Town of Bluffton

Stormwater Design Manual

Effective Date November 10, 2011

www.townofbluffton.sc.gov

Town of Bluffton
Stormwater Design Manual

Introduction i

Chapter 1: Policy and Permit Requirements 1-1

Chapter 2: Hydrology and Runoff Determination 2-1

Chapter 3: Storm Sewer Collection Systems 3-1

Chapter 4: Culverts and Bridges 4-1

Chapter 5: Open Channel 5-1

Chapter 6: Storage Facilities 6-1

Chapter 7: Structural Best Management Practices 7-1

Chapter 8: Water Quality Analysis 8-1

Chapter 9: Erosion and Sedimentation Control 9-1

This project was funded wholly or in part by the US Environmental Protection Agency (US EPA) under a Section 319 Grant through the SC Department of Health and Environmental Control (SC DHEC), as well as the National Oceanic and Atmospheric Administration (NOAA), Office of Ocean and Coastal Management (OCRM), and SC DHEC through a SC Coastal Nonpoint Program Capacity Building Grant for Local Governments by award number NA09NOS4190092.

PURPOSE

Town of Bluffton (Town) is located in an important coastal area that offers a wide range of historical, cultural, and recreational attractions. The Town's residents desire to maintain the character of their communities and sustain and improve the excellent quality of life that the area provides. Equally important to the residents of the Town is the protection of the environment that helps the Town remain the unique place that it is. Preservation of the Town's unique aquatic ecosystems, especially the May, Okatie, Colleton, and New River watersheds are particularly important.

It is the purpose of the Town Stormwater Management Program (Program) to meet the current and anticipated needs of the Town. As part of it duties, the Department of Environmental Protection administers stormwater services for the protection of public and private properties, rivers, estuaries, and other water bodies from the potential damage resulting from uncontrolled stormwater releases and non-point source pollution.

The Town's Stormwater Design Manual (Manual) is a comprehensive tool for the application of stormwater management controls and stormwater Best Management Practices (BMPs) aimed at improving the water quality of stormwater. It contains rules and minimum requirements for the planning, design, construction, operation, and maintenance of all drainage facilities within Town and all areas subject to its jurisdiction. Definitions, formulas, criteria, procedures, and data presented herein have been developed to support the Unified Development Ordinance Article 5. If a conflict arises between the technical data and the Ordinance, the Ordinance shall govern.

GOALS AND OBJECTIVES

The goals and objectives of the Program and the Manual are as follows:

1. Protect human life and health.
2. Prevent new development and redevelopment from creating a demand for public investment in flood-control and water quality improvement works.
3. Improve the quality of stormwater runoff discharge to surface waters and groundwater.
4. Provide an effective stormwater management system that will not result in excessive public or private funds being used for maintenance and replacement of existing components of the stormwater system.
5. Facilitate the design of drainage systems that are consistent with good engineering practice and design, water quality protection, and in accordance with the Town's overall planning efforts and stormwater management planning.
6. Provide a mechanism that allows for development with minimum adverse effects to the natural environment.
7. Encourage preservation of the natural drainage systems in an aesthetically pleasing condition as best possible.
8. Minimize private and public property damage resulting from erosion, sedimentation, and flooding.
9. Ensure Town compliance with all applicable Federal and State regulations.

JURISDICTION

Upon adoption of the Stormwater Management Ordinance by Town Council, the specifications and standards contained herein shall be applicable to all development and redevelopment within the regulatory jurisdiction of Town.

DISCLAIMER

This Manual is established to provide the Town, its Administrator(s), their duly appointed representatives, property owners, developers, engineers, surveyors, and builders a better understanding of acceptable engineering methods to meet the intent of the Town 's Program and Stormwater Management Ordinance. Design of stormwater management for development requires the experienced judgment of the designer. The Town accepts no responsibility for any loss, damage or injury as a result of the use of the Manual.

The application of the Stormwater Management Ordinance and the provisions expressed herein are the minimum stormwater management requirements and shall not be deemed a limitation or repeal of any other powers granted by statute. In addition, if site characteristics indicate that complying with the Town's minimum stormwater management requirements will not provide adequate design or protection for local property or residents, the Town, as part of its review process will require the owner and operator of these facilities to exceed the minimum stormwater management practices, control techniques design and engineering methods and such other programs and controls as are required.

1.0 DESIGN SPECIFICATIONS

This Manual and the design criteria presented within represent good engineering practices and should be utilized in the preparation of stormwater management plans. The criteria are intended to establish requirements, minimum standards, and methods for sound planning, design, and review process. Alternative methods of design may be submitted to the Town for its consideration if it can be demonstrated that the design meets the intent of the Unified Development Ordinance, Article 5.

> "**Stormwater Management Plan**" means the set of drawings and other documents that comprise all of the information and specifications for the programs, drainage systems, structures, BMPs, concepts, and techniques for the control of stormwater.

The design criteria shall be revised and updated as necessary to reflect advances in drainage engineering and urban stormwater management. The Town and its design professionals will utilize the Manual in the planning of new facilities and in their review of proposed work done by developers, private parties, and other governmental agencies.

The strict application of this Manual in the overall planning of new development is protective of water quality and can be practically applied. In the planning of drainage and water quality improvements, the use of the criteria and standards herein may be adjusted as determined by the Town.

1.0.1 Minimum Design Criteria

(a) Each chapter contained in this Manual illustrates and describes in detail the design criteria established for a particular element of a post-construction stormwater system. The design professional is referred to these chapters for further direction of the design criteria and its application.

(b) The design and management of construction site runoff control measures for all qualifying developments as defined in the Ordinance shall be in accordance with SCDHEC NPDES General Permit for Stormwater Discharges from Large and Small Construction Activities, the SCDHEC Erosion and Sediment Reduction and Stormwater Management regulations and its most current version of standards, where applicable. The Town reserves the right to require erosion and sediment control at lower level of application and additional or higher standard of measures and make their requirement a condition of a development permit approval. The design professional should review Chapter 9 for further explanation.

1.0.2 Scope of Stormwater Management Plan

(a) For land disturbing activities, regardless of size, all of the requirements of a stormwater management plan apply, except as detailed in item (b) below.

(b) For single-family residential development lots, which are not part of a larger common plan of development, the person responsible for the land disturbing activity shall conform to the Residential Stormwater requirements as defined below. Submittal of a stormwater management plan is not required. By obtaining a Town Building Permit, the property owner grants the right to the Administrator(s) to conduct on-site inspections. Residential Stormwater requirements are as follows:

Residential Stormwater Requirements

1. Temporary vegetative and structural stormwater management control measures shall be in place prior to land disturbance activity, and shall conform to the requirements of the Manual and the SCDHEC Erosion and Sediment Reduction and Stormwater Management regulations.

2. Permanent vegetative and structural stormwater management control measures shall be in place prior to receiving Certificate of Occupancy. Runoff control measures and practices single-family residential development lots, which are not part of a larger common plan of development, shall include at a minimum:

    a. Permeable pavement for driveways, patios and other impervious areas where suitable soils exist. Excess runoff from permeable pavement areas should be routed to vegetated and/or landscaped areas prior to leaving the development.
    b. Disconnection of roof drains and gutters vegetative areas for infiltration.
    c. Diversion of all runoff to vegetated and/or landscaped areas prior to leaving the development.

(c) In developing stormwater management plans for residential subdivisions, each individual lot in a residential subdivision development shall be required to obtain and comply with the subdivision's overall stormwater management plan, including specified structural BMPs for addressing stormwater quality. Hydrologic parameters that reflect the fully built-out subdivision development shall be used in all engineering calculations.

(d) If individual lots or areas in a residential subdivision are being developed by different property owners or developers, all land disturbing activities related to the residential subdivision shall be covered by the approved Development Plan for the residential subdivision.

1.0.3 Minimum Runoff Control Requirements

The following outlines the general requirements for controlling stormwater runoff rate and pollutant discharge.

1. For single-family residential development lots, which are not part of a larger common plan of development:
    a. Control of the peak runoff discharge and an engineered collection and conveyance of stormwater is not required, if the General Requirements of the Ordinance and the Residential Stormwater Requirements of this Chapter are met.

2. For all other developments (including commercials, residential subdivision, multi-family residential, etc).

    For developments less than 20 acres:
    a. Control post development peak runoff discharge to pre-development runoff rates for the 2-, 10-, and 25-year storm events.
    b. The 100-year storm event must be accommodated through the development without causing damage to structures or exceeding the flood inundation limits allocated for this storm event. Engineered collection and conveyance of stormwater for more frequent storm events must designed to the criteria established in the Chapters of this Manual.

c. Post construction water quality control shall meet the water quality performance standards by implementing approved BMPs that meet targeted goals for BMP pollutant removal efficiency. The attainment of water quality performance standards shall be demonstrated through a specified calculation methodology submitted by the developer and approved by the Town. The specified calculation methodology for these developments is detailed in the Chapter 8 of this Manual.

For developments greater than or equal to 20 acres:

a. Control post development peak runoff discharge to pre-development runoff rates for the 2-, 10-, and 25-year storm events.
b. The 100-year storm event must be accommodated through the development without causing damage to structures or exceeding the flood inundation limits allocated for this storm event. Engineered collection and conveyance of stormwater for more frequent storm events must designed to the criteria established in the Chapters of this Manual.
c. Post construction water quality control shall meet the water quality performance standards by implementing BMPs that provide for a post development pollutant discharge equal to or less than the pre-development pollutant discharge. A Modeling Plan, submitted by the Developer and approved by the Town, shall demonstrate the attainment of water quality performance standards through the use of BMPs. The Modeling Plan submittal shall include the following but not necessarily be limited to: an explanation of the analysis approach, demonstration that pre-development stormwater loads are not exceeded, identification of pollutants and effectiveness of BMPs, description of model(s) to be used and methodology for model application, expected range of accuracy in result prediction, and sources of all data to be used for modeling.

1.1 SUBMITTAL REQUIREMENTS

1.1.1 Stormwater Plan Requirements

Stormwater management plans are an essential component of the general Development Plan and Application. Stormwater management plans are to be submitted to the Town at the time of the Development Plan Application; they alone do not constitute the entire Development Plan. Stormwater management plans shall include as a minimum the following:

(a) A vicinity map indicating a north arrow, scale, boundary lines of the site, and other information necessary to locate the development site.
(b) Description of the existing and proposed topography of the development site except in the case of individual lot grading plans in single-family subdivisions.
(c) Site map of physical improvements on the site including both existing and proposed development.
(d) Natural Resource Conservation Service Soil Survey map indicating a north arrow, scale, and boundary lines of the site.
(e) Location, dimensions, elevations, and characteristics of all stormwater management facilities.
(f) Identification of all areas within the site included in the land disturbing activities and documentation of the total disturbed area calculations.
(g) The location of temporary and permanent vegetative and structural stormwater management control measures.

(h) Anticipated starting and completion dates of the various stages of land disturbing activities and the expected date the final stabilization will be completed. Location of temporary and permanent vegetative and structural stormwater management control measures for each respective stage of construction.
(i) A determination that the development is in compliance with the Stormwater Management and Flood Damage Protection requirements of the Unified Development Ordinance, Article 5.
(j) Designation of all easements (rights-of-way) needed for inspection and maintenance of the drainage systems and stormwater management facilities.
(k) BMPs to control the water quality of the runoff during the land disturbing activities and during the life of the development.
(l) Proposed site(s) for water quality testing and monitoring.
(m) Construction and design details for structural stormwater controls.
(n) Certification by the person responsible for the land disturbing activity that the activity will be accomplished according to the approved stormwater management plan and those responsible personnel will be assigned to the project.

1.1.2 Narrative

A narrative shall be included with the stormwater management plan submitted for review. This narrative shall detail, in addition to the items stated above, the general intent of the development highlighted on the proposed development plans. If the development is to be phased, a detailed description of the proposed phases should be included. The narrative should also describe the proposed stormwater management system detailing the measures included in the system such as detention, infiltration, or filtration controls and the function of each. Description of site conditions around points of all surface water discharge including vegetation and method of flow conveyance from the land disturbing activity shall be included in the narrative.

1.1.3 Design Calculations

Drainage areas contributing to each inlet, pipe, culvert, ditch, or swale shall be delineated and tabulated. Existing stormwater conveyance systems shall be shown on the drawings with details and capacities of each system included with the calculations. All engineering calculations needed to design the system and associated structures shall be submitted, including pre- and post-development flow velocities, peak rates of discharge, and inflow and outflow hydrographs of stormwater runoff at all existing and proposed points of discharge from the site.

In determining downstream effects from stormwater management structures and the development, hydrologic-hydraulic engineering studies may extend downstream to a point as determined by the Administrator(s). All stormwater management facilities and all major portions of the conveyance system through the proposed development (i.e., channels and culverts) will be analyzed, using the design and 100-year storms, for design conditions and operating conditions that can reasonably be expected during the life of the facility. The expected timing of flood peaks through the downstream drainage system should be assessed when planning the use of detention facilities. The results of the analysis shall be included in the hydrologic-hydraulic portion of the design study.

If the stormwater management plan and/or report indicate that there may be a drainage or flooding problem at the exit from the proposed development or at any point downstream as determined by the Town Administrator(s), the Administrator(s) may require:

1. Water surface profiles plotted for the pre- and post-development conditions for design storm frequencies from the 2-, 10-, 25-, and 100-year events.

2. Elevations and description of all structures potentially affected by the design storm frequencies from the 2-, 10-, 25-, and 100-year events.

1.1.4 Approvals

The stormwater management plan will not be considered approved without the inclusion of the Division of Stormwater Management (Division) approval stamp, signature, and date. The stamp of approval on the plans is solely an acknowledgement of satisfactory compliance with the requirements of these regulations. The approval stamp does not constitute a representation or warranty to the applicant or any other person concerning the safety, appropriateness or effectiveness of any provision, or omission from the stormwater management plan.

Prior to the Planning and Growth Management Department issuing a Development Permit, other certifications and permits as required for the plan shall be submitted to their respective agencies for review and approval. A copy of the most recent approval of those certifications and permits should be submitted to the Division for filing with the plan.

1.2 OPERATION AND MAINTENANCE PLAN

The effectiveness of each of the BMPs and stormwater management facilities described in the proceeding sections depends upon appropriate application of design and regular maintenance. Many of the health and safety concerns that may arise when the BMPs or stormwater management facilities are installed can be addressed by a scheduled maintenance plan. Therefore, the stormwater management plan must contain a maintenance plan component, including schedule, for each BMP and facility incorporated into the stormwater system. The maintenance plan must address both maintenance and monitoring procedures as outlined in Article 5 of the Ordinance which, when followed, will limit:

- Conditions of blocking, hindering or obstructing the natural or intended flow of surface waters;
- Improper operation of stormwater detention or impoundment device or any structure or device used for the improvement of the quality of surface runoff;
- Any condition that would damage the Town 's stormwater collection system or that would harm the quality of the Town 's waters; and
- Any conditions specifically declared to be dangerous to the public health, safety, and general welfare of the Town's inhabitants.

Failure to properly operate and maintain BMPs and stormwater management facilities in accordance with the stormwater management plan is a violation of the Ordinance.

1.2.1 BMP Ownership

Sustaining the record and communication of ownership of structural BMPs and stormwater management facilities is important to determining the responsible party for maintenance. The Town's Ordinance maintains a process for which BMP and facility ownership may be transferred. Prior to the transfer of any lot or development site to be served by a structural BMP or facility, the applicant or owner of the site must execute an operation and maintenance agreement that shall be binding on all subsequent owners of

the site, portions of the site, and lots or parcels served by the structural BMP and stormwater management facilties. It is the responsibility of the current owner to communicate this transfer of responsibility. Until the transference of all property, sites, or lots served by the structural BMP or facility, the applicant or owner shall have primary responsibility for carrying out the provisions of the maintenance agreement.

All inspection reports shall be on forms supplied by the Administrator(s). An original inspection report shall be provided to the Administrator(s) beginning one year from the date of as-built certification and each year thereafter on or before the date of the as-built certification. Failure to perform required or emergency maintenance or failure to maintain and provide the required records of that maintenance is a violation of the Town's Ordinance.

1.3 FEE SCHEDULE

A fee schedule may be developed by the Town Council for the Town to perform stormwater management plan review. The fees may be administered individually or incorporated into any existing fee schedules already in use by the Town. The Town may update the fee schedule from time to time. The fee schedule will be posted at Town Hall, or other administrative offices of the Town.

2. 0 HYDROLOGY AND RUNOFF DETERMINATION

By definition, hydrology is the scientific study of water and its properties, distribution, and effects on the earth's surface, soil, and atmosphere. Hydrologic analyses include estimation of peak runoff rates, volumes, and time distribution of stormwater runoff flows and are fundamental in the design of stormwater management facilities. This chapter addresses the movement of water over land resulting directly from precipitation in the form of stormwater runoff.

Land development changes how a watershed responds to precipitation. The most common effects are reduced infiltration and decreased travel time. Increased impervious surfaces and runoff velocities increase peak flow discharge volumes and rates. Total runoff volume is determined by the total drainage area of the receiving watershed, its infiltration characteristics, and the amount of precipitation.

2.1 GENERAL DESIGN CRITERIA

The following guidelines should be followed when selecting hydrologic computation standards:

- The design storm duration shall be the 24-hour rainfall event, using the National Resource Conservation Service (NRCS), formerly known as Soil Conservation Service (SCS), Type III rainfall distribution with a maximum 6-minute time increment.
- If the contributing drainage area is 20 acres or less, the design requires a single culvert or channel, and if no storage design or runoff volume is required, the Rational Method or the NRCS (SCS) Method of runoff calculation shall be acceptable.
- If the contributing drainage area is greater than 20 acres, or if storage or runoff volume design is required, only the NRCS (SCS) Method of runoff calculation shall be acceptable.
- Hydrologic analysis shall use the Antecedent Moisture Condition III (AMC III) for all irrigated ground cover.
- All permanent ponding/storage shall be considered impervious surface for the purposes of hydrologic analysis and routing.

2.2 DRAINAGE AREA CHARACTERISTICS

Drainage area (also referred to as watershed or drainage basin) is an area receiving precipitation, which generates runoff flow that drains into a single point of discharge. The use of contour maps is recommended for drainage area delineation. Identifying the area from which the runoff will be evaluated is one of the basic requirements for design of stormwater management facilities.

Watershed runoff discharge is determined by the following characteristics:

- Size
- Shape
- Slope
- Soils
- Land-Use
- Ponding/Storage
- Soil Moisture

2.2.1 Size

Size of the watershed dictates how much of the precipitation will fall on the area contributing to the flow at the point of interest.

2.2.2 Shape

Shape of the basin influences the timing of the peak flow hydrograph (i.e.; how fast water is being transported from the edge of the watershed to the point of interest). Long, narrow or short, wide basins usually have larger runoff peak rates than basins which are approximately equal in length and width.

2.2.3 Slopes

Slope of a basin significantly impacts the runoff rate. Runoff from a drainage area with steep slopes will travel along its flow path faster than it will from an area with shallow, rolling topography. How the basin slope directly impacts the time of concentration is shown in Section 2.4.1.

2.2.4 Soils

Soil properties influence the relationship between rainfall and runoff with their various rates of infiltration. Based on the infiltration rates, the NRCS (SCS) has divided the soils into four hydrologic soil groups (HSG) A, B, C, and D. The soils in hydrologic group A have the most (0.30 – 0.45 in/hr) infiltration capacity while soils in the hydrologic group D have the least (less than 0.05 in/hr).

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 and 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 consist primarily of either 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 clay layer at or near the surface, and shallow soils over nearly impervious underlying material.

A list of soils for the Town of Bluffton (Town) and their hydrologic classification (HSG) is presented in Table 2-1. Soil Survey maps can be obtained from local NRCS (SCS) office. Note: A/D and B/D indicate the drained/un-drained situation.

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 hydrologic soil group selection. Also runoff curve numbers (CN) vary with the antecedent soil moisture conditions (AMC). See Section 2.4.2 for more details antecedent soil moisture conditions.

TABLE 2-1
HYDROLOGIC SOILS GROUPS (HSG) FOR BLUFFTON, SOUTH CAROLINA

Series Name (Map Symbol)	HSG	Series Name (Map Symbol)	HSG
Argent (Ae)	D	Onslow (On) #	C
Baratari (Ba)#	B/D*	Polawana (Po) #	D
Bladen (Bd)	D	Ridgeland (Rd)	B/D*
Bohicket(BK)	D	Rosedhu (Ro) #	B/D*
Cape Fear (Ca)	D	Santee (Sa)	D
Capers (CE)	D	Seabrook (Sk) #	C
Chisolm (CmB)	A	Seewee (Sw)	B
Coosaw (Cs)	B	Tomotley(To) #	B/D*
Deloss (De) #	D	Wahee (Wa)	D
Eulonia (Ee)	C	Wando (Wd) #	A
Levy (LE)	D	Williman (Wn)	B/D*
Murad (Mu)	B	Yonges (Yo)	D
Okeetee (Oe)	D		

Source: South Carolina Department of Health and Environmental Control, Stormwater Management and Sediment Control Handbook, August 2003
*B/D soils should be assumed to be D soils.
#More recent soil publications may reflect a different series name for these map symbols.

2.2.5 Land Use

Also known as land use cover, land use represents the surface characteristic of the drainage area and consists of a combination of pervious and impervious surfaces.

Two widely used methods characterizing the land surface are Rational Method's "C" value and NRCS (SCS) method's curve number (CN). The surface identifying number of the drainage area may be calculated as a composite number from the sub-areas of the uniform land cover.

2.2.6 Ponding/Storage

If part (up to 5%) of drainage area is available for the runoff retention/detention, this should be included in the peak runoff analysis. The Rational Method is not an appropriate hydrologic methodology for drainage areas with these considerations.

2.2.7 Soil Moisture

Antecedent moisture condition is the index of runoff potential before a storm event. Refer to Section 2.4.2 for more details antecedent moisture condition.

2.3 PRECIPITATION DATA

Rainfall is the primary form of precipitation in the Town. The period in which rainfall begins, intensifies, peaks, and subsides during one storm is categorized as a storm event. Individual storm events are generally defined, or separated, by a minimum 6-hour time interval without precipitation. Variants of an event may be complicated by tropical storm "bands". The storm magnitude is estimated from the storm event volume, duration, and intensity. Statistical calculations are used to establish the probability of a specific storm event. The following items characterize a storm event.

- Depth/Volume/Duration
- Intensity
- Spatial Distribution
- Recurrence Interval

2.3.1 Depth/Volume/Duration

The depth of precipitation over an area generates a certain volume. This volume can be derived from the rainfall intensity that falls during the storm duration (or time period) over the area. Rainfall depths are typically provided based on a return period or the period a certain magnitude will occur in any given year. Table 2-2 presents the 24-hour storm events within the Town for return periods of 2-, 5-, 10-, 25-, 50-, and 100-year.

TABLE 2-2
RAINFALL DEPTH (Inches) FOR BLUFFTON, SOUTH CAROLINA

	Return Period [years]					
Town of Bluffton	2	5	10	25	50	100
	4.5	5.8	6.9	8.4	9.7	11.0

Source: South Carolina DHEC Storm Water Management BMP Handbook, Appendix F, July 2005. (Beaufort County values presented here).

