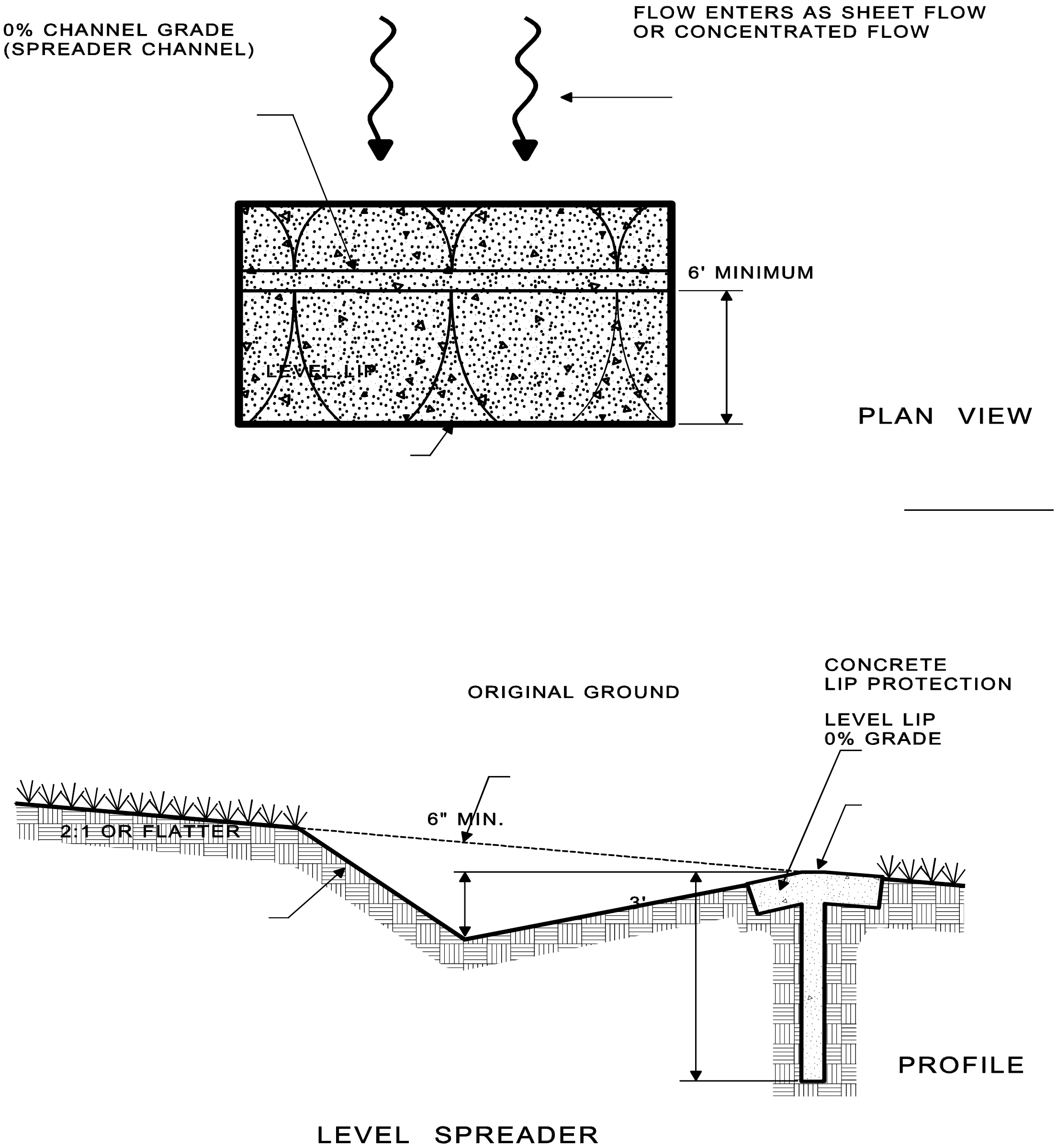


Figure C-5 Diversion Structure



Source: Virginia Erosion and Sediment Control Handbook, Virginia Soil and Water Conservation Commission, 1980

Figure C-6 Concrete Level Spreader

STRUCTURAL STORMWATER CONTROL DESIGN EXAMPLES

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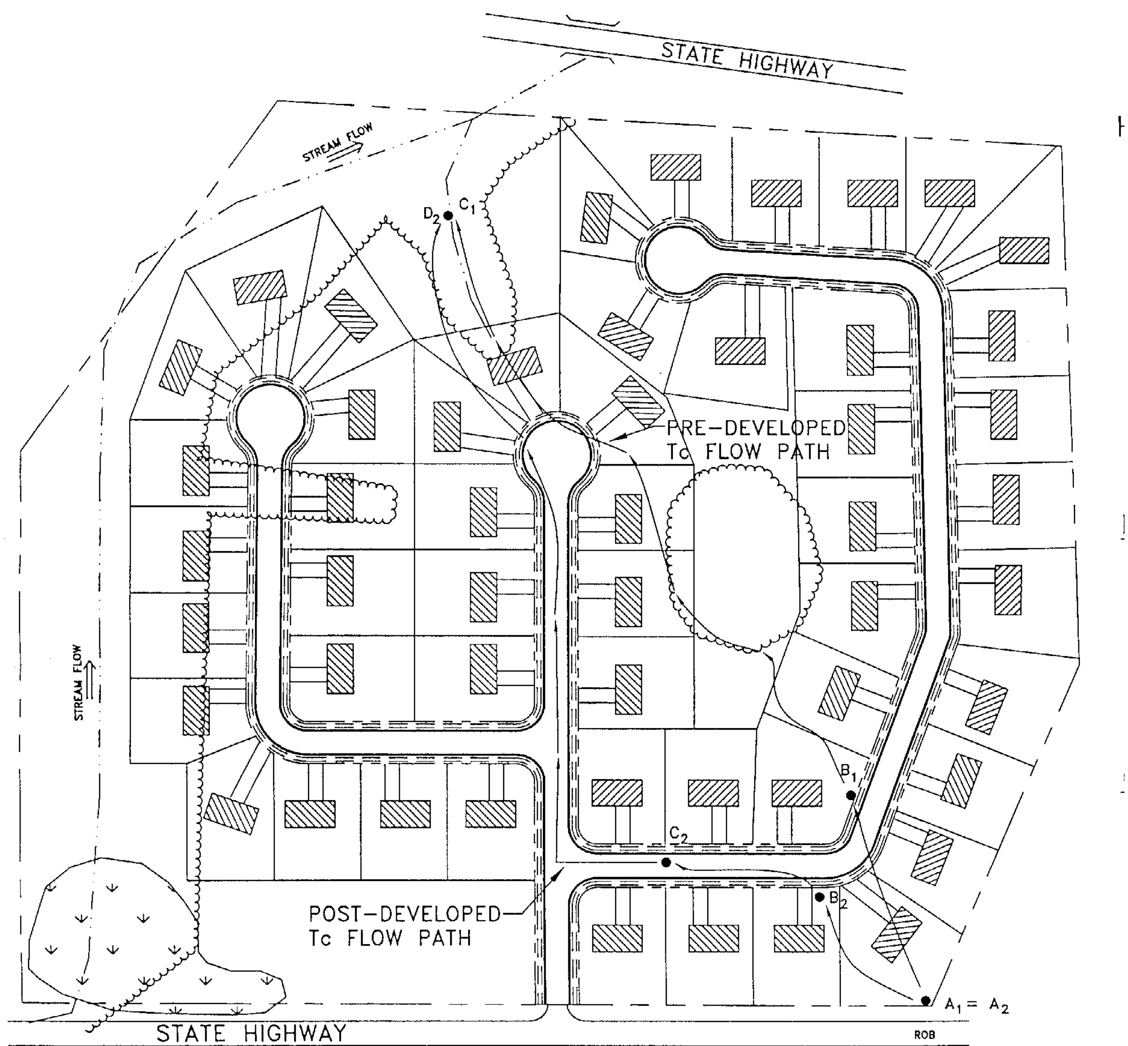
APPENDIX D-1

Stormwater Pond Design Example

The following design example is for a wet extended detention (ED) stormwater pond



Base Data	Hydrologic Data		
Site Area = Total Drainage Area (A) = 38.0 ac		Pre	Post
Measured Impervious Area=13.8 ac; or I=13.8/38=36.3%	CN	65	78
Soils Types: 60% “C”, 40% “B”	t_c	.32 hr	.17 hr
Zoning: Residential (½ acre lots)			

Figure 1. Peachtree Meadows Site Plan

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 -- Compute runoff control volumes from the Unified Stormwater Sizing Criteria

More details 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

$$WQ_v = (1.2")(R_v)(A)$$
$$= (1.2")(0.38)(38.0\text{ ac})(1\text{ft}/12\text{in})$$
$$= \underline{1.44\text{ ac-ft}}$$

Develop Site Hydrologic and Hydrologic Input Parameters

Per Figures 2 and 3. Note that any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations

Condition	**Area**	**CN**	**Tc**
	Ac		*hrs*
pre-developed	38	65	0.32
post-developed	38	78	0.17

Perform Preliminary Hydrologic Calculations

Condition	**$Q_{1\text{-yr}}$**	**$Q_{1\text{-yr}}$**	**$Q_{25\text{-yr}}$**	**$Q_{100\text{-yr}}$**
Runoff	*Inches*	*cfs*	*cfs*	*cfs*
pre-developed	0.7	22	101	147
post-developed	1.4	67	202	267

Compute Channel Protection Volume, (Cp_v)
For stream channel protection, provide 24 hours of extended detention for the 1-year event.

Utilize SCS approach to Compute Channel Protection Storage Volume

See Section 2.1

- 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 (Type II Storm)

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
- $Vs/Vr = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$

- Where Vs equals channel protection storage (Cp_v) and Vr equals the volume of runoff in inches.
- Vs/Vr = 0.65
- Therefore, Vs = Cp_v = 0.65(1.4")(1/12)(38 ac) = 2.9 ac-ft (126,324 cubic feet)

Define the average CP_v-ED Release Rate

- The above volume, 2.9 ac-ft, is to be released over 24 hours.
- (2.9 ac-ft $\times$ 43,560 ft^2/ac) / (24 hrs $\times$ 3,600 sec/hr) = 1.46 cfs

Compute Overbank Flood Protection Volume, (Q_{p25})

- For a Q_{in} of 202 cfs, and an allowable Q_{out} of 101 cfs, and a runoff volume of 552,584 cubic feet (12.69 ac-ft) the Vs necessary for 25-year control is 3.55 ac-ft, under a developed CN of 78. Note that 6.5 inches of rain fall during this event, with approximately 4.0 inches of runoff.
- 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} = 3.55 $\times$ 1.15 = 4.1 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	3.55	
Q_f	Extreme Flood Protection	NA	Provide safe passage for the 100-year event in final design

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.
woods (good cond.)		B	55	6.00 Ac.
		AREA SUBTOTALS:		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	dense grass	'n'=0.24	150 Ft.	2.50% 0.27 Hrs
Shallow Flow	unpaved		500 Ft. 3.23 F.P.S.	4.00% 0.04 Hrs.
Channel Flow				
Total Area in Acres =	38.00 Ac.	Total Sheet Flow= 0.27 Hrs.	Total Shallow Flow= 0.04 Hrs.	Total Channel Flow = 0.00 Hrs.
Weighted CN =	65			
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.

Figure 2. Peachtree Meadows Pre-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	Manning 'n' Wetted Per	Flow Avg	Slope Tt (Hrs)
Sheet Flow	short grass	'n'=0.15	100 Ft.	2.50% 0.13 Hrs
Shallow	paved		300 Ft. 2.87 F.P.S.	2.00% 0.03 Hrs.
Channel Hydraulic Radius	X-S estimated	'n'=0.013 WP estimated	600 Ft. 16.21 F.P.S.	2.00% 0.01 Hrs.
Total Area in Acres =	38.00 Ac.	Total Sheet Flow= 0.13 Hrs.	Total 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	191,988 Cu.
2 Year	4.1 In.	2.0 In.	95.8 CFS	269,103 Cu.
5 Year	4.8 In.	2.5 In.	127 CFS	350,750 Cu.
10 Year	5.5 In.	3.2 In.	159 CFS	435,668 Cu.
25 Year	6.5 In.	4.0 In.	202 CFS	552,584 Cu.
50 Year	7.2 In.	4.7 In.	234 CFS	642,337 Cu.
100 Year	7.9 In.	5.3 In.	267 CFS	733,444 Cu.

Figure 3. Peachtree Meadows Post-Development Conditions

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. 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 3.1 and 3.2.1 were assessed and a pond was found to be suitable.

Step 3 -- Confirm local design criteria and applicability

There are no additional requirements for this site.

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:
$0.5 \times$ (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: $0.5 \times$ (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.

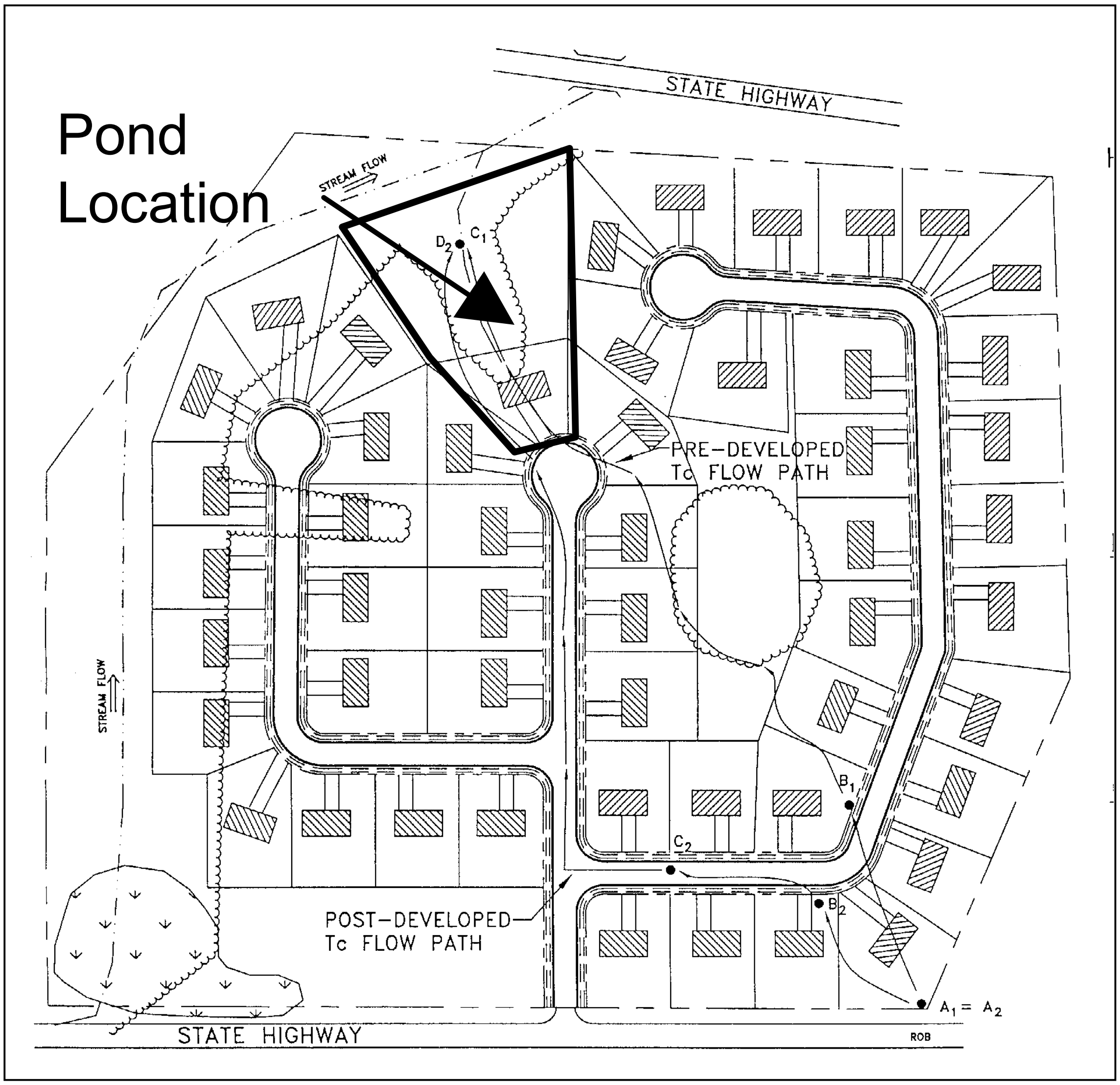


Figure 4. Pond Location on Site

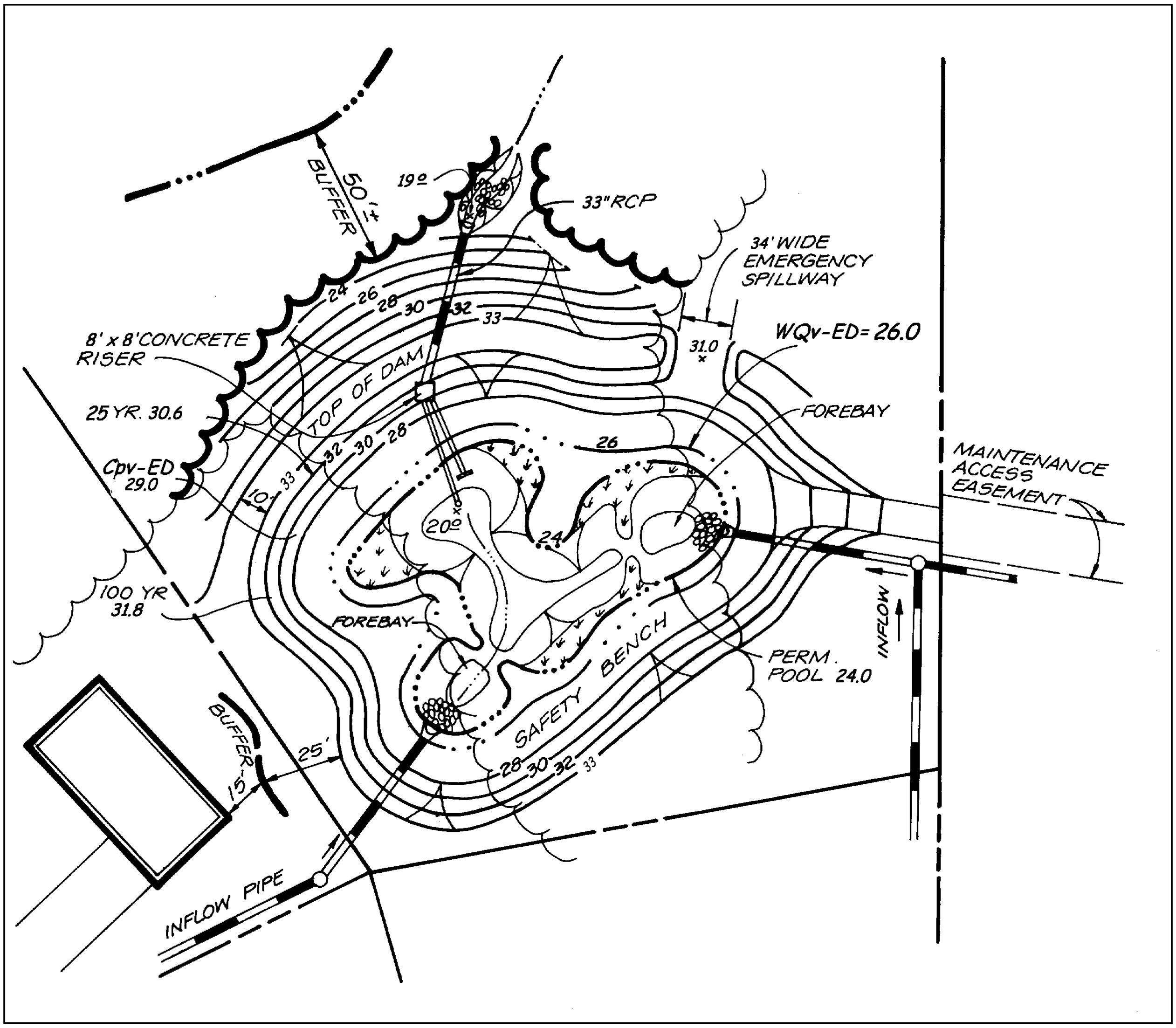


Figure 5. Plan View of Pond Grading (Not to Scale)

Elevation MSL	Average Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3	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

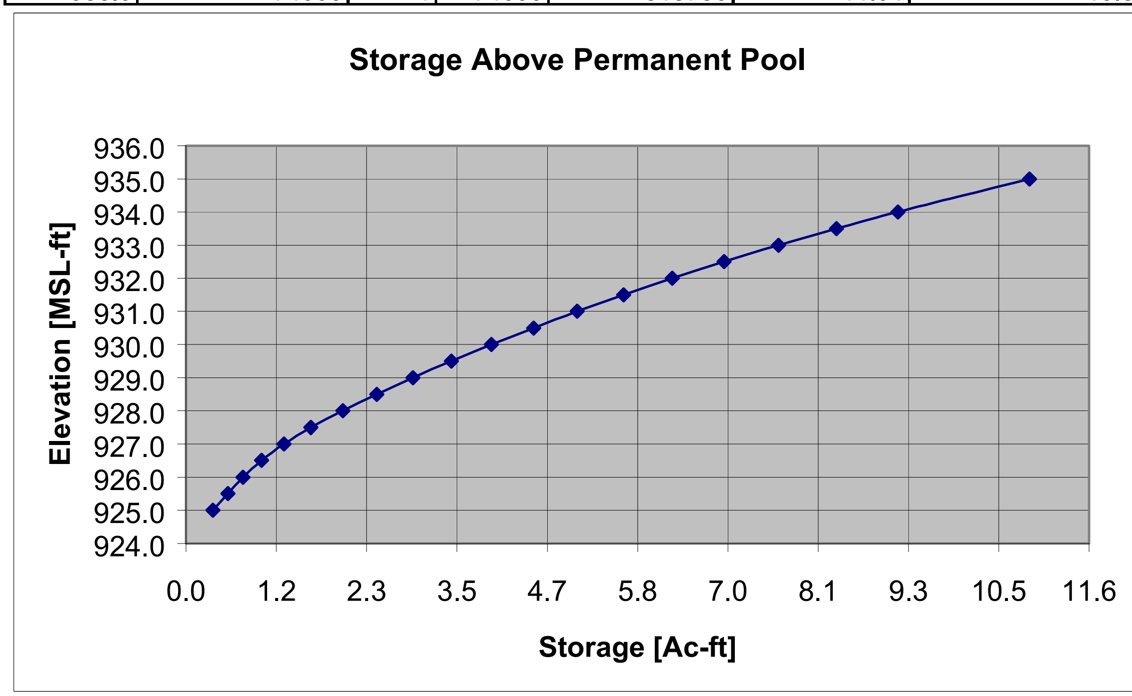


Figure 6. Storage-Elevation Table/ Curve

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 (WQ_v-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 WQ_v of 1.44 ac-ft)

Compute the required WQ_v-ED orifice diameter to release 0.72 ac-ft over 24 hours

- Avg. ED release rate = (0.72 ac-ft)(43,560 ft^2/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 / $[(0.6)((2)32.2\ ft/s^2)(1.0\ ft))^{0.5}]$ 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

- $Q_{WQv\text{-}ED} = CA(2gh)^{0.5} = (0.6)\ (0.075\ ft^2)\ [((2)(32.2\ ft/s^2))^{0.5}]\ (h^{0.5})$,
- $Q_{WQv\text{-}ED} = (0.36)\ h^{0.5}$, 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)

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 = 101 cfs,
- The post developed inflow = 202 cfs, from Table 1,
- From previous estimate $Q_{p\text{-}25}$ = 3.55 ac-ft. Adding 15% to account for ED storage yields a preliminary volume of 4.1 ac-ft.
- From elevation-storage table (Figure 6), read elevation 930.1.

Size 25 year slot to release 101 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 $Q_{p\text{-}25}$ = 101 cfs - (.88 + 1.32) = 98.8 cfs, say 99 cfs.
- Max head = (930.1 – 929) = 1.1'
- Use weir equation to compute slot length
  - $Q = CLH^{3/2}$
  - L = 99 cfs / (3.1) $(1.1^{3/2})$ = 27.7 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 > 99 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 101 cfs at elevation 930.1

- Check inlet condition: (use Section 4.3 culvert charts)
  - H_w = 930.1-919.5 = 10.6 ft
  - Try 33" diameter RCP, Using Figure 4.3-1 with entrance condition 1
  - 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 WQv-ED 3.7" eq dia		Riser						Barrel				Emergency Spillway		Total Discharge
				Cpv-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 = 267 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 = 267. (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 embankment height, Pond will remain Category II or lower.

As reported in Table 1, the preliminary maximum storage volume required is about 3.5 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.	4.1	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.

Step 12 -- Prepare Vegetation and Landscaping Plan

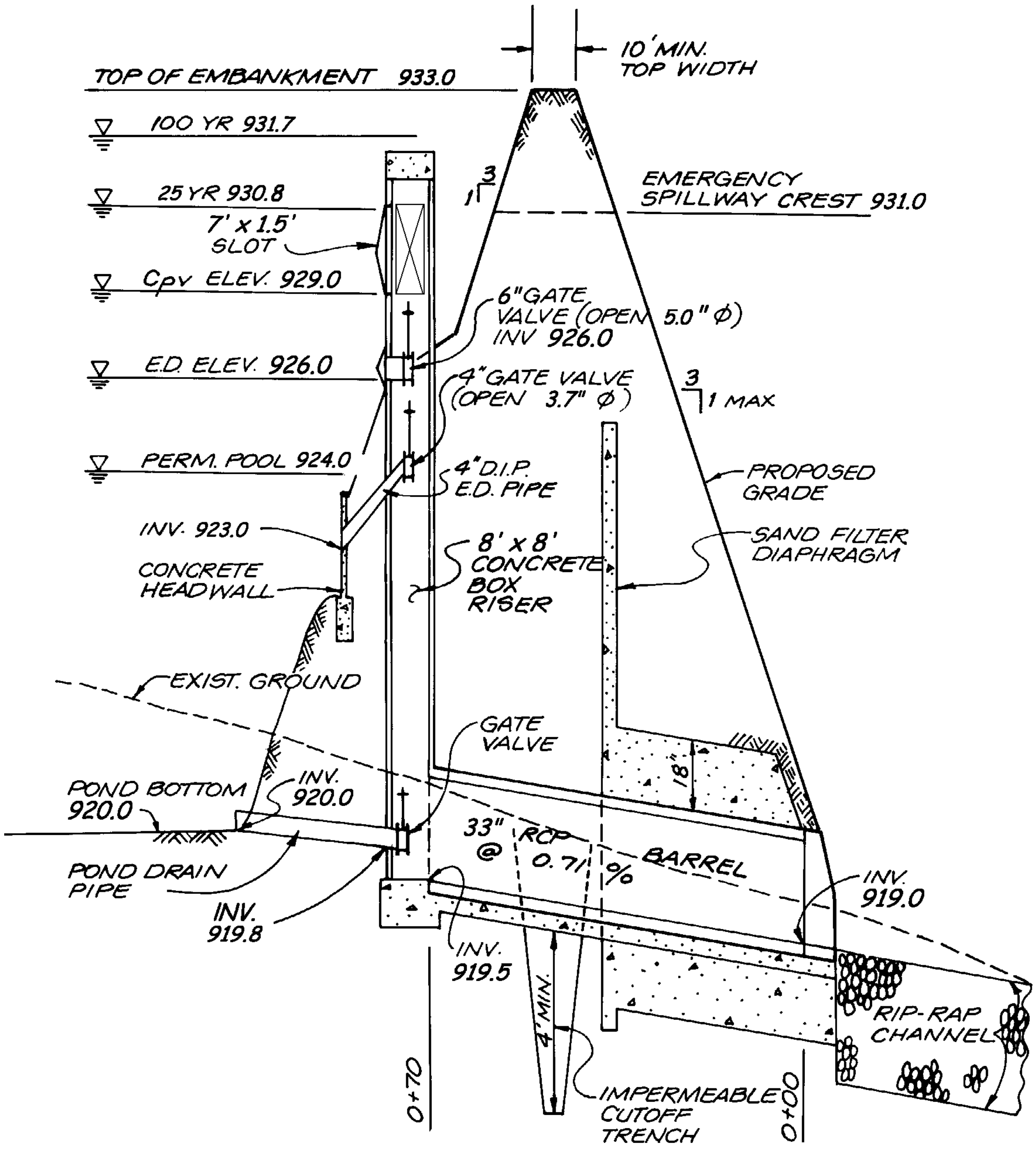


Figure 8. Profile of Principle Spillway

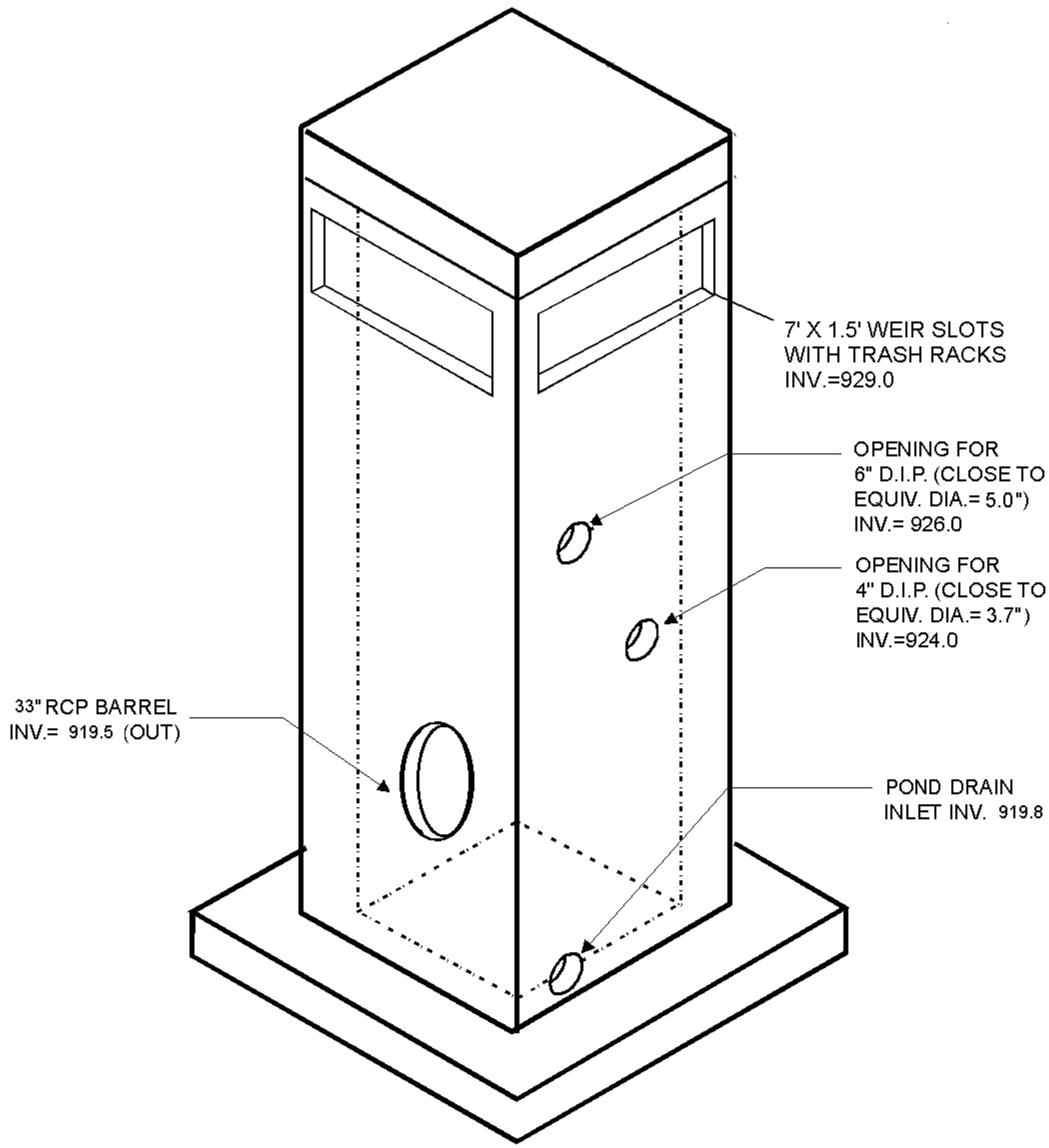


Figure 9. Schematic of Riser Detail

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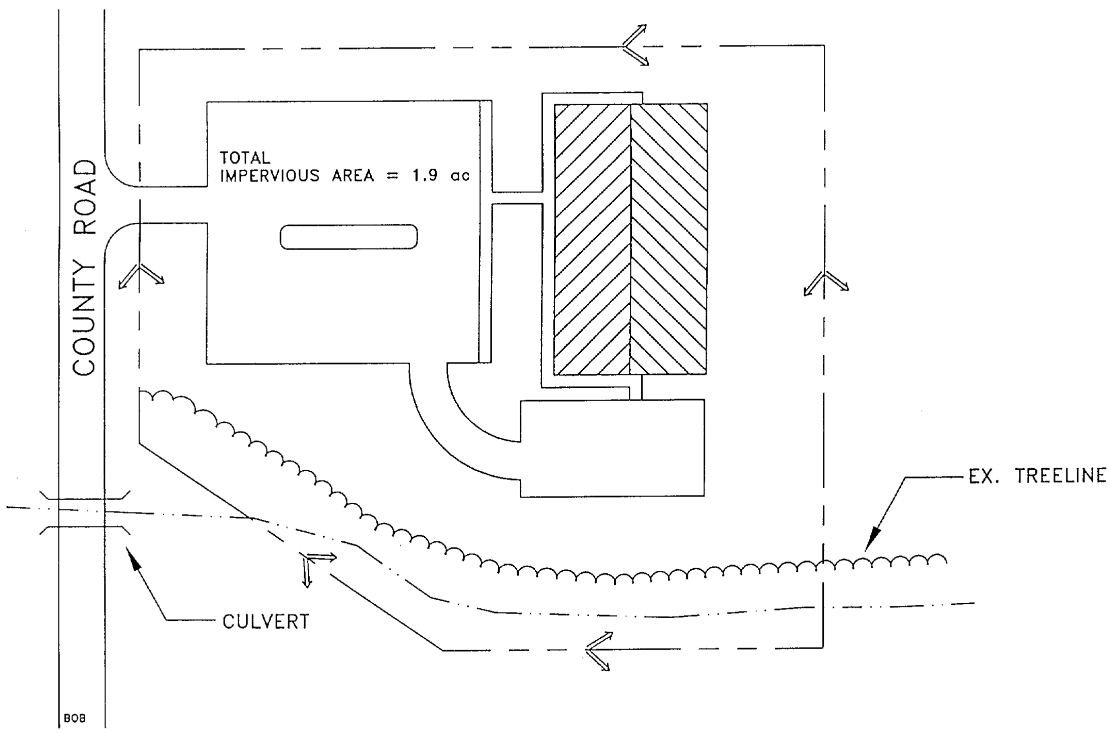
APPENDIX D-2

Bioretention Area Design Example



Base Data	Hydrologic Data		
Site Area = Total Drainage Area (A) = 3.0 ac		**Pre**	**Post**
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%	CN	70	88
Soils Type "C"	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 treatment 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 treatment 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.