2.3.2 Intensity

Intensity represents the rate at which rainfall occurs. The average intensity for a period is the total rainfall event depth divided by the time over which the rainfall occurred. The rainfall intensity values may be also computed using the following equation:

$$i = a / (b + t_c)^c \qquad (2\text{-}1)$$

Where:

i = rainfall intensity, in inches per hour
t_c = time of concentration, in minutes
a , b, and c are coefficients

The coefficients for the 2-, 5-, 10-, 25-, 50-, and 100-year rainfalls and the intensity values for a time of concentration of 5, 10, and 15 minutes for the Town are listed in Table 2-3.

TABLE 2-3
RAINFALL INTENSITY COEFFICIENTS FOR BLUFFTON, SOUTH CAROLINA

Frequency (years)	a	b	c	i (in/hr) t_c=5min	i (in/hr) t_c=10min	i (in/hr) t_c=15min
2	252.6	33.7	1.02	6.00	5.30	4.74
5	263.4	32.0	1.01	6.79	5.97	5.33
10	271.2	30.9	1.01	7.42	6.50	5.79
25	282.3	29.2	0.99	8.40	7.33	6.51
50	290.4	28.0	0.99	9.20	8.01	7.09
100	297.5	26.9	0.98	9.98	8.65	7.64

Source: South Carolina Department of Transportation, September 1997 (Hilton Head values presented here.)

2.3.3 Spatial Distribution

Spatial distribution relates to whether the rainfall depth (volume or intensity) at various locations in a drainage basin are equal for the same event. In practice, spatial variations for a relatively small drainage area can be neglected.

2.3.4 Recurrence Interval

The probability that a rainfall event of a certain magnitude will occur in any given year is expressed in terms of recurrence interval (also called return period or event frequency). The recurrence interval is the average length of time expected to elapse between rainfall events of equal or greater magnitude. The recurrence interval is expressed in years, but is actually based on a storm event's exceedance probability. For example, the 100-year storm, also known as one percent annual chance storm, is the storm that has one percent chance of being equal or exceeded in any given year. The relationship between recurrence interval and exceedance probability is given by

$$T = 1/P \qquad (2\text{-}2)$$

Where:

T = return period, in years
P = exceedance probability

Storms of different magnitude with different recurrence intervals will be set as design criteria for individual parts of stormwater management facilities. For example, a culvert under a freeway will be designed to safely carry flow from a 100-year storm event, but a culvert under residential flow is only required to carry flow from a 25-year storm event.

Sheet Flow

Sheet flow is the flow of water over a plane surface usually taking place in the headwater of the basin. The sheet flow occurs over a distance of up to 300 feet before it forms rills or paths. A maximum of 100 feet shall be used for the design of stormwater systems.

$$T_t = [0.007\ (nL)^{0.8} / (P_2)^{0.5}\ (s)^{0.4}] \qquad (2\text{-}4)$$

Where:

T_t	=	travel time, in hours
n	=	Manning's roughness coefficient for shallow depths of about 0.1 foot
L	=	flow length, in feet
P_2	=	2-year 24-hour rainfall, in inches
s	=	land slope, in feet per foot

TABLE 2-4
ROUGHNESS COEFFICIENTS (MANNING'S "n") FOR SHEET FLOW

Surface Description	"n"[1]
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

Source: NRCS (SCS), TR-55, Second Edition, June 1986.
[1]The "n" values are a composite of information compiled 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 hat will obstruct sheet flow.

2.3.5 Form of Rainfall Data

Rainfall data for probable precipitation depths are based upon historical records. Calculation of resulting peak runoff flow or flow volume is a primary objective of data collection and analysis. Several forms of data useful for peak runoff flow calculation or computer simulation are shown below.

- **Intensity-Duration-Frequency (IDF) Curves** illustrate the average rainfall intensities corresponding to various durations and storm recurrence intervals, such as 100-, 50-, 25-, 10-, 2-year storms.
- **NRCS (SCS) Peak Discharge Method** produces a peak discharge and requires the 24-hour total rainfall depths for the selected recurrence interval.
- **NRCS (SCS) Unit Hydrograph** can be used with any rainfall distribution, however, the 24-hour total rainfall depths and 24-hour rainfall temporal distribution (Type III) will be used in this manual.

2.4 RUNOFF DETERMINATION

The travel time, or time of concentration, of the watershed is directly related to the slope, flow path length, depth of flow, and roughness of the flow surfaces due to the type of ground cover. The time of concentration is used in Rational as well as NRCS (SCS) Methods for the peak flow determination.

2.4.1 Time of Concentration (T_c)

<u>TR-55 Method</u>

TR-55 method (Natural Resource Conservation Service (NRCS), Urban Hydrology for Small Watersheds, Technical Release No. 55, June 1986) is used to compute T_c by summing all the travel times of consecutive flow segments of the drainage conveyance system along the path extending from the hydraulically most distant point of the drainage area to the point of interest within this area.

$$T_c = T_1 + T_2 + T_3 + \ldots + T_n \qquad (2\text{-}3)$$

Where:

T_1 = time of travel through one segment, in hours
N = number of segments

Water moves through the drainage area as sheet flow, shallow concentrated flow, open channel, or a combination of these.

Shallow Concentrated Flow

Average velocities for estimating travel time for shallow concentrated flow can be computed from the following equations. These equations can also be used for slopes less then 0.005 ft/ft.

$$V = 16.1345(S)^{0.5} \quad \text{for unpaved surface} \qquad (2\text{-}5)$$
$$V = 20.3282(S)^{0.5} \quad \text{for paved surface} \qquad (2\text{-}6)$$

Where:

V = average velocity, in feet per second
S = slope of hydraulic grade line (watercourse slope), in feet per foot

These two equations are based on the solution of Manning's Equation with different assumptions for "n" (Manning's roughness coefficient) and "r" (hydraulic radius, in feet). For unpaved areas "n" is 0.05 and "r" is 0.4 feet; for paved areas, "n" is 0.025 and "r" is 0.2 feet.

After determining average velocity, travel time for the shallow concentrated flow segment can be estimated by dividing the flow length by the velocity.

Open Channel Flow

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's Equation for water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bankfull elevation. Manning's Equation for this is:

$$V = 1.49 / n\,(R)^{2/3}\,(s)^{1/2} \qquad (2\text{-}7)$$

Where:

V = average velocity, in feet per second
n = Manning's roughness coefficient
s = slope of the hydraulic grade line, in feet per foot
R = hydraulic radius, in feet and is defined by the equation

$$R = a/p_w \qquad (2\text{-}8)$$

a = cross sectional flow area, in square feet
p_w = wetted perimeter, in feet

After average velocity is computed using above equation, T_t for the channel segment can be estimated by dividing the flow length by the velocity. Velocity in channels should be calculated from Manning's Equation. Cross sections from all channels that have been field checked should be used in the calculations. This is particularly true of areas below dams or other flow control structures.

2.4.2 Flow Determination Methods

This section describes the recommended procedures for calculating the runoff generated from a project site. Correct utilization of these procedures should result in the best available estimation of existing and projected runoff. Their use will also provide the consistency of results necessary when applied to project sites throughout the Town. All hydrologic computational methods shall be accomplished using a method acceptable by the Town.

The following guidelines should be followed when selecting hydrologic computation standards:

- The design storm duration shall be the 24-hour rainfall event, using the NRCS (SCS) Type III rainfall distribution with a maximum 6-minute time increment.
- If the contributing drainage area is 20 acres or less, the design requires a single culvert or channel, and if no storage design or runoff volume is required, the Rational Method or the NRCS (SCS) Method of runoff calculation shall be acceptable.
- If the contributing drainage area is greater than 20 acres, or if storage or runoff volume design is required, only the NRCS (SCS) Method of runoff calculation shall be acceptable.

<u>Rational Method</u>

The Rational Method formula is utilized to determine peak flow rates in urban areas and small watersheds for the following situations:

- Total drainage area of 20 acres or less.
- No storage or volume design required.
- Design involves only the sizing of a single culvert or channel.

The Rational Method allowed only for small, highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters with individual outfall pipes. The Rational Method calculates peak discharge only as opposed to developing a runoff hydrograph for an area. It makes a basic assumption that the design storm has constant rainfall intensity for a time period equaling the drainage area time of concentration (T_c), the time required for water to flow from the most remote point of the basin to the point of interest.

$$Q = C\,i\,A \qquad (2\text{-}9)$$

Where:

Q	=	maximum rate of runoff, in cubic feet per second
C	=	runoff coefficient representing a ratio of runoff to rainfall, Table 2-6
i	=	average rainfall intensity for a duration equal to the T_c, in inches per hour
A	=	drainage area contributing to the design location, in acres

Less Frequent Storms

The runoff coefficients given in Table 2-6 are applicable for storms equal to or more frequent than the 10-year frequency (such as 2-, 5-, and 10-year storms). Less frequent, higher intensity storms, such as 25-, 50-, and 100-year storms require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on the runoff. Adjustment of the

Rational Method for use with major storms can be made by multiplying the runoff coefficient by a frequency factor, C_f, shown in Table 2-5.

For infrequent storm events, the rational equation is then expressed as:

$$Q = C\,C_f\,i\,A \qquad (2\text{-}10)$$

Where:

Q	=	maximum rate of runoff, in cubic feet per second
C	=	runoff coefficient based on 5- to 10-year storms, Table 2-6
C_f	=	frequency factor based on recurrence interval (dimensionless), Table 2-5
i	=	average rainfall intensity, in inches per hour
A	=	drainage area contributing to the design location, in acres

TABLE 2-5
RATIONAL METHOD RUNOFF COEFFICIENT FREQUENCY FACTORS

Recurrence Interval (years)	Frequency Factor, C_f
25	1.1
50	1.2
100	1.25

Note: The product of C_f times C shall not exceed 1.0.

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 professional. While engineering judgment will always be required in the selection of runoff coefficients, recommended runoff coefficients for the Rational Method representing the integrated effects of many drainage basin parameters are listed in Table 2-6.

Composite Coefficients

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 2-6 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as roofs, asphalt and concrete streets, drives and walks. The composite procedure can be applied to an entire drainage area or to typical sample block 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.

TABLE 2-6
RECOMMENDED RUNOFF COEFFICIENT "C" VALUES
RATIONAL METHOD

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
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 and cemeteries	0.25
Playgrounds	0.35
Railroad yard areas	0.40
Unimproved areas (forest)	0.30
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

Rainfall Intensity, i

Rainfall intensity is the average rainfall rate (typically reported in inches per hour) for duration equal to the time of concentration for a selected return period. Once a return period has been selected for design and a time of concentration calculated, the rainfall intensity can be determined from Rainfall-Intensity-Duration data. Table 2-7 lists the rainfall intensity data. Straight-line interpolation can be used to obtain rainfall intensity values for storm durations between the values given in Table 2-7.

The NRCS (SCS) method includes the following basic steps:

- Determination of CN representing different land uses within the drainage area.
- Calculation of T_c to the study point.
- Selection of design storm event.
- Use of the Type III rainfall distribution to determine the total and excess rainfall quantity.
- Use of the unit hydrograph approach including development of triangular and composite hydrographs for the drainage area.

The NRCS (SCS) method applicable to the Town is based on a storm event, which has a Type III time distribution. To use this distribution, the user has to obtain the 24-hour rainfall volume. A relationship between accumulated rainfall and accumulated runoff was derived by NRCS (SCS) from experimental plots for numerous soils and vegetative cover conditions. The following NRCS (SCS) runoff equation is used to estimate direct runoff from a 24-hour storm event. The equation is:

$$Q = (P - 0.2S)^2 / (P + 0.8S) \qquad (2\text{-}11)$$

Where:

Q = total runoff volume for the specified storm event, in inches
P = rainfall volume for the specified storm event, in inches from Table 2-2.
S = potential maximum retention after runoff begins, in inches and is defined by the following equation:

$$S = (1000/CN) - 10 \qquad (2\text{-}12)$$

Where:

CN = NRCS (SCS) curve number

Initial Abstractions (I_a) are all losses in the watershed before runoff begins. These abstractions include water retained in surface depressions, water intercepted by vegetation, evaporation and infiltration. I_a is highly variable but is generally correlated with soil and cover parameters. Through the study of many small agricultural watersheds, I_a is approximated by the following empirical equation:

$$I_a = 0.2S \qquad (2\text{-}13)$$

CNs represents the combined hydrologic effect of the soil type, land use, hydrologic soil group (HSG), and antecedent moisture condition. It may be necessary to create a composite CN by weighting distinct land use-HSG combinations and summing them for the total drainage area. The CN indicates the runoff potential of soil which is not frozen. Higher CN reflects a higher runoff potential.

Another factor of consideration is whether impervious areas are directly connected to the system or if the system is unconnected and flows from impervious areas spread over pervious areas before reaching the outfall point. The CN is similar to the Rational Method's "C" coefficient in that it is based on the surface condition of the drainage area. NRCS (SCS) TR-55 presents a method for analyzing unconnected impervious areas, as discussed later in this section.

TABLE 2-7
RAINFALL INTENSITY FOR BLUFFTON, SC

Rainfall Intensity (inches/hour)[1]						
Storm Duration	**Storm Recurrence Interval**					
	2-year	**5-year**	**10-year**	**25-year**	**50-year**	**100-year**
5 minutes	7.18	8.35	9.34	10.48	11.38	12.25
10 minutes	5.75	6.68	7.45	8.33	9.05	9.73
15 minutes	4.81	5.63	6.28	7.04	7.64	8.19
30 minutes	4.81	5.63	6.28	7.04	7.64	8.19
1 hour	2.08	2.56	2.96	3.46	3.89	4.31
6 hours	0.52	0.65	0.77	0.92	1.06	1.02
12 hours	0.30	0.38	0.45	0.55	0.63	0.72
24 hours	0.18	0.23	0.27	0.33	0.38	0.43
24-hour Volume (in)[2]	4.5	5.8	6.9	8.4	9.7	11.0

[1]Source: National Weather Service, Precipitation-Frequency Atlas of the United States, NOAA Atlas 14, Volume 2, Version 2, August 2003.
[2]Source: South Carolina DHEC Storm Water Management BMP Handbook, Appendix F, July 2005. (Beaufort County values presented here).

NRCS (SCS) Curve Number (CN) Method

The NRCS (SCS) hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. Details of the methodology can be found in the NRCS (SCS) TR-55 and the NRCS Engineering Field Manual for Conservation Practices.

The NRCS (SCS) curve number method begins with a rainfall amount uniformly imposed on the watershed over a specified time distribution. Mass rainfall is converted to mass runoff by using a runoff CN that is based on soil type, plant cover, impervious areas, interception, and surface storage.

Runoff is then transformed into a hydrograph by using unit hydrograph theory and routing procedures that depend on runoff travel time through segments of the watershed. The NRCS (SCS) Method is used to determine stormwater runoff peak flow rates, runoff volumes, and the generation of hydrographs for the routing of storm flows in urban areas and project sites. The following are requirements of its use:

- NRCS (SCS) CN Method is required for total drainage areas greater than 20 acres.
- NRCS (SCS) CN Method may be used for total drainage areas less than 20 acres.
- Runoff volume.
- Routing.
- Design of storage facilities and outlet structure.

The following conditions apply when using the NRCS (SCS) CNs, based on HSG and surface cover, as shown in Table 2-8, to estimate runoff:

- CNs are based on AMC II.
- Understand that initial abstraction (I_a) consists of interception, initial infiltration, surface depression storage, and evapotranspiration.
- Runoff from frozen ground cannot be estimated using this procedure.
- The curve number method becomes less accurate when runoff is less than 0.5 inches. When this situation occurs, use of another procedure is recommended as check. If a discrepancy exists, then other procedures should be explored for approval by the Administrator(s).
- This procedure applies only to direct runoff.
- If the weighted CN is less than 30, use CN = 30 for runoff computations.

Antecedent moisture condition is the index of runoff potential before a storm event. The AMC is an attempt to account for the variation in CN at a particular site for various storm conditions. The CNs listed in Table 2-8 are for average AMC II, which are used primarily for design applications. The three AMC classifications are:

AMC I – Little rain or drought conditions preceding studied rainfall event. The CNs for AMC I can be calculated using the following equation:

$$CN_{(AMC\ I)} = 4.2\ CN_{(AMC\ II)} / (10 - 0.058\ CN_{(AMC\ II)}) \qquad (2\text{-}14)$$

AMC II – Standard CNs developed from rainfall and runoff data.

AMC III – Considerable rainfall prior to studied rainfall event. The CNs for AMC III can be calculated using the following equation:

$$CN_{(AMC\ III)} = 23\ CN_{(AMC\ II)} / (10 + 0.13\ CN_{(AMC\ II)}) \qquad (2\text{-}15)$$

<u>Urban Modifications</u>

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 CNs for urban areas.

It is possible that CN values from urban areas could be reduced through the use of structural stormwater BMPs and strategic placement of vegetated areas to disconnect impervious surfaces for infiltration of runoff.

An impervious area is considered connected if runoff from it flows directly into the storm drainage system. It is also considered connected if runoff from the area occurs as concentrated shallow flow that runs over a pervious area and then into a drainage system.

TABLE 2-8
RUNOFF CURVE NUMBER FOR URBAN AREAS AND AGRICULTURAL LANDS[1]
NRCS (SCS) CN METHOD

Cover Description Cover Type and Hydrologic Condition	Average Percent Impervious Area[2]	Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Fully developed urban areas (vegetation established)					
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 sewers (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 agricultural land					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94
Pasture, grassland, or range: continuous forage for grazing[4]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow: continuous grass, protected from grazing and generally mowed for hay	-	30	58	71	78
Brush-brush, weed, grass mixture with brush the major element[5]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[7]30	48	65	73

TABLE 2-8 continue
RUNOFF CURVE NUMBER FOR URBAN AREAS AND AGRICULTURAL LANDS[1]
NRCS (SCS) CN METHOD

Cover Description Cover Type and Hydrologic Condition	Average Percent Impervious Area[2]	Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Woods-grass combination (orchard or tree farm)[6]	Poor Fair Good	57 43 32	73 65 58	82 76 72	86 82 79
Woods[7]	Poor Fair Good	45 36 [4]30	66 60 55	77 73 70	83 79 77
Farmsteads-buildings, lanes, driveways, and surrounding lots.	-	59	74	82	86

Source: NRCS (SCS), TR-55, second Edition, June 1986

[1] Average runoff condition, and $I_a = 0.2S$

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the NRCS (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.

[4] Poor: <50% ground cover or heavily grazed with no mulch.
Fair: 50% to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.

[5] Poor: <50% ground cover
Fair: 50% to 75% ground cover
Good: >75% ground cover

[6] CNs shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CNs for woods and pastures.

[7] Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.

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 2-8 are not applicable, compute a composite CN using NRCS TR-55 Chapter 2 and Figure 2-3 of TR-55 for reference.

If runoff from unconnected impervious areas is spread over a pervious area as sheet flow, then all or part of the impervious area is not directly connected to the drainage system. Use NRCS TR-55 Chapter 2 for reference in determining a modified CN for the disconnected impervious areas.

Simplified NRCS (SCS) Method

Overview

The following NRCS (SCS) procedures were taken from the NRCS (SCS) TR-55 which presents simplified procedures to calculate storm runoff volume, peak rate of discharges and hydrographs. These procedures are applicable to small drainage areas and include provisions to account for urbanization. The following procedures outline the use of the NRCS (SCS) TR-55 method.

Peak Discharges

The NRCS (SCS) peak discharge method is applicable for estimating the peak runoff rate from watersheds with a homogeneous land use. The following method is based on the results of computer analyses performed using TR-20, "Computer Program for Project Formulation Hydrology," NRCS (SCS) 1992.

The peak discharge equation is:

$$Q_p = q_u A Q F_p \qquad (2\text{-}16)$$

Where:

Q_p	=	peak discharge, in cubic feet per second
q_u	=	unit peak discharge, in cubic feet per second per square mile per inch ($cfs/mi^2/in$)
A	=	drainage area, in square miles
Q	=	runoff, in inches
F_p	=	pond and swamp adjustment factor

The input requirements for this method are as follows:

1. T_c, in hours
2. Drainage area, in square miles
3. Type III rainfall distribution
4. 24-hour design rainfall
5. CN value
6. 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

Computations for the peak discharge method proceed as follows:

1. The 24-hour rainfall depth is in Table 2-7 for the selected return frequency.
2. The runoff CN, is estimated from Table 2-8 and direct runoff, Q, is estimated from Equation 2-11. Determine if urban modifications of CN are appropriate.
3. The CN value is used to determine the initial abstraction, I_a, from Table 2-8, and the ratio I_a/P is then computed. (P = accumulated 24-hour rainfall or potential maximum runoff.)

4. The watershed time of concentration is computed using the procedures in Section 2.4.1 and is used with the ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 2-1. If the ratio I_a/P lies outside the range shown in Figure 2-1, either the limiting values or another peak discharge method should be used.
5. The pond and swamp adjustment factor, F_p, is estimated using Table 2-9.
6. The peak runoff rate is computed using Equation 2-16.

TABLE 2-9

Pond and Swamp Areas (%)*	F_p
0.0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

*Percent of entire drainage basin

Limitations

The accuracy of the peak discharge method is subject to specific limitations, including the following:

a. The watershed must be homogeneous in a hydrologic sense and therefore describable by a single CN value.
b. The watershed may have only one main stream, or if more than one, the individual branches must have nearly equal time of concentrations.
c. Hydrologic routing cannot be considered.
d. The pond and swamp adjustment factor, F_p, applies only to areas located away from the main flow path.
e. Accuracy is reduced if the ratio I_a/P is outside the range given in Figure 2-1.
f. The weighted CN value must be greater than or equal to 40 and less than or equal to 98.
g. The same procedure should be used to estimate existing and developed time of concentration when computing existing and developed peak discharge.
h. The watershed time of concentration must be between 0.1 and 10 hours.

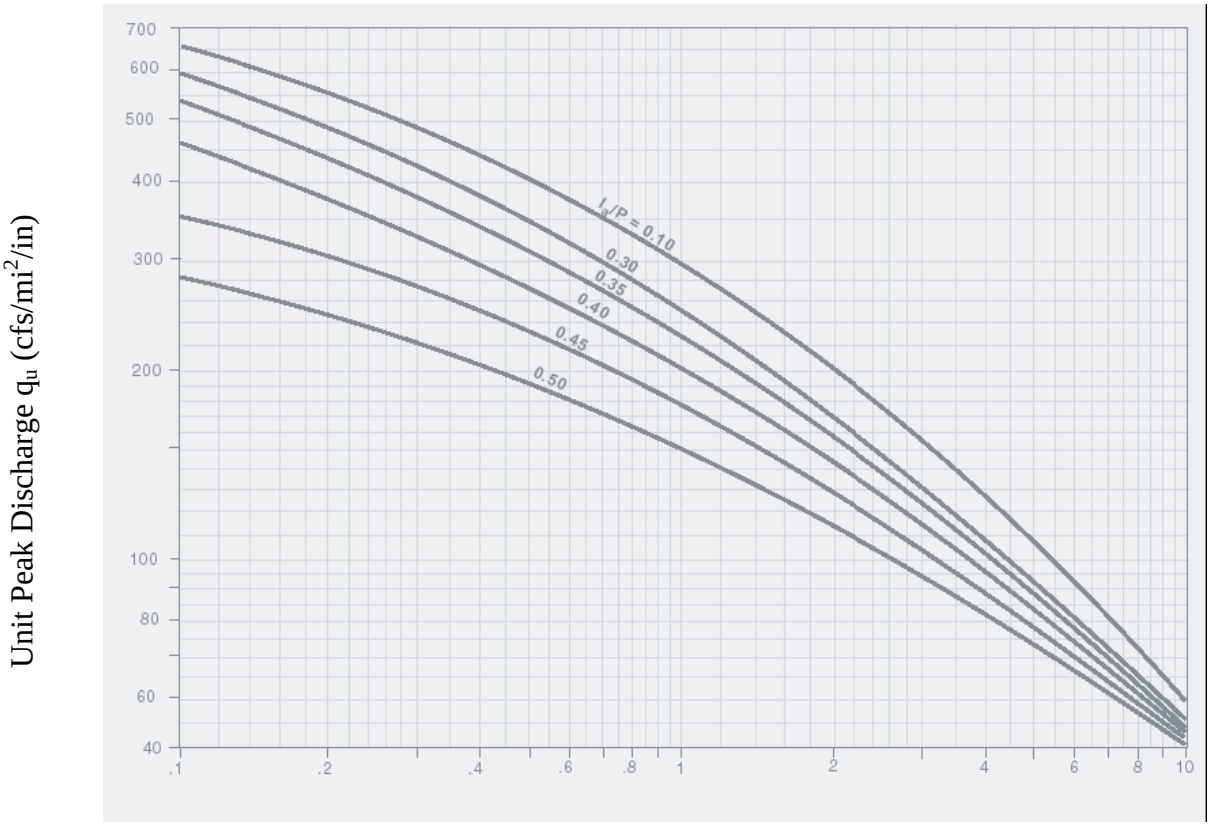


Figure 2-1 NRCS (SCS) Type III Unit Peak Discharge Graph

Hydrograph Generation

In addition to estimating the peak discharge, the NRCS (SCS) method can be used to estimate the entire hydrograph from a drainage area. The NRCS (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, NRCS (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 or from drainage area which is 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 NRCS (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.

3.0 STORM SEWER COLLECTION SYSTEM

Effective drainage of roadway pavements is essential to the maintenance of roadway service level and to traffic safety. Water on the pavement can interrupt traffic, 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.

Pavement drainage requires consideration of surface drainage, gutter flow, inlet capacity, and storm sewer capacity. The design of these elements is dependent on storm frequency and the allowable inundation of stormwater on the pavement surface. This chapter presents guidance for the design of these elements. Most of the information presented in this chapter was originally published in HEC-12, Drainage of Highway Pavements (Federal Highway Administration (FHWA) 1984) and AASHTO's Model Drainage Manual (American Association of State Highways Transportation Officials (AASHTO 1991).

3.1 GENERAL DESIGN CRITERIA

Design criteria for the collection and conveyance of runoff on public roadways are typically based on roadway classification and reasonable frequency of traffic interference. Depending on the roadway classification, certain lanes can incur more inundation of standing water on each side of an inlet (spread) and still pass traffic safely with minor interference.

Inlets

- The 25-year design storm shall be used to design inlets
- Total spread (both sides of an inlet) for a travel lane should not exceed half the travel land width plus gutter width, where a gutter section is used.
- For arterial and multi-lane collector roadways, spread should not exceed one-half of a travel lane on a two-lane roadway and may encroach a full travel lane on a four-lane roadway.
- No curb overtopping may occur in sags.
- Depth of spread should not exceed 6 inches.
- Height for the curb opening on inlets shall not exceed 6 inches.
- Inlet spacing should not exceed 400 feet.
- All curb inlets shall be stenciled with the phrase "Drains to River" or similar phrase.

Storm Sewers

- The 50-year, design storm shall be used on storm sewers crossing under arterials and multi-lane collector roadways.
- The 25-year, design storm shall be used on all remaining storm sewer systems.
- Maximum tail water conditions and associated tidal influences shall be considered when designing a stormwater system
- Minimum maintenance easement widths shall be the greater value of either 20 feet or the sum of the pipe diameter (in feet) plus 4 feet plus 2 times the depth from the pipe invert to the existing grade.

If the storm sewer system is part of a roadway to be accepted by the South Carolina Department of Transportation (SCDOT), the system design is governed by SCDOT design criteria.

3.2 COLLECTION SYSTEM

The roadway collection system is comprised of roadway sections including gutters, and inlets. Gutters collect runoff from the roadway and direct it to the inlets for capture. The inlets collect part of the gutter flow for transport to a subsurface conveyance system. The inlet design and location limit the amount of inundation on a roadway. This inundation is commonly referred to as spread. The control of spread is an important consideration for the minimization of interference to traffic and the safe passage of emergency-response vehicles.

3.2.1 Gutter Capacity and Types

There are various types of gutter sections available for use in roadway drainage design today. The gutter geometry may be determined by the need for additional carrying capacity or the requirement for safe passage of pedestrian traffic. The gutter may have a straight transverse slope (uniform), a composite transverse slope, or transverse slope composed of two straight lines (V-shape). When the allowable spread for the roadway has been determined, the gutter capacity (and part of the roadway that may also be used to convey runoff) can be computed using a modified version of Manning's Equation. The equation is modified because the hydraulic radius term does not adequately describe the spread flow cross section, especially when the top width of flow is significantly larger than the depth.

3.2.2 Standard Gutter Sections

A straight transverse slope section has uniform cross slope for the roadway and gutter. These gutter sections, or standard gutter, as they are commonly known, usually resemble a triangle with the curb forming the vertical leg. For standard gutters, the modified version of the Manning's Equation is presented in terms of cross slope and width of flow at the curb as:

$$Q = \left[\frac{0.56}{n}\right] S_x^{5/3} s^{1/2} T^{8/3} \qquad (3\text{-}1)$$

Where:

Q	=	discharge, in cubic feet per second
n	=	Manning's roughness coefficient
S_x	=	cross slope of the roadway, in feet per foot
s	=	longitudinal slope, in feet per foot
T	=	width of flow (spread), in feet

The resistance of the curb face is neglected in the equation since resistance is negligible when the cross slope is 10 percent or less.

In some instances it may be necessary to calculate the depth of the spread to ensure curb overtopping is not occurring. The depth of flow in a standard gutter can be calculated by the following equation.

$$d = TS_x \tag{3-2}$$

Where:

d	=	depth of flow at curb or deepest point, in feet
T	=	width of flow (spread), in feet
S_x	=	cross slope of the roadway, in feet per foot

Manning's "n" values for different roadway and gutter roughness conditions are presented in Table 3-1.

TABLE 3-1
MANNING'S ROUGHNESS COEFFICIENT "n" FOR ROADWAYS AND GUTTERS

Surface Type	"n" Value
Concrete gutter troweled finish	0.012
Asphalt pavement: Smooth texture Rough texture	 0.013 0.016
Concrete gutter with asphalt pavement: Smooth Rough	 0.013 0.015
Concrete pavement: Float finish Broom finish	 0.014 0.016
Brick	0.016

3.2.3 Composite Gutter Sections

To increase the capacity of a gutter, the gutter cross slope may be steepened with respect to the cross slope of the roadway. These gutters are termed composite gutters. For composite gutters, the capacity is determined for the depressed section and the area above the depressed section separately. The following series of equations demonstrate the calculations required for determining the capacity for these types of gutters.

$$Q = Q_s + Q_w \tag{3-3}$$

Where:

Q	=	discharge, in cubic feet per second
Q_s	=	discharge in roadway section, in cubic feet per second
Q_w	=	discharge in depressed section, in cubic feet per second

Equation 3-3 has been simplified to the following Equation 3-4 which relates the ratio of frontal flow to total gutter flow with the term 1-E_o. "E_o" is further defined in terms of cross slopes and widths in Equation 3-5. Equation 3-5 is an expansive equation that can be solved simply using data from Figure 3-1.

$$Q = \frac{Q_s}{1 - E_0} \tag{3-4}$$

Where:

$$E_0 = \frac{1}{1 + \dfrac{S_w / S_x}{\left[1 + \dfrac{S_w / S_x}{(T/W) - 1}\right]^{8/3} - 1}} \tag{3-5}$$

E_o	=	ratio of frontal flow to total gutter flow
S_w	=	cross slope of depressed section, in feet per foot
S_x	=	cross slope of roadway, in feet per foot
W	=	width of depressed section, feet
T	=	width of flow (spread), in feet

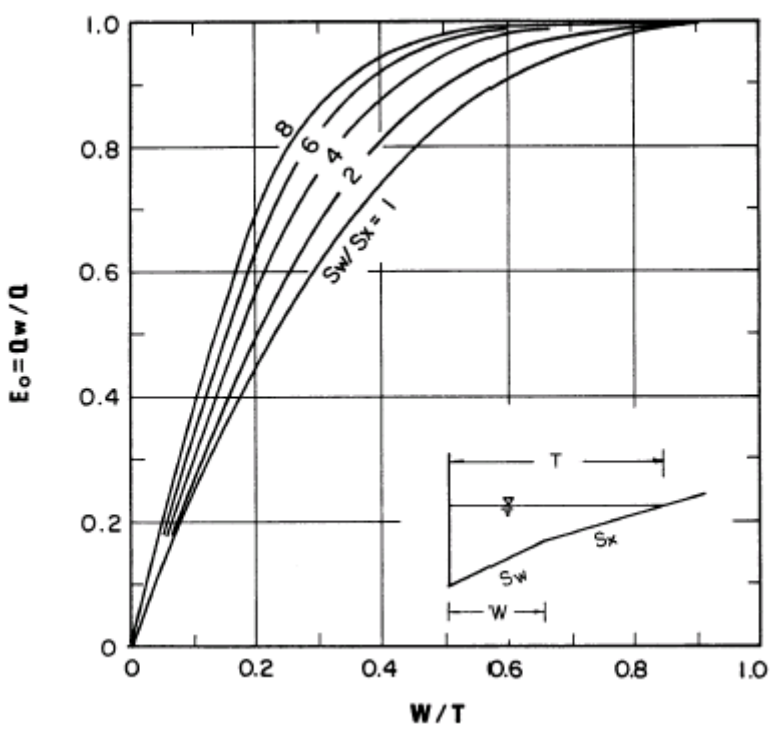


Figure 3-1 Ratio of Frontal Flow to Total Gutter Flow

If the cross slope of the depressed section is not known, or provided, it may be calculated from the roadway geometry with the following equation:

$$S_w = S_x + \frac{a}{W} \tag{3-6}$$

Where:

S_w	=	cross slope of depressed section, in feet per foot
S_x	=	cross slope of roadway, in feet per foot
a	=	local depression, feet
W	=	width of depressed section, feet

Flow depth at the curb, d, may be calculated from the following equation using the spread, T, cross slope of the roadway, S_x, and depth of the depressed section, a, See Figure 3-2.

$$d = a + TS_x \tag{3-7}$$

Where:

d	=	depth of flow at curb, feet
S_x	=	cross slope of roadway, in feet per foot
a	=	local depression, feet
T	=	width of flow (spread), in feet

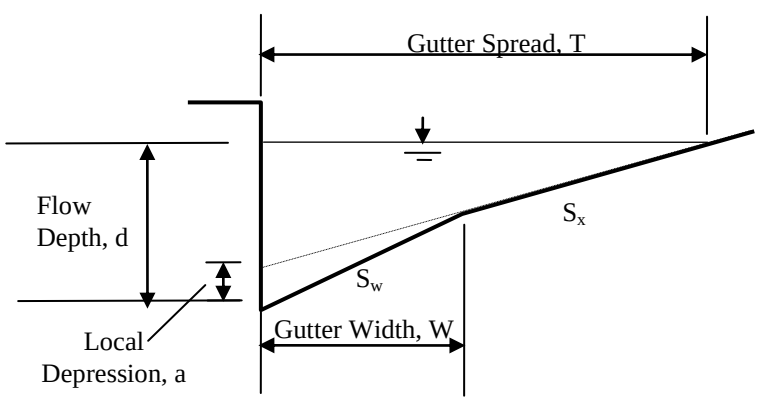


Figure 3-2 Typical Composite Gutter Section

3.2.4 Valley Gutter Sections

Gutters with transverse slopes composed of two straight lines (V-shape) are commonly referred to as valley gutters. The valley gutters may also be composed of a smooth parabolic cross section. The smooth parabolic gutters are commonly referred to as roll back or mountable types. The capacity for all valley type gutters is approximated by the same methodology. For valley gutters an adjusted cross slope of the gutter section is calculated by the following equation.

$$S_a = \frac{S_{x1} S_{x2}}{S_{x1} + S_{x2}} \tag{3-8}$$

Where:

S_a	=	adjusted cross slope, in feet per foot
S_{x1}	=	cross slope of interior gutter side, in feet per foot
S_{x2}	=	cross slope of exterior gutter side, in feet per foot

Use Equation 3-1, and the adjusted cross slope, S_a, from Equation 3-8 as cross slope of depressed section, S_w, to determine the capacity of the valley gutter. If it is necessary to determine the spread associated with a given flow capacity, use Equations 3-1 and 3-8 to calculate a preliminary spread, T′, first. If "T′" is less than T_a+T_b, then "T" is equal to "T′" and the spread is being conveyed only in the valley gutter section. If "T′" is greater than T_a+T_b, then part of the spread is also being conveyed in the roadway section. To determine the final spread, T, that is also being partially conveyed in the roadway, requires an assumption of "T′" and iteration between Equations 3-1 and 3-8 until "T′" converges.

Gutter calculations can be determined easily using available software packages. The Federal Highway Administration's HY-22 and Visual Urban HY-22 (for Windows) are free of charge, downloadable software packages that perform this and other calculations presented in this chapter.

3.2.5 Inlet Capacity and Types

Gutter inlets can be divided into the following three major classes, each with many variations: (1) curb-opening inlets, (2) grate inlets, and, (3) combination inlets. The Town has approved the use of specific inlet designs. The placement of two inlets side by side is referred to as a double inlet and is acceptable.

Brief descriptions of the approved inlet types follow:

- Curb-Opening inlets. These inlets are vertical openings in the curb covered by a top slab.
- Grate inlets. These inlets consist of an opening in the gutter covered by one or more grates.
- Combination inlets. These units consist of both a curb-opening and a grate inlet acting as a single unit.

Other inlets deemed practical and technically sound to meeting the criteria of this chapter may be submitted prior to design commencement for approval by the Administrator(s) on a case by case basis. Additional design consideration may be required where safe passage for bicycle and pedestrian traffic is important factor. The designer should consult SCDOT guidance where local guidance is not available.

Inlet efficiency, E, is defined by the following equation:

$$E = \frac{Q_i}{Q} \tag{3-9}$$

Where:

E	=	efficiency of inlet
Q_i	=	intercepted flow by inlet, in cubic feet per second
Q	=	total gutter flow, in cubic feet per second

The discharge that bypasses the inlet, Q_c, is termed carry-over or bypass. The interception capacity of all inlet configurations increases with increasing flow rates; however, inlet efficiency generally decreases with increasing flow rates.

Factors affecting gutter flow also affect inlet interception capacity. Interception capacity of a curb-opening is largely dependent on flow depth at the curb and curb-opening length. Curb-opening inlet interception capacity and efficiency are increased by the use of a gutter depression at the curb-opening or a depressed gutter to increase the proportion of the total flow adjacent to the curb. The amount of the depression has more effect on the capacity than the arrangement of the depressed area with respect to the inlet.

The interception capacity of a grate inlet depends on the amount of runoff flowing over the grate, the size and configuration of the grate, and the velocity of flow in the gutter. The efficiency of a grate is dependent on the same factors and total flow in the gutter.

The interception capacity of a combination inlet consisting of a grate and a curb-opening does not differ materially from that of a grate. Interception capacity and efficiency are dependent on the same factors which affect grate capacity and efficiency. However, as the depth of water in the gutter increases, the curb-opening becomes a major factor in the interception capacity. Curb-opening inlet interception capacity and efficiency are increased by the use of a gutter depression at the curb-opening or a depressed gutter to increase the proportion of the total flow adjacent to the curb. The amount of the depression has more effect on the capacity than the arrangement of the depressed area with respect to the inlet.

A combination inlet, consisting of a curb-opening inlet placed upstream of a grate, has a capacity equal to that of the curb-opening length upstream of the grate plus that of the grate, taking into account the reduced spread and depth of flow over the grate because of the interception by the curb-opening. This inlet configuration has the added advantage of intercepting debris that might otherwise clog the grate and deflect water away from the inlet.

A combination inlet consisting of a slotted inlet upstream of a grate might appear to have advantages when 100-percent interception is necessary. However, grates intercept little more than

frontal flow and would usually need to be more than 3 feet wide to contribute significantly to the interception capacity of the combination inlet. A more practical solution would be to use a slotted inlet of sufficient length to intercept total flow.

Most investigators have pointed out that the capacity of an inlet is greatly increased by allowing a small percentage of the flow to bypass the inlet. For a given gutter discharge, the catch of each additional increment of width declines rapidly.

3.2.6 Inlet Clogging

All types of inlets are subject to clogging. Attempts to simulate clogging tendencies in the laboratory have not been successful, except to demonstrate the importance of parallel bar spacing in debris handling efficiency. Grates with wider spacing of longitudinal bars pass debris more efficiently. Problems with clogging are largely local since the amount of debris varies significantly from one neighborhood to another. Some neighborhoods may contend with only a small amount of debris while others experience extensive clogging of drainage inlets. Clogging shall be considered in the design of all grate and combination inlets in sag conditions.

The width of the grate should be adjusted to account for clogging. Common practice has been to apply a 25 percent reduction to the width of the inlet. Therefore, a 2-foot by 3-foot grate whose unclogged effective perimeter is 7 feet (2 feet +3 feet +2 feet) would be reduced to 6 feet (1.5 feet + 3 feet +1.5 feet) under clogged conditions. Notice only the width (2-feet) is reduced by 25 percent. In sags on arterials and multi-lane connectors where ponding may cause violation of spread depth requirements, it is advised to design flanking inlets on either side located 0.2 feet above the invert elevation of the sag inlet to promote drainage.

3.2.7 Curb Opening Inlets

Curb-opening inlets may be located for roadway drainage where the expected 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.

Curb-opening is commonly constructed with a depression beginning W feet away from the curb, where W is the width of the curb inlet, and dropping 1-inch per foot below the plane of the pavement. Transitions at the two ends extend W feet from the end of the opening. The equations given apply only if the cross-section of the street has a uniform slope to the face of the curb.

<u>Capacity of Curb-Opening Inlets on Grade</u>

The ratio of frontal flow to total gutter flow, E_o, is expressed by the following equation for either straight cross slopes or depressed gutter sections.

$$E_o = \frac{Q_w}{Q} = 1 - (1 - W / T)^{2.67} \qquad (3\text{-}10)$$

Where:

E_o	=	ratio of frontal flow to total gutter flow
Q_w	=	flow in width W, in cubic feet per second
Q	=	total gutter flow, in cubic feet per second

W	=	width of depressed gutter or grate, in feet
T	=	total spread of water in the gutter, in feet

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by the following equation:

$$L_T = 0.6\,Q^{0.42} S^{0.3} \left[\frac{1}{nS_x} \right]^{0.6} \qquad (3\text{-}11)$$

Where:

L_T	=	curb-opening length required for 100-percent interception, in feet
Q	=	flow in gutter at inlet, in cubic feet per second
S	=	longitudinal gutter slope, in feet per foot
n	=	Manning's roughness coefficient
S_x	=	cross slope of pavement, in feet per foot

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by the following equation and Figure 3-3.

$$E = 1 - \left[1 - \frac{L_i}{L_T} \right]^{1.8} \qquad (3\text{-}12)$$

Where:

E	=	efficiency of inlet or percentage of interception
L_i	=	curb-opening length, in feet
L_T	=	curb-opening length required for 100-percent interception, in feet

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, calculated using the following equation.

$$S_e = S_x + S_w' E_o \qquad (3\text{-}13)$$

Where:

S_e	=	equivalent cross slope, in feet per foot
S_x	=	cross slope of pavement, in feet per foot
S'_w	=	cross slope of the gutter measured from the cross slope of the pavement, where S_x = a / (12W)
a	=	gutter depression, in inches
W	=	width of gutter, in feet
E_o	=	ratio of frontal flow to total gutter flow

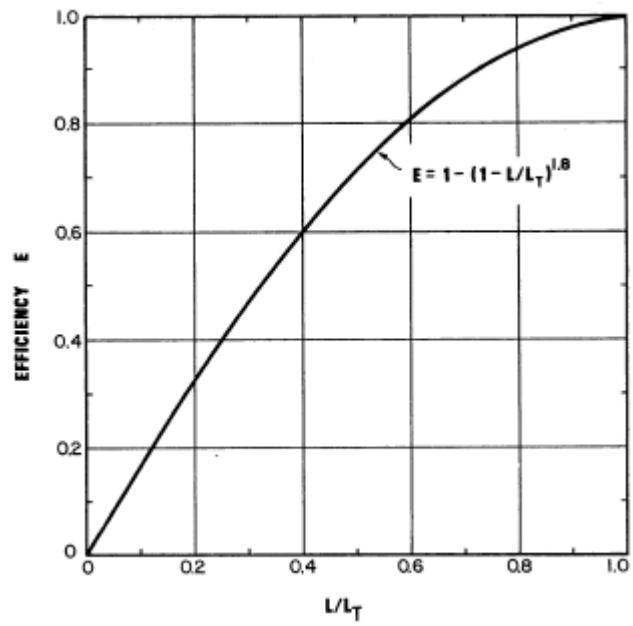


Figure 3-3 Curb-Opening Inlet Interception Efficiency

It is apparent from examination of Figure 3-3 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.

Using the equivalent cross slope, S_e, Equation 3-11 becomes:

$$L_T = 0.6\,Q^{0.42}\,S^{0.3}\left[\frac{1}{nS_e}\right]^{0.6} \qquad (3\text{-}14)$$

Equation 3-14 is applicable with either straight cross slopes or composite cross slopes.

The length of inlet required for a specific interception is determined by rearranging Equation 3-12.

$$L_i = L_T - L_T(1-E)^{0.55} \qquad (3\text{-}15)$$

Where:

L_i	=	curb-opening length required to intercept a specific percent of the flow in the gutter, in feet
L_T	=	curb-opening length required for 100-percent interception, in feet
E	=	efficiency of inlet or percentage of interception

The depressed curb-opening inlet has 54 percent more capacity than the undepressed curb-opening and intercepts 80 percent of the total flow.

Capacity of Curb-Opening Inlets in Sag

The capacity of a curb-opening inlet in sag depends on water depth at the curb as well as the curb-opening length and height. The inlet operates as a weir up to the depth 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 flow in this range can be approximated by solving for the 1.0 and 1.4 height opening values and interpolating.

The weir location for a depressed curb-opening inlet is at the edge of the gutter, and the effective weir length is dependent on the width of the depressed gutter and the length of the curb-opening. The weir location for a curb-opening inlet that is not depressed is at the lip of the curb-opening, and its length is equal to that of the inlet. Limited experiments and extrapolation of the test results on depressed inlets indicate that the weir coefficient for curb-opening inlets without depression is approximately equal to that for a depressed curb-opening inlet.