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

The layout of the Etowah Recreation Center is shown in Figure 1.

Step 1 -- Compute runoff control volumes from the Unified Stormwater Sizing Criteria

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, Rv

$R_v = 0.05 + (63.3)(0.009) = 0.62$

- Compute WQ_v

$$\begin{aligned} WQ_v &= (1.2")(R_v)(A) / 12 \\ &= (1.2")(0.62)(3.0ac)(43{,}560ft^2/ac)(1ft/12in) \\ &= \underline{8{,}102}\ ft^3 \end{aligned}$$

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 2.1 to determine pre- and post-development peak discharges for the 1-yr, 25-yr, and 100-yr 24-hour return frequency storms.

- Per attached TR-55 calculations (Figures 2 and 3)

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{25\text{-year}}$	$Q_{100\ year}$
		Inches	*cfs*	*cfs*	*cfs*
Pre-developed	70	0.9	2.3	9.0	12.0
Post-Developed	88	2.1	8.1	19.0	25.0

- Utilize modified TR-55 approach to compute channel protection storage volume

Initial abstraction (Ia) for CN of 88 is 0.27: [Ia = (200/CN - 2)]

Ia/P = (0.27)/ 3.4 inches = 0.08
T_c = 0.20 hours
q_u = 850 csm/in

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

For a Type II rainfall distribution,

$Vs/Vr = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$
Where Vs equals channel protection storage (Cp_v) and Vr equals the volume of runoff in inches.

Vs/Vr = 0.65

Therefore, Vs = Cp_v = 0.65(2.1")(1/12)(3 ac) = 0.34 ac-ft = 14,810 ft^3

Determine Overbank Flood Protection Volume (Q_{p25}):

For a Q_{in} of 19 cfs, and an allowable Q_{out} of 9 cfs, the Vs necessary for 25-year control is 0.38 ac-ft or 16,553 ft^3, under a developed CN of 88. Note that 6.5 inches of rain fall during this event, with 5.1 inches of runoff.

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
WQ_v	Water Quality	8,102	
Cp_v	Channel Protection	14,810	
Q_{p25}	Overbank Flood Protection	16,553	
Q_f	Extreme Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE SUMMARY				
JOB:	Etowah Recreation Center			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)
woods (good cond.)		C	70	3.00 Ac.
		AREA SUBTOTALS:		3.00 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	dense grass	'n'=0.24	150 Ft.	1.50% 0.33 Hrs
Shallow Flow	unpaved		500 Ft. 2.28 F.P.S.	2.00% 0.06 Hrs.
Channel Flow				
Total Area in Acres =	3.00 Ac.	Total Sheet Flow= 0.33 Hrs.	Total Shallow Flow= 0.06 Hrs.	Total Channel Flow = 0.00 Hrs.
Weighted CN =	70			
Time Of Concentration =	0.39 Hrs.			
Pond Factor =	1	RAINFALL TYPE II		
STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	3.4 In.	0.9 In.	2.3 CFS	10,049 Cu. Ft
2 Year	4.1 In.	1.4 In.	3.5 CFS	15,064 Cu. Ft
5 Year	4.8 In.	1.9 In.	5 CFS	20,574 Cu. Ft
10 Year	5.5 In.	2.4 In.	7 CFS	26,459 Cu. Ft
25 Year	6.5 In.	3.2 In.	9 CFS	34,748 Cu. Ft
50 Year	7.2 In.	3.8 In.	10 CFS	41,221 Cu. Ft
100 Year	7.9 In.	4.4 In.	12 CFS	47,868 Cu. Ft

Figure 2. Etowah Recreation Center Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:	Etowah Recreation Center			EWB
DRAINAGE AREA NAME:	Post-Development 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 (good cond.)		C	74	0.50 Ac.
woods (good cond.)		C	70	0.60 Ac.
impervious		C	98	1.90 Ac.
		AREA SUBTOTALS:		3.00 Ac.
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 4.1 In	Cross Section	Wetted Per	Avg Velocity	Tt (Hrs)
Sheet Flow	dense grass	'n'=0.24	50 Ft.	1.50%
				0.14 Hrs
Shallow Flow	paved		600 Ft.	2.00%
			2.87 F.P.S.	0.06 Hrs.
Channel Flow		'n'=0.024	50 Ft.	2.00%
Hydraulic Radius =0.75	X-S estimated	WP estimated	7.25 F.P.S.	0.00 Hrs.
Total Area in Acres =	3.00 Ac.	Total Sheet Flow=	Total Shallow Flow=	Total Channel Flow =
Weighted CN =	88			
Time Of Concentration =	0.20 Hrs.	0.14 Hrs.	0.06 Hrs.	0.00 Hrs.
Pond Factor =	1	RAINFALL TYPE II		
STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	3.4 In.	2.1 In.	8.1 CFS	23,320 Cu. Ft.
2 Year	4.1 In.	2.8 In.	10.6 CFS	30,527 Cu. Ft.
5 Year	4.8 In.	3.5 In.	13 CFS	37,890 Cu. Ft.
10 Year	5.5 In.	4.2 In.	16 CFS	45,356 Cu. Ft.
25 Year	6.5 In.	5.1 In.	19 CFS	55,422 Cu. Ft.
50 Year	7.2 In.	5.8 In.	22 CFS	63,030 Cu. Ft.
100 Year	7.9 In.	6.5 In.	25 CFS	70,676 Cu. Ft.

Figure 3. Etowah Recreation Center Post-Developed Conditions

Step 2 -- Determine if the development site and conditions are appropriate for the use of a bioretention area.

Site Specific Data:

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 -- Confirm local design criteria and applicability

There are no additional local criteria that must be met for this 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 (19.0 cfs), and *no special flow diversion structure is incorporated.*

Step 6 -- Determine size of bioretention ponding / filter area

$$A_f = (WQ_v)(d_f) / [(k)(h_f + d_f)(t_f)]$$

Where: A_f = surface area of filter bed (ft^2)
d_f = filter bed depth (ft)
k = coefficient of permeability of filter media (ft/day)
h_f = average height of water above filter bed (ft)
t_f = design filter bed drain time (days) (48 hours is recommended)

$A_f = (8{,}102\ ft^3)(5') / [(0.5'/day)(0.25' + 5')(2\ days)]$ (With $k = 0.5'/day$, $h_f = 0.25'$, $t_f = 2$ days)

A_f = 7,716 sq ft

Step 7 -- Set design elevations and dimensions of facility

Assume a roughly 2 to 1 rectangular shape. Given a filter area requirement of 7,716 sq ft, say facility is roughly 65' by 120'. See Figure 5. Set top of facility at 921.0 feet, with the berm at 922.0 feet. The facility is 5' deep, which will allow 3' of freeboard over the seasonally high water table. See Figure 6 for a typical section of the facility.

Step 8 -- Design conveyance to facility (off-line systems)

This facility is not designed as an off-line system.

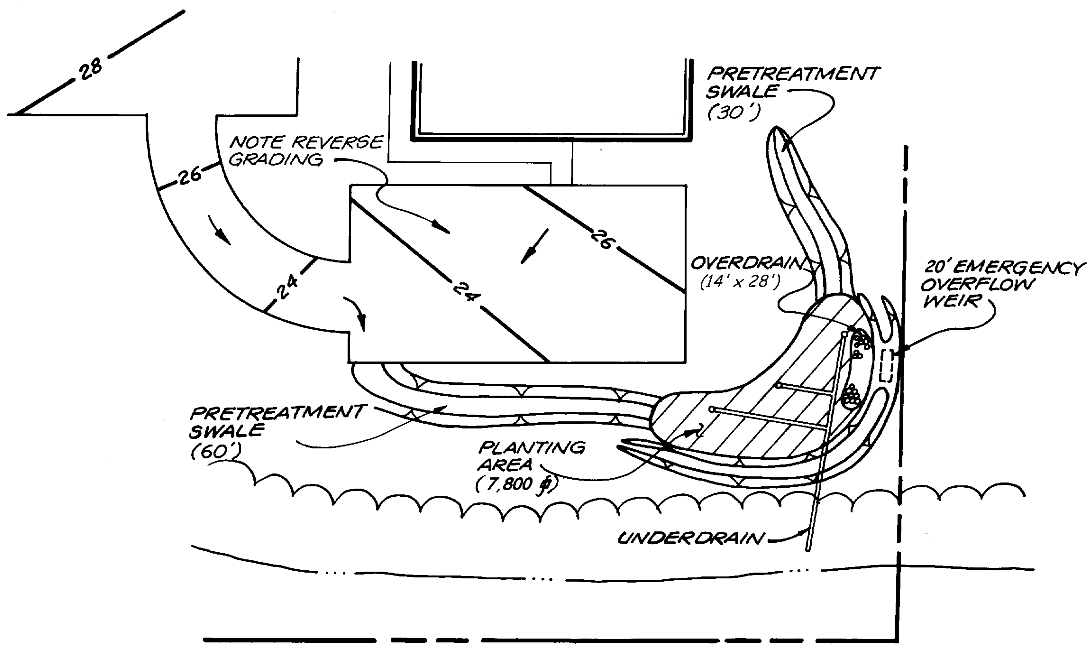


Figure 5. Plan View of Bioretention Facility

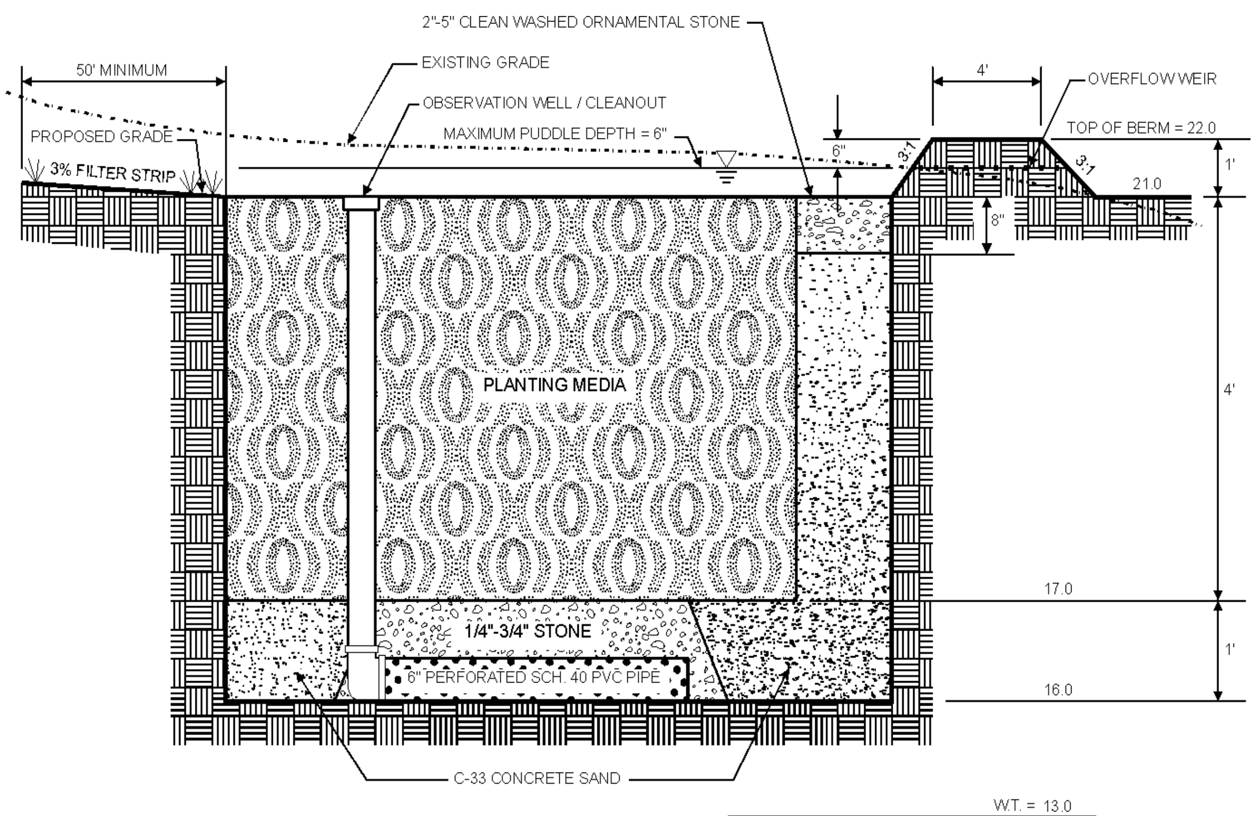


Figure 6. Typical Section of Bioretention Facility

Step 9 -- Design pretreatment

Pretreat with a grass channel, based on guidance provided in Table 2, below. For a 3.0 acre drainage area, 63% imperviousness, and slope less than 2.0%, provide a 90' grass channel at 1.5% slope. The value from Table 2 is 30' for a one acre drainage area.

Table 2 Pretreatment Grass Channel Guidance for 1.0 Acre Drainage Area
(Adapted from Claytor and Schueler, 1996)

Parameter	≤ 33% Impervious		Between 34% & 66% Impervious		≥ 67% Impervious		Notes
Slope	≤2%	≥2%	≤2%	≥2%	≤2%	≥2%	Max slope = 4%
Grassed channel min. length (feet)	25	40	30	45	35	50	Assumes a 2' wide bottom width

Step 10 -- Size underdrain area

Base underdrain design on 10% of the A_f or 772 sq ft. Using 6" perforated plastic pipes surrounded by a three-foot-wide gravel bed, 10' on center (o.c.). See Figures 5 and 6 .

(772 sq ft)/3' per foot of underdrain = 257', say 260' of perforated underdrain

Step 11 – Design emergency overflow

To ensure against the planting media clogging, design a small ornamental stone window of 2" to 5" stone connected directly to the sand filter layer. This area is based on 5% of the A_f or 386 sq ft. Say 14' by 28'. See Figures 5 and 6.

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 = 19.0 cfs
H = 6"

Solve for L: $L = Q / [(C)(H^{3/2})]$ or (19.0 cfs) / $[(2.65)(.5)^{1.5}]$ = 20.3' (say 20')

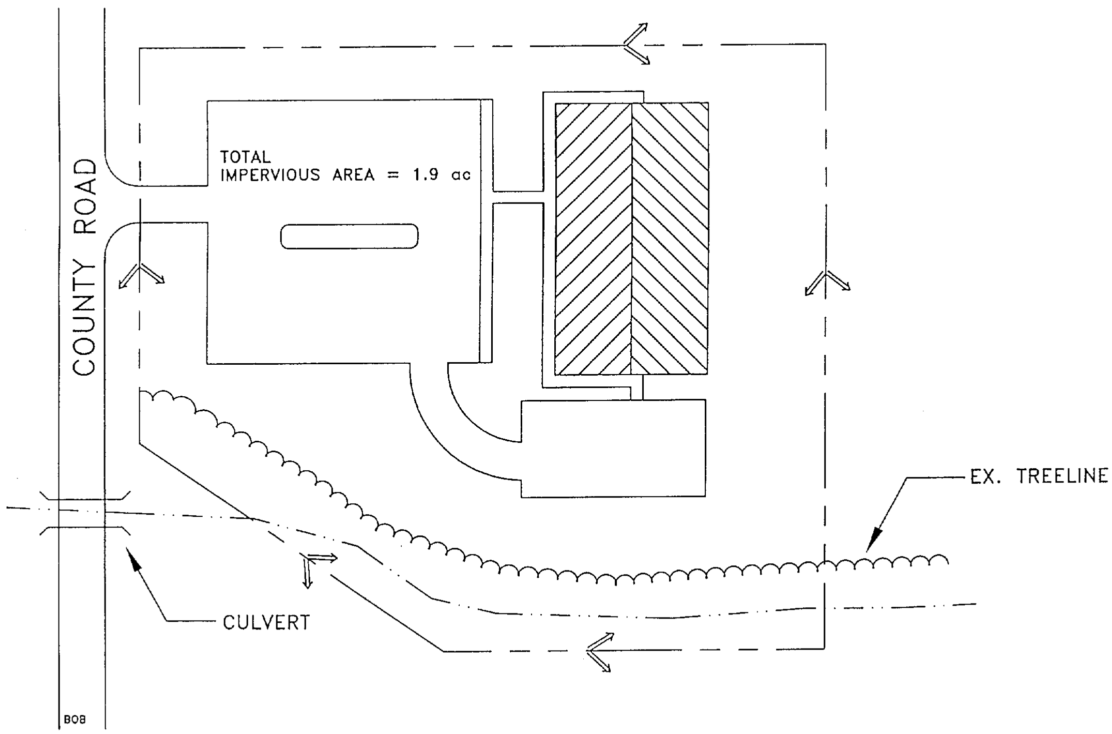
Outlet protection in the form of riprap or a plunge pool/stilling basin should be provided to ensure non-erosive velocities. See Figures 5 and 6.

Step 12 – 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 F.

APPENDIX D-3

Sand Filter Design Example



Base Data	Hydrologic Data		
		Pre	**Post**
Site Area = Total Drainage Area (A) = 3.0 ac	CN	57	83
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%	t_c	.36	.15
Soils Type “B”			

Figure 1. Georgia Pines Community Center Site Plan

This example focuses on the design of a surface sand filter 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 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).

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

The layout of the Georgia Pines Community Center is shown in Figure 1.

Step 1 -- Compute runoff control volumes from the Unified Stormwater Sizing Criteria

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, Rv

  R_v = 0.05 + (63.3) (0.009) = 0.62

- Compute WQ_v

  WQ_v = (1.2”) (R_v) (A) / 12
  = (1.2”) (0.62) (3.0ac) (43,560ft^2/ac) (1ft/12in)
  = 8,102 ft^3 = 0.186 ac-ft

Compute Stream Channel Protection Volume, (Cp_v):

For stream channel protection, provide 24 hours of extended detention for the 1-year event.

- Develop Site Hydrologic and Hydrologic Input Parameters and Perform Preliminary Hydrologic Calculations

  Per Figures 2 and 3. Note that any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{25\text{-year}}$	$Q_{100\ \text{year}}$
		Inches	*cfs*	*cfs*	*cfs*
Pre-developed	57	0.5	0.6	6.0	9.0
Post-Developed	83	1.9	5.5	17.0	22.0

- 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
  From TR-55, Exhibit 4-III (NRCS, 1986):
  q_u = 590 csm/in

  Knowing q_u and T (extended detention time), find qo/qi for a Type II rainfall distribution.

Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.03

$Vs/Vr = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$
Where Vs equals channel protection storage (Cp_v) and Vr equals the volume of runoff in inches.

Vs/Vr = 0.64

Therefore, Vs = Cp_v = 0.64(1.9")(1/12)(3 ac) = 0.30 ac-ft = 13,068 ft^3

- Define the average ED Release Rate

The above volume, 0.30 ac-ft, is to be released over 24 hours.
(0.30 ac-ft × 43,560 ft^2/ac) / (24 hrs × 3,600 sec/hr) = 0.15 cfs

Determine Overbank Flood Protection Volume (Q_{p25}):

For a Q_{in} of 17 cfs, and an allowable Q_{out} of 6 cfs, the Vs necessary for 25-year control is 0.52 ac-ft or 22,677 ft^3, under a developed CN of 83. Note that 7.9 inches of rain fall during this event, with 5.9 inches of runoff.

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	13,068	
Q_{p25}	Overbank Flood Protection	22,677	
Q_f	Extreme Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE				
JOB:	Georgia Pines Center			EWB
DRAINAGE AREA NAME:	Pre-Developed			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.)		B	58	2.40 Ac.
meadow (good cond.)		B	55	0.60 Ac.
		AREA		3.00 Ac.
Time of Concentration	Surface Cover	Manning	Flow	Slope
2-Yr 24 Hr Rainfall = 4.8	Cross	Wetted Per	Avg	Tt (Hrs)
Sheet	dense grass	'n'=0.24	150 Ft.	1.50%
				0.30 Hrs
Shallow	unpaved		500 Ft.	2.00%
			2.28	0.06
Channel				
Total Area in Acres =	3.00 Ac.	Total Sheet Flow=	Total Flow=	Total Channel Flow =
Weighted CN =	57			
Time Of Concentration =	0.36	0.30	0.06	0.00
Pond Factor =	1	RAINFALL TYPE		
STORM	*Precipitation* (P) inches	*Runoff* (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	3.6 In.	0.5 In.	0.6 CFS	4,943 Cu.
2 Year	4.8 In.	1.0 In.	1.8 CFS	10,887 Cu.
5 Year	6.0 In.	1.7 In.	3 CFS	18,252 Cu.
10 Year	6.7 In.	2.1 In.	4 CFS	23,186 Cu.
25 Year	7.9 In.	2.9 In.	6 CFS	32,076 Cu.
50 Year	8.9 In.	3.6 In.	8 CFS	39,672 Cu.
100 Year	9.8 In.	4.4 In.	9 CFS	47,613 Cu.

Figure 2. Georgia Pines Community Center Pre-Developed Conditions

PEAK DISCHARGE				
JOB:	Georgia Pines Center			EWB
DRAINAGE AREA	Post-Development			3-Jan-00
COVER	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2.1.5-1	AREA (In acres)
open space (good		B	61	0.50 Ac.
woods (good		B	55	0.60 Ac.
impervious		B	98	1.90 Ac.
		AREA		3.00 Ac.
Time of	Surface Cover	Manning	Flow	Slope
2-Yr 24 Hr Rainfall = 4.8	Cross	Wetted Per	Avg	Tt (Hrs)
Sheet Flow	short grass	'n'=0.15	50 Ft.	1.50% 0.09 Hrs
Shallow	paved		600 Ft. 2.87 F.P.S.	2.00% 0.06 Hrs.
Channel Hydraulic Radius	X-S	'n'=0.024 WP estimated	50 Ft. 7.25 F.P.S.	2.00% 0.00 Hrs.
Total Area in Acres	3.00 Ac.	Total Sheet Flow=	Total Flow=	Total Channel Flow =
Weighted CN =	83			
Time Of Concentration	0.15 Hrs.	0.09 Hrs.	0.06 Hrs.	0.00 Hrs.
Pond Factor =	1	RAINFALL TYPE		
STORM	*Precipitation* (P) inches	*Runoff* (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	3.6 In.	1.9 In.	5.5 CFS	21,159 Cu.
2 Year	4.8 In.	3.0 In.	8.6 CFS	32,602 Cu.
5 Year	6.0 In.	4.1 In.	12 CFS	44,555 Cu.
10 Year	6.7 In.	4.8 In.	14 CFS	51,881 Cu.
25 Year	7.9 In.	5.9 In.	17 CFS	64,262 Cu.
50 Year	8.9 In.	6.8 In.	20 CFS	74,281 Cu.
100 Year	9.8 In.	7.7 In.	22 CFS	84,372 Cu.

Figure 3. Georgia Pines Community Center Post-Developed Conditions

Step 2 -- Determine if the development site and conditions are appropriate for the use of a surface sand filter.

Site Specific Data:

Existing ground elevation at the facililty 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 -- Confirm local design criteria and applicability.

There are no additional requirements for this site.

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 5)

  Low point at parking lot is 23.5. Subtract 2' to pass Q_{25} 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, h_f, is (h_f) = 4.33' / 2, and h_f = 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 SCS 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 procedure can be used to estimate peak discharges for small storm events. It relies on the volume of runoff computed using the Small Storm Hydrology Method (Pitt, 1994) and utilizes the NRCS, TR-55 Graphical Peak Discharge Method (USDA, 1986). A brief description of the calculation procedure is presented below.

- Using the water quality volume (WQ_v), a corresponding Curve Number (CN) is computed utilizing the following equation:

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

where P = rainfall, in inches (use 1.2" for the Water Quality Storm)
and Q = runoff volume, in inches (equal to WQ_V ÷ area)

- Once a CN is computed, 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 2.1 (either Type II or Type III in the State of Georgia).

  - Read initial abstraction (I_a), compute I_a/P
  - Read the unit peak discharge (q_u) for appropriate t_c

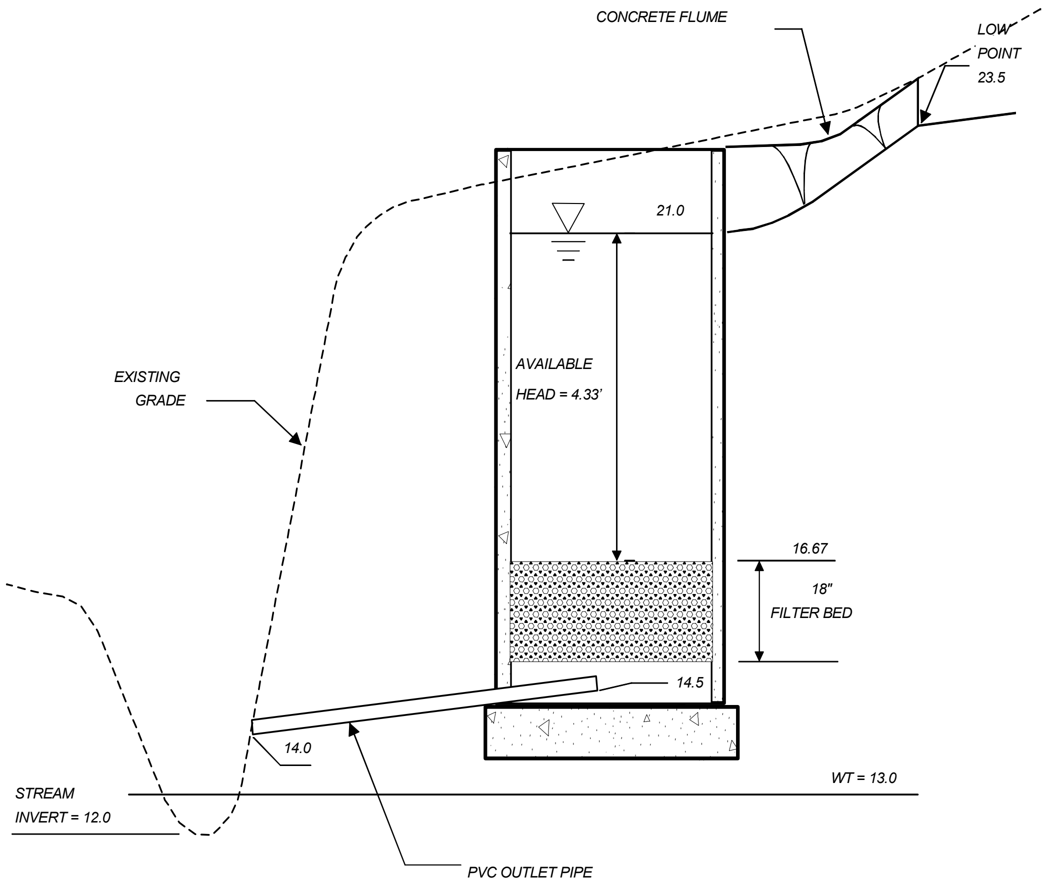


Figure 5. Available Head Diagram

- Using the water quality volume (WQ_v), compute the water quality peak discharge (Q_{wq})

$$\mathbf{Q_{wq} = q_u*A*WQ_V}$$

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 = WQ_v ÷ area = (8,102 ft^3 ÷ 3 ac ÷ 43,560 ft^2/ac × 12 in/ft) = 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 I_a/P = 0.105/1.2 = 0.088. From Section 2.1, q_u = 625 csm/in, and therefore Q_{wq} = (625 csm/in) (3.0 ac/640ac/sq mi.) (0.74") = <u>2.2 cfs</u>.

Step 5 -- Size flow diversion structure (see Figure 6):

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 (17.0 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.

$Q = CLH^{3/2}$

$17 = 3.1 (L) (2')^{1.5}$

L = 1.94'; 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 7):

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)

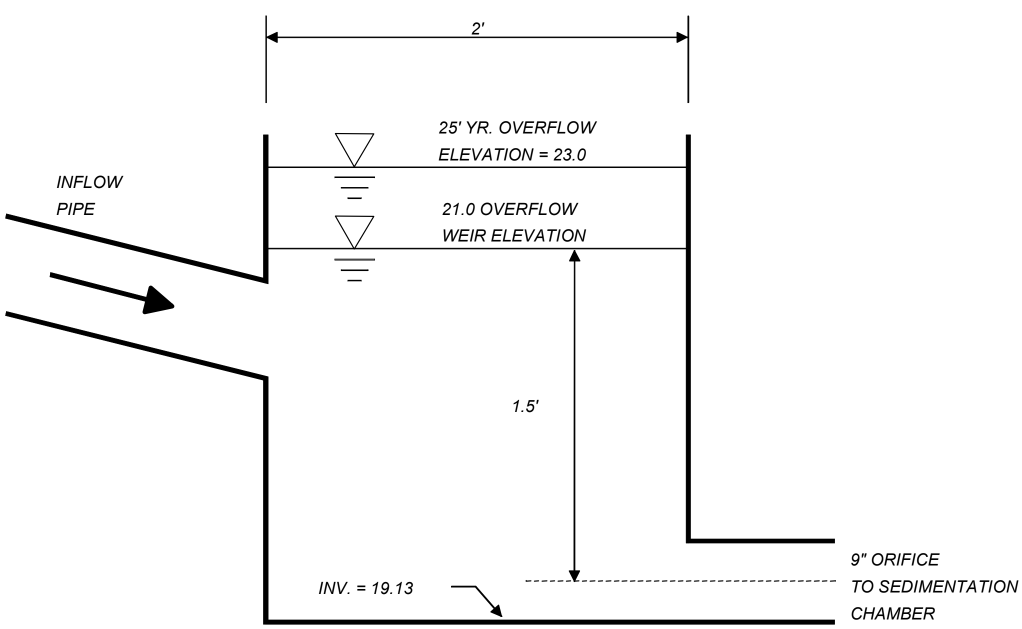


Figure 6. Flow Diversion Structure

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} = ¾(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 ($V_{f\text{-}temp}$): $V_{f\text{-}temp} = 2h_fA_f$
$V_{f\text{-}temp}$ = 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

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 8). 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 = $0.6*A*(2*32.2\ ft/s^2*2.17\ ft)^{1/2}$
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 9 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).

$Q = CA(2gh)^{½}$
$Q = 0.6(0.44\ ft^2)[(2)(32.2\ ft/s^2)(3.5\ ft)]^{½}$
$Q = 3.96$ 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'
$4.0 = 3.1 * L * (0.5\ ft)^{3/2}$
L = 3.65 ft, <u>Use L = 3.75 ft.</u>

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'
$4.0 = 3.1 * L * (0.5\ ft)^{3/2}$
L = 3.65 ft, <u>Use L = 3.75 ft.</u>

Adequate outlet protection and energy dissipation (e.g., riprap, plunge pool, etc.) should be provided for the downstream overflow channel.

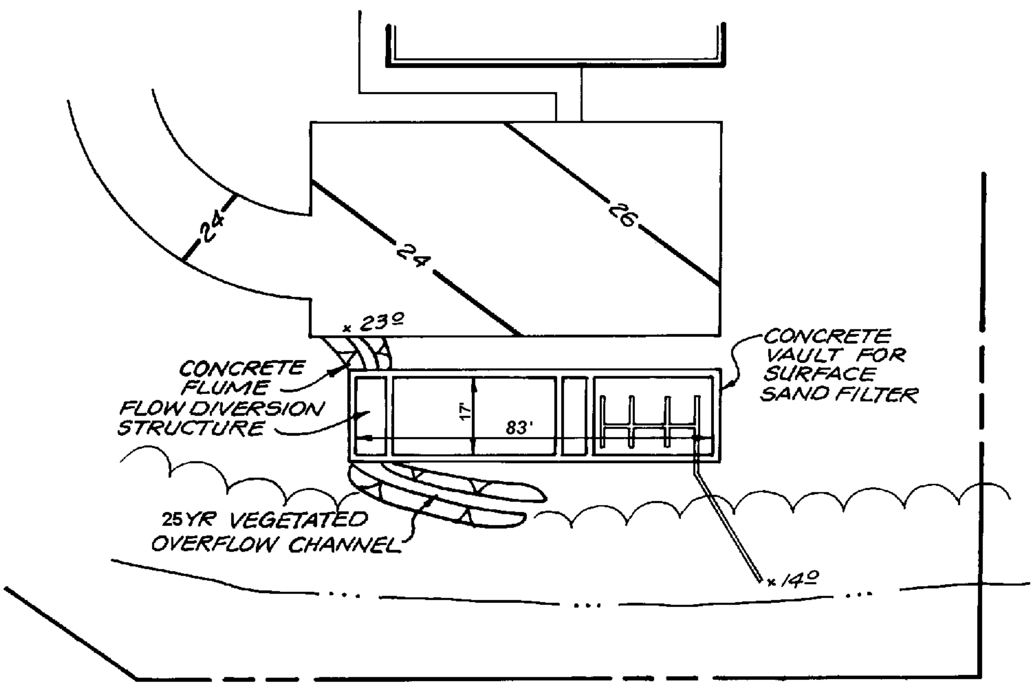


Figure 7. Surface Sand Filter Site Plan

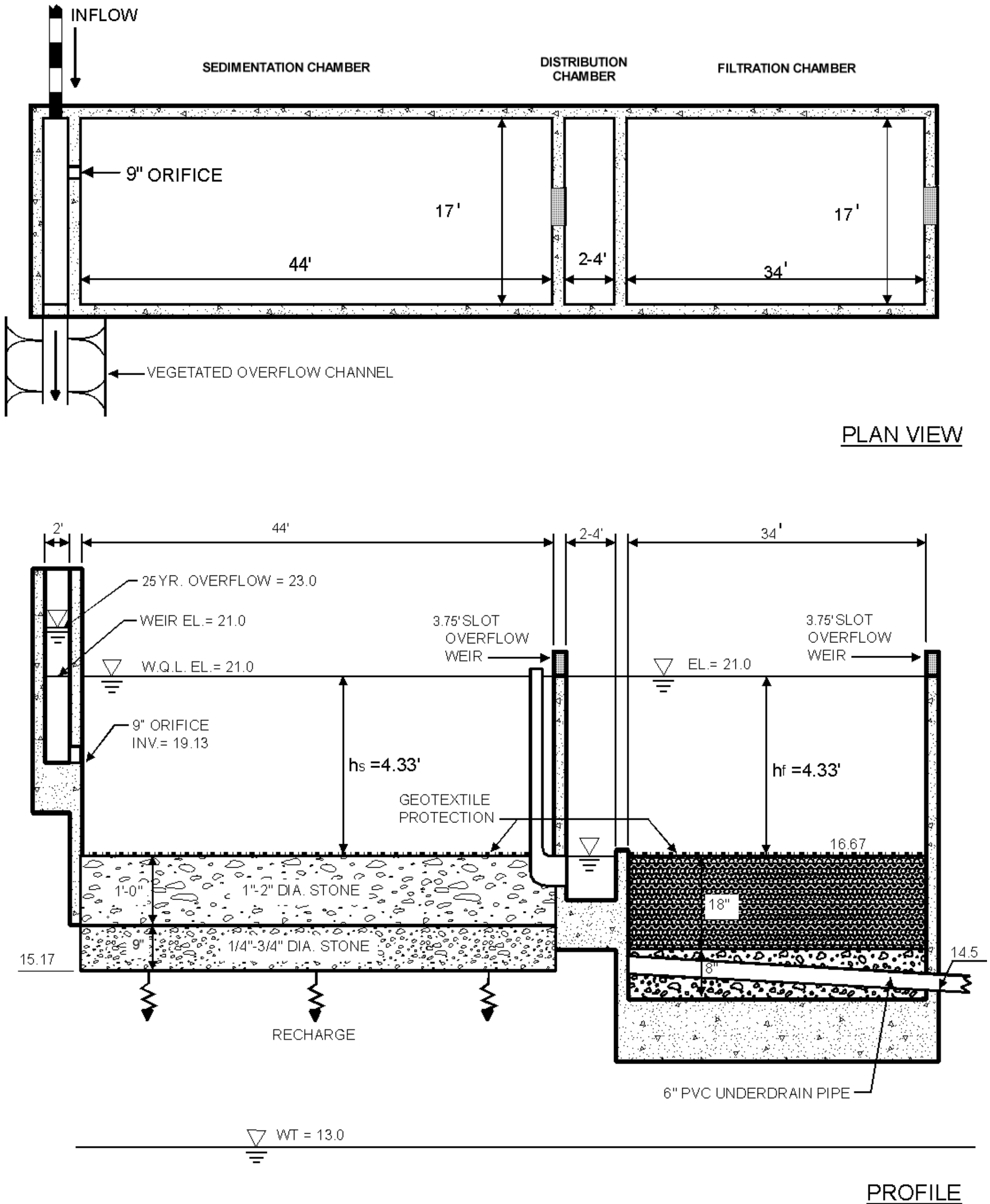


Figure 8. Plan and Profile of Surface Sand Filter

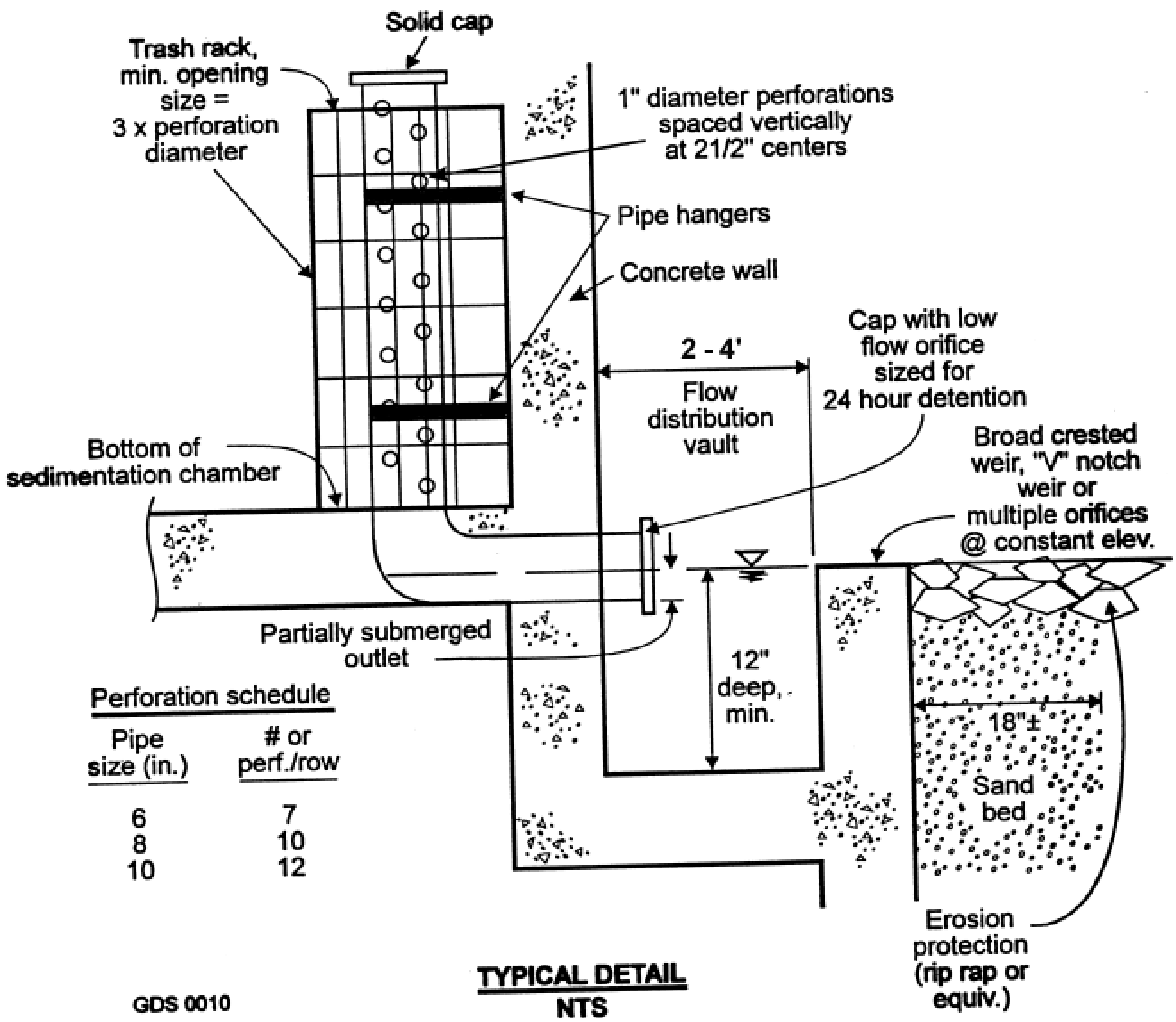
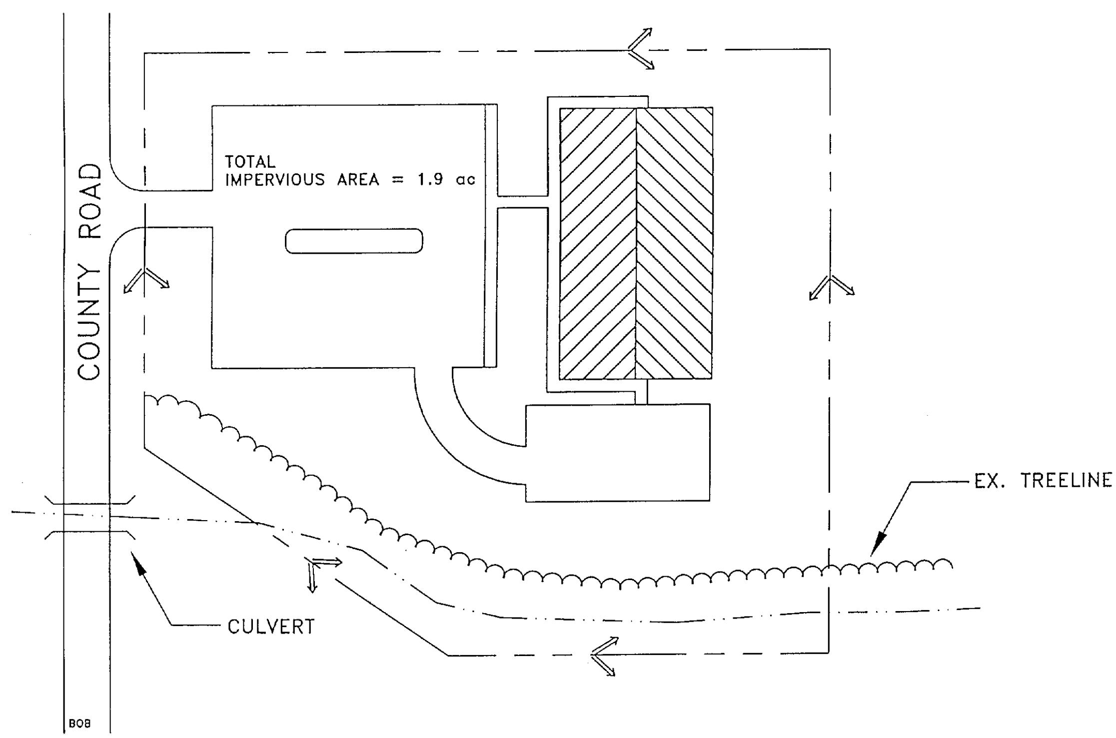


Figure 9. Perforated Stand Pipe Detail

APPENDIX D-4

Infiltration Trench Design Example



Base Data	Hydrologic Data		
Site Area = Total Drainage Area (A) = 3.0 ac		**Pre**	**Post**
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%	CN	57	83
Soils Type "B"	t_c	.36	.15

Figure 1. Georgia Pines Community Center Site Plan

This example focuses on the design of an infiltration trench 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 infiltration trenches 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).
The layout of the Georgia Pines Community Center is shown in Figure 1.

Step 1 -- Compute runoff control volumes from the Unified Stormwater Sizing Criteria

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, Rv

  R_v = 0.05 + (63.3) (0.009) = 0.62

- Compute WQ_v

$$WQ_v = (1.2\text{"}) (R_v) (A) / 12$$
$$= (1.2\text{"}) (0.62) (3.0ac) (43{,}560ft^2/ac) (1ft/12in)$$
$$= \underline{8{,}102}\ ft^3 = \underline{0.186}\ ac\text{-}ft$$

Compute Stream Channel Protection Volume, (Cp_v):

For stream channel protection, provide 24 hours of extended detention for the 1-year event.

- Develop Site Hydrologic and Hydrologic Input Parameters and Perform Preliminary Hydrologic Calculations

  Per Figures 2 and 3. Note that any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{25\text{-year}}$	$Q_{100\ year}$
		Inches	*cfs*	*cfs*	*cfs*
Pre-developed	57	0.5	0.6	6.0	9.0
Post-Developed	83	1.9	5.5	17.0	22.0

- 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
  From TR-55, Exhibit 4-III (NRCS, 1986):
  q_u = 590 csm/in

  Knowing q_u and T (extended detention time), find qo/qi for a Type II rainfall distribution.

Peak outflow discharge/peak inflow discharge $(q_o/q_i) = 0.03$

$Vs/Vr = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$
Where Vs equals channel protection storage (Cp_v) and Vr equals the volume of runoff in inches.

$Vs/Vr = 0.64$

Therefore, $Vs = Cp_v = 0.64(1.9")(1/12)(3 \text{ ac}) = 0.30 \text{ ac-ft} = 13{,}068 \text{ ft}^3$

- <u>Define the average ED Release Rate</u>

The above volume, 0.30 ac-ft, is to be released over 24 hours.
$(0.30 \text{ ac-ft} \times 43{,}560 \text{ ft}^2/\text{ac}) / (24 \text{ hrs} \times 3{,}600 \text{ sec/hr}) = 0.15 \text{ cfs}$

Determine Overbank Flood Protection Volume (Q_{p25}):

For a Q_{in} of 17 cfs, and an allowable Q_{out} of 6 cfs, the Vs necessary for 25-year control is 0.52 ac-ft or <u>22,677</u> ft^3, under a developed CN of 83. Note that 7.9 inches of rain fall during this event, with 5.9 inches of runoff.

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	13,068	
Q_{p25}	Overbank Flood Protection	22,677	
Q_f	Extreme Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE				
JOB:	Georgia Pines Center			EWB
DRAINAGE AREA NAME:	Pre-Developed			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.)		B	58	2.40 Ac.
meadow (good cond.)		B	55	0.60 Ac.
		AREA		3.00 Ac.
Time of Concentration 2-Yr 24 Hr Rainfall = 4.8	Surface Cover Cross	Manning Wetted Per	Flow Avg	Slope Tt (Hrs)
Sheet	dense grass	'n'=0.24	150 Ft.	1.50% 0.30 Hrs
Shallow	unpaved		500 Ft. 2.28	2.00% 0.06
Channel				
Total Area in Acres =	3.00 Ac.	Total Sheet Flow=	Total Flow=	Total Channel Flow =
Weighted CN =	57	0.30	0.06	0.00
Time Of Concentration =	0.36			
Pond Factor =	1	RAINFALL TYPE		
STORM	*Precipitation* (P) inches	*Runoff* (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	3.6 In.	0.5 In.	0.6 CFS	4,943 Cu.
2 Year	4.8 In.	1.0 In.	1.8 CFS	10,887 Cu.
5 Year	6.0 In.	1.7 In.	3 CFS	18,252 Cu.
10 Year	6.7 In.	2.1 In.	4 CFS	23,186 Cu.
25 Year	7.9 In.	2.9 In.	6 CFS	32,076 Cu.
50 Year	8.9 In.	3.6 In.	8 CFS	39,672 Cu.
100 Year	9.8 In.	4.4 In.	9 CFS	47,613 Cu.

Figure 2. Georgia Pines Community Center Pre-Developed Conditions

PEAK DISCHARGE				
JOB:	Georgia Pines Center			EWB
DRAINAGE AREA	Post-Development			3-Jan-0
COVER	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2.1.5-1	AREA (In acres)
open space (good		B	61	0.50 Ac.
woods (good		B	55	0.60 Ac.
impervious		B	98	1.90 Ac.
		AREA		3.00 Ac.
Time of	Surface Cover	Manning	Flow	Slope
2-Yr 24 Hr Rainfall = 4.8	Cross	Wetted Per	Avg	Tt (Hrs)
Sheet Flow	short grass	'n'=0.15	50 Ft.	1.50% 0.09 Hrs
Shallow	paved		600 Ft. 2.87 F.P.S.	2.00% 0.06 Hrs.
Channel Hydraulic Radius	X-S	'n'=0.024 WP estimated	50 Ft. 7.25 F.P.S.	2.00% 0.00 Hrs.
Total Area in Acres	3.00 Ac.	Total Sheet Flow= 0.09 Hrs.	Total Flow= 0.06 Hrs.	Total Channel Flow = 0.00 Hrs.
Weighted CN =	83			
Time Of Concentration	0.15 Hrs.			
Pond Factor =	1	RAINFALL TYPE		
STORM	*Precipitation* (P) inches	*Runoff* (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	3.6 In.	1.9 In.	5.5 CFS	21,159 Cu.
2 Year	4.8 In.	3.0 In.	8.6 CFS	32,602 Cu.
5 Year	6.0 In.	4.1 In.	12 CFS	44,555 Cu.
10 Year	6.7 In.	4.8 In.	14 CFS	51,881 Cu.
25 Year	7.9 In.	5.9 In.	17 CFS	64,262 Cu.
50 Year	8.9 In.	6.8 In.	20 CFS	74,281 Cu.
100 Year	9.8 In.	7.7 In.	22 CFS	84,372 Cu.

Figure 3. Georgia Pines Community Center Post-Developed Conditions

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%

Step 3 -- Confirm local design criteria and applicability

Table 3, 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 3 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' Elevation of BMP location: 20'. The difference is 7'. 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 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 4 -- Compute WQ_v peak discharge (Q_{wq})

- <u>Compute Water Quality Volume:</u>

  WQ_v previously determined to be 8,102 cubic feet.

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 SCS 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 or leads to the design of undersized grass channels.

The following procedure can be used to estimate peak discharges for small storm events. It relies on the volume of runoff computed using the Small Storm Hydrology Method (Pitt, 1994) and utilizes the NRCS, TR-55 Graphical Peak Discharge Method (USDA, 1986). A brief description of the calculation procedure is presented below.

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

where P = rainfall, in inches (use 1.2" for the Water Quality Storm)
and Q = runoff volume, in inches (equal to $WQ_V \div$ area)

- Once a CN is computed, the time of concentration (t_c) is computed (based on the methods identified in TR-55, Chapter 3: "Time of concentration and travel time").

- 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 TR-55, Chapter 4: "Graphical Peak Discharge Method"). Use appropriate rainfall distribution type (either Type II or Type III in State of Georgia).

  - Read initial abstraction (I_a), compute I_a/P
  - Read the unit peak discharge (q_u) from Exhibit 4-II or 4-III for appropriate t_c
  - Using the water quality volume (WQ_v), compute the peak discharge (Q_{wq})

$$\mathbf{Q_{wq} = q_u{*}A{*}WQ_V}$$

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 = $WQ_v \div$ area = (8,102 $ft^3 \div$ 3 ac $\div$ 43,560 ft^2/ac $\times$ 12 in/ft) = 0.74"
CN = $1000/[10+5P+10Q-10(Q^2+1.25{*}Q{*}P)^{1/2}]$
= $1000/[10+5{*}1.2+10{*}0.74-10(0.74^2+1.25{*}0.74{*}1.2)^{1/2}]$
= 95.01
<u>Use CN = 95</u>

For CN = 95 and the T_c = 0.15 hours, compute the Q_{wq} for a 1.2" storm. With the CN = 95, a 1.2" storm will produce 0.74" of runoff. I_a = 0.105, therefore I_a/P = 0.105/1.2 = 0.088. q_u = 625 csm/in, and therefore:

Q_{wq} = (625 csm/in) (3.0 ac/640ac/sq mi.) (0.74") = <u>2.2 cfs</u>.

Step 5 -- Size the infiltration trench

The area of the trench can be determined by the following equation:

$$A = \frac{WQ_v}{(nd + kT/12)}$$

Where:

A = Surface Area
n = porosity
d = trench depth (feet)
k = percolation (inches/hour)
T= Fill Time (time for the practice to fill with water), in hours

Assume that:

n = 0.32
d = 5 feet (see above; feasibility criteria)
k = 1 inch/hour (see above; site data)
T = 2 hours

Therefore:

$A = 8{,}102\ ft^3 / (0.32 \times 5 + 1 \times 2/12)ft$
$A = 4{,}586\ ft^2$

Since the width can be no greater than 25' (see above; feasibility), determine the length:

$L = 4{,}586\ ft^2 / 25\ ft$
L = 183 feet

Assume that 1/3 of the runoff from the site drains to Point A and 2/3 drains to Point B. Use an L-shaped trench in the corner of the site (see Figure 4 for a site plan view). The surface area of the trench is proportional to the amount of runoff it drains (e.g., the portion draining from Point A is half as large as the portion draining Point B).

Step 6 -- Size the flow diversion structures

Since two entrances are used, two flow diversions are needed.

For the entire site:

$Q_{25\text{-year}}$ = 17 cfs (See Figure 3)
Peak flow for WQ_v = 2.2 cfs. (Step 3).

For the first diversion (Point A)

Assume peak flow equals 1/3 of the value for the entire site.
Thus, $Q_{25\text{-year}}$ = 17/3 = 5.7 cfs
Peak flow for WQ_v = 2.2/3 = 0.73 cfs

Size the low flow orifice to pass 0.73 cfs with 1.5' of head using the Orifice equation.

$Q=CA(2gh)^{½}$; $0.73\ cfs = 0.6A(2 \times 32.2\ ft/s^2 \times 1.5')^{½}$
A=0.12 sq. ft. = $\pi d^2/4$; d = 0.4'; use 6" pipe with 6" gate valve

Size the 25-year overflow weir crest at 22.5'. Use a concrete weir to pass the 25-year flow (5.7 – 0.73 = 5 cfs). Assume 1 foot of head to pass this event. Size using the weir equation.

$Q = CLH^{1.5}$; $L= Q/(CH^{1.5})$
$L = 5\ cfs/ (3.1)(1)^{1.5} = 1.6'$; <u>use 1.6' </u>(see Figure 5)

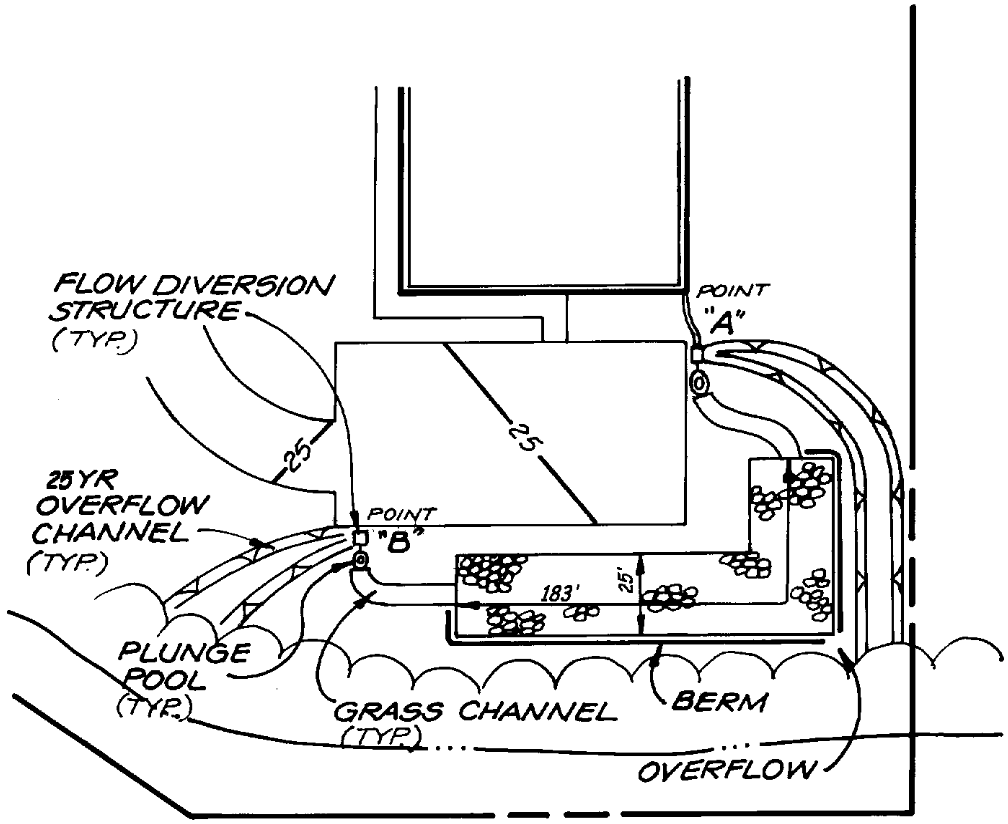


Figure 4. Infiltration Trench Site Plan

Size the second diversion (Point B) using the same techniques.
Peak flow equal 2/3 of the value for the entire site. Thus:

$Q_{25\text{-year}}$ = 17*0.67 = 11.4 cfs
Peak flow for WQ_v = 2.2*0.67 = 1.47 cfs

Size the low flow orifice to pass 1.47 cfs with 1.5' of head using the Orifice equation.

$Q=CA(2gh)^{½}$; 1.47 cfs = $0.6A(2 \times 32.2\ ft/s^2 \times 1.5')^{½}$
A=0.25 sq. ft. = $\pi d^2/4$; d = 0.56'; use 8" pipe with 8" gate valve

Size the 25-year overflow weir crest at 22.0'. Use a concrete weir to pass the 25-year flow (11.4 –1.47 = 9.9 cfs). Assume 1 foot of head to pass this event. Size using the weir equation.