The equation for the interception capacity of a depressed curb-opening inlet operating as a weir is:

$$Q_i = 2.3(L + 1.8W)\, d^{1.5} \qquad (3\text{-}16)$$

Where:

Q_i	=	intercepted flow by inlet, in cubic feet per second
L	=	length of curb opening, in feet
W	=	lateral width of depression, in feet
d	=	depth at curb measured from the normal cross slope, in feet, where $d = TS_x$

The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression. Thus, Equation 3-16 for a depressed curb-opening inlet is valid when:

$$d \leq h + a/12 \qquad (3\text{-}17)$$

Where:

d	=	depth at curb measured from the normal cross slope, in feet
h	=	height of curb-opening inlet, in feet
a	=	depth of depression, in inches

The weir equation for curb-opening inlets without a depression, W = 0, becomes Equation 3-18. The inlet operates as a weir when $d \leq h$.

$$Q_i = 2.3\, L d^{1.5} \qquad (3\text{-}18)$$

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 times the height that is for d > 1.4h. The interception capacity can be computed by the following equation:

$$Q_i = 0.67hL(2gd_o)^{0.5} = 0.67A[2g(d_i - h/2)]^{0.5} \quad (3\text{-}19)$$

Where:

Q_i	=	intercepted flow by inlet, in cubic feet per second
h	=	height of curb-opening inlet, in feet
d_o	=	effective head on the center of the orifice throat, in feet
A	=	clear area of opening, in square feet
d_i	=	depth at lip of curb-opening, in feet
h	=	height of curb-opening orifice, in feet, where h = TS_x + a/12

Equation 3-19 is applicable to depressed and undepressed curb-opening inlets, and the depth at the inlet, d_i, includes any gutter depression.

The height of the orifice, h, in Equation 3-19 assumes a vertical orifice opening. As illustrated in Figure 3-4, other orifice throat locations can change the effective depth on the orifice and the dimension $d_o = d_i - h/2$. A limited throat width could reduce the capacity of the curb-opening inlet by causing the inlet to go into orifice flow at depths less than the height of the opening.

The orifice equation for curb-opening inlets with a horizontal or inclined throat is:

$$Q = 0.67hL(2gd_o)^{1/2} \quad (3\text{-}20)$$

Where:

Q	=	flow in gutter at inlet, in cubic feet per second
h	=	orifice throat width, in feet
L	=	length of curb-opening, in feet
g	=	acceleration of gravity, 32.2 feet per second squared
d_o	=	effective head on the center of the orifice throat, in feet, where $d_o = d_i - h/2$ (see Figure 3-4)

3.2.8 Grate Opening Inlets

Grate inlets intercept all of the gutter flow passing over the grate (frontal flow) if the grate is sufficiently long and the gutter flow velocity is low. Only a portion of the frontal flow is intercepted if the velocity is high or the grate is short and splash-over occurs. Part of the flow along the side of the grate will also be intercepted, dependent on the cross slope of the pavement, the length of the grate, and flow velocity.

Grate Inlets on a Grade

The ratio of frontal flow to total gutter flow, E_o, for a straight cross slope is:

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{8/3} \tag{3-21}$$

Where:

Q	=	total gutter flow, in cubic feet per second
Q_w	=	frontal flow through width W, in cubic feet per second
W	=	width of grate, in feet
T	=	total spread of water in gutter, in feet

The ratio of side flow, Q_s, to total gutter flow is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \tag{3-22}$$

Where:

Q_s	=	side flow, in cubic feet per second
Q	=	total gutter flow, in cubic feet per second
Q_w	=	frontal flow through width W, in cubic feet per second
E_o	=	ratio of frontal to total gutter flow

The ratio of intercepted frontal flow to total frontal flow, R_f, is expressed by the following equation. This ratio is equivalent to frontal flow interception efficiency.

$$R_f = 1 - 0.09(V - V_o) \tag{3-23}$$

Where:

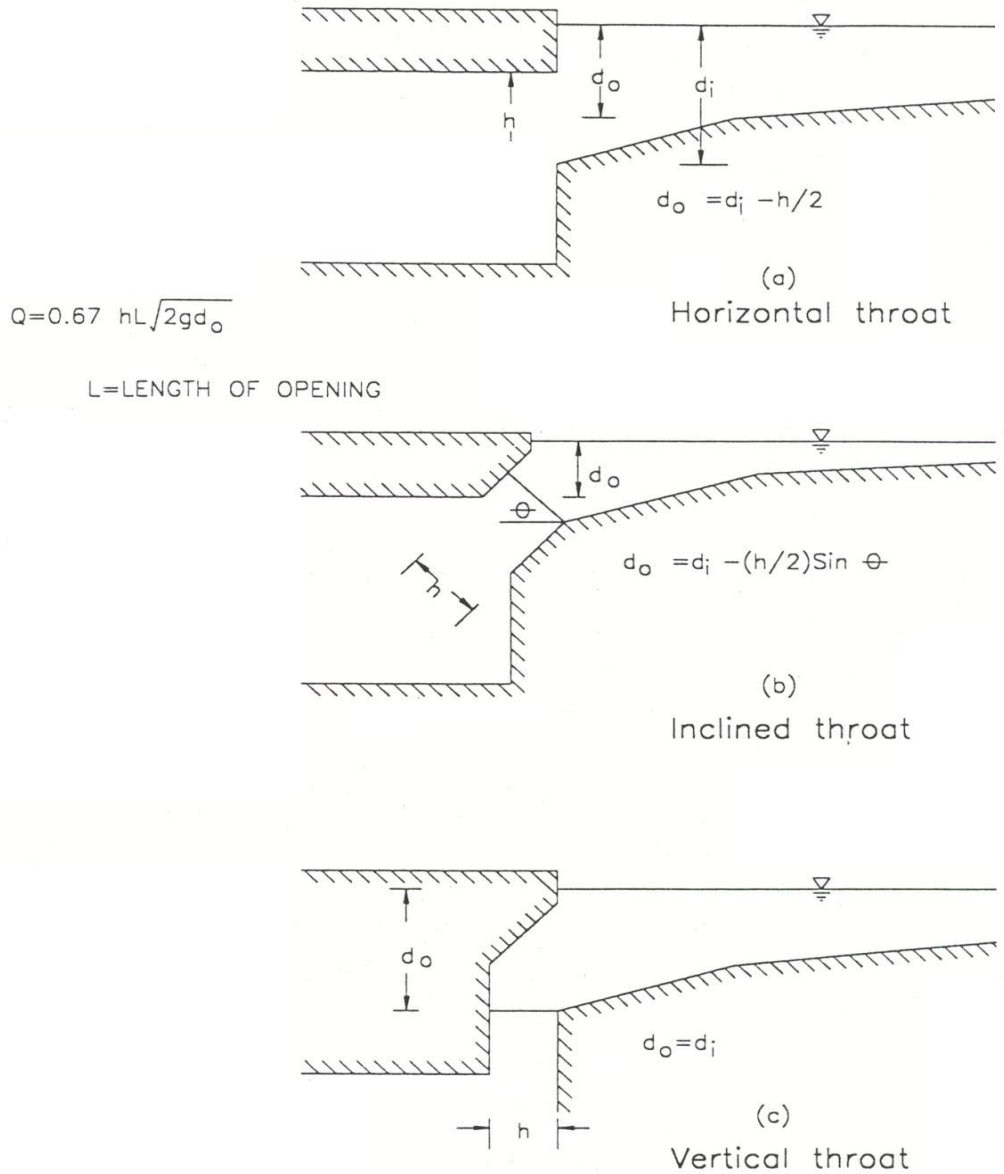
R_f	=	frontal flow interception efficiency,
V	=	velocity of flow in the gutter, in feet per second
V_o	=	gutter velocity where splash-over first occurs, in feet per second

The ratio of intercepted side flow to total side flow, R_s, or side flow interception efficiency, is expressed by the following equation.

$$R_s = \frac{1}{1 + \frac{0.15V^{1.8}}{S_x L^{2.3}}} \tag{3-24}$$

Where:

R_s	=	side flow interception efficiency
V	=	velocity of flow in the gutter, in feet per second
S_x	=	cross slope, in feet per foot
L	=	length of the grate, in feet



Source: Federal Highway Administration, HEC No. 12, Drainage of Highway Pavements.

Figure 3-4 Curb-Opening Inlets

$$R_f = 1 - 0.09(V - V_o) \tag{3-23}$$

Where:

R_f	=	frontal flow interception efficiency,
V	=	velocity of flow in the gutter, in feet per second
V_o	=	gutter velocity where splash-over first occurs, in feet per second

The ratio of intercepted side flow to total side flow, R_s, or side flow interception efficiency, is expressed by the following equation.

$$R_s = \frac{1}{1 + \frac{0.15V^{1.8}}{S_x L^{2.3}}} \tag{3-24}$$

Where:

R_s	=	side flow interception efficiency
V	=	velocity of flow in the gutter, in feet per second
S_x	=	cross slope, in feet per foot
L	=	length of the grate, in feet

The efficiency, E, of a grate is expressed as:

$$E = R_f E_o + R_s (1 - E_o) \tag{3-25}$$

Where:

E	=	grate inlet on grade efficiency
R_f	=	frontal flow interception efficiency
E_o	=	ratio of frontal flow to total gutter flow
R_s	=	side flow interception efficiency

The first term on the right side of Equation 3-25 is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term becomes insignificant in event of high velocities and short grates.

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate inlet on grade multiplied by the total gutter flow as shown in the following equation.

$$Q_i = EQ = Q[R_f E_o + R_s (1 - E_o)] \tag{3-26}$$

Where:

Q_i	=	intercepted flow, in cubic feet per second
E	=	grate inlet on grade efficiency

Q	=	total gutter flow, in cubic feet per second
R_f	=	frontal flow interception efficiency
E_o	=	ratio of frontal flow to total gutter flow
R_s	=	side flow interception efficiency

Grate Inlets in Sag

A grate inlet in sag operates first as a weir having a crest length approximately equal to the outside perimeter, P, along which the flow enters. Bars are disregarded and the side against the curb is not included in computing "P." Weir operation continues to a depth, d, of about 0.4 feet above the top of grate and the discharge intercepted by the grate is:

$$Q_i = 3.0Pd^{3/2} \tag{3-27}$$

Where:

Q_i	=	rate of discharge into the grate opening, in cubic feet per second
P	=	perimeter of grate opening, in feet, disregarding bars and neglecting the side against the curb
d	=	depth of water at grate (for d ≤ 0.4 feet), in feet

When the depth at the grate, d, exceeds about 1.4 feet, the grate begins to operate as an orifice and the discharge intercepted by the grate is:

$$Q_i = 0.67A_g(2gd)^{1/2} = 5.37A_g d^{1/2} \tag{3-28}$$

Where:

Q_i	=	rate of discharge into the grate opening, in cubic feet per second
A_g	=	clear opening of the grate, in square feet
g	=	acceleration of gravity, 32.2 feet per second squared
d	=	depth of ponded water above grate (for d > 1.4 feet), in feet

Between depths of about 0.4 feet and about 1.4 feet over the grate, the operation of the grate inlet is indefinite due to vortices and other hydraulic disturbances. The capacity of the grate is somewhere between that given by the above equations. The flow in this range can be approximated by solving for both d = 0.4 and d = 1.4 feet and interpolating between calculated values.

3.2.9 Combination Opening Inlets

Combination Inlets on a Grade

The interception capacity of combination inlets where the curb opening and the grate are placed side-by-side does not increase appreciably over the capacity of a grate alone. The combination inlet capacity is computed by neglecting the capacity of the curb opening in this situation. A curb opening at combination inlets is sometimes placed upstream of the grate. The curb opening of such design

intercepts debris which might otherwise clog the grate. This type of combination inlet where the curb opening (or slotted drain) is upstream of the grate has an interception capacity equal to the sum of the interception capacities of two inlets except that the frontal flow, and thus the interception capacity of the grate, is reduced by interception by the curb opening. Therefore, the individual capacities are calculated based on two different flows as presented in Figure 3-5.

Gutter flow, Q, is the flow upon which curb opening interception capacity calculation is based and "Q_i" is the flow intercepted by the curb opening. Therefore, the flow approaching the grate, Q', is equal to Q - Q_i. The grate flow interception capacity is calculated based on "Q'".

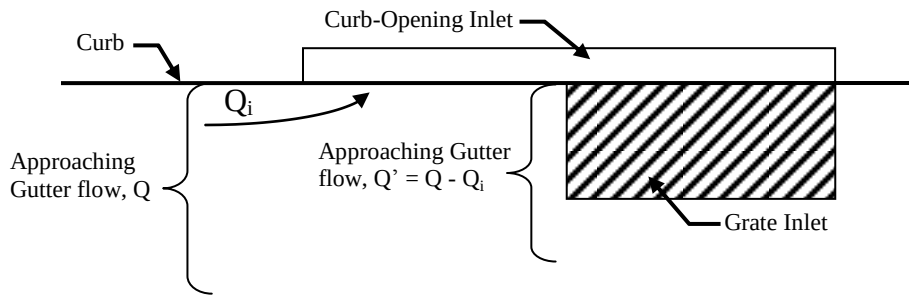


Figure 3-5 Combination Inlet Flow Schematic

Combination Inlets in Sag

Combination inlets consisting of a grate and a curb opening are favorable in sags where clogging of grates can create ponding that is hazardous to traffic. The interception capacity of the combination inlet is essentially equal to that of a grate alone in weir flow. In orifice flow, the capacity is equal to the capacity of the grate plus the capacity of the curb opening.

Where depth at the curb is such that orifice flow occurs, the interception capacity of the inlet is computed by adding Equations 3-20 and 3-28.

$$Q_1 = 0.67 A_g (2gd)^{1/2} + 0.67 hL[2g(d_i - h/2)]^{1/2} \qquad (3\text{-}29)$$

Where:

A_g	=	clear area of the grate, in square feet
g	=	acceleration of gravity, 32.2 feet per second squared
d	=	depth at the curb, in feet
h	=	height of curb opening, orifice, in feet
L	=	length of curb opening, in feet
d_i	=	depth at lip of curb opening, in feet

3.3 CONVEYANCE SYSTEM

It is the purpose of this section to consider the significance of the storm sewer's hydraulic elements and their appurtenances to a storm drainage system. Hydraulically, storm drainage systems consist of pipes (opened or closed) which convey unsteady and non-uniform free flowing stormwater. Steady flow conditions may or may not be uniform.

All storm sewer systems shall be designed by the application of Manning's Equation when flowing in open channel conditions. The hydraulic grade line shall be checked to determine if the open channel flow assumption is valid. In the preparation of hydraulic designs, a thorough investigation shall be made of all existing structures and their performance on the waterway of interest.

The design of a storm drainage system shall be governed by the following seven conditions:

- The system must accommodate the surface runoff resulting from the selected design storm with no damage to physical facilities and minimum interruption of normal traffic.
- Runoff resulting from major storms must be anticipated and discharged free from impedance without damage to physical facilities (such as conveyance past finished floor elevations of buildings and under roadways without washing out embankments and subgrades).
- The storm drainage system must have a maximum reliability of operation with respect to being structurally sound to its environment where it is placed and performing hydraulically to its intended function for the entire life of design.
- The construction costs of the system must be reasonable with relationship to the importance of the facilities it protects.
- The storm drainage system must require minimum maintenance (cleaning and clearing obstructions) and must be accessible for maintenance operations.
- The storm drainage system must be adaptable to future expansion with minimum additional cost and designed to accommodate build-out conditions in the upstream reaches of the drainage area.
- Site design, swales and natural flow features should be utilized to reduce the need for extensive storm sewer systems whenever possible.

3.3.1 Storm Sewer Pipe System

<u>Pipe Sizes and Material Types</u>

Pipes which are to become an integral part of the public storm sewer system shall have a minimum diameter of 15 inches for gravity flow. If alternate shapes are required for utility clearance or special conditions, the designer must contact the Town for approval. All reinforced concrete storm sewers shall meet, at a minimum, the requirement of AASHTO M170 Classes III-V. All pipe design and installation must meet the manufacturers' recommendations for minimum depth of cover.

The pipe design life is a key consideration in selection of pipe material. Pipe design life shall be a minimum 50 years as certified by the manufacturer. The engineer must meet all manufacturers'

requirements on which the design life is based. For example, bedding requirements are critical to meeting the pipe design life.

In selecting a roughness coefficient, "n", consideration shall be given to the average conditions during the useful life of the structure. An increased "n" value shall be used primarily in analyzing old pipes where alignment is poor and joints have become rough. If, for example, concrete pipe is being designed for a location and there is reason to believe that the roughness would increase due to erosion or corrosion of the interior surface, slight displacement of joints, or entrance of foreign material, a roughness coefficient should be selected which, in the judgment of the designer, will represent the average condition. Any selection of "n" values below the minimum or above the maximum, either for monolithic concrete structures, concrete pipe, or corrugated metal pipe, shall have the written approval of the Town. The coefficients of roughness listed in Table 3-2 shall be used in the application of Manning's Equation for storm sewer sizing.

TABLE 3-2
MANNING'S ROUGHNESS COEFFICIENTS FOR STORM SEWERS

Materials of Construction	Design Coefficient[1]
Concrete Pipe	0.012 - 0.015
Corrugated Metal	0.024 - 0.027

[1] Designer may select a single representative "n"

Manhole Location

Manholes shall be located at pipe junctions; changes in alignment, size, and slope; and ends of curved sections.

Manholes shall be located at intervals not to exceed 300 feet for pipe 30 inches in diameter or smaller.

Manholes for pipe larger than 30 inches in diameter, along straight alignments, shall be located at points where design requirements indicate entrance into the pipe is desirable. In no case shall the distance between openings or entrances to the storm sewer system be greater than 1,000 feet.

Alignment

In general, storm sewer alignment between manholes shall be straight. Long radius curves may be allowed to conform to street alignment. Short radius curves may be used on larger pipes in order to reduce head losses at junctions. Curves may be produced by angling the joints or by fabricating beveled ends. Angled joints shall be kept at a minimum to maintain a tight joint. Pipe deflection shall not exceed manufacturers' recommendations, unless precast or cast-in-place bends are specifically designed for deflection. All curved alignments must be approved by the Town prior to installation.

Minimum Grades

Storm sewers should operate with flow velocities sufficient to prevent excessive deposition of solid material, which would result in clogging. The controlling velocity occurs near the bottom of the pipe and is considerably less than the mean velocity. Storm sewers shall be designed to have a minimum mean velocity flowing full of 2.0 feet per second, the lower limit of scouring velocity. Table 3-3 indicates the grades for both smooth wall pipe (n = 0.012) and corrugated wall pipe (n = 0.024) to produce a velocity of 2.0 feet per second. Any variance must be approved by the Administrator(s), or their designated representative. Outlets of pipes on minimum grade should be designed to avoid sedimentation at the outfall.

Maximum velocities in pipes are important mainly because of the possibilities of excessive erosion on the storm sewer inverts. Table 3-4 lists the limits of maximum velocity. Energy dissipaters shall be required at outfalls when pipe flow velocities exceed channel scour velocities.

TABLE 3-3
MINIMUM SLOPE REQUIRED FOR SCOURING VELOCITY[1]

Pipe Size (inches)	Smooth Wall Pipe ("n"=0.012) Slope (ft/ft)	Corrugated Wall Pipe ("n"=0.024) Slope (ft/ft)
12	0.0016	0.0066
15	0.0012	0.0049
18	0.0010	0.0038
21	0.0008	0.0031
24	0.0007	0.0026
27	0.0006	0.0022
30	0.0005	0.0019
36	0.0004	0.0015
42	0.0003	0.0012
48	0.0003	0.0010
54	0.0002	0.0009
60	0.0002	0.0008
66	0.0002	0.0007
72	0.0002	0.0006
78	0.0001	0.0005
84	0.0001	0.0005
96	0.0001	0.0004

[1] Assume pipe flowing full

TABLE 3-4
MAXIMUM VELOCITY IN STORM SEWERS

Description	Maximum Permissible Velocity
Culverts (all types)	15 ft/s
Storm Sewers (collectors)	15 ft/s
Storm Sewers (mains)	12 ft/s

3.3.2 Flow in Storm Sewers

All storm sewers shall be designed by the application of the Continuity Equation and Manning's Equation, either through the appropriate charts and nomographs or by direct solutions of the equations as follows:

$$Q = AV \tag{3-30}$$

$$Q = \frac{1.49}{n} A R^{2/3} S_f^{1/2} \tag{3-31}$$

Where:

Q	=	pipe flow, in cubic feet per second
A	=	cross-sectional area of pipe, in square feet
V	=	velocity of flow, in feet per second
n	=	Manning's roughness coefficient of pipe
R	=	hydraulic radius = A/WP, in feet
WP	=	wetted perimeter, in feet
S_f	=	friction slope of pipe, in feet per foot

There are several general rules to be observed when designing storm sewer sections. These rules are as follows:

- Select pipe size and slope so that the velocity of flow will increase progressively, or at least not appreciably decrease, at inlets, bends, or other changes in geometry or configuration.

- Design system so that the contents of a larger pipe do not discharge into a smaller one, even though the capacity of the smaller pipe may be greater due to steeper slope.

- Match the inside top surface of the soffits rather than the flow lines of the pipes at points where pipe size changes from a smaller to a larger pipe (when necessary, working with minimum slopes, match the point of each pipe that represents 0.8 the distance of the diameter measured from the invert.)

- Check pipes at the time of their design with reference to critical slope. If the slope of the energy line is greater than critical slope, the unit will likely be operating under entrance control instead of the originally assumed normal flow. Pipe slope should be kept below

critical slope if at all possible. This also removes the possibility of a hydraulic jump within the line.

3.3.3 Energy Gradient and Profile of Storm Sewers

When using Bernoulli's Equation in the hydraulic design of storm sewers, all energy losses must be accounted for. These losses are commonly referred to as head losses, and are classified as either friction losses or minor losses. Friction losses are due to forces between the fluid and the boundary material, while minor losses are a result of the geometry of sewer appurtenances such as manholes, bends, and either expanding or contracting transitions. Minor losses can constitute a significant portion of the total head loss.

When storm sewer systems are designed as flowing full, the designer shall establish the head losses caused by flow resistance in the pipe, changes of momentum, and interference at junctions and structures. This information is then used to establish the design water surface elevation at each structure.

It is not necessary to compute the energy grade line of a pipe section if all three of the following conditions are satisfied:

- The slope(s) and the pipe size(s) are chosen so that the slope is equal to or greater than the friction slope.
- The inside top surfaces (soffit) of successive pipes are lined up at size changes.
- The water surface at the point of discharge will not rise above the top of the outlet.

In such cases, the pipe should not operate under pressure and the slope of the water surface under capacity discharge will approximately parallel the slope of the pipe invert, assuming the minor losses are reasonable.

In the absence of these conditions or when it is desired to check the system against a larger flood than that used in sizing the pipes, the hydraulic and energy grade lines shall be computed and plotted. The friction head loss shall be determined by direct application of Manning's Equation. Minor losses due to turbulence at structures shall be determined by the procedure described below. If there is a possibility that the storm sewer system will be extended at some future date, present and future operation of the system must be considered.

The final hydraulic design of a system should be based on the procedures set forth in this Manual. The pipes are treated as either open channel flow or pipe flowing full flow, as the case may be. For open channel flow, the energy grade line is used as a base for calculation, while the hydraulic grade line is used for pipe flowing full flow. The following Friction Head Loss procedure is applicable to storm sewers flowing with a free water surface, or open channel flow. The basic approach to the design of open channel flow in storm sewers is to calculate the energy grade line along the system. It is assumed that the energy grade line is parallel to the pipe grade and that any losses other than pipe friction may be accounted for by assuming point losses at each manhole.