$Q = CLH^{1.5}$; $L= Q/(CH^{1.5})$
$L = 9.9\ cfs/ (3.1)(1)^{1.5} = 3.2'$; <u>use 3.2'</u> (see Figure 5)

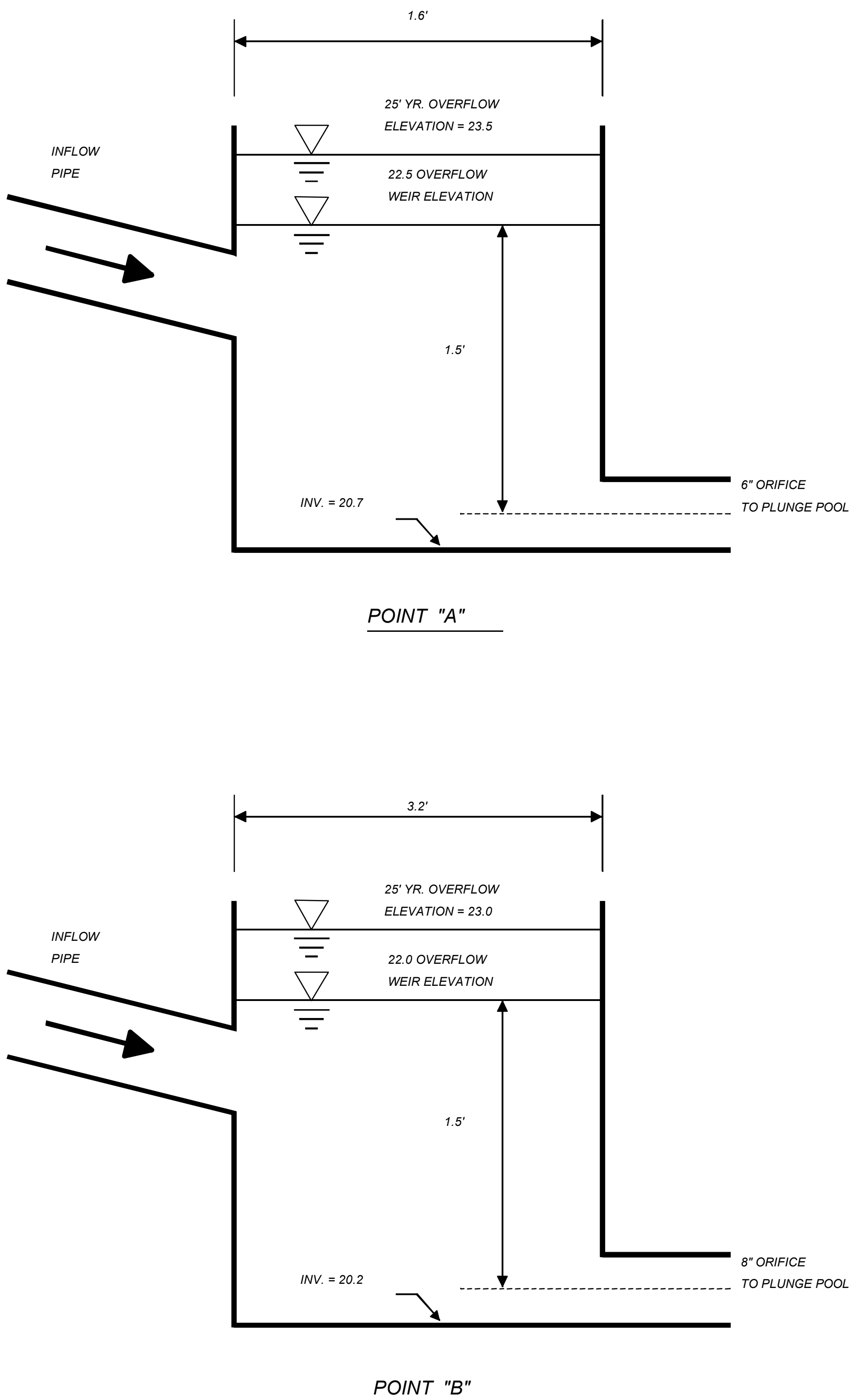


Figure 5. Flow Diversion Structures

Step 7. Size pretreatment volume and design pretreatment measures

As rule of thumb, size pretreatment to treat 25% of the WQ_v. Therefore, treat $8{,}102 \times 0.25 = 2{,}026\ ft^3$.

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 6). 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)(3,883 ft^2) = 207 ft^3

*Plunge Pool*s

Use a 5'X10' plunge pool at Point A and a 10'X10' plunge pool at Point B with average depths of 2'.

Total WQ_{pool}= (10 ft)(10+5 ft)(2 ft) = 300 ft^3

Grass Channel

Thus, the grass channel needs to treat at least (2,026 - 207 - 300)ft^3 = 1,519 ft^3

Use a Manning's equation nomograph or software to size the swale.

The channel at point A should treat one third of 1,519 ft^3 or 501 ft^3

- Assume a trapezoidal channel with 4' channel bottom, 3H:1V side slopes, and a Manning's n value of 0.15. Use a nomograph to size the swale; assume a 1% slope.
- Use a peak discharge of 0.73 cfs (Peak flow for one third of WQ_v, or 2,674 ft^3)
- Compute velocity: V=0.5 ft/s
- To retain the 1/3 of the WQ_v (2,674 ft^3) for 10 minutes, the length would be 300 feet.
- Since the swale only needs to treat 25% of the water quality volume minus the treatment provided by the plunge pool and the gravel layer, or 501 ft^3, the length should be pro-rated to reflect this reduction.

  Therefore, adjust length:

  L= (300 ft)(501 ft^3/2,674 ft^3) =56 feet. Use 60 feet.

The channel at point B should treat two thirds of 1,519 ft^3, or 1,018 ft^3

- Assume a trapezoidal channel with 5' channel bottom, 3H:1V side slopes, and a Manning's n value of 0.12. Use a nomograph to size the swale; assume a 0.5% slope.
- Use a peak discharge of 1.47 cfs (Peak flow for two thirds of WQ_v, or 5,428ft^3)
- Compute velocity: V=0.5 ft/s
- To retain the 2/3 of the WQ_v (5,428 ft^3) for 10 minutes, the length would be 300 feet.
- Since the swale only needs to treat 25% of the water quality volume minus the treatment provided by the plunge pool and the gravel layer, or 1,018 ft^3, the length should be pro-rated to reflect this reduction.

  Therefore, adjust length:

  L= (300 ft)(1,018 ft^3/5,428 ft^3) = 56 feet. Use 60 feet.

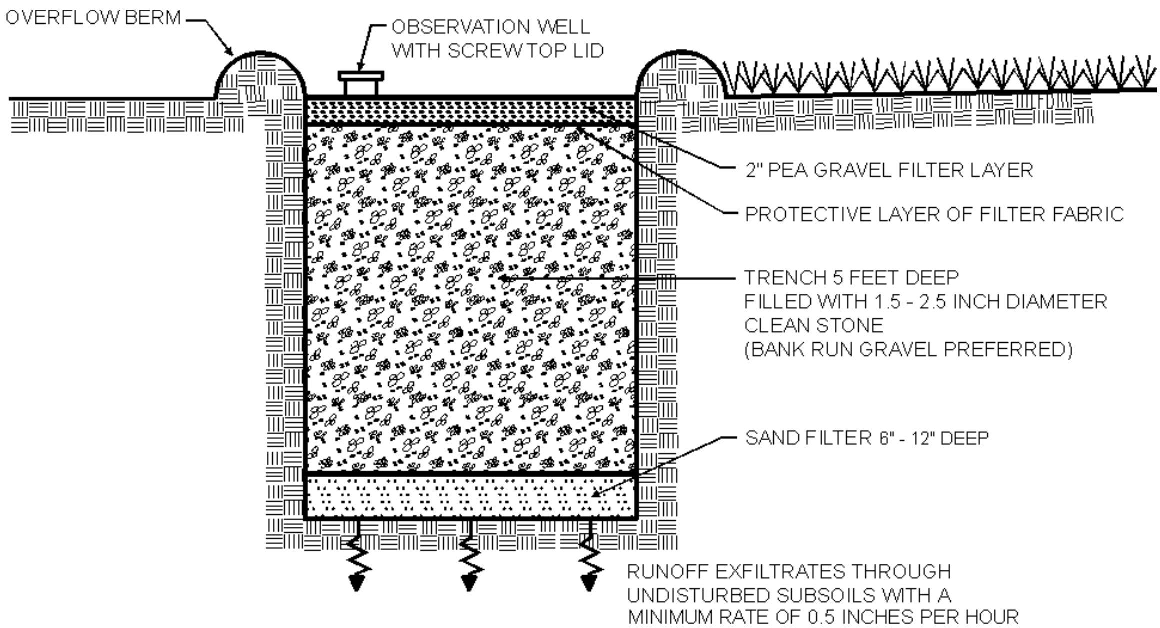


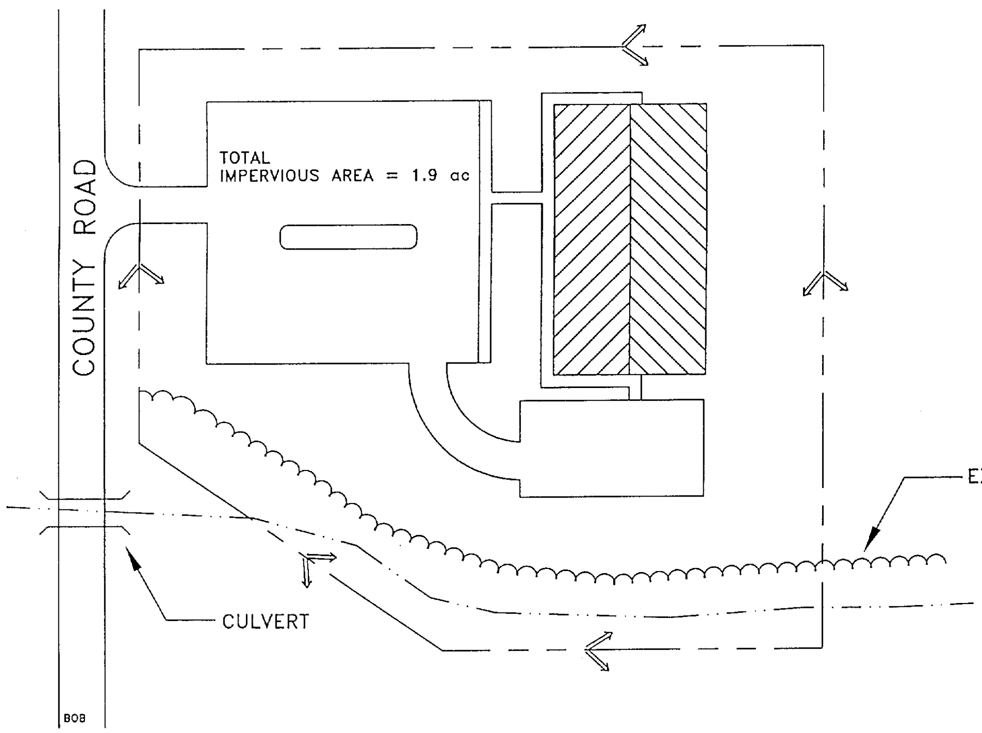
Figure 6. Infiltration Trench Cross Section

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.

APPENDIX D-5

Enhanced Swale Design Example



Base Data	Hydrologic Data		
Site Area = Total Drainage Area (A) = 3.0 ac		**Pre**	**Post**
Impervious Area = 1.9 ac; or I =1.9/3.0 = 63.3%	CN	70	88
Soils Type "C"	t_c	.39	.20

Figure 1. Etowah Recreation Center Site Plan

This example focuses on the design of a dry 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).

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

The layout of the Etowah Recreation Center is shown in Figure 1.

Two swales will be designed to carry flow to the existing stream, one around each side of the development.

Step 1 -- Compute runoff control volumes from the Unified Stormwater Sizing Criteria

Compute Water Quality Volume (WQ_v):

- Compute Runoff Coefficient, Rv

  $R_v = 0.05 + (63.3)(0.009) = 0.62$

- Compute WQ_v

  WQ_v = (1.2”) (R_v) (A) / 12
  = (1.2”) (0.62) (3.0ac) (43,560ft^2/ac) (1ft/12in)
  = 8,102 ft^3 = 0.19 ac-ft

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 2.1 to determine pre- and post-development peak discharges for the 1-yr, 25-yr, and 100-yr 24-hour return frequency storms.

- Per attached TR-55 calculations (Figures 2 and 3)

Condition	CN	$Q_{1\text{-year}}$	$Q_{1\text{-year}}$	$Q_{25\text{-year}}$	$Q_{100\text{ year}}$
		Inches	*cfs*	*cfs*	*cfs*
Pre-developed	70	0.9	2.3	9.0	12.0
Post-Developed	88	2.1	8.1	19.0	25.0

- Utilize modified TR-55 approach to compute channel protection storage volume

  Initial abstraction (Ia) for CN of 88 is 0.27: [Ia = (200/CN - 2)]

  Ia/P = (0.27)/ 3.4 inches = 0.08
  T_c = 0.20 hours
  q_u = 850 csm/in

  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

For a Type II rainfall distribution,

$Vs/Vr = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$
Where Vs equals channel protection storage (Cp_v) and Vr equals the volume of runoff in inches.

Vs/Vr = 0.65

Therefore, Vs = Cp_v = 0.65(2.1")(1/12)(3 ac) = 0.34 ac-ft = 14,810 ft^3

Determine Overbank Flood Protection Volume (Q_{p25}):

For a Q_{in} of 19 cfs, and an allowable Q_{out} of 9 cfs, the Vs necessary for 25-year control is 0.38 ac-ft or 16,553 ft^3, under a developed CN of 88. Note that 6.5 inches of rain fall during this event, with 5.1 inches of runoff.

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
WQ_v	Water Quality	8,102	
Cp_v	Channel Protection	14,810	
Q_{p25}	Overbank Flood Protection	16,553	
Q_f	Extreme Flood Protection	NA	Provide safe passage for the 100-year event in final design

PEAK DISCHARGE SUMMARY				
JOB:	Etowah Recreation Center			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)
woods (good cond.)		C	70	3.00 Ac.
		AREA SUBTOTALS:		3.00 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	dense grass	'n'=0.24	150 Ft.	1.50% 0.33 Hrs
Shallow Flow	unpaved		500 Ft. 2.28 F.P.S.	2.00% 0.06 Hrs.
Channel Flow				

Total Area in Acres =	3.00 Ac.	Total Sheet Flow=	Total Shallow Flow=	Total Channel Flow =
Weighted CN =	70			
Time Of Concentration =	0.39 Hrs.	0.33 Hrs.	0.06 Hrs.	0.00 Hrs.
Pond Factor =	1	RAINFALL TYPE II		

STORM	*Precipitation* (P) inches	*Runoff* (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	3.4 In.	0.9 In.	2.3 CFS	10,049 Cu. Ft
2 Year	4.1 In.	1.4 In.	3.5 CFS	15,064 Cu. Ft
5 Year	4.8 In.	1.9 In.	5 CFS	20,574 Cu. Ft
10 Year	5.5 In.	2.4 In.	7 CFS	26,459 Cu. Ft
25 Year	6.5 In.	3.2 In.	9 CFS	34,748 Cu. Ft
50 Year	7.2 In.	3.8 In.	10 CFS	41,221 Cu. Ft
100 Year	7.9 In.	4.4 In.	12 CFS	47,868 Cu. Ft

Figure 2. Etowah Recreation Center Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:	Etowah Recreation Center			EWB
DRAINAGE AREA NAME:	Post-Development 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 (good cond.)		C	74	0.50 Ac.
woods (good cond.)		C	70	0.60 Ac.
impervious		C	98	1.90 Ac.
		AREA SUBTOTALS:		3.00 Ac.
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 4.1 In	Cross Section	Wetted Per	Avg Velocity	Tt (Hrs)
Sheet Flow	dense grass	'n'=0.24	50 Ft.	1.50%
				0.14 Hrs
Shallow Flow	paved		600 Ft.	2.00%
			2.87 F.P.S.	0.06 Hrs.
Channel Flow		'n'=0.024	50 Ft.	2.00%
Hydraulic Radius =0.75	X-S estimated	WP estimated	7.25 F.P.S.	0.00 Hrs.
Total Area in Acres =	3.00 Ac.	Total Sheet	Total Shallow	Total Channel
Weighted CN =	88	Flow=	Flow=	Flow =
Time Of Concentration =	0.20 Hrs.	0.14 Hrs.	0.06 Hrs.	0.00 Hrs.
Pond Factor =	1	RAINFALL TYPE II		
STORM	Precipitation (P) inches	Runoff (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	3.4 In.	2.1 In.	8.1 CFS	23,320 Cu. Ft.
2 Year	4.1 In.	2.8 In.	10.6 CFS	30,527 Cu. Ft.
5 Year	4.8 In.	3.5 In.	13 CFS	37,890 Cu. Ft.
10 Year	5.5 In.	4.2 In.	16 CFS	45,356 Cu. Ft.
25 Year	6.5 In.	5.1 In.	19 CFS	55,422 Cu. Ft.
50 Year	7.2 In.	5.8 In.	22 CFS	63,030 Cu. Ft.
100 Year	7.9 In.	6.5 In.	25 CFS	70,676 Cu. Ft.

Figure 3. Etowah Recreation Center Post-Developed Conditions

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.

Step 3 -- Confirm local design criteria and applicability

There is a local requirement that the 25-year storm is contained within the top of banks of all channels, including these enhanced swale controls.

No additional local criteria are applicable.

Step 4 -- Determine pretreatment 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

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.

Step 5 -- Determine swale dimensions

Required: bottom width, depth, length, and slope necessary to store WQ_v with less than 18" of ponding (see Figure 5 for representative site plan).

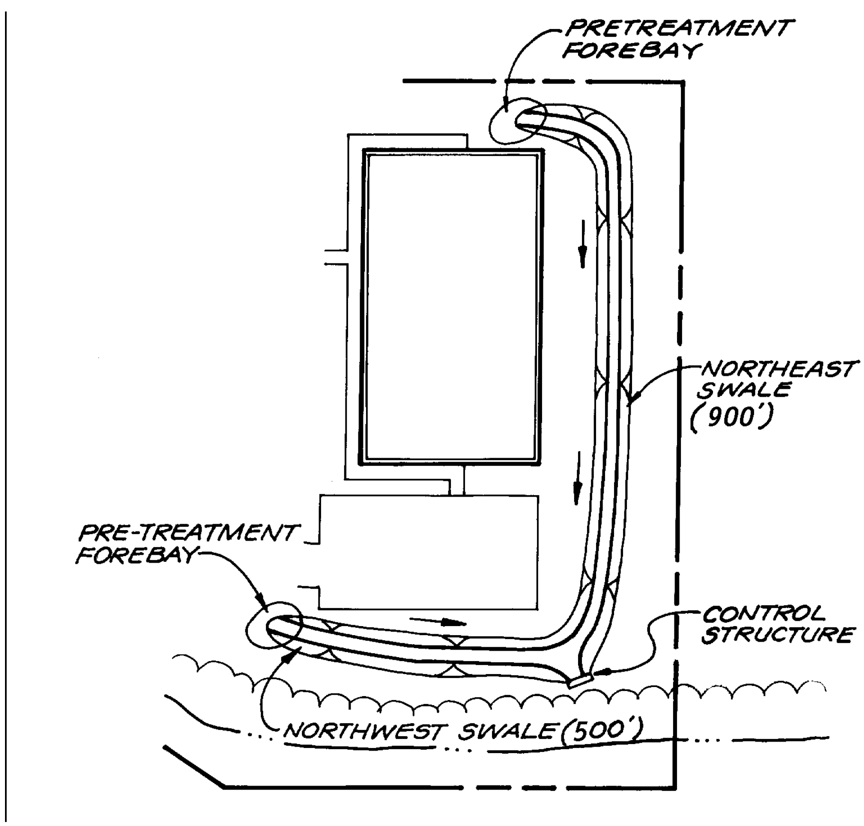


Figure 5. Enhanced Dry Swale Site Plan

Assume a trapezoidal channel with 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 5 and 6. 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 a trapezoidal section with a bottom width of 6', a WQ_v 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 7).

(6.2 sq ft) (1,400 ft) = 8,680cubic feet [> WQ_v of 8,102 ft^3; OK]

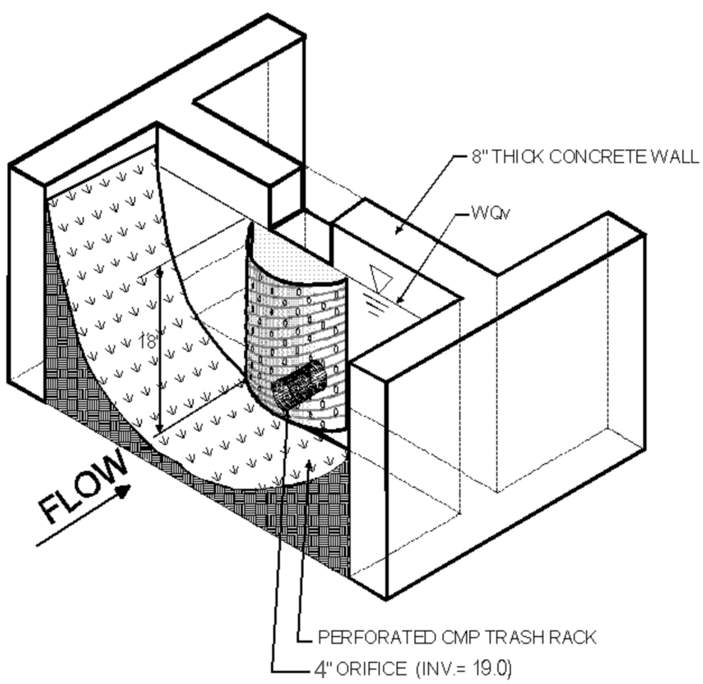


Figure 6 Control Structure at End of Swale

Step 6 -- Compute number of check dams (or similar structure) required to detain WQ_v (see Figure 7)

For the northwest fork, 500 ft @ 1.0% slope, and maximum depth at 18”, place checkdams at: 1.5’/0.01 = 150' <u>place at 150’, 4 required</u>

For the northeast fork, 900 ft @ 1.0% slope, and maximum 18" depth, place checkdams at 1.5’/0.01 = 150’ <u>place at 150’, 6 required</u>

Step 7 -- 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 (<u>k = 1.5’ per day</u>). Provide 6” perforated underdrain pipe and gravel system below soil bed (see Figure 7)

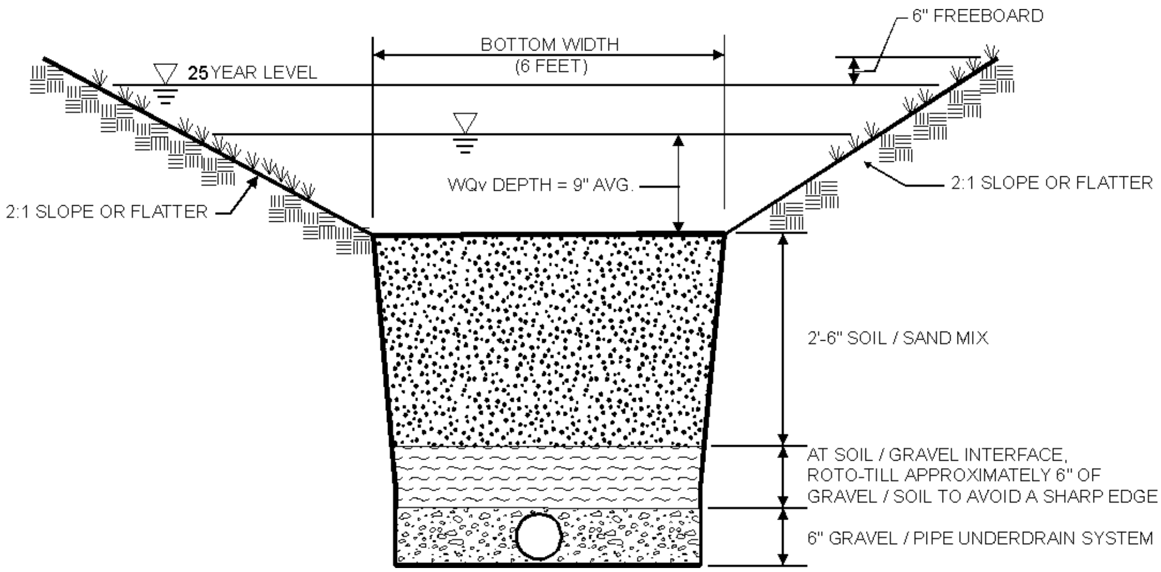


Figure 7 Trapezoidal Dry Swale Section

Step 8 -- 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 19.0 cfs, assume that 30% goes through northwestern swale (5.7 cfs) and 70% goes through the northeastern swale (13.3 cfs). Design for the larger amount (13.3 cfs). From separate computer analysis, with a slope of 1.0%, the 25-year velocity will be 3.3 feet-per-second at a depth of .65 feet, provide an additional .5’ of freeboard above top of checkdams or about 1.2' (total channel depth = 2.7’).

Find 25-year overflow weir length required: (weir eq. $Q = CLH^{3/2}$), where C = 3.1, Q_{25} = 19 cfs, H =1.2; Rearranging the equation yields:

$L = 19\ \text{cfs} / (3.1 * 1.2^{1.5}) = 4.7'$ <u>Use 5 ft</u>

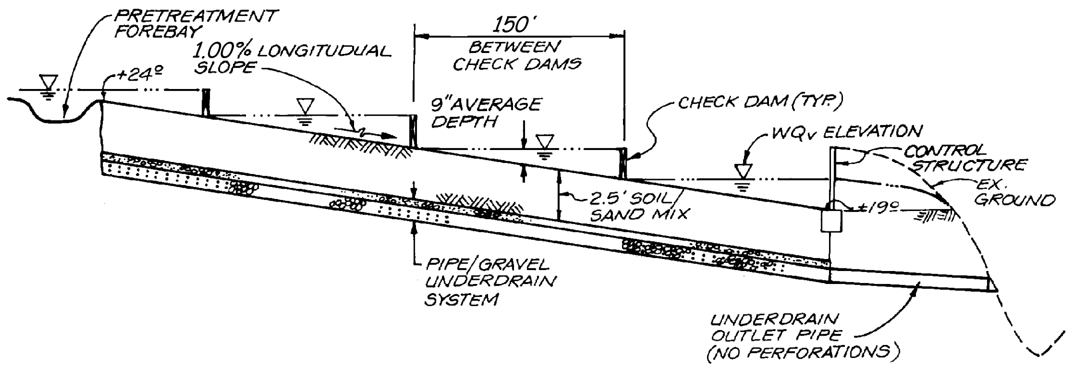


Figure 8 Profile of Northwest Fork Dry Swale

Step 9 -- Design low flow orifice at downstream headwall and checkdams (See Figure 6)

Design orifice to pass 8,102 cubic feet in 6 hours.

8,102 cubic feet/ [(6 hours) (3600 sec/hour)] = 0.4 cfs
Use Orifice equation: $Q = CA(2gh)^{1/2}$

Assume h = 1.5'

$A = (0.4 \text{ cfs}) / [(0.6) ((2) (32.2 \text{ ft/s}^2) (1.5'))^{1/2}]$

A = 0.068 sq ft, dia = 0.29 feet or 3.6" <u>use 4" orifice</u>

Provide 3" v-notch slot in each check dam

Step 10 – Design inlets, sediment forebay(s), and underdrain system

See Figure 8

Step 11 – Prepare Vegetation and Landscaping Plan

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APPENDIX

E

STRUCTURAL CONTROL MAINTENANCE CHECKLISTS

Table of Contents

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Operation and Maintenance Inspection Report for Stormwater Management Ponds

(Adapted from Watershed Management Institute, Inc.)

Inspector Name ___________________________ Project Location
Inspection Date ___________________________ _______________________________________
Stormwater Pond _______________________________________
Normal Pool ______ _______________________________________
Normally Dry ______ Watershed ____________________________

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
Pond Components				
1. Embankment and Emergency spillway				
a. Adequate vegetation and ground cover			A	
b. Embankment erosion			A	
c. Animal burrows			A	
d. Unauthorized plantings			A	
e. Cracking, bulging, or sliding of dam				
i. Upstream face			A	
ii. Downstream face			A	
iii. At or beyond toe Upstream			A	
Downstream			A	
iv. Emergency spillway			A	
f. Pond, toe & chimney drains clear and functioning			A	
g. Leaks on downstream face			A	
h. Abutment protection or riprap failures			A	
i. Visual settlement or horizontal misalignment of top of dam			A	
j. Emergency spillway clear of debris			A	
k. Other (specify)			A	
2. Riser and principal spillway				
Type: Reinforced concrete ______ Corrugated pipe ______ Masonry ______				
a. Low flow orifice obstructed			A	
b. Low flow trash rack i. Debris removal necessary			A	
ii. Corrosion control			A	
c. Weir trash rack i. Debris removal necessary			A	
ii. Corrosion control			A	

Inspection Items		Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
d.	Excessive sediment accumulation inside riser			**A**	
e.	Concrete/Masonry condition Riser and barrels i. Cracks or displacement			**A**	
	ii. Minor spalling (<1”)			**A**	
	iii. Major spalling (rebars exposed)			**A**	
	iv. Joint failures			**A**	
	v. Water tightness			**A**	
f.	Metal pipe condition			**A**	
g.	Control valve i. Operational/exercised			**A**	
	ii. Chained and locked			**A**	
h.	Pond drain valve			**A**	
	i. Operational/exercised			**A**	
	ii. Chained and locked			**A**	
i.	Outfall channels flowing			**A**	
j.	Other (specify)			**A**	
3.	Permanent pool (wet ponds)				
a.	Undesirable vegetative growth			**M**	
b.	Floating or floatable debris removal required			**M**	
c.	Visible pollution			**M**	
d.	High Water Marks			**M**	
e.	Shoreline problems			**M**	
f.	Other (specify)			**M**	
4.	Sediment forebays				
a.	Sedimentation Noted			**M**	
b.	Sediment removal when depth < 50% design depth			**M**	
5.	Dry pond areas				
a.	Vegetation adequate			**M**	
b.	Undesirable vegetative growth			**M**	
c.	Undesirable woody vegetation			**M**	
d.	Low flow channels clear of obstructions			**M**	
e.	Standing water or wet spots			**M**	
f.	Sediment and/or trash accumulation			**M**	
g.	Other (specify)			**M**	
6.	Condition of outfalls into pond				
a.	Riprap failures			**A,S**	
b.	Slope erosion			**A,S**	
c.	Storm drain pipes			**A,S**	
d.	Endwalls/headwalls			**A,S**	
e.	Other (specify)			**A,S**	

Inspection Items			Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
7.	Other					
	a.	Encroachments on ponds or easement area			**M**	
	b.	Complaints from residents (describe on back)			**M**	
	c.	Aesthetics i. Grass height			**M**	
		ii. Graffiti removal necessary			**M**	
		iii. Other (specify)			**M**	
	d.	Any public hazards (specify)			**M**	
	e.	Maintenance access			**M**	
8.	Constructed wetland areas					
	a.	Vegetation healthy and growing			**A**	
	b.	Evidence of invasive species			**A**	
	c.	Excessive sedimentation in wetland area			**A**	

Inspection Frequency Key A=Annual, M=Monthly, S=After major storm

Summary

1. Inspectors Remarks: ____
____
____
____
____
____
____

2. Overall condition of Facility (Check one)
   ______ Acceptable
   ______ Unacceptable

3. Dates any maintenance must be completed by:
____
____
____

____
Inspectors Signature

Operation and Maintenance Inspection Report for Filtration Facility

(Adapted from Watershed Management Institute, Inc.)

Inspector Name ______________________ Project Location
Inspection Date ______________________ ______________________
Watershed ______________________ ______________________
As-built Plans available? ____________ ______________________

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
1. Debris removal				
Adjacent area clear of debris			M	
Inlets and outlets clear of debris			M	
Filtration facility free of debris			M	
2. Vegetation				
Adjacent area stabilized			M	
Grass mowed			M	
Any evidence of erosion			M	
3. Oil and grease				
Any evidence of filter clogging			M	
4. Water retention where required				
Water holding chambers at normal pool			M	
No evidence of leakage			M	
5. Sediment deposition				
Filtration chamber clean of sediments			A	
Water chambers not more than ½ full of sediments			A	
6. Structural components				
Any evidence of structural deterioration			A	
Grates in good condition			A	
Any evidence of spalling or cracking of structural parts			A	
7. Outlets/overflow spillway				
Good condition (no need for repair)			A	
Any evidence of erosion			A	
8. Overall function of facility				
Any evidence of flow bypassing facility			A	
Any noticeable odors outside of facility			A	

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
9. Pump (Where applicable) Catalog cuts and wiring diagram for pump available			A	
Waterproof conduits for wiring appear to be intact			A	
Panel box is well marked			A	
Any evidence of pump failure (excess water in pump well, etc.)			A	

Inspection Frequency Key A=Annual, M=Monthly, S=After major storm

Necessary Action:

If any of the items above where answered Yes for "Maintenance Needed", a time frame needs to be established for repair or correction.

No action necessary. Continue routine inspections.
Correct noted facility deficiencies by (date) _________________________

Facility repairs were previously indicated and completed. Site reinspection is necessary to verify corrections or improvements.

Site reinspection completed on (date)_______________________________

Site reinspection was satisfactory.
Next routine inspection is scheduled for approximately (date):_______________________

____
Inspectors Signature

Operation and Maintenance Inspection Report for Infiltration Trenches

(Adapted from Watershed Management Institute, Inc.)

Inspector Name ______________________ Project Location

Inspection Date ______________________ ______________________

Watershed ______________________ ______________________

As-built Plans available? ____________ ______________________

	Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
1.	Debris removal				
	Trench surface clear of debris			**M**	
	Inlets clear of debris			**M**	
	Inflow pipes clear of debris			**M**	
	Overflow spillway clear of debris			**M**	
2.	Sediment traps, forebays, or pretreatment swales				
	Obviously trapping sediment			**A**	
	Greater than 50% of original storage volume remaining			**A**	
3.	Vegetation				
	Mowing done when necessary			**M**	
	Fertilized per specification			**M**	
	Any evidence of erosion			**M**	
	Contributing drainage area stabilized			**M**	
4.	Dewatering				
	Trench dewaters between storms			**M**	
5.	Sediment removal of trench				
	Any evidence of sedimentation in trench			**A**	
	Does sediment accumulation currently require removal			**A**	
6.	Inlets				
	Good condition			**A**	
	Any evidence of erosion			**A**	
7.	Outlets/overflow spillway				
	Good condition (no need for repair)			**A**	
	Any evidence of erosion			**A**	
8.	Aggregate repairs				
	Surface of aggregate clean			**A**	
	Top layer of stone in need of replacement			**A**	
	Trench in need of rehabilitation			**A**	

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
9. Vegetated surface Evidence of erosion present			**M**	
Perforated inlet functioning adequately			**M**	
Does water stand on vegetated surface			**M**	
Does good vegetative cover exist			**M**	
10. Overall function of facility Any evidence of flow bypassing facility			**S**	

Inspection Frequency Key A=Annual, M=Monthly, S=After major storm

Necessary Action:

If any of the items above where answered Yes for "Maintenance Needed", a time frame needs to be established for repair or correction.

No action necessary. Continue routine inspections.
Correct noted facility deficiencies by (date) ________________________

Facility repairs were previously indicated and completed. Site reinspection is necessary to verify corrections or improvements.

Site reinspection completed on (date)______________________________

Site reinspection was satisfactory.
Next routine inspection is scheduled for approximately (date):______________________

____
Inspectors Signature

Operation and Maintenance Inspection Report for Enhanced Swales / Grass Channels / Filter Strips

(Adapted from Watershed Management Institute, Inc.)

Inspector Name ____________________________ Project Location

Inspection Date ____________________________ ____

Watershed ____________________________ ____

As-built Plans available? ________________ ____

Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
1. Debris removal				
Facility and adjacent area clear of debris			**M**	
Inlets and outlets clear of debris			**M**	
Any dumping of yard wastes into facility			**M**	
Has litter (branches, etc.) been removed			**M**	
2. Vegetation				
Adjacent area stabilized			**M**	
Grass mowed			**M**	
Plant height not less than design water depth			**M**	
Fertilized per specifications			**M**	
Any evidence of erosion			**M**	
Is plant composition according to approved plans			**M**	
Any unauthorized or inappropriate plantings			**M**	
Any dead or diseased plants			**M**	
Any evidence of plant stress from inadequate watering			**M**	
Any evidence of deficient stakes or wires			**M**	
3. Oil and grease				
Any evidence of filter clogging			**M**	
4. Dewatering				
Facility dewaters between storms			**M**	
5. Check dams/energy dissipators/sumps				
Any evidence of sedimentation buildup			**A,S**	
Are sumps greater than 50% full of sediment			**A,S**	
Any evidence of erosion at downstream toe of drop structures			**A,S**	

	Inspection Items	Checked? Yes / No	Maintenance Needed? Yes / No	Inspection Frequency	Comments
6.	Sediment deposition				
	Swale clean of sediments			**A**	
	Sediments should not be > than 20% of swale design depth			**A**	
7.	Outlets/overflow spillway				
	Good condition (no need for repair)			**A,S**	
	Any evidence of erosion			**A,S**	
	Any evidence of blockages			**A,S**	
8.	Integrity of facility				
	Has facility been blocked or filled inappropriately			**A**	
9.	Bioretention Planting Soil				
	Any evidence of planting soil erosion			**A**	
10.	Organic Layer				
	Mulch covers entire area (NO voids) and to specified thickness			**A**	
	Mulch is in good condition			**A**	

Inspection Frequency Key A=Annual, M=Monthly, S=After major storm

Necessary Action:

If any of the items above where answered Yes for "Maintenance Needed", a time frame needs to be established for repair or correction.

No action necessary. Continue routine inspections.
Correct noted facility deficiencies by (date) ________________________

Facility repairs were previously indicated and completed. Site reinspection is necessary to verify corrections or improvements.

Site reinspection completed on (date)______________________________

Site reinspection was satisfactory.
Next routine inspection is scheduled for approximately (date):______________________

____
Inspectors Signature

APPENDIX

F

LANDSCAPING AND AESTHETICS GUIDANCE

Introduction

Landscaping is a critical element in the design of stormwater facilities for water quantity and quality management, serving both functional and aesthetic purposes. Plants and vegetation perform a number of functions in stormwater controls and conveyance facilities, including:

- Slowing and retarding flow by increasing hydraulic roughness
- Preventing the erosion of bare soil
- Enhancing infiltration of runoff into the soil
- Providing pollutant removal through vegetative uptake
- Preventing access to deep open water areas
- Contributing to wildlife and fish habitat
- Improving the overall appearance of stormwater facilities

The purpose of this Appendix is to provide guidance on landscaping and plant selection for stormwater facilities and structural controls, as well as provide an overview on developing aesthetically-pleasing stormwater facilities. This appendix is divided into the following sections:

- ❑ Subsection F.1 covers general landscaping guidance that should be considered when landscaping any stormwater facility.

- ❑ Subsection F.2 discusses the physical site factors and considerations involved in selecting plant material for stormwater facility landscaping.

- ❑ Subsection F.3 includes key factors to consider in selecting plant material for stormwater landscaping are reviewed, including hardiness, physiographic regions, inundation tolerance, and other factors.

- ❑ Subsection F.4 outlines more specific guidance on landscaping criteria and plant selection for individual structural stormwater control designs, including:

    - Stormwater Ponds and Wetlands
    - Bioretention Areas
    - Infiltration Trench and Surface Sand Filter Facilities
    - Enhanced Swales and Grass Channels
    - Filter Strips and Stream Buffers

- ❑ Subsection F.5 contains a detailed plant list of trees and shrubs that may be used when preparing a vegetation and landscaping planting plan for a stormwater facility.

- ❑ Subsection F.6 provides examples of aesthetics and good landscaping in structural control design.

F.1 General Landscaping Guidance

Below are general guidelines that should be followed in the landscaping of any stormwater control or conveyance facility.

DO NOT:

- Plant trees, scrubs or any type of woody vegetation on an embankment
- Plant trees and shrubs within 15 feet of the toe of slope of a dam.
- Plant trees or shrubs known to have long tap roots within the vicinity of the earthen dam or embankment, or subsurface drainage facilities.
- Plant trees and shrubs within 25 feet of a principal spillway structure (e.g., riser)
- Plant trees and shrubs within 25 feet of perforated pipes.
- Block maintenance access to structures with trees or shrubs.

DO:

- Take into account site characteristics and plant selection guidelines (see subsections F.2 and F.3, respectively) when selecting plants for stormwater facilities.
- Consider how plant characteristics will affect the landscape and the performance of a structural stormwater control or conveyance.
- Carefully consider the long-term vegetation management strategy for the structural control, keeping in mind the maintenance legacy for the future owners.
- Preserve existing natural vegetation when possible.
- Avoid the overuse of any plant materials.
- Have soils tested to determine if there is a need for amendments.
- Select plants that can thrive in on-site soils with no additional amendments or a minimum of amendments.
- Consider water availability, particularly for wetland and water-intensive plantings.
- Decrease the areas where turf is used. Use low maintenance ground cover to absorb run-off.
- Plant stream and edge of water buffers with trees, shrubs, ornamental grasses, and herbaceous materials where possible, to stabilize banks and provide shade.
- Provide slope stabilization methods for slopes steeper than 2:1, such as planted erosion control mats. Also, use seed mixes with quick germination rates in this area. Augment temporary seeding measures with container crowns or root mats of more permanent plant material.
- Utilize erosion control mats and fabrics to protect in channels that are subject to frequent wash outs.
- Stabilize all water overflows with plant material that can withstand strong current flows. Root material should be fibrous and substantial but lacking a tap root.
- Sod area channels that are not stabilized using erosion control mats.
- Divert flows temporarily from seeded areas until stabilized.
- Check water tolerances of existing plant materials prior to inundation of area.
- Stabilize aquatic and safety benches with emergent wetland plants and wet seed mixes.
- Provide a 15-foot clearance from a non-clogging, low flow orifice.

- Limit herbaceous embankment plantings to 10 inches in height, to allow visibility for the inspector who is looking for burrowing rodents that may compromise the integrity of the embankment.
- Shade inflow and outflow channels, as well as the southern exposures of pond, to reduce thermal warming
- Avoid plantings that will require routine or intensive chemical applications (i.e. turf area).
- Maintain and frame desirable views. Be careful not to block views at entrances, exits, or difficult road curves. Screen or buffer unattractive views into the site.
- Use plants to prohibit pedestrian access to pools or slopes that may be unsafe.
- Keep maintenance area open to allow future access for pond maintenance.
- Provide a planting surface that can withstand the compaction of vehicles using maintenance access roads.
- Make sure the facility maintenance agreement includes a maintenance requirement of designated plant material.
- Provide signage for:
  - Stormwater management facilities to help educate the public
  - Wildflower areas to designate limits of mowing
  - Preserving existing natural vegetation

F.2 Site Considerations

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

Soil Characteristics

Plant establishment and growth can be limited by a number of different soil characteristics including:

- Soil texture
- PH -- whether acid, neutral, or alkali
- Nutrient levels -- nitrogen, phosphorus, potassium
- Minerals -- such as chelated iron, lime
- Salinity
- Toxicity

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 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 aggregation of small soil particles into larger particles. The amount of aggregation in a soil is strongly influenced by the amount of organic matter present.

Soil samples should be analyzed by experienced and qualified individuals who can explain the results and provide information on any soil amendments that are required. Soil fertility can often be corrected by applying fertilizer or by increasing the level of organic matter in the soil. Soil pH can be corrected with applications of lime. Where poor soils can't be amended, seed mixes and plant material must be selected to establish ground cover as quickly as possible.

Areas that have recently been involved in construction 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 two inches, preferably to a four-inch depth. Hard soils may require discing to 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. If the area is to be sodded, discing will allow the roots to penetrate into the soil.

Whenever possible, topsoil should be spread to a depth of four inches (two inch minimum) over the entire area to be planted. This provides organic matter and important nutrients for the plant material. This also allows the stabilizing 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. 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.

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.

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.

Orientation

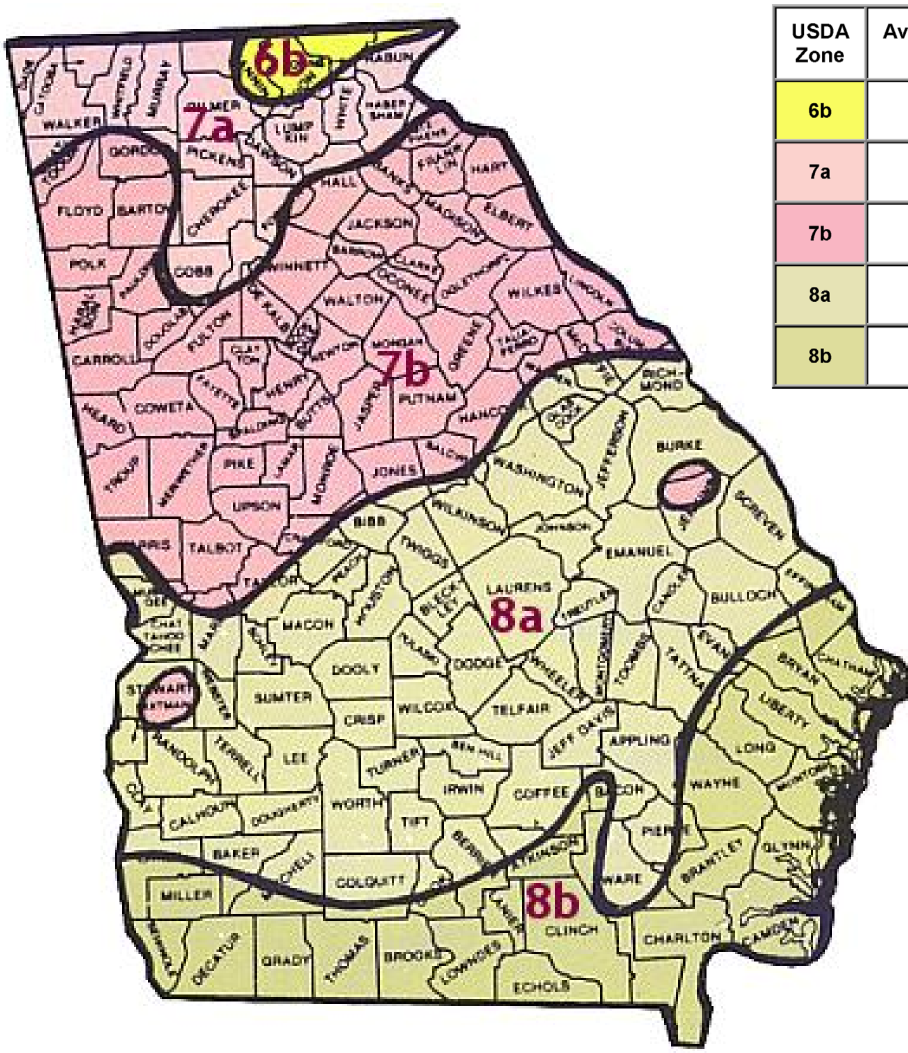
Slope exposure should be considered for its effect on plants. A southern-facing slope receives more sun and is warmer and drier, while the opposite is true of a northern slope. Eastern- and western-facing slopes are intermediate, receiving morning and afternoon sun, respectively. Western-facing slopes tending to receive more wind.

F.3 Plant Selection for Stormwater Facilities

F.3.1 Hardiness Zones

Hardiness zones are based on historical 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 (-17.8 to -20.5 C)
7a	0 to 5 F (-15.0 to -17.7 C)
7b	5 to 10 F (-12.3 to -14.9 C)
8a	10 to 15 F (-9.5 to -12.2 C)
8b	15 to 20 F (-6.7 to -9.4 C)

Figure F-1 USDA Plant Hardiness Zones in Georgia

F.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, a successful stormwater management facility planting design can be achieved through mimicking these natural associations. The five physiographic regions are described below with associated vegetation listed as general planting guidance.

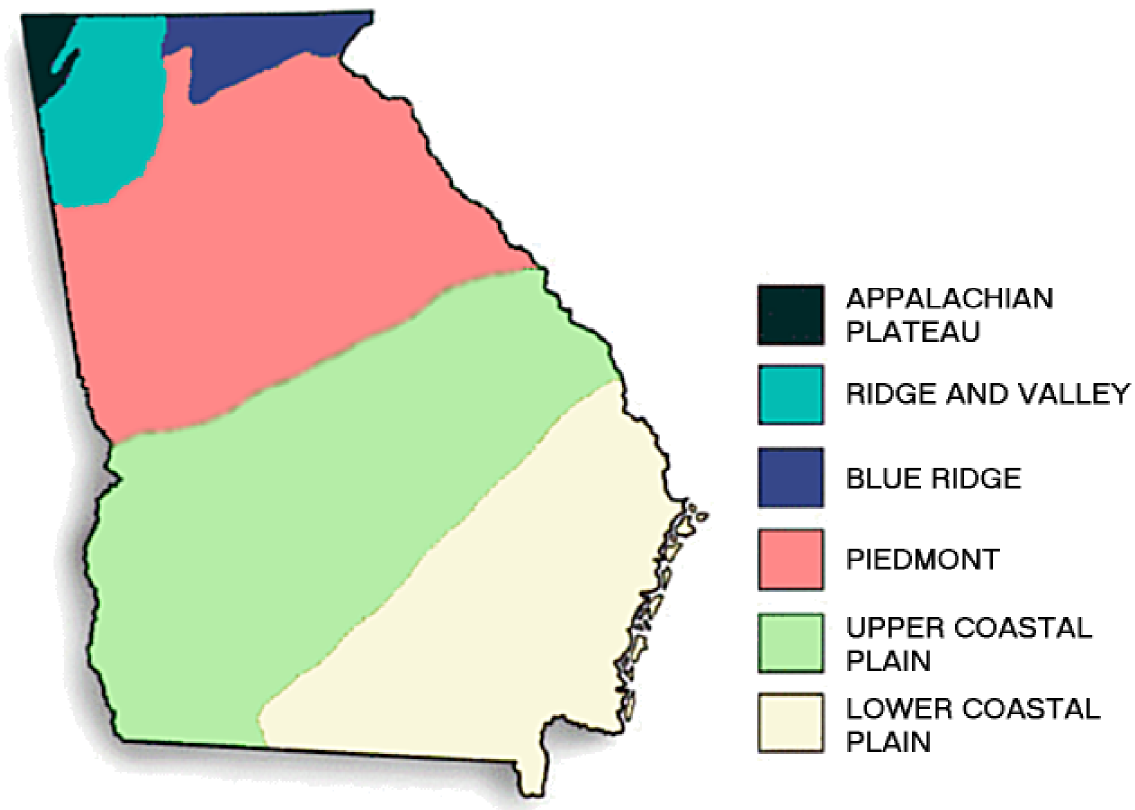


Figure F-2 Physiographic Provinces of Georgia
(Adapted from: Georgia Wildlife Web)

Coastal Plain – The Georgia Coastal Plain province is a low, flat region of well-drained, gently rolling hills and poorly drained flatwoods. The Coastal Plain extends east and south of the Fall Line Hills, the old Mesozoic shoreline still marked by a line of sand hills. Its soils, sands, and sandy clays are of marine origin and are usually acidic. They possess a low native fertility due to excessive leaching. Its elevation ranges from sea level to 225 m (750 ft). The Coastal Plain is sometimes divided into upper and lower sections, the upper section being near the Fall Line and the lower section being the mainland along the Atlantic coast.

On well-drained soils of the Coastal Plain, the dominant plant species are Long-leaf Pine, Loblolly Pine, and several species of oak. On poorly drained soils, the dominant species are Long-leaf Pine and Slash Pine with a dense ground cover of Saw Palmetto, Gallberry, and Wire-grass.

These plants are adapted to a humid subtropical climate of mild winters, hot summers, high rainfall, and frequent ground fires. Where the soil is poorly drained, Pond Pines are dominate. The Southern Mixed Hardwood community includes oaks, Sweet Gum, magnolias, Red Bay, and Pignut Hickory. Such hardwood communities are found bordering freshwater streams and floodplain swamps and in low, fertile areas near the coast. Wooded swamps composed of Cypress, Tupelo, and Red Maple trees are found adjacent to swamps, ponds, and lakes as well as along sluggish, meandering streams. The major plant communities on the Barrier Islands are maritime oak forests and pine forests. Major cities and urban areas in the Coastal Plain include Albany, Savannah and Valdosta. Columbus, Macon and Augusta all straddle the Fall Line between the Piedmont and Coastal Plain.

Piedmont – The Piedmont province contains a series of rolling hills and occasional isolated mountains. Rivers and ravines are found throughout this province. This is an area of oak-hickory-pine forests and mixed deciduous forests. Oak-hickory-pine forests are the most widespread type of forest in the southeastern United States. The dominant trees include oaks, hickories, Short-leaf Pine, and Loblolly Pine. Pines occur in the less favorable or disturbed areas of the Piedmont. In river valleys, mixed deciduous forests of hardwood trees such as Sweet Gum, Beech, Red Maple, elms, and birches are found.

Common understory species in the Piedmont include the sweet fern, flowering dogwood, sassafras, blueberry, pink azalea, hydrangea, spicebush and arrowwood. The Atlanta metro area and Athens are both located in the Piedmont province.

Blue Ridge – The Blue Ridge province occupies the northeastern portion of Georgia. It consists of an irregular sequence of mountains, ridges, and basins. Elevations reach 480 - 1,410 m (1,600 - 4,700 ft). The Blue Ridge Mountains and Cohutta Mountains form most of this province, with the McCaysville Basin separating them. Portions of the Piedmont Province extend into this province as well. Distinctly different elevations result in considerable variety in vegetation.

Ridge and Valley – The Ridge and Valley province occupies most of northwestern portion of Georgia. It includes the Chickamauga Valley, Armuchee Ridges, and the Great Valley. These form a series of parallel valleys separated by ridges in the northwest corner of the state. Lowland areas are about 210 - 240 m (700 - 800 ft) above sea level, but the higher ridges may be above 480 m (1,600 ft). Plant species vary from area to area, based on local soil type, elevation, moisture, and disturbances. Major cities in the Ridge and Valley province include Rome, Dalton and metro Chattanooga.

Appalachian Plateau – This mountainous province is found in extreme northwestern Georgia. Its most prominent features are Lookout and Sand Mountains. A variety of vegetation types occur in this area, depending on elevation, but Appalachian Oak Forests cover most of the Province. For example, forests on north-facing ravines between 800 and 1,200 m (2,640 - 3,960 ft) include Basswood, Sugar Maple, Tulip Poplar, Beech, Birch, and Hemlock trees. More northern species of evergreens and shrubs appear in the forests above 1,200 m (3,960 ft). The understory may include rhododendrons, native azaleas, and Mountain Laurel.

Floodplain Plant Communities – Floodplain areas are a microclimatic area that results in a characteristic plant community that is similar in all five physiographic provinces. Floodplain plant communities are an important reference community since many stormwater practices are located with this area. Floodplains occur along streams in both steep and level areas. The most noteworthy plants found along floodplains are River Birch, Willows, Poplars, Silver Maple, Sweet Gum (Coastal Plain and Piedmont); Sycamore, Box Elder, Green Ash, American Elm, Swamp White Oak, Bur Oak (Piedmont); and Honeylocust and Hackberry. Shrubs commonly found in floodplains include Shrub Willows, Ninebark, Silkey Cornel, Buttonbush, Spicebush, Black Alder, Winterberry, Black Elderberry, and Alders.

F.3.3 Other Considerations in Plant Selection

Use or Function

In selecting plants, consideration must be given to their desired function in the stormwater management facility. Is the plant needed as ground cover, soil stabilizer, biofilter or source of shade? Will the plant be placed for functional or aesthetic purposes? Does the adjacent use provide conflicts or potential problems and require a barrier, screen, or buffer? Nearly every plant and plant location should be provided to serve some function in addition to any aesthetic appeal.

Plant Characteristics

Certain plant characteristics are so obvious, they may actually be overlooked in the plant selection. These are:

- Size
- Shape

For example, tree limbs, after several years, can grow into power lines. A wide growing shrub may block maintenance access to a stormwater facility. Consider how these characteristics can work for you or against you, today and in the future.

Other plant characteristics must be considered to determine how the plant grows and functions seasonally, and whether the plant will meet the needs of the facility today and in the future. Some of these characteristics are:

- Growth Rate
- Regeneration Capacity
- Maintenance Requirements (e.g. mowing, harvesting, leaf collection, etc.)
- Aesthetics

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 facility can result in maintained or increased values of a property.

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.

Native versus Nonnative Species

This Manual encourages the use of native plants in stormwater management facilities, since they are best suited to thrive under the physiographic and hardiness conditions encountered at a site. Unfortunately, not all native plants provide the desired landscape or appearance, and may not always be available in quantity from local nurseries. Therefore, naturalized plants that are not native species, but can thrive and reproduce in the new area may be a useful alternative.

Because all landscaping needs may not be met by native or naturalized plants, some ornamental and exotic species are provided in this guide that can survive under difficult conditions encountered in a stormwater management facility. Since many stormwater facilities are adjacent to residential areas, the objectives of the stormwater planting plan may shift to resemble the more controlled appearance of nearby yards, or to provide a pleasing view. 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.

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 within the facility.

The six zones shown in Table F-1 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.

F.4 Specific Landscaping Criteria for Structural Stormwater Controls

F.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 F-1 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 F-1 Hydrologic 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, 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.

Table F-2 provides a list of selected wetland plants for Georgia stormwater ponds and wetlands for hydrologic zones 1-4

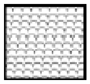
Table F-2 Wetland Plants (Herbaceous Species) for Stormwater Facilities

Scientific Name	Common Name	Hydrologic Zone
Acorus calumus	Sweetflag	2
Andropogon glomeratus	Bushy Broom Grass	3
Andropogon virginicus	Broom Grass	4
Canna flaccida	Golden Canna	2
Carex spp.	Caric Sedges	2
Chasmanthium latifolium	Upland Sea-Oats	3
Coreopsis leavenworthii	Tickseed	2
Coreopsis tinctoria	Dwarf Tickseed	3
Crinum americanum	Swamp Lily	2
Cyperus odoratus	Flat Sedge	2
Eleocharis cellulosa	Coastal Spikerush	2
Eleocharis interstincta	Jonited Spikerush	2
Eupatorium fistolosum	Joe Pye Weed	4
Helianthus angustifolius	Swamp Sunflower	2
Hibiscus coccinieus	Swamp Hibiscus	2
Iris louisiana	Louisiana Iris	2
Iris virginica	Southern Blue-Flag	2
Juncus effusus	Soft Rush	2
Leersia oryzoides	Rice Cut Grass	2

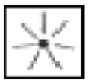
Table F-2 continued

Scientific Name	Common Name	Zone
Liatris spicata	Spiked Gayfeather	3
Lobelia cardinalis	Cardinal Flower	3
Nuphar luteum	Spatterdock	1
Nymphaea mexicana	Yellow Water Lily	1
Nymphaea odorata	Fragrant Water Lily	1
Osmunda cinnamomea	Cinnamon Fern	3
Osmunda regalis	Royal Fern	3
Panicum virgatum	Switchgrass	2
Peltandra virginica	Green Arum	2
Polygonum hydropiperoides	Smartweed	2
Pontederia cordata	Pickerelweed	2
Pontederia lanceolata	Pickerelweed	2
Rudbeckia hirta	Black-eyed Susan	4
Sagittaria lancifolia	Lance-leaf Arrowhead	2
Sagittaria latifolia	Duck Potato	2
Saururus cernuus	Lizard's Tail	2
Scirpus americanus	Three-square	2
Scirpus californicus	Giant Bulrush	2
Scirpus validus	Softstem Bulrush	2
Sorgham nutans	Yellow Indian Grass	4
Thalia geniculata	Alligator Flag	2
Typha spp.	Cat-tail	2
Vernonia gigantea	Ironweed	4
Woodwardia virginica	Virginia Chain Fern	2

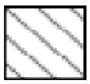
Source: Aquascape, Inc.