3.3.4 Friction Head Loss

The pipe friction can be evaluated by modifying the Manning's Equation.

$$S_f = \left[\frac{Qn}{1.49AR^{2/3}} \right]^2 \tag{3-32}$$

Where:

S_f	=	friction slope of pipe, in feet per foot
Q	=	pipe flow, in cubic feet per second
n	=	Manning's roughness coefficient
A	=	cross-sectional area of pipe, in square feet
R	=	hydraulic radius, A/WP, in feet
WP	=	wetted perimeter, in feet

The pipe friction head loss is equal to the friction slope of the pipe multiplied by the pipe length.

$$h_f = S_f L \tag{3-33}$$

Where:

h_f	=	pipe friction head loss, in feet
S_f	=	friction slope of pipe, in feet per foot
L	=	length of pipe, in feet

3.4 OUTLET PROTECTION

The function of outlet protection is to dissipate the energy of concentrated stormwater flows thereby reducing erosion or scouring at stormwater outlets and paved channel sections. In addition, outlet protection lowers the potential for downstream erosion. This type of protection can be achieved through a variety of techniques, including permanent turf reinforcement mats, stone or riprap, concrete aprons, paved sections, or other structural measures.

3.4.1 Design Criteria for Outlets

The outlet design for pipes and channel sections applies to the immediate outlet area or reach below the pipe or channel and does not apply to continuous lining and protection of channels or streams. Notably, pipe or channel outlets at the top of cut slopes or on slopes steeper than 10 percent should not be protected using just outlet protection. This causes re-concentration of the flow when the flow leaves the protection area resulting in large velocities.

Outlet protection may be designed according to the following criteria:

Flow Velocity

The flow velocity at the outlet when flowing at design capacity shall not exceed the permissible velocity of the receiving unprotected grass-lined channel.

Tailwater Depth

The depth of tailwater immediately below the pipe outlet must be determined for the design capacity of the pipe. Manning's Equation may be used to determine tailwater depth. If the tailwater depth is less than half the outlet pipe diameter, it should be classified as a minimum tailwater condition. If the tailwater depth is greater than half the outlet pipe diameter, it should be classified as a maximum tailwater condition. Pipes which discharge onto level areas with no defined channel may be assumed to have a Minimum Tailwater Condition classification.

Protection Length

The required protection length, La, according to the tailwater condition, should be determined from the appropriate graphs provided in Figures 3-6 and 3-7 of this chapter:

Minimum Tailwater Condition - Use Figure 3-6
Maximum Tailwater Condition - Use Figure 3-7

Protection Width

When the pipe discharges directly into a well-defined channel, the protection should extend across the channel bottom and up the channel banks to an elevation one foot above the maximum tailwater depth or to the top of the bank, whichever is less.

If the outlet discharges onto a flat area with no defined channel, the width of the protection should be determined as follows:

- The upstream end of the protection, adjacent to the outlet, should have a width three times the diameter of the outlet pipe (3D).
- For a minimum tailwater condition, the downstream end of the protection should have a width equal to the pipe diameter plus the length of the apron (D + La).
- For a maximum tailwater condition, the downstream end of the protection should have a width equal to the pipe diameter plus 0.4 times the length of the apron (D +0.4* La).

Bottom Grade

The protection shall be constructed with no slope along its length (0 percent grade) where applicable. The downstream invert elevation of the protection should be equal to the elevation of the invert of the receiving channel. There shall be no overfalling at the end of the protection.

Side Slopes

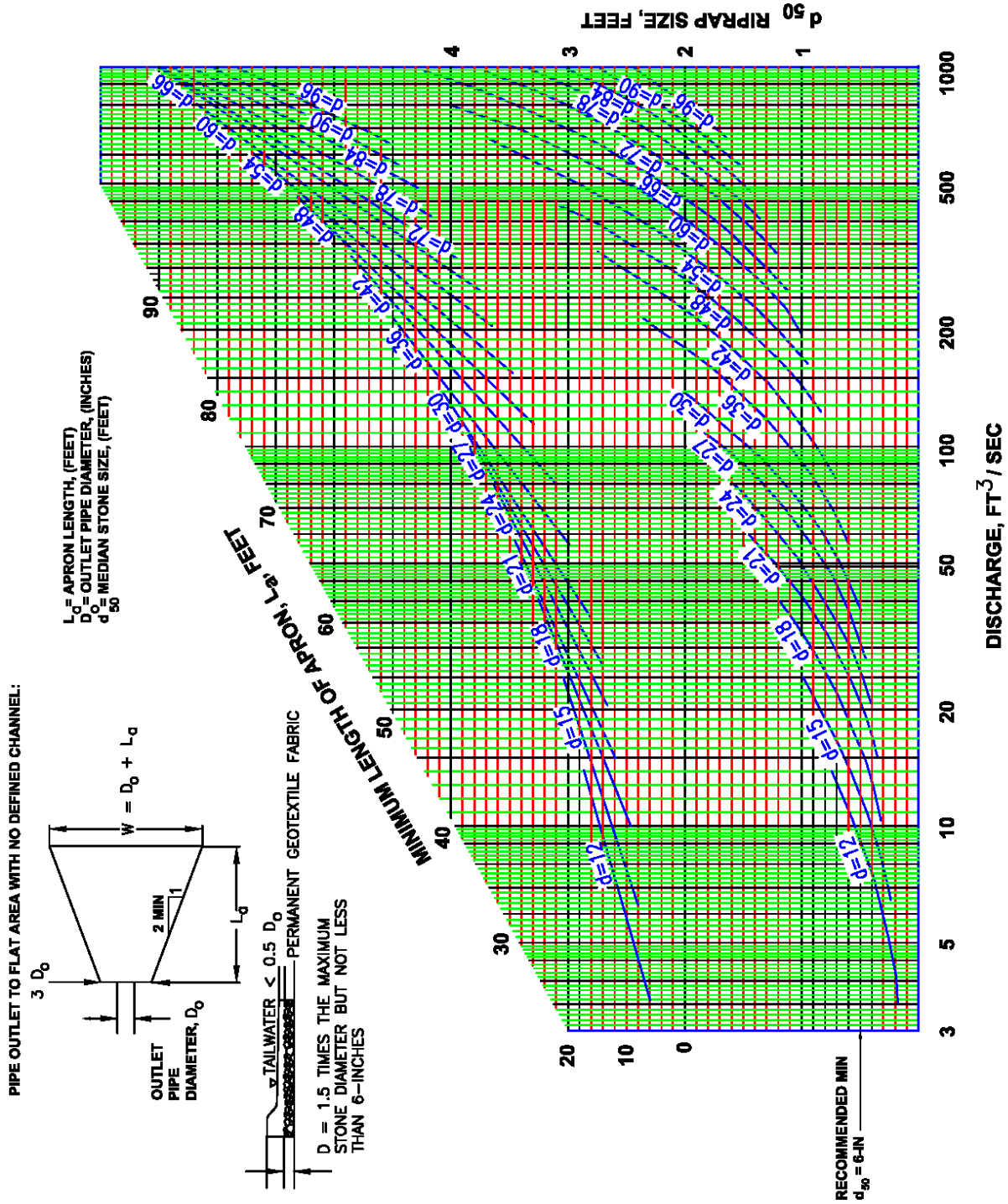
If the outlet discharges into a well-defined channel, the receiving side slopes of the channel should not be steeper than 3:1 (H:V).

Alignment

The protection should be located so there are no bends in the horizontal alignment.

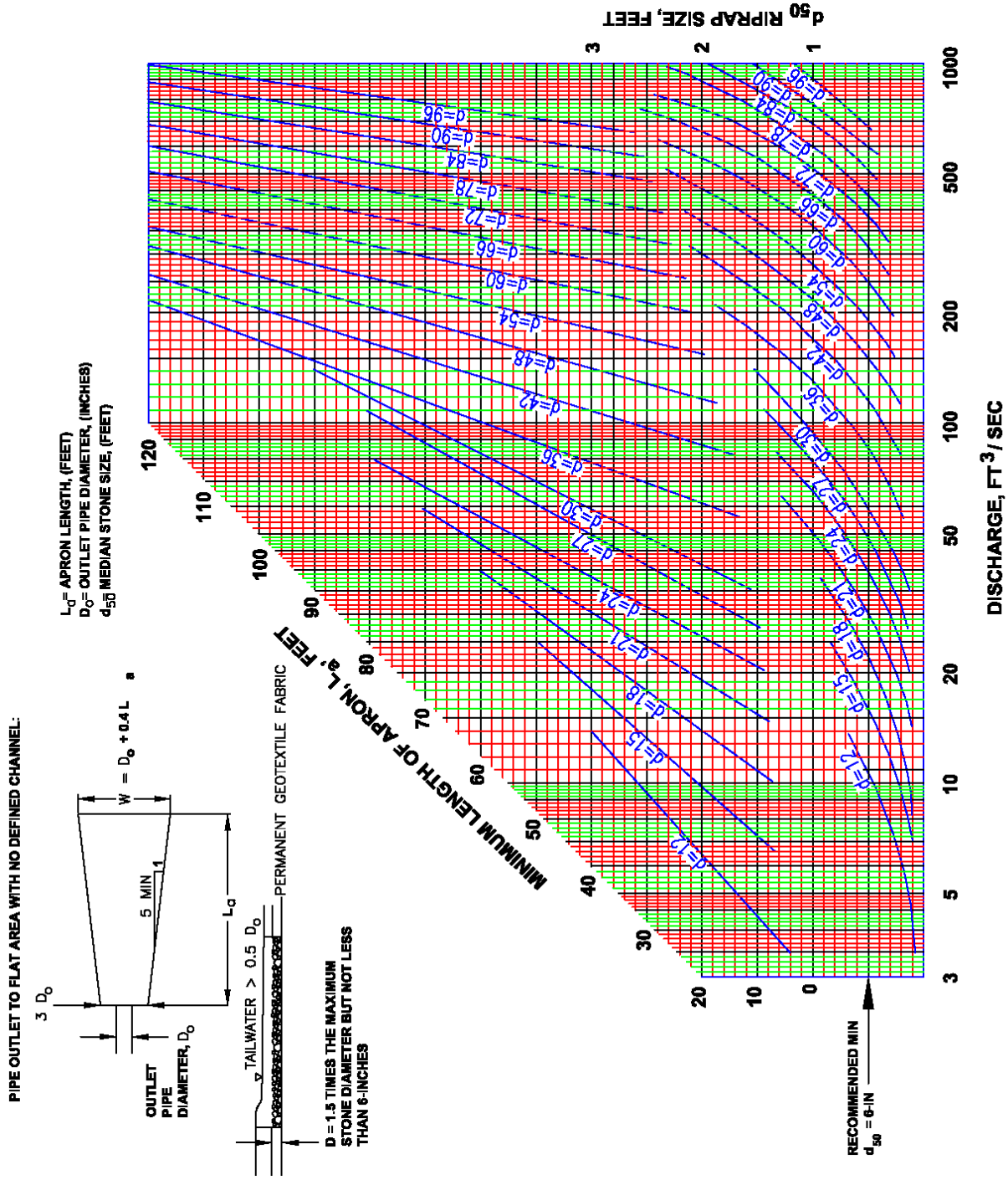
Materials

- The preferred protection shall be lined with an appropriate permanent turf reinforcement matting (TRM). The shear stress and maximum velocity shall be calculated to determine which type of TRM is applicable for the given situation.
- When conditions are too severe for TRM, the protection may be lined with riprap or other appropriate means. The median-sized stone for riprap may be determined from the curves in Figure 3-6 and Figure 3-7 according to the tailwater condition.
- In all cases, a non-woven geotextile filter cloth should be placed between the riprap and the underlying soil to prevent soil movement into and through the riprap.



DESIGN OF OUTLET PROTECTION FROM A ROUND PIPE FLOWING FULL MINIMUM TAILWATER CONDITIONS (T_W < 0.5 DIAMETER)

Figure 3-6 Design of Outlet Protection – Minimum Tailwater Conditions



DESIGN OF OUTLET PROTECTION FROM A ROUND PIPE FLOWING FULL
MAXIMUM TAILWATER CONDITIONS (T_W > 0.5 DIAMETER)

Figure 3-7 Design of Outlet Protection – Maximum Tailwater Conditions

4.0 CULVERTS AND BRIDGES

The function of a culvert or bridge is to safely pass the peak flow generated by the design storm under a roadway, railroad, or other feature. The culvert or bridge design shall not cause excessive backwater or velocities. The design of a culvert must take into account the different engineering and technical aspects of the culvert site and adjacent areas which may be impacted by the design.

4.1 DESIGN CRITERIA

The following design criteria apply for bridge and culvert design crossings.

4.1.1 Storm Event Design Frequency

- Residential and collector roadways cross-drain culverts – 25-year storm
- Arterial road culverts – 50-year storm event
- All culvert analyses shall demonstrate passage of the 100-year storm event without damage to physical facilities (such as conveyance past finished floor elevations of buildings and under roadways without washing out embankments and subgrades)

4.1.2 Discharge Velocities

Inlet and outlet flow velocities shall not impact channel stability. Scour analyses shall be performed for critical culvert and bridge structures, as identified by the Administrator(s), municipality, state or, federal jurisdictions to determine necessary channel and structure protection. At a minimum, all inlet and outlet locations and other locations impacted by flow velocities near structures shall include design of channel protection where erosive flow velocities occur.

Culverts shall be designed to have a minimum mean velocity flowing full of 2.0 feet per second, the lower limit of scouring velocity.

4.1.3 Culvert Material Types

Material for culverts under roadways or under driveways within right-of-way shall consist of reinforced concrete pipe (RCP). The minimum allowed strength of RCP shall be Class III.

Material for culverts under privately owned and maintained drives not in a public right-of-way may consist of reinforced concrete pipe (RCP), aluminum or aluminized steel corrugated pipe with approved gasket joints, solid or corrugated PVC walls with wall stiffness of 46 or greater or centrifugally cast fiberglass reinforced polymer mortar (CCFRPM) sewer pipe. Joint construction and pipe installation shall be in accordance with sewer pipe standards.

Material for culverts for installations in grassed areas, in areas that do not have traffic bearing loads, or that do not have surcharge loads from embankment may consist of high density polyethylene (HDPE) pipe with smooth interior liner. HDPE pipe connections shall be watertight in areas where either elevated seasonal high ground water may be evident or where hydraulic surcharge of the system will occur.

The Manning's "n" values for culvert materials are presented in Table 4-1.

TABLE 4-1
MANNING'S "n" VALUES

Conduit Physical Shape	Wall & Joint Description	Manning's "n"
Concrete Pipe	Good joints, smooth walls	0.013
	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 with Annular Corrugations	2 2/3 inch x 1/2 inch corrugations	0.024
	6 inch x 1 inch corrugations	0.025
	5 inch x 1 inch corrugations	0.026
	3 inch x 1 inch corrugations	0.028
	6 inch x 2 inch structural plate	0.035
	9 inch x 2 1/2 inch structural plate	0.035
Corrugated Metal Pipes Helical Corrugations, Full Circular Flow	2 2/3 inch x 1/2 inch corrugated 24 inch plate width	0.012
Spiral Rib Metal Pipe	3/4 inch x 3/4 inch in recesses at 12 inch spacing, good joints	0.013
High Density Polyethylene (HDPE)	Corrugated Smooth Liner	0.011
	Corrugated Interior	0.024
Polyvinyl Chloride (PVC)		0.011

Note: For further information concerning Manning's "n" values for selected conduits consult Federal Highway Administration, Hydraulic Design of Highway Culverts, HDS No. 5, page 163, 2001.

4.1.4 Geometry

The culvert shall be of adequate length to join or match required headwalls, sloping of embankments, end wall treatments, and any other inlet/outlet protection improvements. The longitudinal slope shall conform to existing naturalized channel slope. The culvert invert shall not impede flows along the bottom of the open channel. Culvert skew shall not exceed approximately 30 degrees. The minimum pipe diameter (round or arch) shall be 15 inches; the minimum box dimension shall be 3 by 6 feet. Bridge passages shall be designed not to substantially impact flow characteristics.

4.1.5 Culvert End Treatments

Culvert headwalls shall be used at pipe inlets larger than 24 inches in diameter or rise. Design of inlets should not impact embankment stability or erosion. A combination of headwall and wingwalls may be allowed where limiting the length of embankment side slopes, property impacts, or other conditions necessitate their use. Exposure of end treatment configurations should not have adverse effect upon adjacent activities or uses. Barriers, buffers, or other means of access restriction shall be designed as necessary.

Hardened concrete inlet aprons may be utilized to provide inlet channel protection. The apron shall extend a minimum length of one pipe diameter along the flow channel and conform to the channel bottom. If the exit velocity is high and/or receiving channel conditions are prone to erosion and destabilization of the channel, the outfall energy dissipators for culverts or channel armor shall be required. Materials may include properly sized stone riprap on geotextile, stilling basins, hardened control devices, or natural structures, designed in accordance with FHWA HEC No. 14, Hydraulic Design of Energy Dissapators for Culverts and Channels, 1983 or as otherwise approved by Administrator(s).

4.1.6 Hydraulic Considerations

If the project location is in the area of a mapped 100-year frequency base flood elevation shown on the Federal Emergency Management Agency's (FEMA) Flood Insurance Rate Map (FIRM), the design must follow the National Flood Insurance Program guidelines.

Headwater Limitations

Headwater (HW) is the depth of water above the culvert invert at the upstream (entrance) end of the culvert. The allowable headwater elevation is determined from an evaluation of land use upstream, roadway or embankment elevation, and acceptable elevation for flood passage. Culverts with a rise of 42 inches or smaller shall not be subjected to an HW of greater than 1.6 times the rise of the culvert for up to a 25-year storm event. Culverts with a rise of 48 inches and larger shall not be subjected to an HW of greater than 0.8 times the rise of the culvert for a 25-year storm event and not greater than 1.25 times the rise of the culvert for a 50-year storm event. Resulting HW of culverts shall not increase, adversely impact, or result in a surface water elevation increase that is unacceptable or greater than 1 foot at areas upstream of the culvert. Design shall demonstrate that flow passes safely around the culvert and that headwater or tailwater elevations from the culvert do not endanger property for events including a 100-year storm event.

Tailwater Considerations

Tailwater is the depth of water above the culvert invert at the downstream (outfall) of the culvert. The tailwater depth for a range of discharges must be determined by way of hydraulic evaluation. There may be a need for calculating backwater curves to establish the tailwater conditions. The following site 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.
- If the culvert discharges to an open channel, the stage-discharge curve for the receiving channel must be determined.

- 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. For tidal culverts discharging into an ocean, the mean high and mean low tide elevations must be considered.
- 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.

Storage

If area upstream of the culvert will be utilized as storage during the design storm, the following must be considered:

- The total area of inundation by design storm, and
- The freeboard and bankfull elevation design criteria for open channels outlined in Chapter 4.1 General Design Criteria shall be met.

4.1.7 Culvert Weep Holes

Weep holes are installed to relieve hydrostatic pressure resulting in uplift forces on the culvert structure and shall be included as necessary. The weep holes should be used in conjunction with filter materials in order to intercept the flow and prevent the formation of piping channels. The filters should be designed as under-drain filter structures to prevent clogging.

4.1.8 Maintenance Easement Width

Minimum maintenance easement widths shall be the greater value of either 20 feet or the sum of the pipe diameter (in feet) plus 4 feet plus 2 times the depth from the pipe invert to the existing grade.

4.1.9 Environmental Considerations

Selected culvert design and location should cause the least impact on the stream, wetlands, and wildlife habitat. This selection analysis shall consider the entire impacted site, including any inlet and outlet channels where the stormwater design substantially impacts the existing hydraulic capacity or surface water elevation.

4.1.10 Regulated Floodway Requirements

The culvert/bridge design must be in compliance with National Flood Insurance Program. It is necessary to consider the 100-year frequency flood at the local identified special flood hazard areas. The design professional should review floodway regulations applicable for the project and impacted area.

4.2 CULVERT FLOW

Culverts shall be selected based on hydraulic performance, site conditions and economy. It is necessary to know the design culvert flow regime to properly access its impact. Culvert selection shall include analysis of both inlet and outlet control.

The culvert flow controls for a straight, uniformly shaped culvert are divided into two basic classes depending on the control section: inlet control and outlet control. For each type of control, different

factors and equations are used to compute the hydraulic capacity of the culvert. Inlet control is restricted due to the opening efficiency and opening size. Conversely, outlet control is restricted by friction and by tailwater effects. Both the inlet flow capacity and the outlet flow capacity must be calculated to compare the values and select which condition is most restrictive.

4.2.1 Inlet Control

For a culvert operating under inlet control, the culvert barrel is capable of conveying a greater discharge than the inlet will accept. The flow control section is just inside the culvert barrel at its entrance. The flow profile passes through critical depth at this location and flow in the barrel is supercritical. Conditions downstream of the entrance have no effect on culvert capacity. The barrel flows partially full over its length and the flow approaches normal depth at the outlet end.

Under inlet control, only the headwater and the inlet configuration affect the hydraulic performance. The headwater elevation at the culvert entrance supplies the energy necessary to force flow through a culvert.

The maximum discharge through a culvert flowing partially full occurs when flow is at critical depth for a given energy head. To assure that flow passes through critical depth near the inlet, the culvert must be laid on a slope equal to or greater than critical slope for the design discharge. Placing culverts which are to flow partially full on slopes greater than critical slope will increase the outlet velocities but will not increase the discharge capacity. The section near the inlet at which critical flow occurs limits the discharge.

The capacity of a culvert flowing partially full with control at the inlet is governed by the following equation when the approach velocity is considered zero (Illustration A in Figure 4-1).

$$HW = d_c + \frac{V_2^2}{2g} + h_e \qquad (4\text{-}1)$$

Where:

HW	=	headwater depth, in feet
d_c	=	critical depth of flow, in feet
V_2	=	critical velocity at entrance of culvert, in feet per second
g	=	acceleration of gravity, 32.2 feet per second squared
h_e	=	entrance head loss, in feet

$$h_e = k_e \left[\frac{V_2^2}{2g} \right]$$

k_e	=	entrance loss coefficient (Table 4-2)

The entrance loss 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 4-2.

TABLE 4-2
ENTRANCE LOSS COEFFICIENTS

Type of Structure and Design of Entrance	Coefficient "k_e"
Pipe, Concrete:	
Projecting from fill, socket end (groove 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-edged	0.5
Rounded (radius = 1/12 D)	0.2
Mitered to conform to fill slope	0.7
End section conforming to fill slope[1]	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe or Pipe-Arch, Corrugated Metal:	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End section conforming to fill slope[1]	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete:	
Headwall parallel to embankment (no wingwalls):	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel:	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel:	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides):	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

Source: Federal Highway Administration, Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5, 2001.

[1] "End sections conforming to fill slope" 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. Some end sections incorporating a closed taper in their design have superior hydraulic performance.