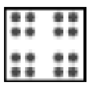
12 to 36 inch depth below normal pool elevation
Water Lily, Deep Water Duck Potato, Spatterdock, Wild Celery, Redhead Grass



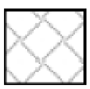
0 to 12 inch depth below normal pool elevation
Arrowhead/Duck Potato, Soft Rush, various Sedges, Softstem Bulrush, Cattail, Switchgrass, Southern Blue Flag Iris, Swamp Hibiscus, Swamp Lily, Pickerelweed, Pond Cypress, various Asters



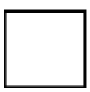
0 to 12 inch elevation above normal pool elevation
Various species from above, Broom Grass, Upland Sea-Oats, Dwarf Tickseed, various Ferns, Hawthorns, Boxelder, Ash, Willow, Red Maple, Willow Oak



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 Q_{p25} 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 F-3 Legend of Hydrologic Zones Around Stormwater Facilities

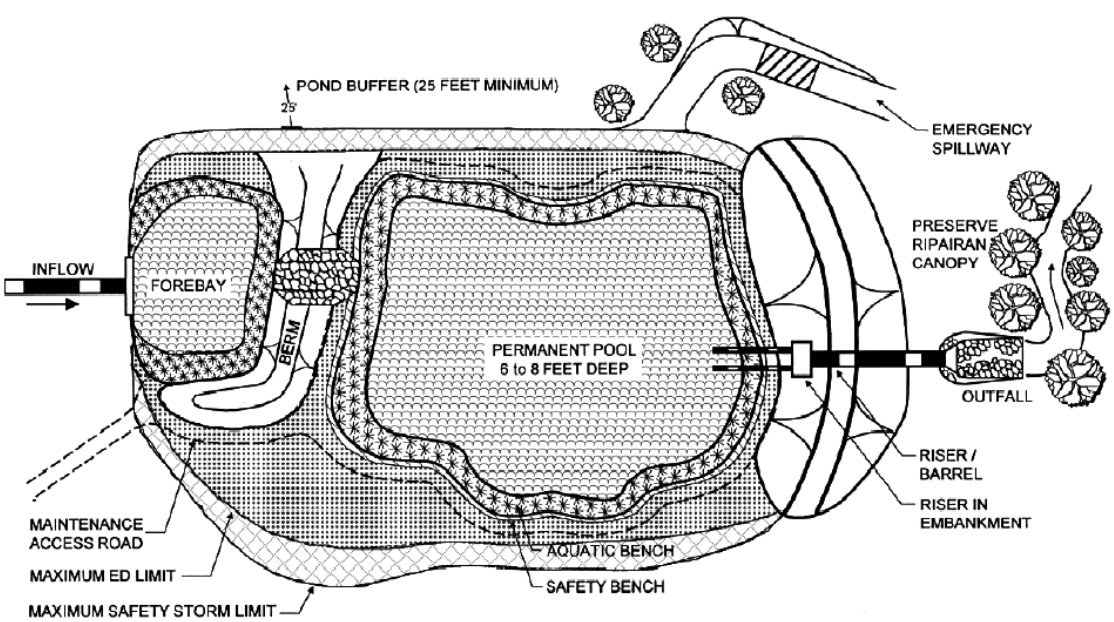


Figure F-4 Plan View of Hydrologic Zones around Stormwater Wet ED Pond

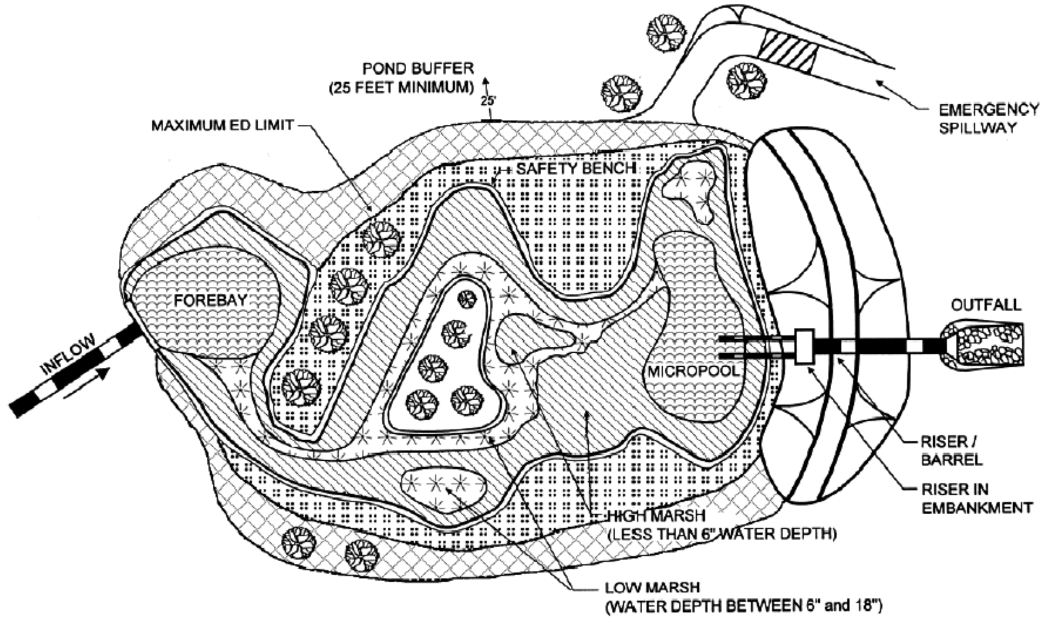


Figure F-5 Plan View of Hydrologic Zones around Stormwater ED Shallow Wetland

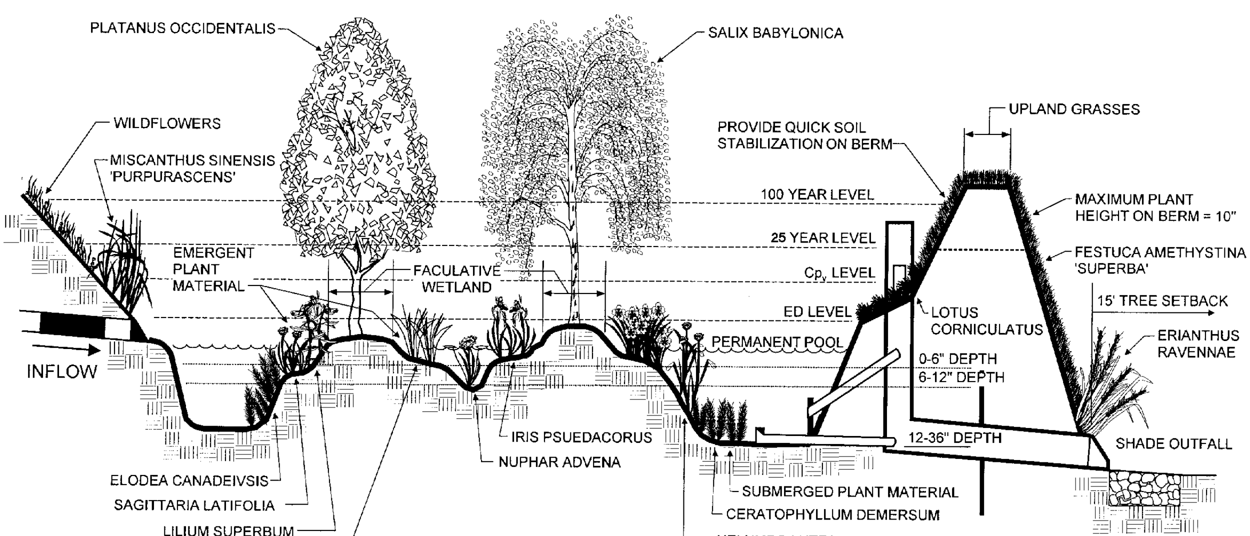


Figure F-6 Section of Typical Shallow ED Wetland

F.4.2 Bioretention Areas

Bioretention areas are structural stormwater controls that capture 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.

Planting Soil Bed Characteristics

The characteristics of the soil for the bioretention facility are perhaps as important as the facility location and size. The soil must be permeable enough to allow runoff to filter through the media, while having characteristics suitable to promote and sustain a robust vegetative cover crop. In addition, much of the nutrient pollutant uptake (nitrogen and phosphorus) is accomplished through adsorption and microbial activity within the soil profile. Therefore, the soils must balance soil chemistry and physical properties to support biotic communities above and below ground.

The planting soil should be a sandy loam, loamy sand, loam, or a loam/sand mix (should contain a minimum 35 to 60% sand, by volume). The clay content for these soils should by less than 25% by volume. Soils should fall within the SM, ML, SC classifications or the Unified Soil Classification System (USCS). A permeability of at least 1.0 feet per day (0.5"/hr) is required (a conservative value of 0.5 feet per day should be used for design). The soil should be free of stones, stumps, roots, or other woody material over 1" in diameter. Brush or seeds from noxious weeds, such as Johnson Grass, Mugwort, Nutsedge, and Canadian Thistle should not be present in the soils. Placement of the planting soil should be in lifts of 12 to 18", loosely compacted (tamped lightly with a dozer or backhoe bucket). The specific characteristics are presented in Table F-3.

Table F-3 Planting Soil Characteristics

Parameter	Value
pH range	5.2 to 7.00
Organic matter	1.5 to 4.0%
Magnesium	35 lbs. per acre, minimum
Phosphorus (P_2O_5)	75 lbs. per acre, minimum
Potassium (K_2O)	85lbs. per acre, minimum
Soluble salts	500 ppm
Clay	10 to 25%
Silt	30 to 55%
Sand	35 to 60%

(Adapted from EQR, 1996; ETAB, 1993)

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.

Planting Plan Guidance

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 F-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 are included in Table F-4. More potential bioretention species can be found in the wetland plant list in subsection F.5.

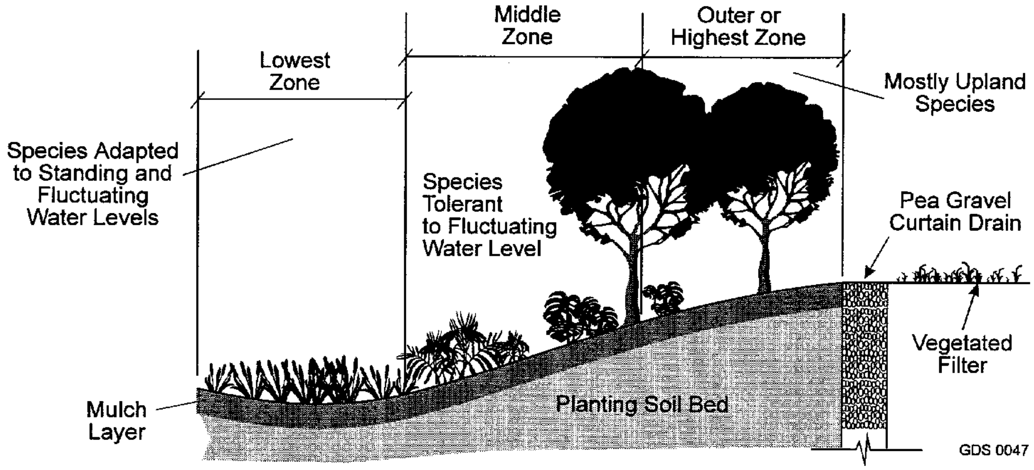


Figure F-7 Planting Zones for Bioretention Facilities

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.

- Native plant species should be specified over exotic or foreign species.
- 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. On average, the trees should be spaced 8 feet apart.
- Plants should be placed at regular intervals 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.
- Trees should be planted primarily along the perimeter of the bioretention area.
- 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.

Plant materials should conform to the American Standard Nursery Stock, published by the American Association of Nurserymen, and should be selected from certified, reputable nurseries. Planting specifications should be prepared by the designer and should include a sequence of construction, a description of the contractor's responsibilities, a planting schedule and installation specifications, initial maintenance, and a warranty period and expectations of plant survival. Table F-5 presents some typical issues for planting specifications. Figure F-8 shows an example of a sample planting plan for a bioretention area.

Table F-4 Commonly Used Species for Bioretention Areas

Trees	Shrubs	Herbaceous Species
Acer rubrum Red Maple	*Aesculus pariviflora* Bottlebrush Buckeye	*Andropogon virginicus* Broomsedge
Betula nigra River Birch	*Aronia arbutifolia* Red Chokeberry	*Eupatorium perpurea* Joe Pye Weed
Juniperus virginiana Eastern Red Cedar	*Fothergilla gardenii* Fothergilla	*Hemerocalis spp.* Day Lily
Koelreuteria paniculata Golden Rain Tree	*Hamemelis virginiana* Witch Hazel	*Iris pseudacorus* Yellow Iris
Nyssa sylvatica Black Gum	*Hypericum densiflorum* Common St. Johns Wort	*Lobelia cardinalis* Cardinal Flower
Platanus acerifolia London Plane-Tree	*Ilex glabra* Inkberry	*Panicum virgatum* Switchgrass
Platanus occidentalis Sycamore	*Ilex verticillata* Winterberry	*Pennisetum alopecuroides* Fountaingrass
Quercus palustris Pin Oak	*Juniperus horizontalis* Creeping Juniper	*Rudbeckia laciniata* Greenhead Coneflower
Quercus phellos Willow Oak	*Lindera benzoin* Spicebush	*Scirpus cyperinus* Woolgrass
Salix nigra Black willow	*Myrica pennsylvanica* Bayberry	*Vernonia gigantea* Ironweed

Table F-5 Planting Plan Specification Issues for Bioretention Areas

Specification Element	Elements
Sequence of Construction	Describe site preparation activities, soil amendments, etc.; address erosion and sediment control procedures; specify step-by-step procedure for plant installation.
Contractor's Responsibilities	Specify the contractors responsibilities, such as watering, care of plant material during transport, timeliness of installation, repairs due to vandalism, etc.
Planting Schedule and Specifications	Specify the materials to be installed, the type of materials (e.g., B&B, bare root, containerized); time of year of installations, sequence of installation of types of plants; fertilization, stabilization seeding, if required; watering and general care.
Maintenance	Specify inspection periods; mulching frequency; removal and replacement of dead and diseased vegetation; treatment of diseased trees; watering schedule after initial installation (once per day for 14 days is common); repair and replacement of staking and wires.
Warranty	Specify warranty period, the required survival rate, and expected condition of plant species at the end of the warranty.

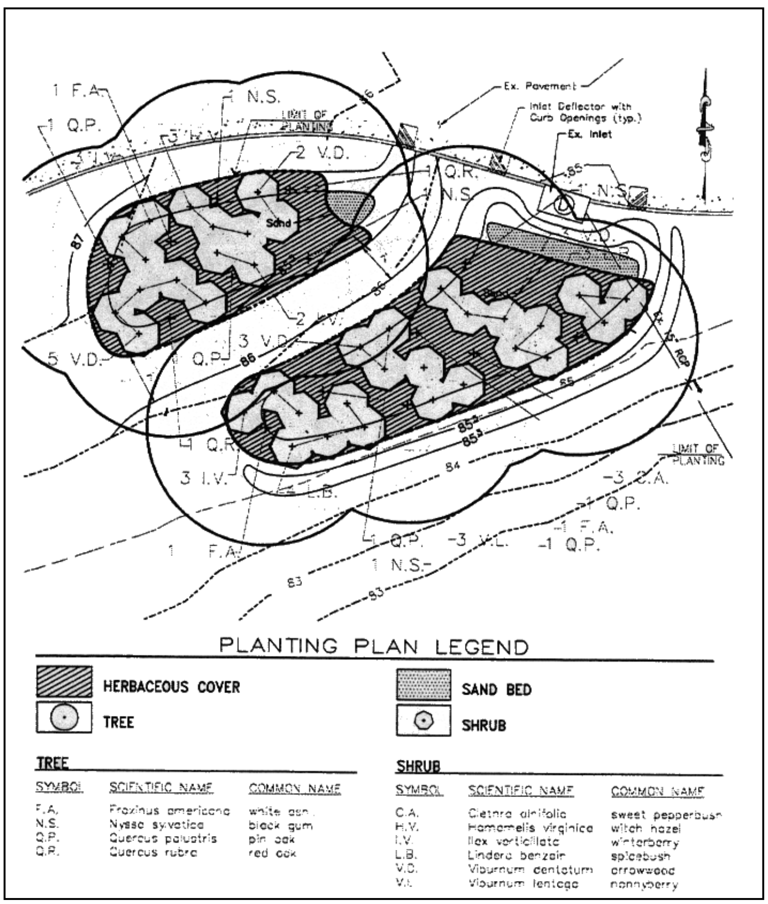


Figure F-8 Sample Bioretention Area Planting Plan
(Source: VDCR, 1999)

F.4.3 Surface Sand Filters and Infiltration Trenches

Both surface sand filters and infiltration trenches can be designed with a grass cover to aid in pollutant removal and prevent clogging. The sand filter or trench 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.

Grass should be capable of withstanding frequent periods of inundation and drought. Vegetated filter strips and buffers should fit into and blend with surrounding area. Native grasses are preferable, if compatible.

Design Constraints:

- Check with your local review authority to see if the planning of a grass cover or turf over a sand filter or infiltration trench is allowed.
- Do not plant trees or provide shade within 15 feet of infiltration or filtering area or where leaf litter will collect and clog infiltration 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.

F.4.4 Enhanced Swales, Grass Channels and Filter Strips

Table F-6 provides a number of grass species that perform well in the stressful environment of an open channel structural control such as an enhanced swale or grass channel, or for grass filter strips. In addition, wet swales may include other wetland species (see F.4.1). Select plant material capable of salt tolerance in areas that may include high salt levels.

Table F.7 Common Grass Species for Dry and Wet Swales and Grass Channels

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 the swale or grass channel environment. Most are salt-tolerant, as well. *Note 2:* Where possible, one or more of these grasses should be in the seed mixes		

F.5 Trees and Shrubs for Stormwater Facilities

The following pages present a detailed list of wetland trees and shrubs that may be used for stormwater management facilities such as stormwater ponds, stormwater wetlands and bioretention areas in Georgia (Source: Garber and Moorhead, 1999)

Table F-7 Wetland indicator status, growth form, flood tolerance and seed dispersal and treatment for selected native Georgia wetland trees and shrubs'

Species	Indicator*	Form	Flood Tolerance**	Seed Dispersal***	Seed Treatments****	Comments
Boxelder *Acer negundo*	FACW	Tree	T	Sept.-Mar.	Cold Strat. 30-40 Days (Mech. Rup. Pericarp)	Can propagate by softwood cuttings
Red Maple *Acer rubrum*	FAC	Tree	T	Apr.-July	Strat. not required	Can propagate by softwood cuttings, tissue culture
Silver Maple *Acer saccharinum*	FACW	Tree	T	Apr.-June	Strat. not req.	
Red Buckeye *Aesculus pavia*	FAC	Shrub	NE	Sept.-Nov.	Strat. not req.	Plant seed as soon as collected. Do not let dry out.
Painted Buckeye *Aesculus sylvatica*	FAC	Shrub	NE	July-Aug.	Cold Strat. 90 Days	
Hazel Alder *Alnus serrulata*	FACW +	Tree	NE	Sept.-Oct.	Cold Strat. 30-60 Days	Can propagate by cuttings, tissue culture
Common Pawpaw *Asimina triloba*	FAC	Tree	I	Sept.-Oct.	Scarification Required Cold Strat. 60-90 Days	
River Birch *Betula nigra*	FACW	Tree	IT	May-June	Cold Strat. 60-90 Days	Can propagate by softwood cuttings
American Hornbeam *Carpinus caroliniana*	FAC	Tree	WT	Oct.-Spring	Cold Strat. 60 Days	
Water Hickory C *Carya aquatica*	OBL	Tree	IT	Oct.-Dec.	Cold Strat. 30-90 Days Warm Strat. 60 Days	
Bitternut Hickory *I Carya cordiformus*	FAC	Tree	NE	Sept.-Dec.	Cold Strat. 90 Days	
Pecan *Carya illinoensis*	FAC +	Tree	IT	Sept.-Dec.	Cold Strat. 30-90 Days	
Shellbark Hickory *Carya laciniosa*	FACW-	Tree	NE	Sept.-Oct.	Cold Strat. 90-120 Days	
Sugarberry *Celtis laevigata*	FACW	Tree	IT	Oct.-Dec.	Cold Strat. 60-90 Days	
Common Buttonbush *Cephalanthus occidentalis*	OBL	Shrub	VT	Sept.-Oct.	Strat. not req.	
Atlantic White Cedar *Chemaecyparis thyoides*	OBL	Tree	T	Oct.-March	Warm Strat. 30 Days Cold Strat. 30 Days	

* *Indicator*: **OBL-obligate**; **FACW-facultative wetland**; **FAC-facultative**; **FACU-facultative upland**.
Indicators may be modified by (+) or (-) suffix; (+)indicates a species more frequently found in wetlands; (-) indicates species less frequently found in wetlands.

** *Flood Tolerance Mature Plants*:

VT-Very Tolerant: Survives flooding for periods of two or more growing seasons.
T-Tolerant: Survives flooding for one growing season.
I-Intermediately Tolerant: Survives one to three months of flooding during growing season
WT-Weakly Tolerant: Survives several days to several weeks of growing-season flooding.
IT-Intolerant: Cannot survive even short periods of a few days or weeks of growing-season flooding.
NE-Not established.

*** *Seed Dispersal*: Approximate dates across natural range of a given species.

**** *Seed Treatments*:
Cold stratification: Place moist seeds in polyethylene plastic bags and place in refrigerated storage at 33°-41° F for specified time.
Warm stratification: Place moist seeds in polyethylene plastic bags at 68°-86° F for specified time. Scarification-mechanical or chemical treatment to increase permeability of seed coat.

Table F-7 continued

Species	Indicator*	Form	Flood Tolerance**	Seed Dispersal***	Seed Treatments****	Comments
Slash Pine *Pinus elliottii*	FACW	Tree	IT	Oct.	Cold Strat. 30 Days	
Spruce Pine *Pinus glabra*	FACW	Tree	IT	Oct.-Nov.	Cold Strat. *28* Days	
Pond Pine *Pinus serotina*	FACW +	Tree	T	Spring	Cold Strat. 30 Days	Cones often remain closed after ripening
Loblolly Pine *Pinus taeda*	FAC	Tree	IT	Oct.-Dec.	Cold Strat. 30-60 Days	
American Sycamore *Platanus occidentalis*	FACW +	Tree	T	Feb.-Apr.	Cold Strat. *60-90* Days	
Eastern Cottonwood *Populus deltoides*	FAC +	Tree	VT	May-Aug.	Strat. not req.	Can propagate by cuttings
Swamp Cottonwood *Populus heterophylla*	OBL	Tree	VT	Apr.-July	Strat. not req.	Can propagate by cuttings
Wafer Ash *Ptelea trifoliata*	FAC	Shrub	NE	Sept.	Cold Strat. *90-120* Days	
Swamp White Oak *Quercus bicolor*	FACW +	Tree	T	Aug.-Dec.	Strat. not req.	White oak group, check native range
Cherrybark Oak *Quercus pagoda*	FAC +	Tree	I	Aug.-Dec.Cold Strat.	*30-90* Days	Red Oak group
Laurel Oak *Quercus laurifolia*	FACW	Tree	IT	Aug.-Dec.	Cold Strat. *30-90* Days	Red Oak group
Overcup Oak *Quercus lyrata*	OBL	Tree	T	Aug.-Dec.	Strat. not req.	White Oak group
Swamp Chestnut Oak *Quercus michiauxii*	FACW-	Tree	I	Aug.-Dec.	Strat. not req.	White Oak group
Water Oak *Quercus nigra*	FAC	Tree	T	Aug.-Dec.	Cold Strat. *30-90* Days	Red Oak group
Willow Oak *Quercus phellos*	FACW-	Tree	T	Aug.-Dec.	Cold Strat. *30-90* Days	Red Oak group
Shumard Oak *Quercus shumardii*	FACW-	Tree	IT	Aug.-Dec.	Cold Strat. *30-90* Days	Red Oak group
Coastal Plain Willow *Salix caroliniana*	OBL	Tree	VT	Mar.-Apr.	Strat. not req.	Seed will not remain viable in storage; plant within 10 days after collection.Can propagate by cuttings
Black Willow *Salix nigra*	OBL	Tree	VT	June-July	Not required.	Seed will not remain viable in storage. Plant within 10 days after collection.Can propagate by cuttings
Baldcypress *Taxodium distichum* var. *distichum*	OBL	Tree	VT	Oct.-Nov.	Cold Strat. *90* Days.	Soak seed for S min. in ethyl alcohol be-fore placing in cold stratification.
Pondcypress *Taxodium distichum* var. *nutans*	OBL	Tree	VT	Oct.-Nov.	Cold Strat. *60-90* Days.	Soak seed for *24* to *48* hrs. in 0.0196 cit-ric acid before plac-ing in cold stratification.
American Elm *Ulmus americana*	FACW	Tree	T	Mar.-June	Cold Strat. 60-90 Days	Can propagate by cuttings
Slippery elm *Ulmus rubra*	FAC	Tree	I	Apr.-June	Cold Strat. 60-90 Days	Can propagate by cuttings

Table F-7 continued

Species	Indicator*	Form	Flood Tolerance**	Seed Dispersal***	Seed Treatments****	Comments
Rough-Leaf Dogwood *Cornus drummondii*	FAC	Tree	T	Aug.-Jan.	Warm Strat. 70°-80° 1 Day Cold Strat. 30 Days	
Hawthornes *Crataegus spp.*	FAC	Shrub	IT	Fall-Winter	May Req. Scari-fication Warm Strat. 70°-80° 30-90 Days Cold Strat. 90-180 Days	
Common Persimmon *Diospyros virginiana*	FAC	Tree	T	Oct.-Nov.	Cold Strat. 60-90 Days	
Eastern Burning Bush *Euonymus atropurpuresu*	FAC	Shrub	NE	Sept.-Oct.	Warm Strat. 68°-86° 60 Days Cold Strat. 60 Days	
Carolina Ash *Fraxinus caroliniana*	OBL	Shrub	VT	Sept.- Dec.	Cold Strat. 60 Days	
Green Ash *Fraxinus pennsylvanica*	FACW	Tree	VT	Oct.-Feb.	Cold Strat. 60-90 Days	
Pumpkin Ash *Fraxinus profonda*	OBL	Tree	VT	Oct.-Dec.	Cold Strat. 60 Days	
Waterlocust *Gleditsia aquatica*	OBL	Tree	T	Sept.-Dec.	Req. Scarifica-tion	
Loblolly Bay *Gordonia laisianthus*	FACW	Tree	T	Fall	Not Established	
Decidious Holly *Illex decidua*	FACW	Shrub	VT	Sept.-Mar.	Warm Strat. 68°-Day, 86°-Night 60 Days Cold Strat.-60 Days	
Spicebush *Lindera benzoin*	FACW	Shrub	NE	Sept.-Oct.	Cold Strat. 120 Days	
Sweetgum *Liquidamber styraciflua*	FAC +	Tree	T	Sept.-Nov.	Cold Strat. *30* Days	
Yellow Poplar *Liriodendron tulipifera*	FAC	Tree	I	Oct.-Nov.	Cold Strat. 60-90 Days	
Sweetbay *Magnolia virginiana*	FACW +	Tree	IT	Sept.-Nov.	Cold Strat. 90-180 Days	Can propagate by cuttings
Red Mulberry *Morus rubra*	FAC	Tree	IT	June-Aug.	Cold Strat. 30-90 Days	
Southern Bayberry *Myrica cerifera*	FAC +	Shrub	NE	Aug.-Oct.	Cold Strat. 60-90 Days	
Water Tupelo *Nyssa aquatica*	OBL	Tree	VT	Oct.-Nov.	Cold Strat. 30-120 Days	
Ogeechee Tupelo *Nyssa ogeche*	OBL	Tree	VT	Aug.-Sept.	Cold Strat. 30-120 Days	
Swamp Tupelo *Nyssa sylvatica* var. *biflora*	OBL	Tree	VT	Sept.-Dec.	Cold Strat. 30-120 Days	
Redbay *Persea borbonia*	FACW	Tree	MT	Fall	Not established	

Table F-8 Seedling response of selected species to flooding conditions

Species	Water Level	Seedling Survival*	Comments
Boxelder *Acer negundo*	Total submersion Growing Season	100% at 2 weeks 70% at 3 weeks 36% at 4 weeks 0% at 32 days	Chlorotic leaves after 4 days. Slow recovery.
Red Maple *Acer rubrum*	Partial submersion Growing season	100% at 5 days 90% at 10 days 0% at 20 days	Adventitious roots developed after 15 days Height growth decreased in saturated soil Soil saturation
	Soil saturation Growing season	Growing season 100% at 32 days	
Silver Maple *Acer saccharinum*	Total submersion Growing season	100% at 3 weeks	Lower leaves wilt after 2 days. Slow recovery Height growth better at saturated conditions than field capacity
	Soil saturation Growing season	100% at 60 days	
River Birch *Betula nigra*	Soil saturation Growing season	100% at 32 days	Growth severely stunted
Pecan *Carya illinoensis*	Total submersion Growing season	75% at 4 weeks	
Sugarberry *Celtis laevigata*	Soil saturation Growing season	100% at 60 days	
Common Buttonbush *Cephalanthus occidentalis*	Total submersion Growing season	100% at 30 days	
Green Ash *Fraxinus pennsylvanica*	Total submersion Growing season	100% at 5 days 90% at 10 days 73% at 20 days 20% at 30 days	Lower leaves chlorotic after 8 days Better growth in saturated soil than soil at field capacity
	Partial submersion Growing season	100% at 14 days	
	Soil saturation Growing season	100% at 60 days	
Sweetgum *Liquidambar styraciflua*	Total submersion Growing season	0% at 32 days	
	Partial submersion Growing season	0% at 3 months	

Adapted from Teskey & Hinkley, 1977

* Seeding survival in relation to length of flooding

Table F-8 continued

Species	Water Level	Seedling Survival*	Comments
Yellow Poplar *Liriodendron tulipifera*	Partial submersion Growing Season	0% at 2 months No adverse effects	
Water Tupelo *Nyssa aquatica*	Partial submersion Growing season	90-100% over growing season 32% when seedlings nearly overtopped	Best growth when water table fluctuates
Swamp Tupelo *Nyssa sylvatica* var. *bif ora*	Partial submersion Growing season	90 - 100% over growing season	Poor root growth in stagnant water
	Soil saturation Growing season	90 - 100% over growing season	Best growth in saturated soil
Slash Pine *Pinus elliottii*	Partial submersion Growing season	68% at 2 months 12% at 7 months	Root and shoot growth decreased
Loblolly Pine *Pinus taeda*			Root & shoot growth reduced during flooding Dormant season flooding increased height and diameter growth
American Sycamore *Platanus occidentalis*	Total submersion Growing season	100% at 10 days 0% at 30 days	Growth decreased by saturated soil
	Soil saturation Grnwina season	95% at 32 days	
Eastern Cottonwood *Populus deltoides*	Total submersion Growing season	0% at 16 days	Best growth when water table is 2 feet below surface
	Partial submersion Growing season	90% at 10 days 70% at 20 days 47% at 30 days	High mortality when deeply flooded
Cherrybark Oak *Quercus pagoda*	Total submersion Growing season	87% at 5 days 6% at 10 days 0% at 20 days	Height growth decreased by soil saturation
	Soil saturation Growing season	89% at 15 days 47% at 30 days 13% at 60 days	
Water Oak *Quercus nigra*	Partial submersion Growing season	Survived 2 months	
Willow Oak *Quercus phellos*	Soil saturation Growing season	100% at 50 days	Poorer growth in saturated soil than soil at field capacity
Shumard Oak *Quercus shumardii*	Total submersion Growing season	100% at 5 days 90% at 10 days 6% at 20 days	Height growth poorer in saturated soil than soil at field capacity
	Soil saturation Growing season	100% at 30 days 66% at 60 days	
Black Willow *Salix nigra*	Total submersion Growing season	100% at 30 days	Better height growth in saturated soil than soil at field capacity
	Soil saturation Growing season	100% at 60 days	
Baldcypress *Taxodium distichum* var. *distichum*	Total submersion Growing season	100% at 4 weeks	
American Elm *Ulmus americana*	Total submersion Growing season	100% at 10 days 27% at 20 days 0% at 30 days	Height growth decreased in saturated soil
	Soil saturation Growing season	100% at 15 days 94% at 60 days	

* Seeding survival in relation to length of flooding

F.6 Aesthetic Considerations in Stormwater Facility Design and Landscaping

to be provided

(updates found at www.georgiastormwater.com)

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STORMWATER COMPUTER MODELS

G.1 Types of Models

In urban stormwater management there are typically three types of models used commonly: *hydrologic*, *hydraulic* and *water quality* models. There are also a number of other specialty models to simulate ancillary issues (some of which are sub-sets of the three main categories) such as sediment transport, channel stability, lake quality, dissolved oxygen and evapotranspiration.

Hydrologic Models

Hydrologic models attempt to simulate the rainfall-runoff process to tell us "how much water, how often." They use rainfall information or simulations to provide runoff characteristics including peak flow, flood hydrograph and flow frequencies.

Hydrologic models can be:

- *Deterministic* – giving one answer for a specific input set, or
- *Stochastic* – involving random inputs giving any number of responses for a given set of parameters.

- *Continuous* – simulating many storm events over a period of time, or
- *Single Event* – simulating one storm event.

- *Lumped* – representing a large area of land use by a single set of parameters, or
- *Distributed* – land areas are broken into many small homogeneous areas each of which has a complete hydrologic calculation made on it.

Hydraulic Models

Hydraulic models take a known flow amount (typically the output of a hydrologic model) and provide information about flow height, location, velocity, direction and pressure.

Hydraulic models share some of the differing characteristics of hydrologic models (continuous vs. single event) and add:

- *One-dimensional* – calculating flow information in one direction (e.g. downstream) only, or
- *Multi-dimensional* – calculating flow information in several dimensions (e.g. in and out of the channel and downstream).

- *Steady* – having a single unchanging flow velocity value at a point in the system, or
- *Unsteady* – having changing flow velocities with time.

- *Uniform* – assuming the channel slope and energy slope are equal, or
- *Non-uniform* – solving a more complex formulation of the energy and momentum equations to account for the dynamic nature of flows.

For most problems encountered in hydraulics, a simple one-dimensional, steady model will work well. But if the volume and time distribution of flow are important (for example, in a steeper stream with storage behind a series of high culvert embankments) an unsteady model is needed. If there is a need to predict with accuracy the ebb and flow of floodwater out of a channel (for example in a wide, flat floodplain where there are relief openings under a road) then a 2-dimensional model becomes necessary. If pressure flow and the accurate computation of a hydraulic grade line are important an unsteady, non-uniform model with pressure flow calculating capabilities is needed.

Water Quality Models

The goal in water quality modeling is to adequately simulate the various processes and interactions of stormwater pollution. Water quality models have been developed with an ability to predict loadings of various types of stormwater pollutants.

Water quality models can become very complex if the complete cycle of buildup, wash-off and impact are determined. These models share the various features of hydrologic and hydraulic models in that it is the runoff flow that carries the pollutants. Therefore, a continuous hydrologic model with estimated pollution concentrations becomes a continuous water quality pollution model. Water quality models can reflect pollution from both point and nonpoint sources.

Water quality models tend to have applications that are targeted toward specific pollutants, source types or receiving waters. Some models involve biological processes as well as physical and chemical processes. Often great simplifications or gross assumptions are necessary to be able to model pollutant accumulations, transformations and eventual impacts.

Detailed short time increment predictions of "pollutographs" are seldom needed for the assessment of receiving water quality. Hence, the total storm event loads or mean concentrations are normally adequate. Simple spreadsheet-based loading models involve an estimate of the runoff volume which, when multiplied by an event mean concentration, provide an estimate of pollution loading. Because of the lack of ability to calibrate such models for variable physical parameters, such simple models tend to be more accurate the longer the time period over which the pollution load is averaged. An annual pollutant load prediction may tend toward a central estimate, while any specific storm prediction may be grossly in error when compared to actual loadings because antecedent conditions vary widely from week to week. Simulation models have the ability to adjust a number of loading parameters for calibration purposes and can simulate pollution accumulation over a long period. They can then more reliably predict loadings for any specific storm event.

While calibration data is not always needed in hydrologic or hydraulic models for an acceptably accurate answer, in water quality models the non-calibrated prediction is often off by orders of magnitude. Water quality predictions are not credible without adequate site-specific data for calibration and verification. However, even without specifically accurate loading values relative effects of pollution abatement controls can be tested using uncalibrated models.

Computer Model Applications

Stormwater computer models can also be categorized by their use or application:

Screening-level models are typically equations or spreadsheet models that give a first estimate of the magnitude of urban runoff quality or quantity. At times this is the only level that is necessary to provide answers. This is true either because the answer needs to be only approximate or because there is no data to justify a more refined procedure.

Planning-level models are used to perform "what if" analysis comparing in a general way design alternatives or control options. They are used to establish flow frequencies, floodplain boundaries, and general pollution loading values.

Design-level models are oriented toward the detailed simulation of a single storm event for the purposes of urban stormwater design. They provide a more complete description of flow or pollution values anywhere in the system of concern and allow for adjustment of various input and output variables in some detail. They can be more exact in the impact of control options, and tend to have a better ability to be calibrated to fit observed data.

Operational models are used to produce actual control decisions during a storm event. They are often linked with SCADA systems. They are often developed from modified or strongly calibrated design models, or can be developed on a site-specific basis to appropriately link with the system of concern and accurately model the important physical phenomena.

G.2 The Modeling Process

The overall modeling process involves: (1) development of study or model objectives, (2) identification of resources and constraints, and finally, (3) the selection and implementation of the model itself.

Model Objectives

It is important to know specifically what answers are needed, to what accuracy, and in what format. Requiring a simple peak flow is far different from needing to know the timing of peaks from several different intersecting watersheds. Estimating future floodplain elevations along a reach is a fundamentally different problem than finding the probability of roadway overtopping.

A review of the problem begins the process of determining the model objectives. These objectives also establish a performance or design criteria for the model. Must the system handle the 25-year storm? Are future conditions important? Which ones? Are annual loadings of pollution adequate? Which pollutants?

Those aspects of the system to be modeled will dictate what models are appropriate for use. For example, if storm sewers are present then an open channel model can be ruled out as an appropriate model for the entire system. If a specific type of hydraulic structure is present that a standard model cannot handle, an alternate way to simulate that structure will be necessary.

Model objectives also explain how the numbers generated from the model will relate to the needs of the study. For example, if a cost-benefit analysis is required, the model results must be interpreted in terms of overall life-cycle cost and not simply in terms of discharge rate.

Model Constraints

Availability of data, funds, time and user ability can potentially constrain modeling solutions. The goal of any modeling effort is to develop an approach that stays within the constraints dictated while addressing the needs of the study identified in the previous step. Data collection/availability and cost are usually the chief constraints.

Sources of existing available data should be researched. Look for data that tends to "ground truth" model outputs. Even partial data can be useful if it helps to validate the model or modeling results. After existing data sources have been identified, the need to gather additional data is assessed. Automated processes and systems such as GIS and GPS can reduce both cost and human error. A consideration of the long-term use of data and its maintenance is necessary. For example, if the model is to eventually become an operational model, the ability to maintain the data in a cost effective way becomes of paramount importance.

Accuracy and the corresponding necessary level of detail are of overriding importance. Accuracy depends on both the accuracy of the input data and the degree to which the model adequately represents the hydrologic, hydraulic or water quality processes being modeled. For example, if lumped hydrologic parameters are adequate, then the cost of the modeling effort can be reduced. However, the ability to determine information within the sub-basin represented by a single parameter is lost. Changing model needs from an average 500-acre sub-basin size to a 50-acre size can increase the cost of a model almost ten fold. Is the information derived worth the cost?

Both risk and uncertainty affect the modeler's ability to predict results accurately. Risk is an estimated chance of an occurrence, such as flooding. Uncertainty is the error associated with measuring or estimating key parameters or functions. Uncertainty arises due to errors in sampling, measurement, estimation and forecasting, and modeling. For hydrologic and hydraulic analysis, stage and discharge are of prime importance. Uncertainty in discharge is due to short or nonexistent flood records, inaccurate rainfall-runoff modeling, and inaccuracy in known flood flow regulation where it exists. Stage uncertainty comes from errors and unknowns in roughness, geometry, debris accumulation, sediment effects and others factors.

Accuracy developed in one area can be impacted by rough estimates in another, and the technological gains lost. For example, the gains in accuracy from very precise field surveys of cross sections can be lost if the estimates of roughness coefficients or discharge rates are very approximate.

Sensitivity analysis involves holding all parameters constant except one and assessing the change in output variable of concern with a certain percent change in the input variable. Those variables that are amplified in the output should be estimated with higher accuracy and with a more detailed consideration of the potential range of values and the need for conservative design. The modeler must try to assess how accurate estimates are and to account for risk and uncertainty through estimating the range of potential error and choosing values that balance conservative engineering with cost consciousness. The designer typically develops a "most likely" estimate of a certain design parameter (for example, 10-year storm rainfall or Manning's roughness coefficient) and then uses sensitivity analysis to test the impact of variability in the parameter estimate on the final solution

<u>Selection and Implementation</u>

Once the model objectives and constraints have been evaluated, the model (or models) is selected and the study or design is implemented. Typical steps in model implementation include validation, calibration, verification and production.

Validation involves a determination that the model is structured and coded as intended for the range of variables to be encountered in the study. Validation tests key algorithms for accuracy. For example, if a hydrologic model cannot handle short time steps or long time periods it cannot be used without modification. If a certain model begins to lose accuracy at high or low imperviousness or cannot accurately handle backwater situations, and these will be encountered in practice the model cannot be used. Often validation is a one-time effort, after which the modeler is comfortable with the model's "quirks" and knows how to deal with them. Validation often involves pushing parameters to the limit of reasonable extent to test an algorithm. For example, in a hydrologic model infiltration can be reduced to zero to test if the input and output hydrographs are equal. Or the model can be run with small rainfalls using porous soils to determine if no runoff is generated, or only runoff from directly connected impervious areas.

Calibration is the comparison of a model to field measurements, other known estimates of output (e.g. regression equations), or another model known to be accurate, and the subsequent adjustment of the model to best fit those measurements. Verification then tests the calibrated model against another set of data not used in the calibration. This step is not always possible due to the general shortage of data of any sort in stormwater management. Goodness of prediction is done through a simple comparison of the difference in observed and predicted peaks, pollution loads, flood elevations or volumes divided by the observed values and expressed as a percentage, or as simple ratio. Assessing the goodness of fit of a hydrograph is done by calculating the sum of the squares of the difference between observed and predicted values at discreet time steps.

Once the model is prepared for use, attention shifts to efficient production methods that minimize the potential for errors while maximizing efficiency. Often “production line”-type efforts are used for large modeling projects. However, constant attention must be paid to ensure the execution of correct procedures, detailed documentation of efforts and input/output data sets, and recognition of anomalies that would invalidate a particular model run.

While there is much to be gained from simple user interfaces and black box approaches that simplify the input and output processes, there is an inherent danger that the modeler will not be aware of errors or problems in the modeling process. For example, in hydraulic modeling, shifts from super- to sub-critical flow happen at sharp break points and are reflected in a jump in water surface. If not caught, a model will under predict flow elevation. Numeric instability in mathematical algorithms may give oscillating answers that have nothing to do with reality.
A structured review process must be established to insure reasonableness of output and accuracy of input values have been used. Labeling of data sets should be systematic and exact.

G.3 Summary of Commonly-Used Models

Computer models can be simple, representing only a very few measured or estimated input parameters or can be very complex involving twenty times the number of input parameters. The “right” model is the one that: (1) the user thoroughly understands, (2) gives adequately accurate and clearly displayed answers to the key questions, (3) minimizes time and cost, and (4) uses readily available or collected information. Complex models used to answer simple questions are not an advantage. However, simple models that do not model key necessary physical processes are useless.

There is no one engineering model or software that addresses all hydrologic, hydraulic and water quality situations. Design needs and troubleshooting for watershed and stormwater management occur on several different scales and can be either system-wide (i.e., watershed) or localized. System-wide issues can occur on both large and small drainage systems, but generally require detailed, and often expensive, watershed models and/or design tools. The program(s) chosen to address these issues should handle both major and minor drainage systems. Localized issues also exist on both major and minor drainage systems, but unlike system-wide problems, flood and water quality solution alternatives can usually be developed quickly and cheaply using simpler engineering methods and design tools.

Table G-1 lists several widely used computer programs and modeling packages. The programs were examined for their applicability to both system-wide and localized issues, the methodologies used for computations, and ease-of-use.

For the purposes of this table, major drainage systems are defined as those draining to larger receiving waters. These are typically FEMA-regulated streams, or lakes or reservoirs. Minor drainage systems are smaller natural and man-made systems that drain to the more major streams. Minor drainage systems can have both closed and open-channel components and can include, but are not limited to, neighborhood storm sewers, culverts, ditches, and tributaries.

A brief description of program capabilities and methodologies are presented in a short discussion of each program.

Table G-1 Stormwater Modeling Programs and Design Tools					
	Major System Modeling	**Minor System Modeling**	**Hydrologic Features**	**Hydraulic Features**	**Water Quality Features**
Hydrology Software					
HEC-1	**X**		**X**		
TR-55			**X**		
PondPack		**X**	**X**	**X**	
WMS	**X**		**X**		
Watershed Modeling	**X**		**X**		
Hydraulics Software					
HEC-RAS	**X**			**X**	
WSPRO	**X**			**X**	
EPA SWMM	**X**	**X**	**X**	**X**	**X**
CulvertMaster		**X**		**X**	
FlowMaster		**X**		**X**	
Water Quality Software					
HSPF	**X**		**X**		**X**
BASINS	**X**		**X**	**X**	**X**
QUAL2E	**X**			**X**	**X**
WASP5	**X**			**X**	**X**
SLAMM	**X**		**X**		**X**

Hydrology Programs

HEC-1 - Flood Hydrograph Package

HEC-1 was developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers to simulate the surface runoff response of a watershed to rainfall events. Although it is a DOS-based program, it is still considered by many in the engineering and regulatory communities to be the leading model for major drainage system applications such as Flood Insurance Studies and watershed master planning. HEC-1 is accepted by the Federal Emergency Management Agency, therefore it is the most widely used model for major drainage system analyses.

In a HEC-1 model, the watershed is represented in the model as an interconnected system of hydrologic (e.g., sub-basins, reservoirs, ponds) and hydraulic (e.g., channels, closed conduits, pumps) components. The model computes a runoff hydrograph at each component, combining two or more hydrographs as it moves downstream in the watershed. The model has a variety of rainfall-runoff simulation methods, including the popular SCS Curve Number methodology. The user can define rainfall events using gage or historical data, or HEC-1 can generate synthetic storms. Hydrograph generation is performed using the unit hydrograph technique. Clark, SCS Dimensionless, and Snyder Unit Hydrographs are the available methodologies. Several common channel and storage routing techniques are available as well.

HEC-1 is not considered a "design tool." The program has limited hydraulic capabilities. It does not account for tailwater effects and cannot adequately simulate many urban hydraulic structures such as pipe networks, culverts and multi-stage detention pond outlet structures. However, there are other hydrologic applications developed within HEC-1 that have been utilized with much success. Multiplan-multiflood analyses allow the user to simulate a number of flood events for different watershed situations (or plans). The dam safety option enables the user to analyze the impact dam overtopping or structural failure on downstream areas. Flood damage analyses assess the economic impact of flood damage.

Because it is not a Windows-based program, HEC-1 does not have easy to use input and output report generation and graphical capabilities, and therefore is generally not considered a user-friendly program. Because of its wide acceptance, several software development companies have incorporated the source code into enhanced "shells" to provide a user-friendly interface and graphical input and output capabilities. Examples of these programs include Graphical HEC-1 developed by Haested Methods and WMS developed by the Environmental Modeling Research Laboratory.

The Corps of Engineers has developed a user-friendly, Windows-based Hydrologic Modeling System (HEC-HMS) intended to replace the DOS-based HEC-1 model. The new program has all the components of HEC-1, with more user-friendly input and output processors and graphical capabilities. HEC-1 files can be imported into HEC-HMS. Version 2 of this model has been released, however its acceptance and use is limited at this time. While highly anticipated by the engineering community, widespread use of HEC-HMS has been slow to develop, mainly due to the necessity for the Corps of Engineers to further develop, modify and "debug" the early program. FEMA is expected to approve the model after some length of time.

TR-55 - Technical Release 55

The TR-55 model is a DOS-based software package used for estimating runoff hydrographs and peak discharges for small urban watersheds. The model was developed by the NRCS (formally SCS), and therefore uses SCS hydrograph methodology to estimate runoff. No other methodology is available in the program. Four 24-hour regional rainfall distributions are available for use. Rainfall durations less than 24-hours cannot be simulated. Using detailed input data entered by the user, the TR-55 model can calculate the area-weighted CN, time of concentration and travel time. Detention pond (i.e., storage) analysis is also available in the TR-55 model, and is intended for initial pond sizing. Final design requires a more detailed analysis.

TR-55 is easy-to-use, however because it is DOS-based it does not have the useful editing and graphical capabilities of a Windows-based program. Haestad Methods, Inc., included most of the TR-55 capabilities in its PondPack program described below.

PONDPACK

PondPack, by Haestad Methods, Inc., is Windows-based software developed for modeling general hydrology and runoff from site development. The program analyzes pre- and post-developed watershed conditions and sizes detention ponds. It also computes outlet rating-curves with consideration of tailwater effects, accounts for pond infiltration, calculates detention times and analyzes channels.

Rainfall options are unlimited. The user can model any duration or distribution, for synthetic or real storm events. Several peak discharge and hydrograph computation methods are available, including SCS, the Rational Method and the Santa Barbara Unit Hydrograph procedure. Infiltration can be considered, and pond and channel routing options are available as well. Like TR-55, PondPack allows the user to calculate hydrologic parameters, such as the time of concentration, within the program.

PondPack has limited, but useful hydraulic features, using Manning's equation to model natural and man-made channels and pipes. A wide variety of detention pond outlet structure configurations can be modeled, including low flow culverts, weirs, riser pipes, and even user-defined structures.

WMS - Watershed Modeling System

WMS was developed by the Engineer Computer Graphics Laboratory of Brigham Young University. WMS is a Windows-based user interface that provides a link between terrain models and GIS software, with industry standard lumped parameter hydrologic models, including HEC-1, TR-55, TR-20 and others. The hydrologic models can be run from the WMS interface. The link between the spatial terrain data and the hydrologic model(s) gives the user the ability to develop hydrologic data that is typically gathered using manual methods from within the program. For example, when using SCS methodologies, the user can delineate watersheds and sub-basins, determine areas and curve numbers, and calculate the time of concentration at the computer. Typically, these computations are done manually, and are laborious and time-consuming. WMS attempts to utilize digital spatial data to make these tasks more efficient.

Watershed Modeling

The Watershed Modeling program was developed to compute runoff and design flood control structures. The program can run inside the MicroStation CAD system. Like WMS, this feature enables the program to delineate and analyze the drainage area of interest. Area, curve number, land use and other hydrologic parameters can be computed and/or catalogued for the user, removing much of the manual calculation typically performed by the hydrologic modeler.

Watershed Modeling contains a variety of methods to calculate flood hydrographs, including SCS, Snyder and Rational methods. Rainfall can be synthetic or user-defined, with any duration and return period. Rainfall maps for the entire U.S. are provide to help the user calculate IDF relationships. Several techniques are available for channel and storage routing. The user also has a wide variety of outlet structure options for detention pond analysis and design.

Hydraulics Programs

HEC-RAS - River Analysis System

HEC-RAS is a Windows-based hydraulic model developed by the Corps of Engineers to replace the popular, DOS-based HEC-2 model. RAS has the ability to import and convert HEC-2 input files and expounds upon the capabilities of HEC-2. Since its introduction several years ago, the user-friendly HEC-RAS has become known as an excellent model for simulation of major systems (i.e., open channel flow) and has become the chief model for calculating floodplain elevations and determining floodway encroachments for Flood Insurance Studies. Like HEC-2, HEC-RAS has been accepted for FIS analysis by the FEMA. However, RAS is a much easier model to use than HEC-2 as it has an extremely useful interface that provides the immediate capability to view model input and output data in graphical, tabular, and report formats.

HEC-RAS performs one-dimensional analysis for steady flow water surface profiles, using the energy equation. Energy losses are calculated using Manning's equation and contraction and expansion changes. Rapidly varied flow (e.g., hydraulic jumps) is modeled using the momentum equation. The effects of in-stream structures, such as bridges, culverts, weirs and floodplain obstructions and in-stream changes such as levees and channel improvements can be simulated. The model allows the user to define the geometry of the channel or structure to the level of detail required by the application. One popular and useful feature of the HEC-RAS model is the capability to easily facilitate floodway encroachment analysis. Five encroachment methods are available to the user.

The Corps of Engineers has stated that future versions of the HEC-RAS model will have components for unsteady flow and sediment transport simulations. In the model's original form, RAS does not provide a tie to GIS information. However, the model was designed with GIS applications in mind and future ties between RAS and GIS platforms are anticipated. Several software developers have already released enhanced versions of RAS that provide the capability to import GIS data for channel geometry and export RAS output for floodplain and floodway delineation. Examples of such software include BOSS RMS, developed by BOSS International and SMS (Surface Water Modeling System), distributed by the Scientific Software Group.

WSPRO

WSPRO was developed by the USGS to compute water surface profiles for one-dimensional, gradually varied, steady flow. Like HEC-RAS, WSPRO can develop profiles in subcritical, critical and supercritical flow regimes. WSPRO is designated HY-7 in the Federal Highway

Administration (FHWA) computer program series and its original objective was analysis and design of bridge openings and embankment configurations. Since then, the model has been expanded to model open channels and culverts.

Open channel computations use standard step-backwater techniques. Flow through bridges is simulated using an energy-balancing technique that uses a coefficient of discharge and estimates an effective flow length. Pressure flow under bridges using orifice-type flow equations developed by the FHWA. Culvert flow is simulated using FHWA techniques for inlet control and energy balance for outlet control.

WSPRO is considered a fairly easy-to-use DOS-based model, applicable to water surface profile analysis for highway design, flood insurance studies, and establishing stage-discharge relationships. However, the model in its original form is not Windows based and therefore does not have the useful editing and graphical features found in HEC-RAS. Like HEC-RAS, a third party software developer has designed SMS (Surface Water Modeling Software) to support both pre- and post-processing of WSPRO data.

EPA SWMM - Storm Water Management Model

EPA SWMM was developed by the Environmental Protection Agency (EPA) to analyze storm water quantity and quality problems associated with runoff from urban areas. EPA SWMM has become the model of choice for simulation of minor drainage systems primarily composed of closed conduits. The model can simulate both single-event and continuous events and has the capability to model both wet and dry weather flow. The basic output from SWMM consists of runoff hydrographs, pollutographs, storage volumes and flow stages and depths.

SWMM's hydraulic computations are link-node based, and are performed in separate modules, called blocks. The EXTRAN computational block solves complete dynamic flow routing equations to simulate backwater, looped pipe connections, manhole surcharging and pressure flow. It is the most comprehensive model in its capabilities to simulate urban storm flow and many cities have used successfully for storm water, sanitary, or combined sewer system modeling. Open channel flow can be simulated using the TRANSPORT block, which solves the kinematic wave equations for natural channel cross-sections.

Although evaluated for this study as a hydraulic model, SWMM has both hydrologic and water quality components. Hydrologic processes are simulated using the RUNOFF block, which computes the quantity and quality of runoff from drainage areas and routes the flow to the major sewer system lines. Pollutant transport is simulated in tandem with hydrologic and hydraulic computations and consists of calculation of pollutant buildup and washoff from land surfaces and pollutant routing, scour and in-conduit suspension in flow conduits and channels.

EPA SWMM is a public domain, DOS-based model. For large watersheds with extensive pipe networks, input and output processing can be tedious and confusing. Because of the popularity of the model commercial, third-party enhancements to SWMM have become more common, making the model a strong choice for minor system drainage modeling. Examples of commercially enhanced versions of EPA SWMM include MIKE SWMM, distributed by BOSS International, XPSWMM by XP-Software, and PCSWMM by Computational Hydraulics Inc (CHI). CHI also developed PCSWMM GIS, which ties the SWMM model to a GIS platform.

CULVERTMASTER

CulvertMaster, developed by Haestad Methods, Inc., is an easy-to-use, Windows-based culvert simulation and design program. The program can analyze pressure or free surface flow conditions and in subcritical, critical and supercritical flow conditions, based on drawdown and backwater. A variety of common culvert shapes and section types are available. Tailwater effects are considered and user can enter a constant tailwater elevation, a rating curve, or specify an outlet channel section. Culvert hydraulics are solved using FHWA methodology for inlet and outlet control computations. Roadway and weir overtopping are checked in the solution of the culvert.

CulvertMaster does have a hydrologic analysis component to determine peak flow using the Rational Method, SCS Graphical Peak Methods. The user also has the option of entering a known peak flow rate. The user must enter all rainfall and runoff information (e.g., IDF data, rainfall depths, curve numbers, C coefficients, etc).

FLOWMASTER

FlowMaster, also developed by Haestad Methods, Inc., is a Windows-based hydraulic pipe and channel design program. The user enters known information on the channel section or pipe, and allows the program to solve for the unknown parameter(s), such as diameter, depth, slope, roughness, capacity, velocity, etc. Solution methods include Manning's equation, the Darcy-Weisbach formula, Hazen-Williams formula, and Kutter's Formula. The program also features calculations for weirs, orifices, gutter flow, ditch and median flow and discharge into curb, grated, and slot inlets.

<u>Water Quality Programs</u>

HSPF - Hydrologic Simulation Program FORTRAN

The HSPF model was developed by the EPA for the continuous or single-event simulation of runoff quantity and quality from a watershed. The original model was developed from the Stanford Watershed Model, which simulated runoff quantity only. It was expanded to include quality components, and has since become a popular model for continuous non-point source water quality simulations. Non-point source conventional and toxic organic pollutants from urban and agricultural land uses can be simulated, on pervious and impervious land surfaces and in streams and well-mixed impoundments. The various hydrologic processes are represented mathematically as flows and storages. The watershed is divided into land segments, channel reaches and reservoirs. Water, sediment and pollutants leaving a land segment move laterally to a downstream land segment, a stream or river reach, or reservoir. Infiltration is considered for pervious land segments.

HSPF model output includes time series information for water quality and quantity, flow rates, sediment loads, and nutrient and pesticide concentrations. To manage the large amounts of data associated with the model, HSPF includes a database management system. To date, HSPF is still a DOS-based model and therefore does not have the useful graphical and editing options of a Windows-based program. Input data requirements for the model are extensive and the model takes some time to learn. However the EPA continues to expand and develop HSPF, and still recommends it for the continuous simulation of hydrology and water quality in watersheds.

BASINS - Better Assessment Science Integrating Point and Non-Point Sources

The BASINS watershed analysis system was developed by the EPA for use by regional, state and local pollution control agencies to analyze water quality on a watershed-wide basis. BASINS integrates the ArcView GIS environment, national databases containing watershed data, and modeling programs and water quality assessment tools into one stand-alone program. The program will analyze both point and non-point sources and supports the development of the total maximum daily loads (TMDLs). The assessment tools and models utilized in BASINS include TARGET, ASSESS, Data Mining, HSPF, TOXIROUTE and QUAL2E. The databases, assessment tools and models are directly tied to the ArcView GIS environment.

QUAL2EU - Enhanced Stream Water Quality Model

QUAL2EU was developed by the EPA and intended for use as a water quality planning tool. The model actually consists of four modules: QUAL2E - the original water quality model, QUAL2EU - the water quality model with uncertainty analysis, and pre and post processing modules. QUAL2EU simulates steady state or dynamic conditions in branching streams and well-mixed lakes, and can evaluate the impact of waste loads on water quality. It also can enhance a field sampling program by helping to identify the magnitude and quality characteristics of non-point waste loads. Up to 15 water quality constituents can be modeled. Dynamic simulation allows the user to study the effects of diurnal variations in water quality (primarily DO and temperature). The steady state option allows the user to perform uncertainty analyses.

QUAL2EU is a DOS-based program, and the user will require some length of time to develop a QUAL2EU model, mainly due to the complexity of the model and data requirements for a simulation. However, to ease user interaction with the model an interactive preprocessor (AQUAL2) has been developed to help the user build input data files. A postprocessor (Q2PLOT) also exists that displays model output in textual or graphical formats.

WASP5 - Water Quality Analysis Simulation Program

The WASP5 model was developed by the EPA to simulate contaminant fate in surface waters. Both chemical and toxic pollution can be simulated in one, two, or three dimensions. Problems studied using WASP5 include biochemical oxygen demand and dissolved oxygen dynamics, nutrients and eutrophication, bacterial contamination, and organic chemical and heavy metal contamination. WASP5 has an associated stand alone hydrodynamic model, called DYNHYD5, that simulates variable tidal cycles, wind and unsteady flows. DYNHYD4 supplies flows and volumes to the water quality model. The model is DOS-based, however WASP packages can be obtained from outside vendors that include interactive tabular and graphical pre- and post-processors.

SLAMM - Source Loading and Management Model

The SLAMM model was originally developed as a planning tool to model runoff water quality changes resulting from urban runoff pollutants. The model has been expanded to included simulation of common water quality best management practices such as infiltration BMPs, wet detention ponds, porous pavement, street cleaning, catchbasin cleaning and grass swales. Unlike other water quality models, SLAMM focuses on small storm hydrology and pollutant washoff, which is large contributor to urban stream water quality problems. SLAMM computations are based on field observations, as opposed to theoretical processes.
SLAMM can be used in conjunction with more commonly used hydrologic models to predict pollutant sources and flows.

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APPENDIX

H

GEORGIA SAFE DAM RULES

(Georgia Department of Natural Resources – Environmental Protection Division
Rules Chapter 391-3-8)

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RULES
OF
DEPARTMENT OF NATURAL RESOURCES
ENVIRONMENTAL PROTECTION DIVISION

CHAPTER 391-3-8
RULES FOR DAM SAFETY

TABLE OF CONTENTS

391-3-8-.01 Purpose. Amended. The Purpose of these Rules is to implement the responsibilities assigned to the Environmental Protection Division by the Georgia Safe Dams Act of 1978; Part 3 of Article 5 of O.C.G.A. §§ 12-5. These Rules are promulgated to provide for the inventory, classification, inspection and permitting of certain dams in order to protect the health, safety, and welfare of all the citizens of the State by reducing the risk of failure of such dams to prevent death or injuries to persons.

Authority O.C.G.A. Secs. 12-5-370 through 12-5-385. **Administrative History**. Original Rule entitled "Purpose" was filed as Emergency Rule 391-3-8-0.5 -.01 on August 28, 1978: effective July 28, 1978, the date of adoption, to remain in effect for a period of 120 days or until the effective date of a permanent Rule covering the same subject matter superseding said Emergency Rule, as specified by the Agency. **Amended**: Permanent Rule of the same title adopted superseding Emergency Rule 391-3-5-0.5-0.1. Filed August 31, 1978; effective September 20, 1978. **Amended:** Rule repealed and a new Rule of the same title adopted. Filed October 29, 1985; effective November 18, 1985. **Amended:** F. August 31, 1990; eff. September 20, 1990.

391-3-8-.02 Definitions. **Amended**. For the purpose of these rules and regulations, the term:

(a) "abutment" means the bordering area of the dam site which functions as a support for the ends of the dam structure.

(b) "Act" means the "Georgia Safe Dams Act of 1978" as amended (O.C.G.A. 12-5-370 through 12-5-385).

(c) "appurtenant works" means such structures as spillways, either in the dam or separate therefrom; the reservoir and its rims; low level outlet works; access bridges; and water conduits such as tunnels, pipelines or penstocks, either through the dam or its abutments.

(d) “Category I” means the classification where improper operation or dam failure would result in probable loss of human life. Situations constituting “probable loss of life” are those situations involving frequently occupied structures or facilities, including, but not limited to, residences, commercial and manufacturing facilities, schools and churches.