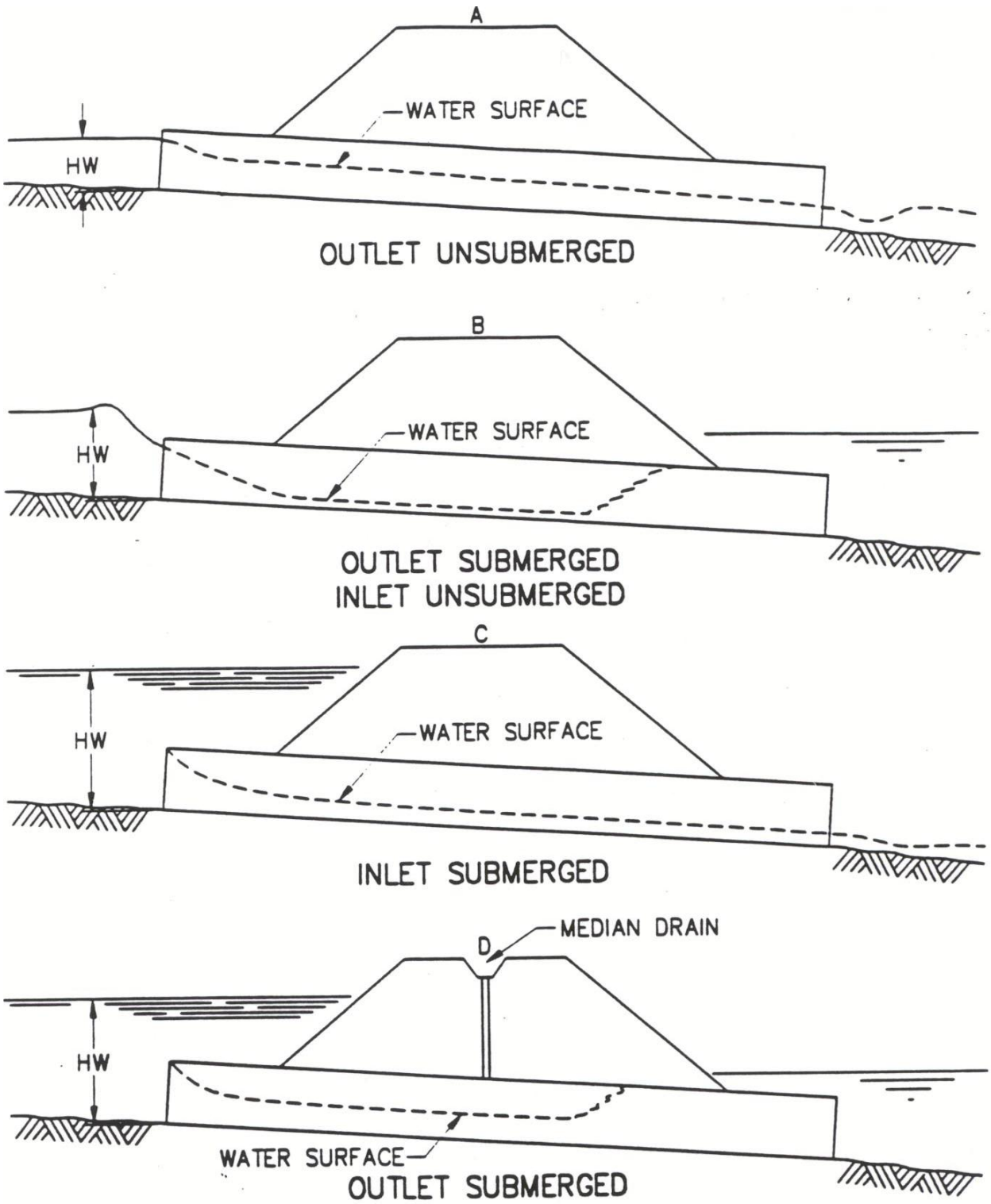


Figure 4-1 Types of Inlet Control

4.2.2 Outlet Control

All the factors affecting the hydraulic performance of a culvert in inlet control also influence culverts in outlet control. In addition, the barrel characteristics (roughness, area, shape, length, and slope) and the tailwater elevation affect culvert performance in outlet control.

The barrel roughness is a function of the culvert material and is represented by Manning's "n" coefficient. The barrel length is the total length extending from the entrance to the exit of the culvert. The barrel slope is the actual slope of the culvert and is often equivalent to the slope of the stream. The tailwater elevation is based upon the downstream water surface elevation measured from the outlet invert. Backwater calculations or normal depth approximations, when appropriate, are two methods used to determine the tailwater elevation. Figure 4-2 depicts several examples of outlet control.

Illustration A in Figure 4-2 represents the classic full flow condition, with both inlet and outlet submerged. The barrel is in pressure flow throughout its length. This condition is often assumed in calculations, but seldom actually exists.

In illustration B of Figure 4-2, the outlet is submerged with the inlet unsubmerged. In this case, the headwater is shallow so that the inlet crown is exposed as the flow contracts into the culvert.

Most culverts flow with free outlet but, depending on topography or downstream constraint, a tailwater elevation sufficient to submerge the outlet may form at some instances. For an outlet to be submerged, the depth at the outlet must be equal to or greater than the diameter of the culvert. The capacity of a culvert flowing full with a submerged outlet is governed by the following equation when the approach velocity is considered zero. Outlet velocity is based on full-pipe flow at the outlet.

$$HW = H + TW - S_o L \qquad (4\text{-}2)$$

Where:

HW	=	headwater depth, in feet
H	=	head for culvert flowing full, in feet
TW	=	tailwater depth, in feet
S_o	=	slope of culvert, in feet per foot
L	=	length of culvert, in feet

Figure 4-2, illustration C shows the entrance submerged to such a degree that the culvert flows full throughout its entire length while the exit is unsubmerged. This is a rare condition. It requires an extremely high headwater to maintain full barrel flow with no tailwater. The outlet velocities are usually high under this condition.

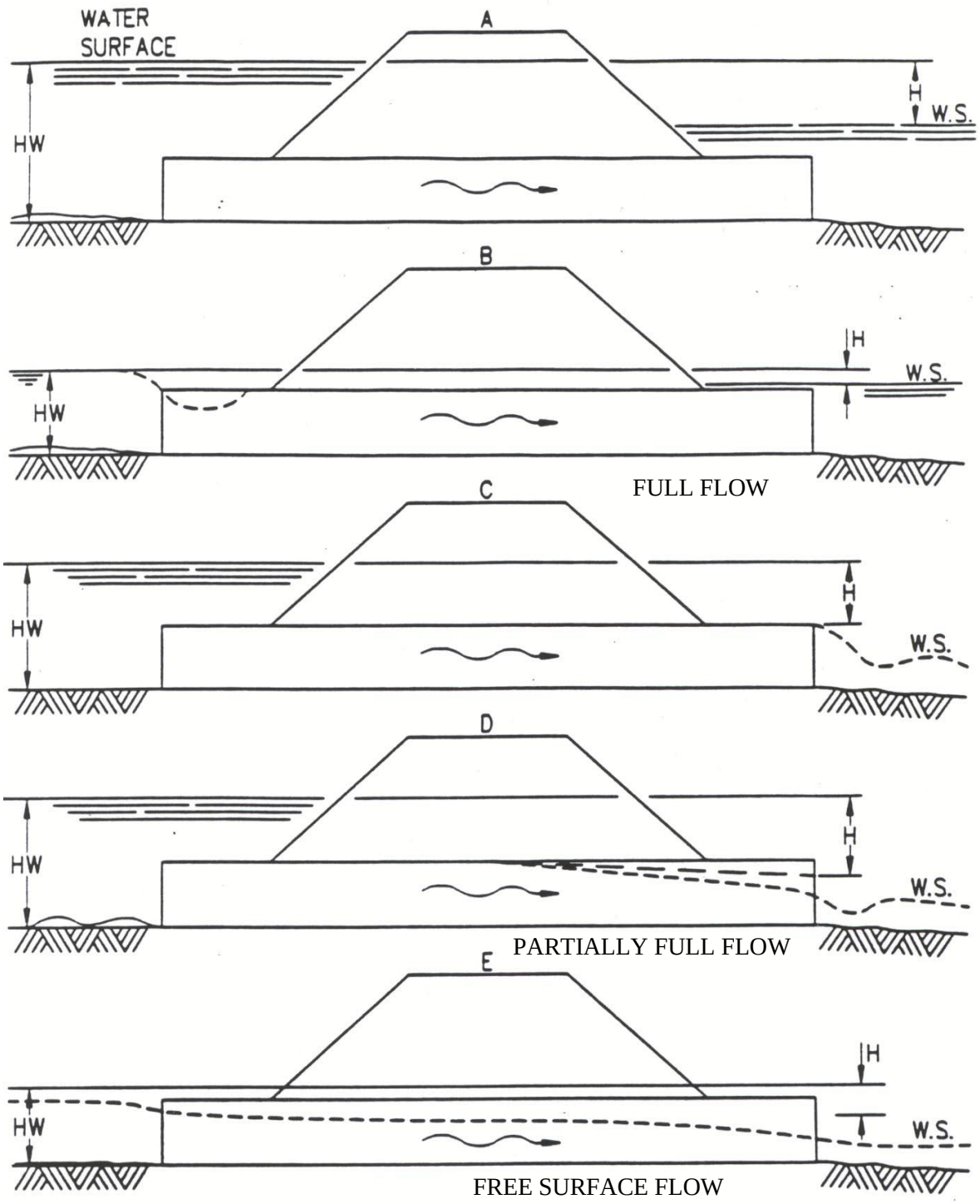


Figure 4-2 Types of Outlet Control

Illustration D in Figure 4-2 is a typical representation of partially full flow. The culvert entrance is submerged by the headwater and the outlet end flows freely with a low tailwater. For this condition, the barrel flows partially full over at least part of its length (subcritical flow) and the flow passes through critical depth just upstream of the outlet.

The capacity of a culvert flowing full over at least part of its length with a submerged entrance (HW $\geq$ 1.2 D) is governed by the following equation when the approach velocity is considered zero.

$$HW = H + P - S_o L \tag{4-3}$$

Where:

HW	=	headwater depth, in feet
H	=	head for culverts flowing full, in feet
P	=	pressure line height, in feet
P	=	$(d_c + D)/2$
d_c	=	critical depth, in feet
D	=	diameter or height of structure, in feet
S_o	=	slope of culvert, in feet per foot
L	=	length of culvert, in feet

Outlet velocity is based on critical depth if tailwater depth is less than critical depth (TW < d_c). If tailwater depth is greater than critical depth (TW > d_c), outlet velocity is based on tailwater depth.

In the condition where neither the inlet nor the outlet end of the culvert are submerged and the flow has free surface, as in Figure 4-2 illustration E, the barrel flows partially full over its entire length and the flow profile can be subcritical or supercritical. The tailwater depth can be above or below critical flow.

If the headwater pool elevation does not submerge the culvert inlet (HW < 1.2D), the slope at design discharge is subcritical ($S_o < S_c$), the tailwater depth is above critical depth (TW $\geq$ d_c), and the control occurs at the outlet. The capacity of the culvert is governed by the following equation when the approach velocity is considered zero:

$$HW = TW + \frac{V_{TW}^2}{2g} + h_e + h_f - S_o L \tag{4-4}$$

Where:

HW	=	headwater, in feet depth must be ≤ 1.2 D
TW	=	tailwater depth, in feet
V_{TW}	=	culvert discharge velocity at tailwater depth, in feet per second
h_e	=	entrance head loss, in feet

$$h_e = k_e \left[\frac{V_E^{\,2}}{2g} \right]$$

V_E	=	velocity just inside the culvert, in feet per second
k_e	=	entrance loss coefficient (Table 4-2)
g	=	acceleration of gravity, 32.2 feet per second squared
h_f	=	friction head loss, in feet

$$h_f = \frac{29n^2L}{R^{1.33}}\left[\frac{V^2}{2g}\right]$$

n	=	Manning's roughness coefficient (Table 4-1)
L	=	length of culvert barrel, in feet
V	=	average culvert velocity, in feet per second
V	=	Q/A
Q	=	discharge, in cubic feet per second
A	=	cross sectional area of flow, in square feet
R	=	hydraulic radius, in feet
R	=	A/WP
WP	=	wetted perimeter, in feet
S_o	=	slope of culvert, in feet per foot

The capacity of a culvert flowing partially full with outlet control and tailwater depth below critical depth (TW < d_c) is governed by the following equation when the approach velocity is considered zero. The entrance is unsubmerged (HW < 1.2D), and the design discharge is subcritical ($S_o < S_c$).

$$HW = d_c + \frac{V_c^2}{2g} + h_e + h_f - S_oL \qquad (4\text{-}5)$$

Where:

HW	=	headwater, in feet must be ≤ 1.2 D, or entrance is submerged.
d_c	=	critical depth, in feet
V_c	=	critical velocity occurring at critical depth (ft/s)
h_e	=	entrance head loss, in feet

$$h_e = k_e\left[\frac{V_c^2}{2g}\right]$$

k_e	=	entrance loss coefficient (Table 4-2)
g	=	acceleration of gravity, 32.2 feet per second squared
h_f	=	friction head loss, in feet

$$h_f = \frac{29n^2L}{R^{1.33}}\left[\frac{V^2}{2g}\right]$$

V	=	average pipe velocity, in feet per second
V	=	Q/A
Q	=	discharge, in cubic feet per second
A	=	cross sectional area of flow, in square feet
n	=	Manning's roughness coefficient (Table 4-1)
L	=	length of culvert barrel, in feet
R	=	hydraulic radius, in feet
R	=	A/WP
WP	=	wetted perimeter, in feet
S_o	=	slope of culvert, in feet per foot

4.2.3 Critical Depth

When the sum of kinetic energy plus potential energy for a specified discharge is at a minimum, critical flow occurs. During critical flow, the maximum discharge through the culvert occurs with any specified total energy head. For a given flow rate, the depth of flow and slope associated with critical flow define the critical depth and critical slope. If a culvert has an unsubmerged outlet, the maximum capacity of the culvert is established when critical flow occurs.

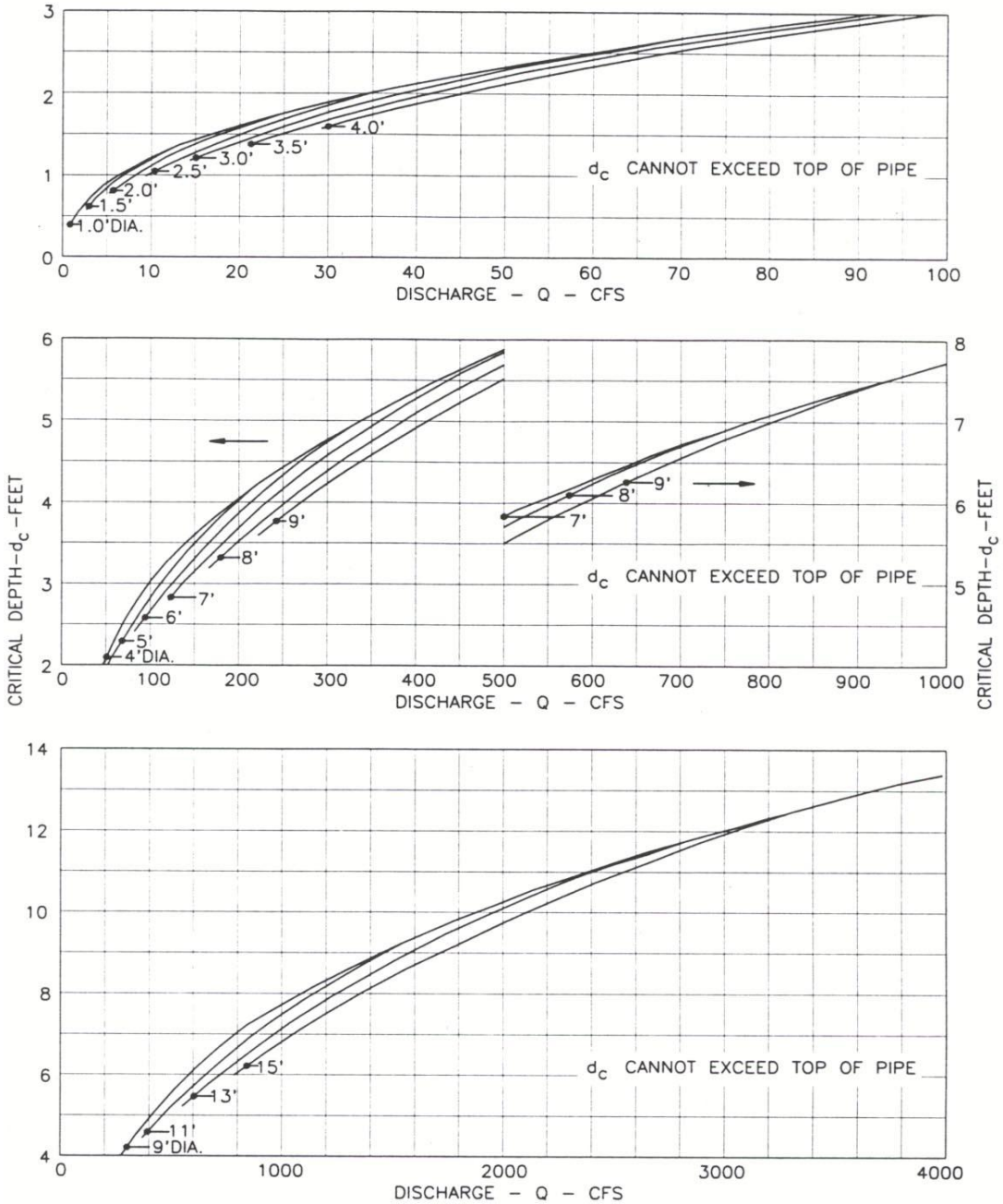
Critical depth for various culvert sections can be determined using the appropriate curve charts on Figures 4-3 and 4-4. These charts are based on the assumption that the culvert flows full over its entire length and is submerged at both ends. The charts can be used to approximate outlet control behavior when the tailwater elevation drops below the crown of the culvert outlet.

4.3 CULVERT SELECTION AND DESIGN

Culvert selection techniques can range from solving empirical formulas, to using nomographs and charts, to comprehensive mathematical analysis for specific hydraulic conditions. The many hydraulic factors involved make precise evaluation time consuming and difficult without the help of computer programs and models. The actual models used for these calculations shall be at the discretion of the design professional with approval from the Administrator(s). Applicable computer models for culvert design include, but are not limited to the following:

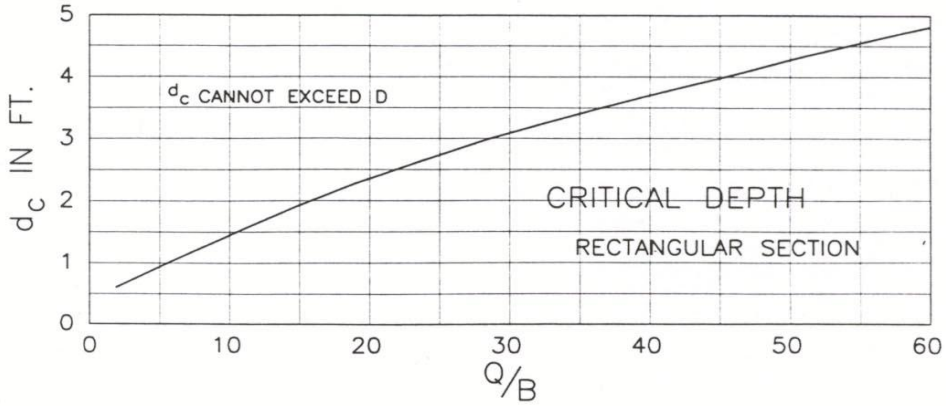
- HY8
- Hydraflow Storm Sewers by Intelisolve
- XPSWMM
- HEC-RAS
- Culvert Master
- ICPR

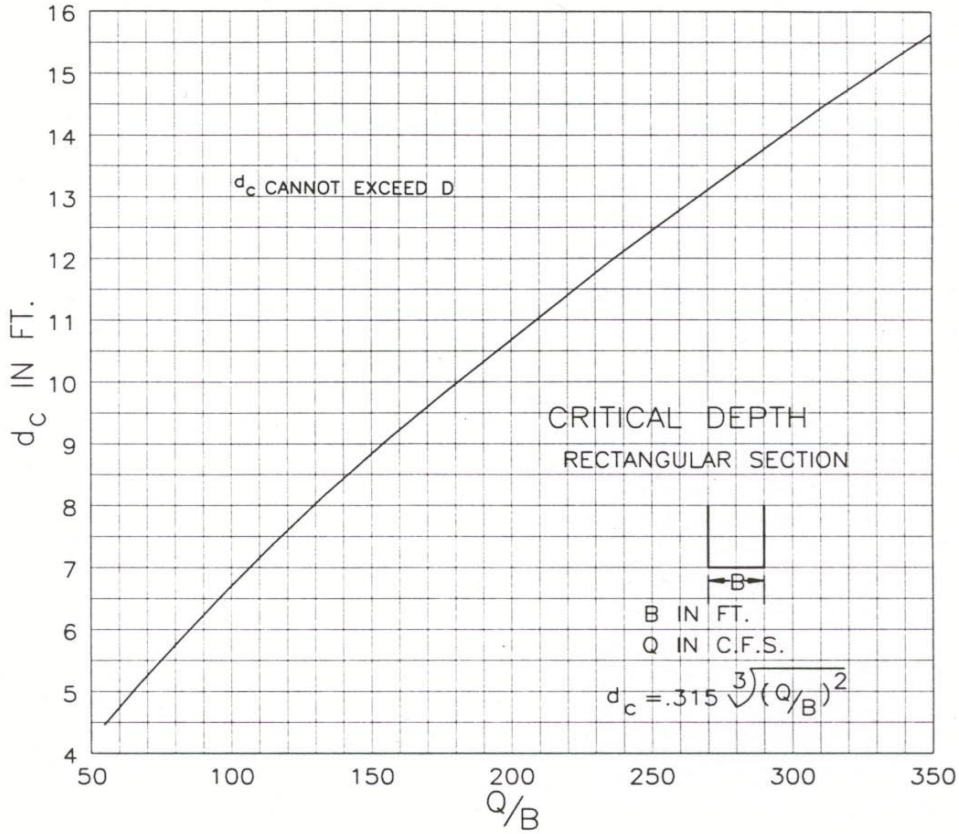
The simple empirical and nomograph methods do not account for all of the factors that impact flow through culverts, but they can be easily used to estimate flow capacities for the conditions they represent.



Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Figure 4-3 Critical Depth, d_c Circular Pipe





Source: Federal Highway Administration, HDS No. 5, Hydraulic Design of Highway Culverts.

Figure 4-4 Critical Depth, d_c Rectangular Section

5.0 OPEN CHANNEL

Open channel is frequently an integral component of an urban drainage system design. The drainage system may include the following types of channels or their combination:

- Natural Channel
- Constructed Channel
- Ditch
- Swale

These types of channel are ruled by common open channel design procedure. A stable open channel brings several advantageous features to the design:

- Relatively low construction cost,
- Potential flood control (instream storage attenuating downstream peak),
- Potential groundwater recharge,
- A habitat for native vegetation and wildlife,
- Areas for recreation, and
- An aesthetically pleasing environment adding social benefits.

The implementation of an open channel into the design requires careful planning and accounting for potential right-of-way constraints, utility conflicts, maintenance cost, and safety issues.

This section provides the necessary criteria and methodology for selection and design of open channels.

5.1 GENERAL DESIGN CRITERIA

5.1.1 Design Storm Frequency

Open channels shall be designed to convey the 10-year event below its freeboard height and the 25-year event below its bankfull elevation. The 100-year design storm shall be routed through the channel system to determine that no finished floor of residential dwellings, public, commercial, and industrial buildings will be inundated by the 100-year flood water surface elevation.

5.1.2 Geometry

The channel geometry design depends on site conditions and conveyance needs. The channel cross section may be trapezoidal, parabolic, V-shaped, or combination of those geometric shapes. Most desirable and commonly used is trapezoidal channel cross section.

- Channel side slopes shall be stable throughout the entire length. The channel side slopes shall be a maximum 2:1 (H:V).

- Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross-slope of 12:1 (H:V).

- If relocation of a stream channel is unavoidable, the geometry, meander pattern, roughness, and slope should conform to the existing conditions, as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.