(e) “Category II” means the classification where improper operation or dam failure would not expect to result in probable loss of human life.

(f) “conduit” means any closed waterway such as a cast-in-place cut-and-cover culvert, a precast or prefabricated pipe imbedded in the dam or foundation of tunnel bored through the abutment used for the purpose or regulating or releasing water impounded by a dam.

(g) “construct” or “construction” means the building, removal or modification of any artificial barrier, together with appurtenant works for the impoundment or diversion of water or liquid substances and shall include any activity which, other than routinely as part of an approved maintenance program, repairs, removes, or restores such artificial barrier, or alters its design, shape or structural characteristics, and shall also include any enlargement of such artificial barrier.

(h) “dam” means, with exception of the exemptions outlined in Rule 391-3-8-.04 herein, the following:

1. Any artificial barrier, including appurtenant works, which impounds or diverts water and which the improper operation or failure of such would result in probable loss of human life as determined pursuant to the Act, and which

(i) is twenty-five (25) feet or more in height from the natural bed of the stream or water course measured at the downstream toe or the lowest elevation of the outside limit of the barrier (whichever is lower) to the maximum water storage elevation; or

(ii) has an impounding capacity at maximum water storage elevation of one hundred (100) acre-feet or more.

2. Any artificial barrier, including appurtenant works, constructed in conjunction with the reclamation of surface mined land, and meeting the requirements of subsection 1., above and when improper operation or failure would result in probable loss of human life.

(i) “small dam” means any artificial barrier meeting the requirements of subsection (h) above with a storage capacity not exceeding 500 acre-feet and a height not exceeding 25 feet.

(j) “medium dam” means any artificial barrier meeting the requirements of subsection (h) above with a storage capacity exceeding 500 acre feet but not exceeding 1000 acre-feet or a height exceeding 25 feet but not exceeding 35 feet.

(k) "large dam" means any artificial barrier meeting the requirements of subsection (h) above and with a storage capacity exceeding 1000 acre-feet but not exceeding 50,000 acre-feet or a height exceeding 35 feet but not exceeding 100 feet.

(l) "very large dam" means any artificial barrier meeting the requirements of subsection (h) above and with a storage capacity exceeding 50,000 acre-feet or a height exceeding 100 feet.

(m) "engineer" means the State Conservation Engineer of the Natural Resources Conservation Service or the engineer of record.

(n) "engineer of record" - means an individual who:

1. Is a licensed engineer registered with the State of Georgia; and
2. Is competent and has relevant experience in areas related to dam investigation, inspection, design, and construction for the type of dam being investigated, inspected, designed, or constructed ; and
3. Understands adverse dam incidents, failures and the potential causes and consequences of dam failures; and
4. Will have responsible charge for the design of a new Category I dam or repair of an existing Category I dam; and
5. Has substantiated their qualifications to the Georgia Safe Dams Program prior to their engagement by an Owner/Operator of an existing or proposed Category I Dam.

(o) "flood control pool" means the storage volume of the entire reservoir at the crest of the emergency spillway.

(p) "flood control zone" means the storage volume available between the normal pool and the flood control pool.

(q) "foundation" means the earth or rock on which the dam rests.

(r) "freeboard" means the difference in elevation between the top of the dam and the maximum reservoir water surface that would result should the inflow design flood occur and should the outlet works function as planned.

(s) "hydrometeorological gauges" means any variety of measuring devices used in determining data concerning rainfall, snow, fog, dew, etc.

(t) "impoundment" means the water or liquid substance that is or will be stored by a dam - commonly referred to as the reservoir.

(u) "maximum water storage elevation" means the elevation of the lowest point of the top of the impoundment structure independent of low points caused by partial failure or collapse.

(v) "normal pool" means the reservoir storage volume at normal storage elevation.

(w) "normal water storage elevation" means the normal elevation of water surface which is obtained by the reservoir when the intake and outlet works are operating as planned during periods of normal precipitation and runoff and not during periods of drought or flood.

(x) "principal spillway" means the spillway which conveys normal runoff out of the reservoir.

(y) "probable maximum precipitation (PMP)" means the greatest amount of rainfall of a six-hour duration which would be expected for a given drainage basin as determined by Hydrometeorological Report No. 52 published by the U.S. Weather Bureau.

(z) "spillway" means the feature of a storage or detention dam which is designed to released surplus water which cannot be contained in the allotted storage space, and at diversion dams is a means to bypass flows exceeding those which are turned into the diversion system.

(aa) "structural height" means the height of the dam measured from the lowest point of the dam's foundation to the highest point on the top of the dam.

(bb) "surcharge zone" means the reservoir of storage volume located between the crest of the emergency spillway (flood control pool) and the maximum water storage elevation.

(cc) "surface mining" means any activity constituting all or part of a process for the removal of mineral ores and other solid matter for sale or for processing or for consumption in the regular operation of a business. However, the removal of mineral ores and other solid matter by tunnels, shafts, and dimension stone quarries shall not be considered surface mining.

Authority Ga. L. 1978, pp. 795-807, et seq.: O. C. G. A. Secs. 12-5-370 through 12-5-385. **Administrative History**. Original Rule entitled "Definitions" was filed as Emergency Rule 391-3-8-0.5-.02 on August 28, 1978, effective July 28, 1978, the date of adoption, to remain in effect for a period of 120 days or until the effective date of a permanent Rule covering the same subject matter superseding said Emergency Rule, as specified by the Agency. **Amended:** Permanent Rule of the same title adopted superseding Emergency Rule 391-3-8-0.5-.02. Filed August 31, 1978; effective September 20, 1978. **Amended**: Filed October 29, 1985; effective November 18, 1985. **Amended:** F. August 31, 1990; eff. September 20, 1990.

391-3-8-.03 Inventory and Classification. Amended.

(1) It shall be the duty of the Director to inventory the dams in this state and to classify each dam into one of the following categories:

(a) Category I Dam

(b) Category II Dam

(2) The inventory shall consist of all dams not excluded under Rule 391-3-8-.04 subsections (a), (b), (c), (e) and (f).

(3) The inventory and classification of dams including proposed structures shall be carried out in accordance with the Safe Dams Program Quality Assurance Plan.

(4) When an existing Category II dam may be reclassified to a Category I dam because of proposed development downstream of the dam, the governing authority issuing the permit for the development shall provide for review by the Safe Dams Program the following information:

(a) location of the Category II dam and the proposed development; and

(b) a surveyed cross-section of the stream valley at the location of the proposed development including proposed finished floor elevations; and

(c) a dam breach analysis using the Dambreak computer model to establish the height of the floodwave in the downstream floodplain. The dambreak modeling shall be completed by an engineer in accordance with the Safe Dams Program Quality Assurance Plan.

(5) If the Director determines that an existing Category II dam will be reclassified to a Category I dam if the proposed development occurs, then the owner of the existing Category II dam may request an inspection from the Director within ten (10) days of notification of the proposed development by the local governing authority. The preliminary visual inspection shall be carried out in accordance with subsection (2) paragraph (b) and subsection (3), paragraph (d) , subparagraph, (ii) (l) and (2) under Rule 391-3-8-.08. Detailed surveys, hydrologic and hydraulic analyses will not be performed, however the Director may provide an opinion on the hydraulic adequacy of the dam.

(6) A written evaluation of the existing Category II dam's compliance with Category I requirements will be provided to the owner of the dam and the local governing authority based on the preliminary visual inspection by the Safe Dams Program.

Authority O.C.G.A. Secs. 12-5-370 through 12-5-385. **Administrative History**. Original Rule entitled "Scope and Exclusions" was filed as Emergency Rule 391-3-8-0.5-.03 on August 28, 1978, effective July 28, 1978, the date of adoption to remain in effect for a period of 120 days until the effective date of a permanent Rule covering the same subject matter superseding said Emergency Rule as specified by the Agency. **Amended:** Permanent Rule of the same title adopted superseding Emergency Rule 391-3-8-0.5-.03. Filed August 31,1978; effective September 20, 1978. **Amended**: Rule renumbered as Rule 391-3-8-.04 and a new rule 391-3-8-.03 entitled "Inventory and Classification" adopted. Filed October 29, 1985; effective November 18, 1985. **Amended:** F. August 31, 1990; eff. September 20, 1990.

391-3-8-.04 Scope and Exclusions. Amended. These rules and regulations shall apply to any dams or artificial barriers existing or constructed in Georgia except for the following:

(a) any dam owned and operated by any department or agency of the United States government;

(b) any dam constructed or financially assisted by the United States Natural Resources Conservation Service or any other department or agency of the United States government when such department or agency designed or approved plans and supervised construction and maintains a regular program of inspection of the dam; provided, however, that this exemption shall cease on November 1, 2000, for all such dams which the supervising federal agency has relinquished authority for the operation and maintenance of such dam to a person unless the supervising federal agency certifies by said date and at least biannually thereafter to the Director that such dams are in compliance with requirements of this part, including minimum spillway design, and with the maintenance standards of the supervising federal agency;

(c) any dam licensed by the Federal Energy Regulatory Commission, or for which a license application is pending with the Federal Energy Regulatory Commission;

(d) any dam classified as a Category II Dam;

(e) any artificial barrier, except as provided in Rule 391-3-8-.02 (h), constructed in connection with and incidental to surface mining, provided that upon completion of mining the impoundment created by the barrier is drained and reclaimed or stabilized as a lake pursuant to a mined land use plan approved by the Director pursuant to the Georgia Surface Mining Act;

(f) any artificial barrier which is 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.

Authority O.C.G.A. Secs 12-5-370 through 12-3-385. **Administrative History**. Original Rule entitled "Application for a Permit" was filed as Emergency Rule 391-3-0.5-.04 on August 28, 1978, effective July 28, 1978, the date of adoption to remain in effect for a period of 120 days or until the effective date of permanent Rule covering the same subject matter superseding this Emergency Rule, as specified by the Agency. **Amended**: Permanent Rule of the same title adopted superseding Emergency Rule 391-3-8-0.5-.04. Filed August 31, 1978; effective September 20, 1978. **Amended:** Rule renumbered as rule 391-3-8-.05 and Rule 391-3-8-.03 entitled "Scope and Exclusions" repealed and a new Rule of the same title adopted as Rule 391-3-8-.04. Filed October 29, 1985; effective November 18, 1985. **Amended:** F. Aug 31, 1990: eff. Sept, 20, 1990.

391-3-8-.05 Application for a Permit. Amended.

(1) No person shall operate or construct a dam as defined by the Act and these Rules without first having obtained a permit from the Division; provided, however, any person who is operating a dam may continue such operation or construction pending final action by the Director on the permit application and provided such application has been filed with the Director within 180 days after notice by the Director that permit is required.

(2) Permit application shall be on forms as may be prescribed and furnished by the Division.

(3) The Director may require the submission of plans, specifications and other information deemed relevant to the application.

(4) If a permit application for the construction of a dam is not approved by the Director, the application shall be returned to the applicant along with the reasons for its disapproval. Such applicants may reapply for said permit by correcting deficiencies in the application and resubmitting the application to the Director.

(5) Permits shall not be transferred from one person to another without the approval of the Director. If the ownership changes from one person to another, the new owner shall immediately notify the Director in writing of such transactions. The Director shall also be notified of any proposed change in the operation of the dam.

(6) Permits shall not be transferred from one dam to another dam.

Authority O.C.G.A. Secs. 12-5-370 through 12-5-385. **Administrative History**. Original Rule entitled "Revocation, Suspension or Modification of Permits" was filed as Emergency Rule 391-3-8-0.5-.05 on August 28, 1978, effective July 28, 1978, the date of adoption, to remain in effect for a period of 120 days or until the effective date of a permanent Rule covering the same subject matter superseding this Emergency Rule, as specified by the Agency. **Amended**: Permanent Rule of the same title adopted superseding Emergency Rule 391-3-8-0.5-.05. Filed August 31, 1978; effective September 20, 1978. **Amended**: Rule renumbered as Rule 391-3-8-.06 and Rule 391-3-8-0.5-.06 and Rule 391-3-8-.04 entitled "Application for Permit" amended and adopted as Rule 391-3-8-.05. Filed October 29, 1985; effective November 18, 1985. **Amended**: F. August 31, 1990; eff. Sept 20, 1990.

391-3-8-.06 Revocation, Suspension or Modification of Permits. Amended. Permits may be revoked, suspended, modified, or denied by the Director for cause including but not limited to the following:

(a) violation of any permit condition;

(b) failure to fully disclose all relevant facts or obtaining a permit through misrepresentation;

(c) violations of the Act or these Rules;

(d) changes in conditions that require such action on a permit in order to insure compliance with the Act or these Rules.

Authority O.C.G.A. Secs 12-5-370 through 12-5-385. **Administrative History**. Original Rule entitled "Dam Removal" was filed as Emergency Rule 391-3-8-0.5-.06 on August 28, 1978, effective July 28, 1978, the date of adoption, to remain in effect for a period of 120 days or until the effective date of a permanent Rule covering the same subject matter superseding said Emergency Rule, as specified by the Agency. **Amended:** Permanent Rule of the same title adopted superseding Emergency Rule 391-3-8-0.5.-.06 Filed August 31, 1978; effective September 20, 1978. **Amended**: Rule renumbered as Rule 391-3-8-.07 and Rule 391-3-8-.05 entitled " Revocation, Suspension or Modification of Permits" renumbered as Rule 391-3-8-.06. Filed October 29, 1985;

effective November 18, 1995. **Amended:** F. Aug 31, 1990; eff. September 20, 1990.

391-3-8-.07 Dam Removal. Amended. No person may remove a dam without the approval of the Director in accordance with the procedures required by Section 8 of the Act.

Authority O. C. G. A. Secs 12-5-370 through 12-5-385. **Administrative History**. Original Rule entitled "Permits for Operation of Existing Dams" was filed as Emergency Rule 391-3-8-0.5-.07 on August 28, 1978, effective July 28, 1978, the date of adoption, to remain in effect for a period of 120 days or until the effective date of a permanent Rule covering the same subject matter superseding said Emergency Rule, as specified by the Agency . **Amended:** Permanent Rule of the same title adopted superseding Emergency Rule 391-3-8-0.5-.07. Filed August 31, 1978; effective September 20, 1978. **Amended:** Rule renumbered as Rule 391-3-8-.08 and Rule 391-3-8-.06 entitled "Dam Removal" renumbered as Rule 391-3-8-.07. Filed October 29, 1985; effective November 18, 1985. **Amended:** F. August 31, 1990; eff. Sept. 20, 1990.

391-3-8-.08 Permits for the Construction and/or Operation of New and Existing Dams. Amended.

(1) New Dams

(a) Applications for a permit to construct and operate a Category I dam shall be accompanied by a statement from an engineer who provides engineering design services for the dam, certifying that he/she has the necessary training and experience to design such dam, and that to the best of his/her knowledge, understanding and belief such design meets the standards of the Act and these Rules. If the design engineer determines that a geological investigation of the dam is advisable, such investigation shall be conducted by a professional geologist registered to practice in the State of Georgia.

(b) As an alternative to a certificate from an engineer, the Director may accept a permit application accompanied by a certificate from the State Conservation Engineer of the Natural Resources Conservation Service stating that the design of the dam meets the standards of this Act and the rules and regulations promulgated hereunder.

(c) Construction of such dams shall be completed in the time frame stated in the special conditions of the Construction and Operation Permit.

(d) Notice by registered mail shall be given to the Director at least 10 days prior to the commencement of construction for permitted dam construction activities.

(2) Existing Dams

(a) Permits for the operation of dams in existence may be issued provided the application for a permit is judged complete and meets the requirements of the Act and these Rules.

(b) When a visual inspection, performed by an engineer, reveals that abnormal stress exists or that the dam was not constructed in accordance with the requirements of the Act or these Rules, a detailed engineering survey meeting the requirements of this section shall be performed prior to final action on the permit application. Such visual inspection may be provided by the applicant, in accordance with Section 12-5-376(g) of the Act, or by the Division, or by another authorized agency under contract with the Director on behalf of the Division.

(3) Applications for permits for existing or proposed dams shall include the following evaluations and information when such information is relevant and available as determined by the Director:

(a) A regional vicinity map showing the location of the existing or proposed dam and the latitude and longitude of the center of the dam expressed to the nearest second, the watershed drainage area, and the downstream area subject to potential damage due to failure or misoperation of the dam or operation equipment including other artificial barriers or downstream fixed improvements which would be affected;

(b) A detailed description of the existing or proposed dam including:

(i) Proposed or as-built drawings indicating plans, elevations and sections of the dam and appurtenant works including details of the discharge facilities such as outlet works, limited service and emergency spillways, flashboards, fuse plugs and other operation equipment;

(ii) the elevation of the top and lowest outside limit of the dam, and the elevation of the lowest upstream and downstream toe;

(iii) the profile of the top of the dam and the dam's structural height;

(iv) the maximum and normal storage elevation, hydraulic heights and freeboard and storage capacity associated with each;

(v) the surface area of the impoundment;

(vi) the top and bottom width of the dam;

(vii) the elevation of the crest, type, width or diameter; length and location of spillways and the number, size and type of gates if the structure is controlled;

(viii) the type, location, entrance and exit inverts of outlet works, and emergency drawdown facilities;

(ix) the location, crest elevation, and description of the invert, sides, and length of limited service and emergency spillways;

(x) the location, and description of flashboards and fuse plugs, including hydraulic head(pool elevation) and other conditions required for breaching along with the assumed results of breaching;

(xi) the type, location, observations and records of hydrometeorological gauges appurtenant to the project;

(xii) the maximum non-damaging discharge causing only negligible damage at potential damage locations downstream;

(xiii) the location and description of any proposed or existing instrumentation including, but not limited to, observation wells, piezometers, settlement devices, seepage outlets and weirs;

(xiv) the location, elevation and description of areas affected by reservoir fluctuation.

(c) Design and safety evaluation reports including:

(i) a hydrological analysis of the proposed or existing dam, reservoir, drainage basin system including computation of the basin P.M.P. or the design storm event, average watershed slope, watershed area, hydrologic soil groups, land use of impoundment watershed, reservoir inflow hydrograph, spillway and exit water-surface profiles, flow rate, expected frequency of emergency spillway use and minimum freeboard;

(ii) analysis and/or evaluation of the proposed or existing dam that indicates that the dam will be stable during construction (new dams), filling (new dams) and under all conditions of reservoir operations including assumed material properties and all pertinent applied loads;

(iii) evaluation of seepage and measures taken to control seepage through the embankment, foundation, and abutments so that no internal erosion will take place and that there will be no significant sloughing in the area where the seepage emerges;

(iv) evaluation of the geology of the site and foundation including any boring logs or laboratory testing with engineering conclusions, foundation data, geological maps, profiles and cross sections, foundation treatment, and any relevant seismic information;

(v) evaluation of materials in the foundation and embankment including results of any laboratory tests, field permeability tests, construction control tests, and assumed design or evaluation properties of materials;

(vi) the properties of concrete including source or proposed source of aggregate, mix design, type of cement and additives, and the result of testing during construction;

(vii) evaluation or design of cover (vegetation, masonry, or riprap) to protect the upstream slope, crest, and downstream slope of the dam and abutments against erosion from wind, waves and runoff;

(viii) the proposed water control plan, including the regulation plan under normal conditions and during flood or other emergency conditions;

(ix) analysis of the anticipated time required to completely drain the flood control zone and normal pool;

(x) the electric and mechanical equipment types and rating of normal and emergency power supplies, hoists, cranes, valves and valve operators, control and alarm systems, and other electrical and mechanical equipment systems that could affect the safe operation of the dam;

(xi) the spillway and tailwater rating curve below the dam site, including the elevation corresponding to the maximum design flood discharge and approximate nondamaging channel capacity;

(xii) evaluation and/or analysis of settlement estimates and steps adopted to compensate for total settlement and to minimize differential settlements;

(d) Other data requirements for new and existing dams:

(i) New Dams

1. the proposed method of construction and quality control provisions for the project, including the responsibilities of the applicant, the design engineer, the builder, and the prescribed order of the work;

2. the proposed dam construction schedule and filling schedule for the reservoir;

3. the proposed inspection and maintenance plan;

4. the proposed instrumentation and monitoring plan including the filling surveillance plan;

5. the estimated life of the dam and reservoir;

6. any other pertinent data as may be required by the Director;

(ii) Existing dams

1. detailed description of the condition of the dam and appurtenant works resulting

from a detailed visual inspection including a description of any signs of structural deterioration and seepage such as, but not limited to, surface cracks, settlement, structural condition of any conduits through the dam, and erosion;

2. the year of construction, and the date and description of any modifications or repairs;

3. the construction history including the diversion scheme, construction sequence, pertinent construction problems, alterations, modifications, and major maintenance repairs;

4. a summary of past major flood events or previous failures or known deficiencies, including any experiences that presented a threat to the safety of the project or to human life and any action taken to correct or eliminate such hazards;

5. the records of performance observations including instrumentation records;

6. the inspection history including the results of the last safety inspection, the organization that performed the inspection, and the date the inspection was performed;

7. Any other pertinent information as may be required by the Director.

Authority O. C. G. A. Secs. 12-5-370 through 12-5-385. **Administrative History**. Original Rule entitled "Design Standards for Existing Dams" was filed as Emergency Rule 391-3-8-0.5-.08 on August 28, 1978, effective July 28, 1978, the date of adoption, to remain in effect for a period of 120 days or until the effective date of a permanent Rule covering the same subject matter superseding Emergency Rule, as specified by the Agency. **Amended:** Permanent Rule of the same title adopted superseding Emergency Rule 391-3-8-0.5-.08. Filed August 31, 1978; effective September 20, 1978. **Amended**: Rule renumbered as Rule 391-3-8-.09 and Rule 391-38-.07 entitled "Permits for Operation of Existing Dams" renumbered as Rule 391-3-8-.08. Filed October 29, 1985: effective November 18, 1985. **Amended** F. Aug. 31. 1990: eff. Sept 20, 1990.

391-3-8-.09 Standards for the Design and Evaluation of Dams. Amended.

(1) The design and/or evlauation of new and existing dams shall conform to accepted practices of the engineering profession and dam safety industry. Design manuals, evaluation guidelines, and procedures used by the following agencies can be considered as acceptable design or evaluation references, except as those references differ from Georgia Law and these regulations:

(a) U.S. Army Corps of Engineers;
(b) Natural Resources Conservaion Service;
(c) U.S. Department of Interior, Bureau of Reclamation;
(d) Federal Energy Regulatory Commission;

(2) Other design and evaluation methods may be used to demonstrate compliance with the objectives of these rules, but are subject to the approval of the Director.

(3) Design and Evaluation of Dams under Paragraph (1) and (2) above shall, as a minimum, consider the following basic principles:

(a) All dams must be stable under all conditions of construction and/or operation of the impoundment. Details of stability evaluation shall be submitted to the Director for approval. Analyses using the methods, guidelines and procedures of the agencies listed in Paragraph (1) yielding the following Minimum Safety Factors can be considered as acceptable stability:

Earthen Embankments	
1. End of Construction	1.3
2. Steady State Seepage	1.5
3. Steady State Seepage with Seismic Loading	1.1
4. Rapid Drawdown (Upstream)	1.3
5. Submerged Toe with Rapid Drawdown	1.3

Concrete Structures (cohesion included)	
1. Normal Reservoir	3.0
2. Normal Reservoir with Seismic Loading	1.0
3. Design Flood	2.0

(b) Details of the engineering evaluation of material properties in the dam or appurtenant structures shall be submitted to the Director for review and approval. Conservative selections for soil strength values shall be used for analyses or evaluations. Details of any foundation investigation and laboratory testing supporting assumed design or evaluation parameters shall be included for review.

(c) All dams and appurtenant structures shall be capable of withstanding seismic accelerations defined in the most current "Map for Peak Acceleration with a 2% exceedance in 50 years" for the contiguous United States published by the United States Geological Survey (a.k.a. NEHRP maps). The minimum seismic acceleration shall be 0.05g . The seismic accelerations may be reduced or seismic evaluation eliminated if the applicant's engineer can successfully demonstrate to the Director by engineering analyses or judgment that smaller seismic accelerations are appropriate or no seismic evaluation is needed.

(d) All dams shall have a means of draining the reservoir to a safe level as demonstrated by the applicant's engineer. The submittal by the applicant's engineer shall include the computation of the maximum time required to drain the reservoir. Exceptions to this rule may be given by the Director based on an engineering evaluation demonstrating the lack of this capability would not endanger the public.

(e) All earthen embankments shall be protected from surface erosion by appropriate vegetation, or some other type of protective surface such as riprap or paving, and shall be maintained in a safe condition. Examples of appropriate vegetation include, but are not limited to, Bermuda, Tall Fescue, Centipede grasses and Lespedeza sericea. Inappropriate vegetation on existing dams such as trees shall be removed only after consultation with the Division or other qualified persons on the proper procedures for removal. Hedges and small shrubs may be allowed on existing dams if they do not obscure inspection or interfere with the operation and maintenance of the dam.

(f) Design Storm. Each dam shall be capable of safely passing the fraction of the flood developed from the PMP hydrograph depending on the subclassification of the dam. The design storm for each subclassification of a dam is as follows:

1. Small Dam	25 percent PMP
2. Medium Dam	33.3 percent PMP
3. Large Dam	50 percent PMP
4. Very Large Dam	100 percent PMP

Based on visual inspection and detailed hydrologic and hydraulic evaluation, including documentation of completed design and construction procedures, up to 10 percent lower requirement (22.5, 30, 45, 90) may be accepted on existing PL566 (including RC&D structures) and PL 534 Project Dams at the discretion of the Director, provided the project is in an acceptable state of maintenance. The design storm may be reduced on existing dams if the applicant's engineer can successfully demonstrate to the Director, by engineering analysis, that the dam is sufficient to protect against probable loss of human life downstream at a lesser design storm. Earth emergency spillways shall not function until the 50 year storm.

(g) Seepage Control. All dams shall be able to prevent the development of instability due to excessive seepage forces, uplift forces, or loss of materials in the embankment, abutments, spillway areas, or foundation. For new dams, seepage analysis for design, and inspection during construction shall be in sufficient detail to prevent the occurrence of critical seepage gradients.

(i) For new dams, the design shall include a seepage control method which meets the minimum acceptable safety standards, as determined by the Division. All internal drainage systems with pipe collection systems shall have cleanouts.

(ii) In existing dams, seepage shall be investigated by an engineer and appropriate control measures shall be taken as necessary.

(h) Monitoring Devices.

(i) Monitoring devices, including but not limited to piezometers, settlement plates, tell-tale stakes, seepage outlets and weirs, and permanent bench marks may be required by the Director for use in the inspection and monitoring of the safety of a dam during operation.

(ii) For new dams or existing dams where appropriate, a reservoir filling monitoring and surveillance plan to be implemented during reservoir filling or re-filling shall be submitted to the Director for approval prior to start of filling or re-filling.

(i). Design Life. The design life for proposed dams and reservoirs shall be adequate for the dams and reservoirs to perform effectively as planned, as determined by the following criteria:

(i) The time required to fill the reservoir with sediment from the contributing watershed; and

(ii) The durability of appurtenances and materials used to construct the dams.

(j) Freeboard. Appropriate freeboard for wave action shall be considered by an engineer through engineering analysis. The required freeboard shall be provided above the maximum reservoir surface elevation that would result from the inflow from the design storm for the structure. The resulting maximum reservoir surface elevation plus freeboard shall determine the elevation of the top of the dam. In lieu of determining the appropriate amount of freeboard by engineering analysis, a minimum of three (3) feet of freeboard shall be provided on earth dams.

(k) Existing concrete and/or masonry dams and appurtenant structures shall be structurally sound and shall have joints free of trees and other vegetation and shall show no signs of significant structural deterioration such as excessive cracks, spalling, efflorescence and exposed reinforcing steel.

(4). Other design standards may be imposed as deemed appropriate by the Director after review of design of new structures or through a visual inspection of an existing structure conducted pursuant to Rule 391-3-8-.08 (2)(b) of these regulations, or based on a review of the detailed engineering study prepared by an engineer.

Authority O. C. G. A. Secs. 12-5-370 through 12-5-385. **Administrative History**. Original Rule entitled "Permits for the Construction and Operation of New Dams" was filed as Emergency Rule 391-3-8-0.5-.09 on August 28, 1978, effective July 28, 1978, the date of adoption, to remain in effect for a period of 120 days or until the effective date of a permanent Rule covering the same subject matter superseding said Emergency Rule, as specified by the Agency. **Amended:** Permanent Rule of the same title adopted superseding Emergency Rule 391-3-5-0.5-.09. Filed August 31, 1978; effective September 20, 1978. **Amended**: Rule renumbered as Rule 391-3-8-.10 and Rule 391-3-8-.08 entitled "Design Standards for Existing Dams" amended and renumbered as Rule 391-3-8-.09. Filed October 29, 1985; effective November 18, 1985. **Amended:** F. August 31, 1990; eff. September 20, 1990.

391-3-8-.10 Inspection and Maintenance Plan Requirements. Amended. Dam Owners and operators of dams shall be responsible for conducting routine inspection and maintenance of dams necessary to:

(a) Prevent the growth of trees or brush on the embankment of the dam and on the spillway system;

(b) Prevent the accumulation of debris, obstructions, or other deleterious materials from the spillway system;

(c) Insure that all gates, orifices, dissipators, trash racks, and other appurtenances that affect the proper operation of the dam and reservoir are kept in good repair and working order, and that spillway and outlet gates necessary to pass flood flows shall be test operated at least once each year. The dam owner shall file an affidavit with the Director certifying that such appurtenances and gates are in good working order;

(d) Maintain adequate and suitable vegetation to prevent erosion of the embankment and earth spillway for the dam;

(e) Determine that any seepage on the downstream slopes of the dam does not exceed normal amounts and does not present a situation indicative of potential dam failure. At any time where there is a questions regarding seepage and potential dam failure, the Director shall be notified in writing and provided a description of the situation, and

(f) Dam owners shall immediately notify the Division when symptoms of failure, including but not limited to, erosion, surface cracks, seepage, settlement, or movement occur.

Authority O. C. G. A. Secs. 12-5-370 through 12-5-385. **Administrative History**. Original Rule entitled "Effective Date" was filed as Emergency Rule 391-3-8-8-0.5-.12 on August 28, 1978, effective July 28, 1978, the date of adoption, to remain in effect for a period of 120 days or until the effective date of a permanent Rule covering the same subject matter superseding said Emergency Rule, as specified by the Agency. **Amended:** Permanent Rule of the same title adopted superseding Emergency Rule 391-3-8-0.5-.12. Filed August 31, 1978; effective September 20, 1978. **Amended:** Rule repealed and Rule 391-3-8-.11 entitled "Inspection and Maintenance Plan Requirements" renumbered as Rule 391-3-8-.12. Filed October 29, 1985; effective November 18, 1985. **Amended:** F. Aug 31, 1990; eff. Sept 20, 1990.

391-3-8-. 11 Effective Date. This Chapter shall become effective on October 26, 1998

Authority O. G. C. A. Secs. 12-5-370 through 12-5-385. Original Rule entitled "Effective Date' adopted F. Aug. 31, 1990; eff. Sept 20, 1990.