5.1.3 Freeboard

Freeboard is extra height of channel lining above the design storm depth where overflow is expected to occur and potentially cause damage. This additional range of inundation creates a safety factor should unexpected obstructions or additional runoff create a rise in the design water surface elevation.

- For channels three feet or less in depth, one half of foot of freeboard shall be provided.
- For channels deeper than three feet and up to five feet in depth, one foot of freeboard shall be provided.
- For channels deeper than five feet in depth, freeboard that is at least equal to 20 percent of the total channel depth shall be provided.

5.1.4 Velocity Limitations

The final design of an open channel should be consistent with the velocity limitations for the selected channel lining to satisfy the condition of non-erosive velocity in the channel.

Maximum velocity values for selected lining categories are presented in Table 5-1. Velocity limitations for vegetative linings are reported in Table 5-2. The manufactured channel lining velocity limitations will be established by manufacturer.

TABLE 5-1
MAXIMUM VELOCITIES FOR COMPARING LINING MATERIALS

Material	Maximum Velocity (ft/s)
Sand	2.0
Silt	3.5
Silty Clay	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

TABLE 5-2
MAXIMUM VELOCITIES FOR VEGETATIVE CHANNEL LININGS

Vegetation Type	Slope Range (%)[1]	Maximum Velocity[2] (ft/s)
Bermuda Grass	0 – 5	6
	5 – 10	5
Bahia	All	4
Tall Fescue Grass	0 – 10	4
Mixtures[3]		
Kentucky Bluegrass	0 – 5	5
Buffalo Grass	5 – 10	4
	>10	3
Grass Mixture	0 - 5[1]	4
	5 – 10	3
Sericea Lespedeza,	0 - 5[4]	2.5
Weeping Lovegrass, Alfalfa	All	
Annuals[5]	0 – 5	2.5
Sod	All	4
Lapped Sod	All	5.5

[1] Do not use on slopes steeper than 10 percent except for side-slope in combination channels.
[2] Use velocities exceeding 5 feet per second only where good stands can be established and maintained.
[3] Mixtures of Tall Fescue, Bahia, and/or Bermuda.
[4] Do not use on slopes steeper than 5 percent except for side-slope in combination channels.
[5]Annuals - used on mild slopes or as temporary protection until permanent covers are established.

5.1.5 Channel Lining

Channel linings include vegetative and engineered materials, further divided into flexible materials (as grass, riprap, articulated concrete, and gabions), and rigid materials (paving blocks and concrete). The channel lining design criteria require that two primary conditions are satisfied:

- The channel system with the lining in place must have capacity for the peak flow expected from the design storm, and
- The channel lining must be resistant to erosion for the design velocity.

5.1.6 Maintenance Corridors and Easements

A maintenance corridor shall be provided with all drainage channels. This corridor shall provide a minimum access width of 20 feet from the channel bank on each side unless otherwise approved by the Town. For small channels with cross sectional widths less than ten feet measured from top of bank to top of bank, this corridor may be provided only on one side. These dedicated maintenance corridors shall be sufficiently cleared and graded to allow easy access for maintenance equipment. If the channel is to be maintained by the Town, the corridor will be within a dedicated easement.

Drainage easements are utilized to provide for the protection and legal maintenance of drainage systems not within a right-of-way. Drainage easements shall be required in subdivisions over any portion of a drainage system not within a right-of-way and necessary for the functioning of the system. Drainage easements for all facilities must be shown on construction drawings and approved by the county engineer. The easements shall be designated prior to issuance of a development permit and recorded in public records. The minimum allowable width of easements are provided in Table 5-3.

TABLE 5-3
MINIMUM EASEMENT WIDTHS FOR OPEN CHANNELS

Open Channel Width	Minimum Easement Width[1]
Bottom width 20 feet or less	15 feet + BW + 2SD (30 feet minimum)[2]
Bottom width 20 to 40 feet	30 feet + BW + 2SD[2]
Bottom width greater than 40 feet	40 feet + BW + 2SD[2]

Source: Beaufort County Code of Ordinance, Section 106-2858
[1] The minimum required width of drainage easements may be increased if deemed necessary by the Town's Administrator(s), only for justifiable reasons.
[2] Where BW = bottom width, S = side slope, D = depth of opening.

5.2 CHANNEL DISCHARGE

Designing a stable channel under dynamic channel conditions requires an understanding of sediment transport, stream channel response, and erosive forces which are at work. Unlined channels must be designed to minimize excessive scour while lined channels must be designed to prevent deposition of sediments. Details of methodology specific to protection from erosive forces can be found in the Federal Highway Administration, Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14 (HEC-14), 1983.

All variables used in fluid mechanics and hydraulics fall into one of three classes: those describing the boundary geometry, those describing the flow, and those describing the fluid. Various combinations of these variables define parameters that describe the state of flow in open channels.

5.2.1 Manning's "n" Values

The Manning's "n" value is an important variable describing material roughness in open channel flow computations. Changes in this variable can significantly affect flow discharge, depth, and velocity estimates. Since Manning's "n" values depend on many physical characteristics of channel surface, care and good engineering judgment must be exercised in the selection process. The composite "n" value should be calculated where the lining material, and subsequently the Manning's "n" value, changes within a cross section.

Recommended Manning's "n" values for artificial channels with rigid, unlined, temporary, and riprap linings are given in Table 5-4.

TABLE 5-4
MANNING'S ROUGHNESS COEFFICIENTS FOR ARTIFICIAL CHANNELS "n"

Lining Category	Lining Type	"n" at various flow depths		
		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[1]	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	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

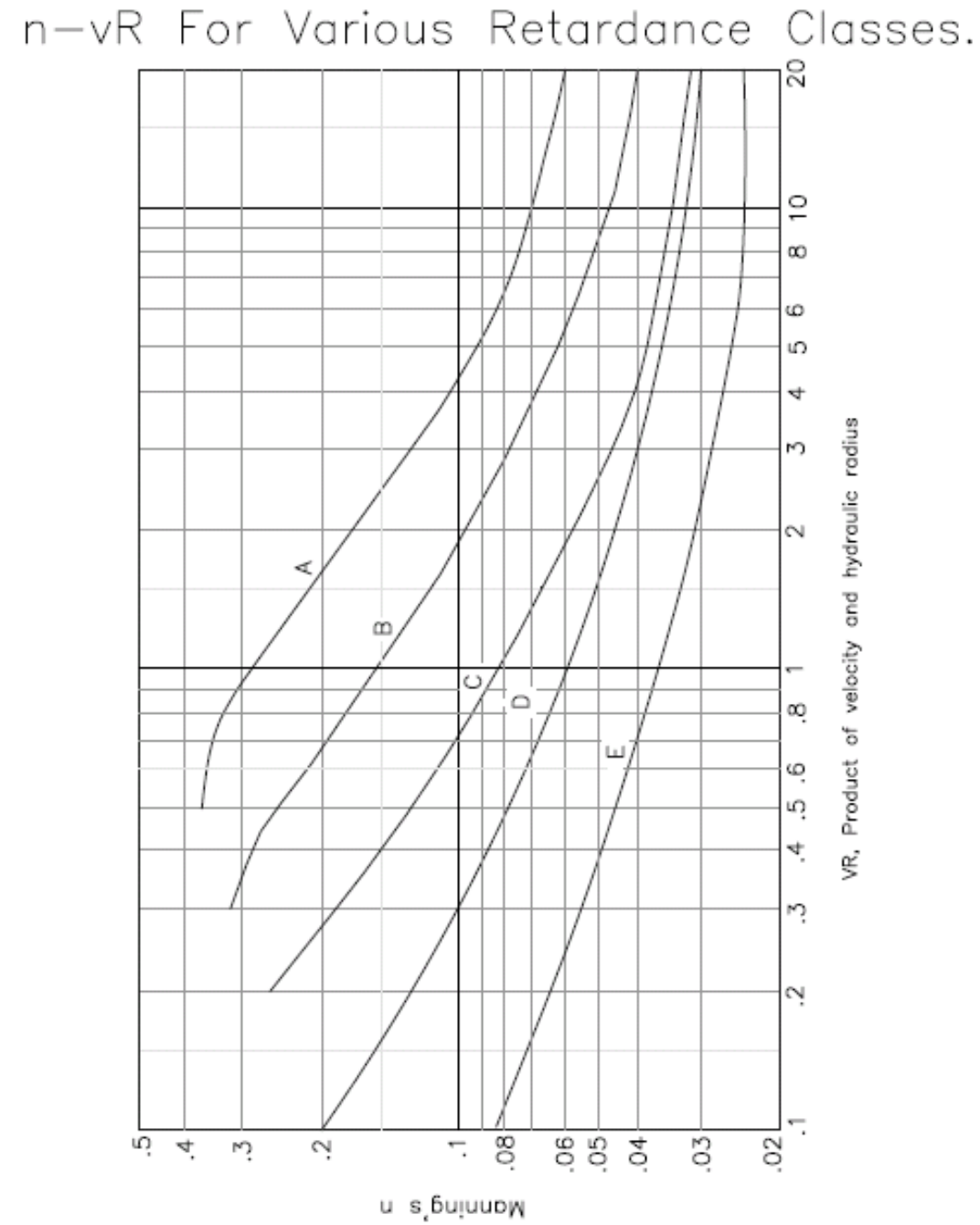
Source: Federal Highway Administration, Design of Roadside Channels with Flexible Linings, HEC-15, 1988.
Note: Values listed are representative values for the respective depth ranges. Manning's "n" varies with the flow depth.
[1]Some "temporary" linings become permanent when buried.

Recommended Manning's values for natural channels which are either excavated or dredged and natural are given in Table 5-5. For natural channels, Manning's "n" values should be estimated using the procedures presented in the publication Guide For Selecting Manning's Roughness Coefficients For Natural Channels And Flood Plains, (FHWA-TS-84-204, 1984).

TABLE 5-5
UNIFORM FLOW VALUES OF ROUGHNESS COEFFICIENT "n"

Type of Channel And Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
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
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
Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
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. Clean bottom, brush on sides, 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)			
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
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

TABLE 5-5 - continued UNIFORM FLOW VALUES OF ROUGHNESS COEFFICIENT "n"			
Type of Channel And Description	**Minimum**	**Normal**	**Maximum**
Floodplains			
Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
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
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
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.			
Regular section with no boulders or brush	0.025	.....	0.060
Irregular and rough section	0.035	.....	0.100



Source: Design Hydrology and Sedimentology for Small Catchments. Haan et. al, 1994.

Figure 5-1 Manning's "n" Values for Vegetated Channels

TABLE 5-6
CLASSIFICATION OF VEGETATIVE 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")
	Kudzu	Dense growth, uncut
	Weeping lovegrass	Good stand, unmowed (average 13")
	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 sericea	After cutting to 2" (very good before cutting)
E	Bermuda grass	Good stand, cut to 1.5"
	Bermuda grass	Burned stubble

Note: Covers classified above have been tested in experimental channels. Covers were green and generally uniform.

5.2.2 Channel Design

The drainage channel design has to provide adequate capacity for flow resulting from the design storm. The hydraulic characteristics of open channels shall be determined by using Manning's and continuity equations. Manning's Equation is commonly expressed as:

$$V = 1.49/n\ R^{2/3}\ S^{1/2} \qquad (5\text{-}1)$$

Where:

V = average flow velocity, in feet per second
n = Manning roughness coefficient
S = channel slope, in feet per foot
R = hydraulic radius, in feet, calculated as

$$R = A/P \qquad (5\text{-}2)$$

Where:

A = flow cross sectional area, in square feet
P = wetted perimeter, in feet (length of boundary between water and channel)

Note: For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line, water surface and channel bottom are equal.

The continuity equation is commonly expressed as:

$$Q = V\ A \qquad (5\text{-}3)$$

Where:

Q = average flow through a cross section, in cubic feet per second
V = average flow velocity over a cross section, in feet per second
A = area of cross section, in square feet

The continuity and Manning's Equations together may be solved for channel flow:

$$Q = 1.49/n\ A\ R^{2/3}\ S^{1/2} \qquad (5\text{-}4)$$

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 flow area) 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 as previously mentioned. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter.

Permissible Velocity

Permissible nonerosive velocity of a channel is dependent upon stability of lining materials and channel vegetation. In stable channels, the flow velocities generated by the design storm event shall not exceed the permissible velocity of the channel liner.

To satisfy this condition it is necessary to calculate the velocity in the design channel for the design storm event and compare it with the permissible velocity for the selected channel lining shown in Tables 5-1 and 5-2. The Manning's "n" roughness coefficients for structural lining are in Table 5-5 and for vegetative linings in Table 5-6. For vegetative lining, the lower retardance (generally C or D) from Table 5-6 is recommended, because this condition is defined when vegetation is cut low or lies down, producing a lower Manning's "n" value, lower flow depths, and higher flow velocities.

Note: A temporary lining may be required to stabilize the channel until the vegetation is established. If a channel requires temporary lining, the designer should analyze shear stress in the channel to select an appropriate liner that provides protection and promotes vegetation establishment. For the design of temporary lining, the tractive force method is recommended.

The channel outfall has to be evaluated for stability and the receiving channel for carrying capacity. If discharge velocities exceed allowable velocities for the receiving stream, an outlet protection design will be required. This is further discussed in more depth in Chapter 3.4 Outlet Protection.

Tractive Force

The design of riprap lined channels and temporary linings is based on shear stress/tractive force analysis. This method assumes that the design flow is uniform and does not vary with time. Since actual flow conditions change through the length of the channel, it is necessary to subdivide the channel into design reaches with uniform flow and work within the tractive force method limitations.

Shear Stress, ·

The critical shear stress or critical tractive force determines a soil's resistance to the shearing forces of concentrated flows. When the shearing forces of the flow exceed the critical tractive force of the soil, erosion takes place.

The slope, flow depth (normal depth, d_n) calculated for the design flow generated by 25-year, storm, and other channel design characteristics are important variables in shear stress calculation.

The governing equation for channel shear stress is:

$$\cdot\cdot = g\cdot\cdot d_n S \qquad (5\text{-}5)$$

Where:

·	=	maximum shear stress, in pounds per square foot
g	=	unit weight of water = 62.4 pounds per cubic foot
d_n	=	maximum normal channel flow depth, in feet
S	=	channel bed slope, in feet per foot

Permissible Shear Stress, $\cdot_d$

Permissible shear stress is the force required to initiate movement of the lining material and is not related to the erodibility of the underlying soil. However, if the lining is eroded or broken, the bed material will be exposed to the erosive force of the flow.

If the permissible shear stress for the lining material in Table 5-7 is greater than the computed shear stress, the proposed riprap or temporary lining is considered acceptable. If the lining is unacceptable, lining with a higher permissible shear stress is necessary. In some cases it may be necessary to alter channel dimensions to reduce the shear stress to the level below the permissible value.

TABLE 5-7
PERMISSIBLE SHEAR STRESSES FOR RIPRAP AND TEMPORARY LINING

Lining Category	Lypeining T	Permissible Unit Shear Stress, $\cdot_d$ [lb/ft^2]
Temporary	Woven Paper Net	0.15
	Jute Net	0.45
	Fiberglass Roving	0.60
	Straw with Net	1.45
	Curled Wood Mat	1.55
	Synthetic Mat	2.00
	D_{50} Stone Size [inch]	
Gravel Riprap	1	0.33
	2	0.67
Rock Riprap	6	2.00
	9	3.00
	12	4.00
	15	5.00
	18	6.00
	21	7.80
	24	8.00

Source: Federal Highway Administration, Design of Roadside Channels with Flexible Linings, HEC-15, 1988.

Normal Depth, d_n

Normal depth is unique for each channel with a particular slope and discharge and can be calculated using Manning's Equation 5-1. This generally requires a trial-and-error solution. Values of the Manning's roughness coefficient for different ranges of depth are provided in Table 5-4 for temporary linings and for riprap. The trial-and-error method for normal depth calculation is described in the section below.

It is recommended that normal depth should be calculated for the following conditions:

- Bare matting with no vegetation,
- Matting with maintained vegetation, and
- Matting with un-maintained vegetation.

Iterative Method for Normal Depth Calculation

The use of Manning's procedure includes iterative calculations to calculate the normal depth of flow in a uniform channel when the channel shape, slope, roughness and design discharge are known.

Using Manning's Equation 5-1 and the continuity Equation 5-3, and solving for flow, Q, Equation 5-4 can be rearranged to the following ratio:

$$A\,R^{2/3} = n\,Q / (1.49\,S^{1/2})$$

To calculate the normal depth of flow (d_n) by the trial and error process, trial values of depth (d_n) are selected to calculate a corresponding flow area (A), wetted perimeter (P), and hydraulic radius (R). For each trial depth selected, a corresponding $AR^{2/3}$ value is calculated. Trial values of the depth are selected until the $AR^{2/3}$ value equals the known ratio calculated by using the known roughness, design discharge, and channel slope.

Computing Shear Stress around a Channel Bend, τ_b

Computing shear stress around a channel bend requires special considerations because the change in flow direction imposes higher shear stress on the channel bottom and banks. The maximum shear stress in a bend is given by the following equation:

$$\tau_b = K_b\,\tau \qquad (5\text{-}6)$$

Where :

τ_b	=	bend shear stress, in pounds per square foot
K_b	=	bend factor
τ	=	computed shear stress for straight channel, in pounds per square foot

The τ_b value is related to the radius of curvature of the channel at its center line, R_c, and the bottom width of the channel, B, Figure 5-2. The length of channel requiring protection downstream from a bend, L_p, is a function of the roughness of the lining material and the hydraulic radius, R, as shown in Figure 5-3.

The design channels usually exhibit high stress around bends and therefore these areas should be carefully evaluated. The channel may be protected by change in cross sectional area that would decrease the shear stress, or upgrade the protective channel lining so it withholds the shear forces.

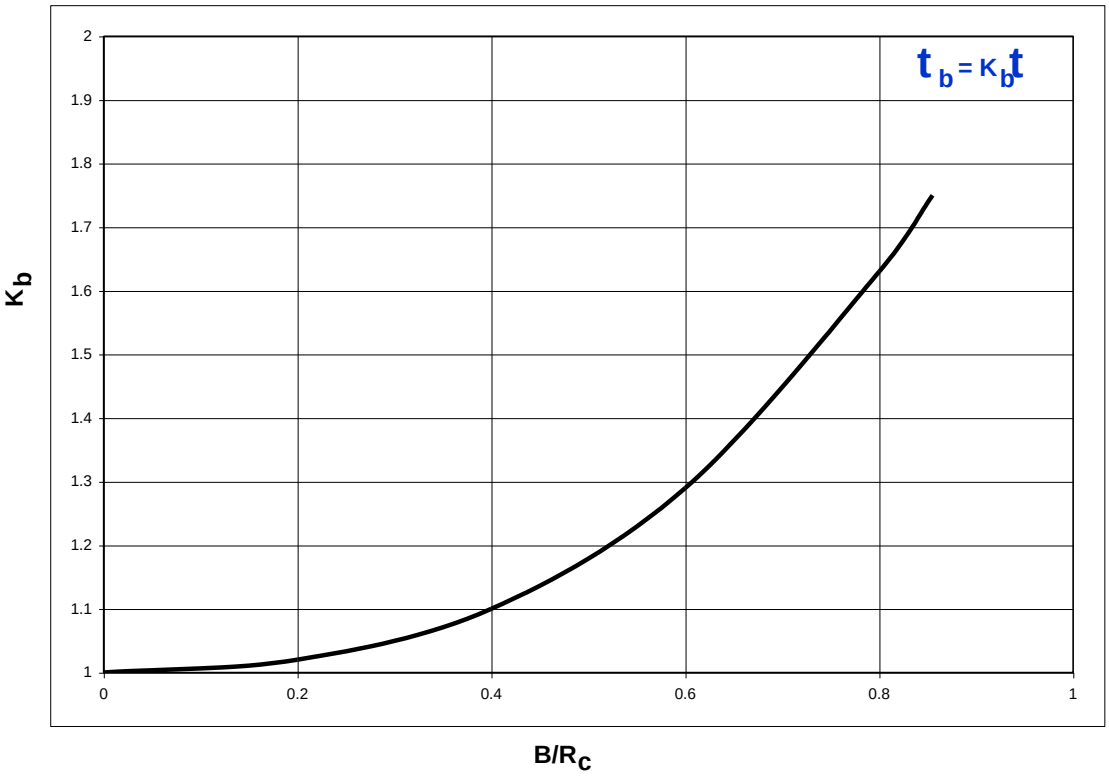


Figure 5-2 K_b Factor for Maximum Shear Stress on Channel Bends
Source: FHWA, HEC-15, April 1988

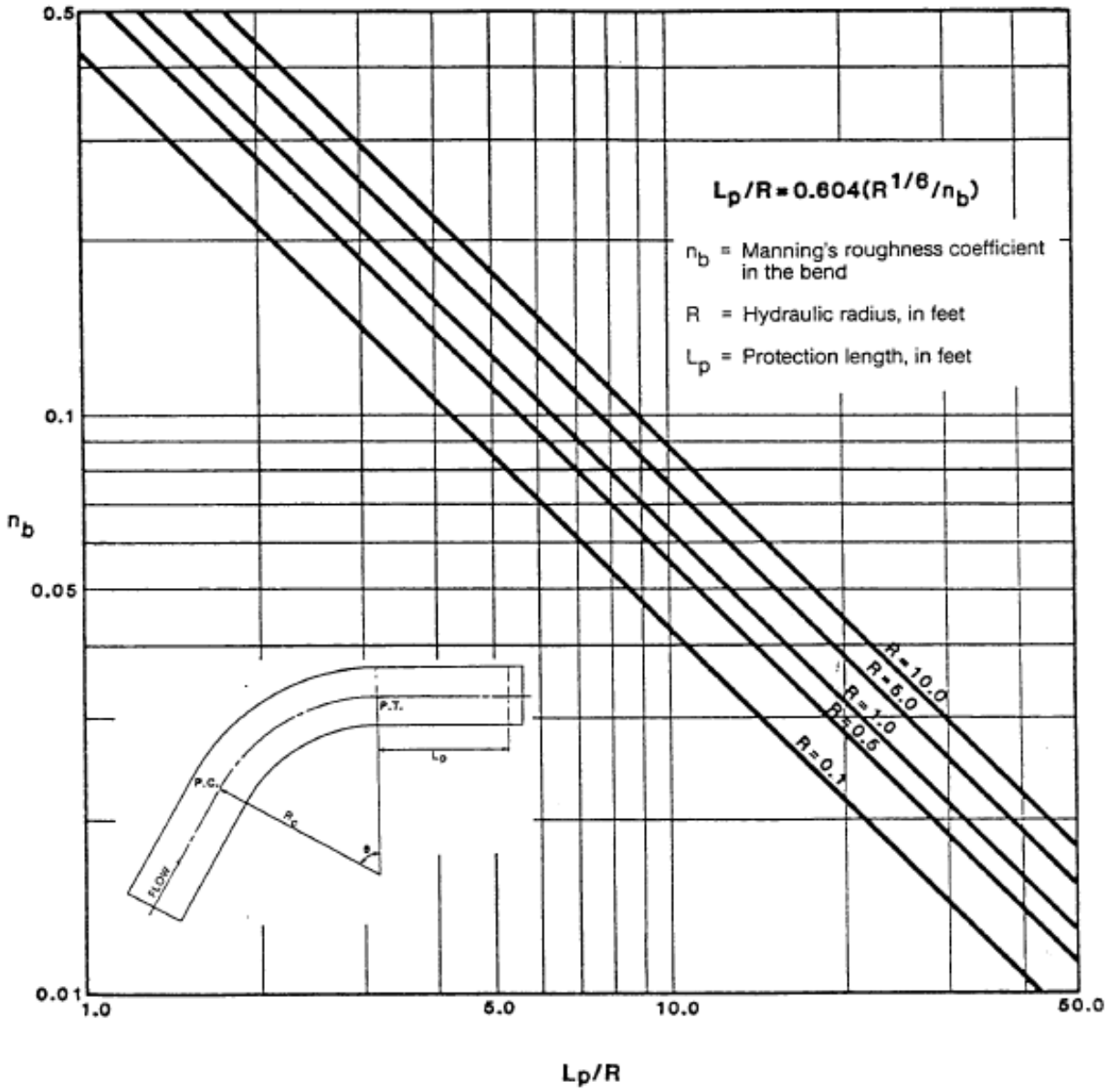


Figure 5-3 Protection Length, L_p, Downstream of Channel Bend
Source: FHWA, HEC-15, April 1988

6.0 STORAGE FACILITIES

Urban stormwater storage facilities are often referred to as either detention or retention facilities. Detention facilities are those permanent stormwater management structures whose primary purpose is to temporarily store stormwater and release the stored stormwater at controlled rates. Therefore, detention facilities are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities are designed to completely drain or drain back to a normal pool elevation after the design storm has passed. Retention facilities are those permanent structures that permanently store a given volume of stormwater. These facilities typically store an additional volume of stormwater for release by infiltration and/or evaporation. They do not pass stormwater purposely but reduce it completely through other means. The term storage facilities will be used in this chapter to include detention and retention facilities.

Temporary storage of stormwater improves the quality of water discharged from the facility. Chapter 7 provides further detail on using storage facilities for water quality treatment of stormwater. Storage facilities can range from small facilities in parking lots or other onsite facilities to large lakes and reservoirs. This chapter provides general design criteria for promoting onsite storage where possible.

It should be noted that the location of storage facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding. Small individual storage facilities generally have only minimal regional flood control benefits, as the localized benefits quickly diminish as the flood wave moves away from the facility and travels downstream. 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 careful consideration to downstream effect should exercised in large or complex watersheds.

6.1 GENERAL DESIGN CRITERIA

The R.72-300 South Carolina Standards for Stormwater Management and Sediment Reduction provides performance criteria for design and operation of storage facilities. The criteria have been compiled and outlined below based on their function in controlling stormwater.

Storage

- Storage volume shall be adequate to attenuate the post-development peak discharge rates to predevelopment discharge rates for the 2-, 10- and 25-year storms.
- Development projects shall use a calculated natural ground cover and vegetated surface condition for the determination of predevelopment discharge rates from the site.
- Parking lot, cul-de-sac, and traffic islands shall be designed to be depressed and open to receive stormwater runoff storage and treatment.

Release Rate

- Control structure release rates shall be less than or equal to predevelopment peak runoff rates for the 2-, 10- and 25-year storms, with emergency overflow adequately designed for the 100-year discharge.
- Design calculations must demonstrate that the facility will limit runoff from the 2-, 10- and 25-year post-development discharge rates to predevelopment peak discharge rates.
- Outlet structures that draw water from or near the normal pool surface of the storage facility shall be used
- Discharge velocities shall be non-erosive for the design storm.

6.2 DESIGN RECOMMENDATIONS

The sizing of a storage facility depends on the amount of storage, its location within the system, and its operational characteristics. An analysis of storage facilities should consist of comparing the design flow at a location downstream of the proposed storage site both with and without storage. Flow in excess of the design storm flow is expected to pass through or around the storage facility safely (i.e., 100-year flood). Additional design recommendations for storage facilities should include:

- Storage volume,
- Grading and depth requirements,
- Safety considerations and landscaping,
- Outlet works, and
- Location.

Note: The same hydrologic procedure shall be used to determine pre- and post-development hydrology.

6.2.1 Storage

Routing calculations must be used to demonstrate that the storage volume is adequate. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project.

6.2.2 Grading and Depth

This section presents a discussion of the general grading and depth criteria for storage facilities followed by criteria related to dry detention basins and wet detention ponds.

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments shall be less than 20 feet in height and shall have side slopes no steeper than 3:1 (H:V). Riprap protected embankments shall be no steeper than 1:2 (H:V). Geotechnical slope stability analysis is required for embankments greater than 10 feet in height and is mandatory for embankment slopes steeper than those given above. Procedures for performing slope stability evaluations can be found in most soil engineering textbooks, including Soil Engineering by Spangler and Handy (1982) and Introductory Soil Mechanics and Foundations by Sowers and Sowers (1970).

Final side slopes for storage facilities shall be a maximum of 2:1 (H:V) and shall extend three feet below the permanent pool water level. Side slopes shall be stabilized with grass or other natural vegetation. During construction, if side slopes do not meet the above requirements then the permanent pool shall be surrounded with a fence of a minimum height of 6 feet until such time as the final side slope is established.

Impoundment depths greater than 25 feet or storage volume greater than 10 acre-feet are subject to the requirements of the South Carolina Safe Dams Act (SDA) unless the facility is excavated for the depth. Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements, and required freeboard. The feature's pleasing aesthetical characteristics are also important in urbanized areas.

6.2.3 Outlet Works

Outlet works selected for storage facilities typically include a principal outlet works and an emergency spillway. These structures are sized to accomplish the design functions of the storage facility. Outlet works can include risers, drop inlets, pipes, various weirs, orifices, and their combinations. The clogging factor of these structures has to be considered in the outlet works structure design.

The outlet works are designed to convey the design storm. The emergency spillway shall safely convey the 100-year flood determined by using a fully developed land use conditions, and with the outlet located above the pond's 25-year water surface elevation.

For large storage facilities under the jurisdiction of the SDA, Dams and Reservoirs Safety Act Regulations (DRSAR) guidelines must be followed for selecting a design flood magnitude. The sizing of the emergency outlet should be always consistent with the potential threat to downstream life and property damage in a case of embankment failure. The hydrologic routing calculations are the only means accepted by Town for sizing the outlet works.

Where the discharge structure consists of a single pipe outlet, the pipe shall have a minimum inside diameter of 12 inches since maintenance of outlets smaller than 12 inches is likely to be a problem. If design release rates call for outlets smaller than 12 inches, release structures such as perforated risers or flow control orifices may be incorporated.

<u>Principal Outlet Works</u>

Discharge for various headwater depths can be controlled by crests (weir control), riser, barrel opening, low orifice control, or barrel pipe (outlet control). Each of these flow controls shall be evaluated when determining the rating curve of the principal outlet. The following weir, orifice, and pipe flow equations can be used to evaluate a single opening outlet structure.

Weir Flow

Weir flow may be computed for a standard, uncontracted, horizontal weir by the following equation:

$$Q = CLH_w^{3/2} \qquad (6\text{-}1)$$

Where:

Q	=	discharge, in cubic feet per second
C	=	weir coefficient
L	=	length of the weir, in feet; for circular riser pipes, L is the pipe circumference
H_w	=	the depth of flow over the weir crest, in feet

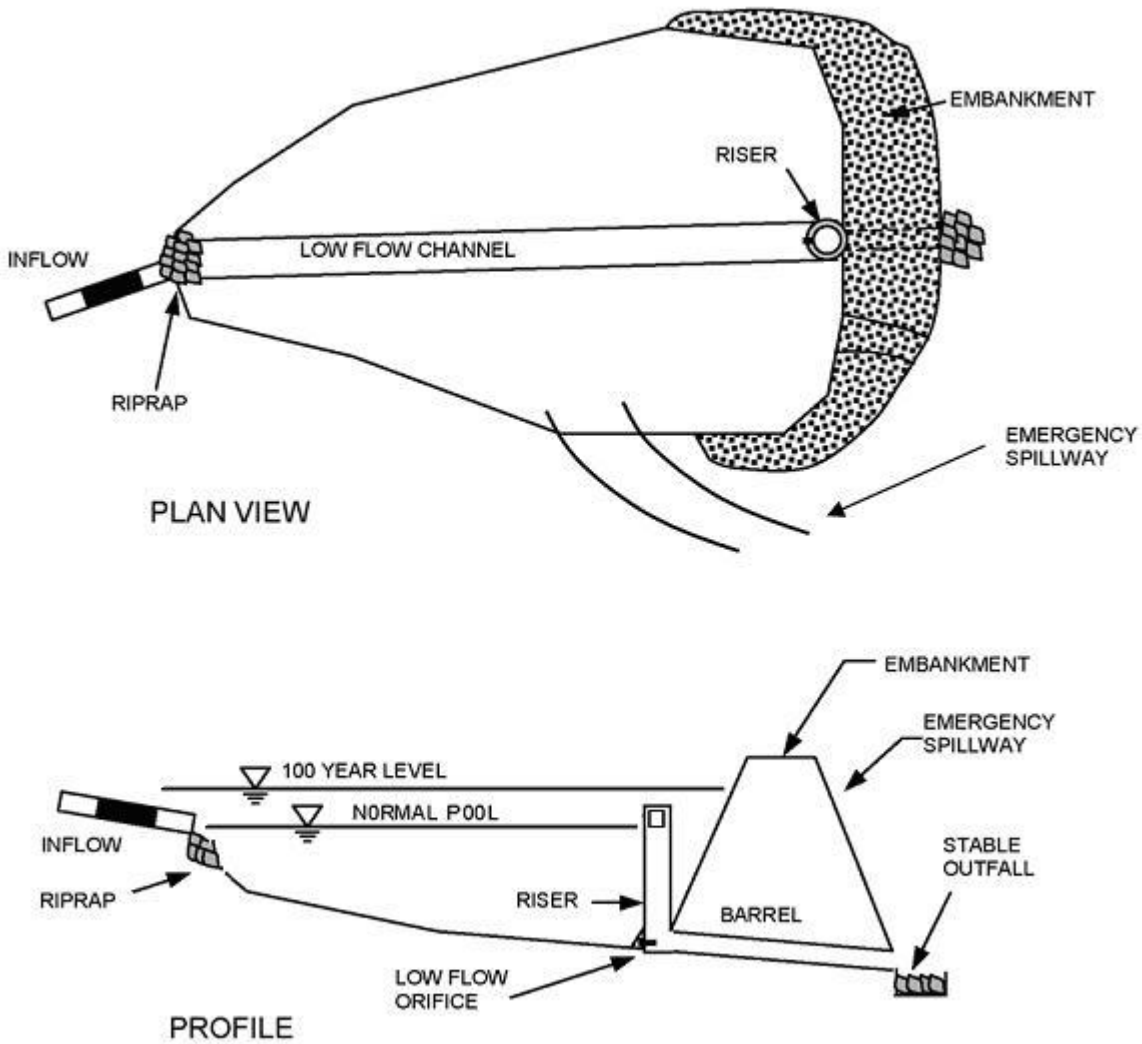


Figure 6-1 Schematic of Stormwater Storage Facility Design Features

The weir coefficient is a function of various hydraulic properties and dimension characteristics of a weir. Experiments have been conducted on various types of weir configurations and formulas have been developed to determine the "C" value. Available empirical formulas are numerous and the design professional is urged to solicit hydraulic textbooks such as Handbook of Hydraulics by Brater and King (1996) and use sound engineering judgment. The effects of submergence must be considered when designing or evaluating a weir flow. A simple check for submergence can be made by comparing the tailwater to the weir crest elevation.

Weir flow may be computed for a V-notch weir may be computed by the following equation:

$$Q = C \tan\left(\frac{\theta}{2}\right) H^{5/2} \qquad (6\text{-}2)$$

Where:

Q	=	discharge, in cubic feet per second
C	=	weir coefficient, usually 2.50
θ	=	angle of the notch at the apex, in degrees
H	=	total energy head, in feet

Orifice Flow

Orifice flow may be computed by the following equation:

$$Q = CA\sqrt{2gH_A} \tag{6-3}$$

Where

Q	=	discharge, in cubic feet per second
C	=	orifice coefficient
A	=	cross-sectional area of the orifice, in square feet
g	=	acceleration of gravity, 32.2 feet per second squared
H_A	=	$H_o/2$
H_o	=	Difference in elevations of the water surfaces on either side of the orifice in feet, when the orifice is submerged. Difference in elevation between the water surface on the upstream side of the orifice and the centroid of the orifice, in feet, if the orifice has a free discharge.

The orifice coefficient is a function of various hydraulic properties and dimensional characteristics. The design professional is urged to solicit hydraulic textbooks such as Handbook of Hydraulics by Brater and King (1996) and use sound engineering judgment.

Pipe Flow

Pipe flow may be computed by the following equation:

$$Q = A\left[\frac{2gH}{1 + k_b + k_e + k_f L}\right]^{0.5} \tag{6-4}$$

Where:

Q	=	discharge, in cubic feet per second
A	=	cross-sectional area of the pipe, in square feet
g	=	acceleration of gravity, 32.2 feet per second squared
H	=	difference between headwater and tailwater elevations, in feet
k_b	=	bend loss coefficient, use 0.6
k_e	=	entrance loss coefficient, use 0.5
k_f	=	friction loss coefficient, use Darcy-Weisbach for concrete pipe flowing at 5 ft/s
L	=	length of pipe, in feet

Emergency Spillways

All detention basins shall include a stabilized emergency spillway designed to convey the 100-year storm event with a minimum of 1 foot of freeboard. The Town may require the design professional to evaluate a more stringent design requirement including a breach analysis if there is a potential for loss of life or significant downstream property damage. The elevation of the 100-year storm event also shall be reviewed to ensure that area structures are not impacted. Certain embankments may be classified as dams and are required to meet rules established by the SCDHEC.

The emergency spillway should be constructed using undisturbed native (in-situ) soils. The geologic and topographic features of a site determine the design features of the spillway, such as position, profile, and length. The cross section dimensions are governed by required hydraulic capacity and are determined by reservoir routing of the design storm. Emergency spillways for detention ponds are often designed as stabilized open channels with linings selected to resist erosion. Spillway channel and lining design should be done according to guidelines in Chapter 5 Open Channel.

Discharge from the emergency spillway shall be directed to the receiving channel without causing erosion along the downstream toe of the dam. An emergency spillway proposed for the protection of an earthen embankment shall be in undisturbed soil, if possible, to avoid flow-imposed stress against a constructed fill. The side slopes of the excavated earth channel shall be no steeper than 3:1 (H:V) and shall be stabilized in accordance with procedures for open channels. Where the site limitations prevent a full channel cut, a wing dike shall be provided to direct spillway flows away from the downstream toe of the dam. Ready access to the emergency spillway system is required.

The configuration of the entrance channel from the reservoir to the control section of the emergency spillway shall provide a smooth transition to avoid turbulent flow over the spillway crest. The outlet of the emergency spillway shall transition to the channel without causing erosion. The slope of the exit channel usually follows the configuration of the abutment. In cases of highly erodible soils, it may be necessary to use structural or vegetative protective lining. As an alternative, increased detention storage reduces the frequency and/or duration of emergency spillway operation and therefore reduces potential erosion problems.

6.2.4 Location

In addition to controlling the peak discharge from the outlet works, storage facilities will change the timing of the entire hydrograph. If several storage facilities are located within a particular basin, it is important to determine what effects a particular facility may have on combined hydrographs in downstream locations. The Administrator(s) may request an analysis of the potential for adverse impacts downstream. At the request of the Town, channel routing calculations should be performed proceeding downstream to a confluence point where the drainage area being analyzed represents ten percent of the total drainage area. For example, if the proposed development occupies a drainage area of seven acres then the analysis must be performed far enough downstream that the resulting contributing drainage of analysis is equal to or greater than seventy (70) acres (seven acres being ten percent of seventy acres). At this point, the effect of the hydrograph routed through the proposed storage facility on the downstream hydrograph should be assessed to ensure detrimental effects on downstream hydrographs are not present.

6.2.5 Dams and Reservoirs Safety Act Regulations

Under the SDA, a regulated dam is an artificial barrier that does or may impound water and that is 25 feet or greater in height or has a maximum storage volume of 10 acre-feet or more. A number of exemptions are allowed from the SDA and any questions concerning a specific design or application should be addressed to the South Carolina Department of Health and Environmental Control (SCDHEC).

According to SDA, all dams and reservoirs subject to regulation shall be classified according to their size and hazard potential. Classifications shall be made in accordance with the Section 72-2. of DRSAR and are subject to final approval by the SCDHEC. It may be necessary to reclassify dams overtime in accordance with changes in DRSAR regulations.

Size classification may be determined by either storage or height, whichever gives the larger size capacity. The hazards potential classification pertains to potential loss of human life or property damage in the event of failure or improper operation of the dam or appurtenant works.

6.3 ONSITE DETENTION

Potential advantages and disadvantages of onsite detention structures should be considered by the design professional in the early stages of development. Discharge rates and outflow velocities are regulated to conform to the capacities and physical characteristics of downstream drainage systems. Energy dissipation and flow attenuation resulting from onsite storage can reduce soil erosion and pollutant loading. By controlling release flows, the impacts of pollutant loading of stored runoff on receiving water quality can be minimized.

The onsite detention structures listed in this section shall be designed for the temporary storage of stormwater. The design professional should ensure that all detained stormwater is emptied in the specified duration to avoid creating hazardous and unsightly conditions, or more importantly, to prevent jeopardizing the future function of the structure itself. Onsite detention systems shall also be designed to provide positive overflow and drainage relief in the event that blockage of primary release locations occurs.

6.3.1 Parking Lots

There are a variety of structural and site design-based strategies available for detaining stormwater in parking lots. Foremost, the use of a pervious ground cover is ideal as it promotes onsite infiltration, natural filtration and ground water recharge. In addition to assorted pervious ground covers such as gravel, cobbles, pervious concrete, and pervious asphalt, a multitude of proprietary porous paving systems are also commercially available. For all parking lots, strategically placed vegetated swales or depressed uncurbed bioretention areas between parking stalls should be constructed to retain and treat any runoff generated onsite. In addition, below-ground proprietary structural storage products that are commercially available can be employed to meet both water quantity and water quality goals if so approved as part of the stormwater system design by the Administrators(s).

Another strategy for stormwater detention on parking lots consists of using the paved areas of the lot to channel runoff to grassed areas or gravel-filled seepage pits (see Figure 6-2). Water from pavement should flow across a grassed vegetative buffer before entering a collection swale, infiltration swale, trench, or basin where the flow will then infiltrate into the ground. Soil conditions and the effects of siltation in reducing infiltration must be considered. Minimum slopes of one-half (0.5) percent are recommended in parking lot detention areas. Additional use of a narrow width of

open graded aggregate in non-pedestrian areas can facilitate sheet flow dispersion and sediment reduction.

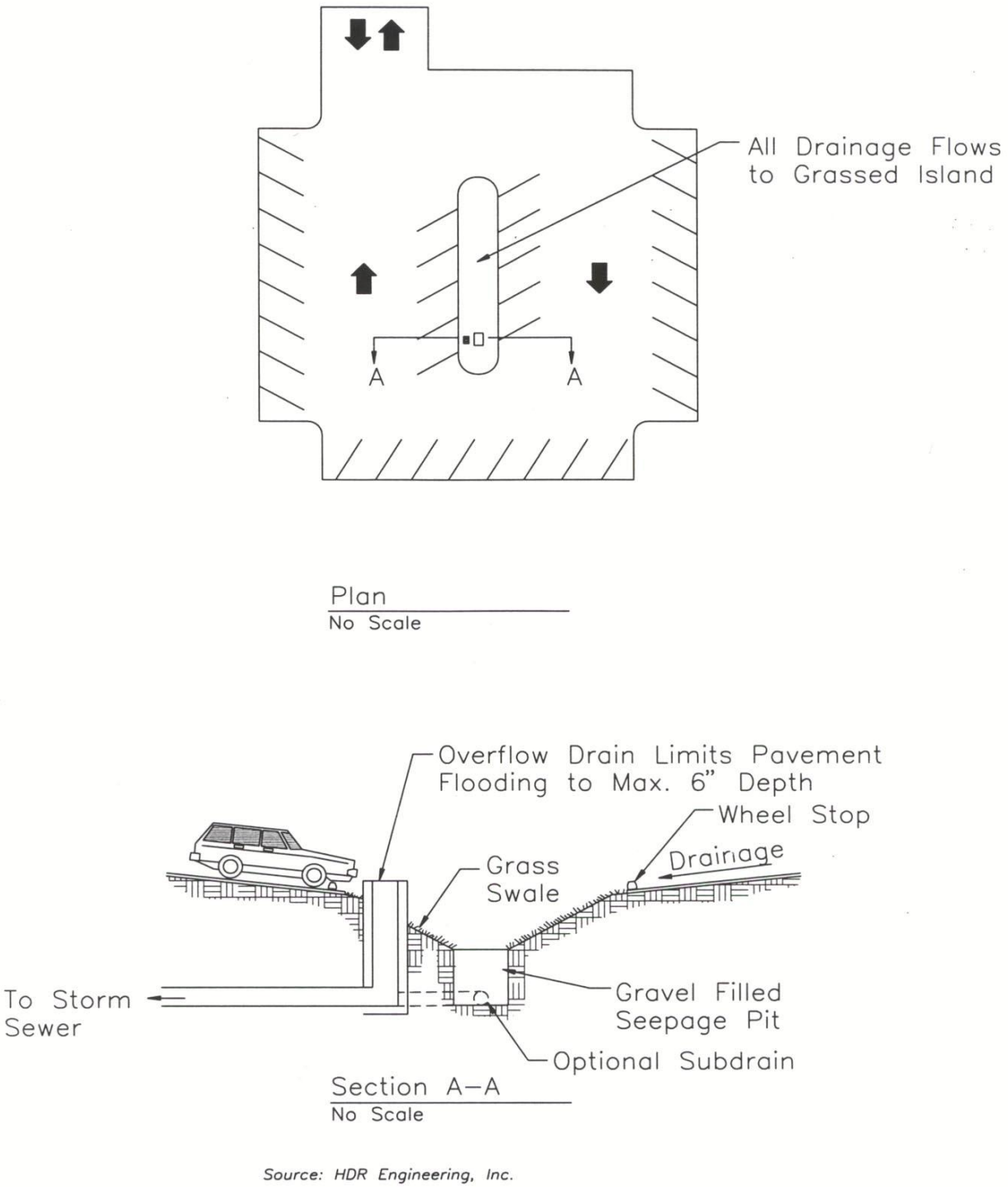
6.3.2 Recreational Areas

Recreational areas such as outdoor athletic fields have a substantial area of grass cover which often provides a high infiltration rate. Generally, storm runoff from such fields is minimal. Grassed recreational fields can be utilized for the temporary detention of storm runoff without adversely affecting their primary function. Officials responsible for the operation and maintenance of the recreational facilities should be consulted to determine the allowable duration of ponding.

Parks, like recreational areas, create little runoff of their own; however, parks provide excellent detention storage potential for runoff from adjacent areas.

6.3.3 Property Line Swales

Planning and layout of new development areas requires adequate surface drainage that directs runoff away from buildings. This is accomplished by sloping the finished grade away from the buildings. When possible, the layout should call for a swale to be located along the back and/or side property line which then drains runoff along sides of properties (See Figure 6-3). Such drainage should be guided away from installation storm sewers and towards natural channels. If storm sewers are the only point of discharge, the drainage route should be as long as possible to allow infiltration. The final grading plan for the lot layout can allow up to six (6) inches of temporary ponding along the property line. Easements shall be provided along property lines to allow maintenance access if this method of stormwater management is utilized.

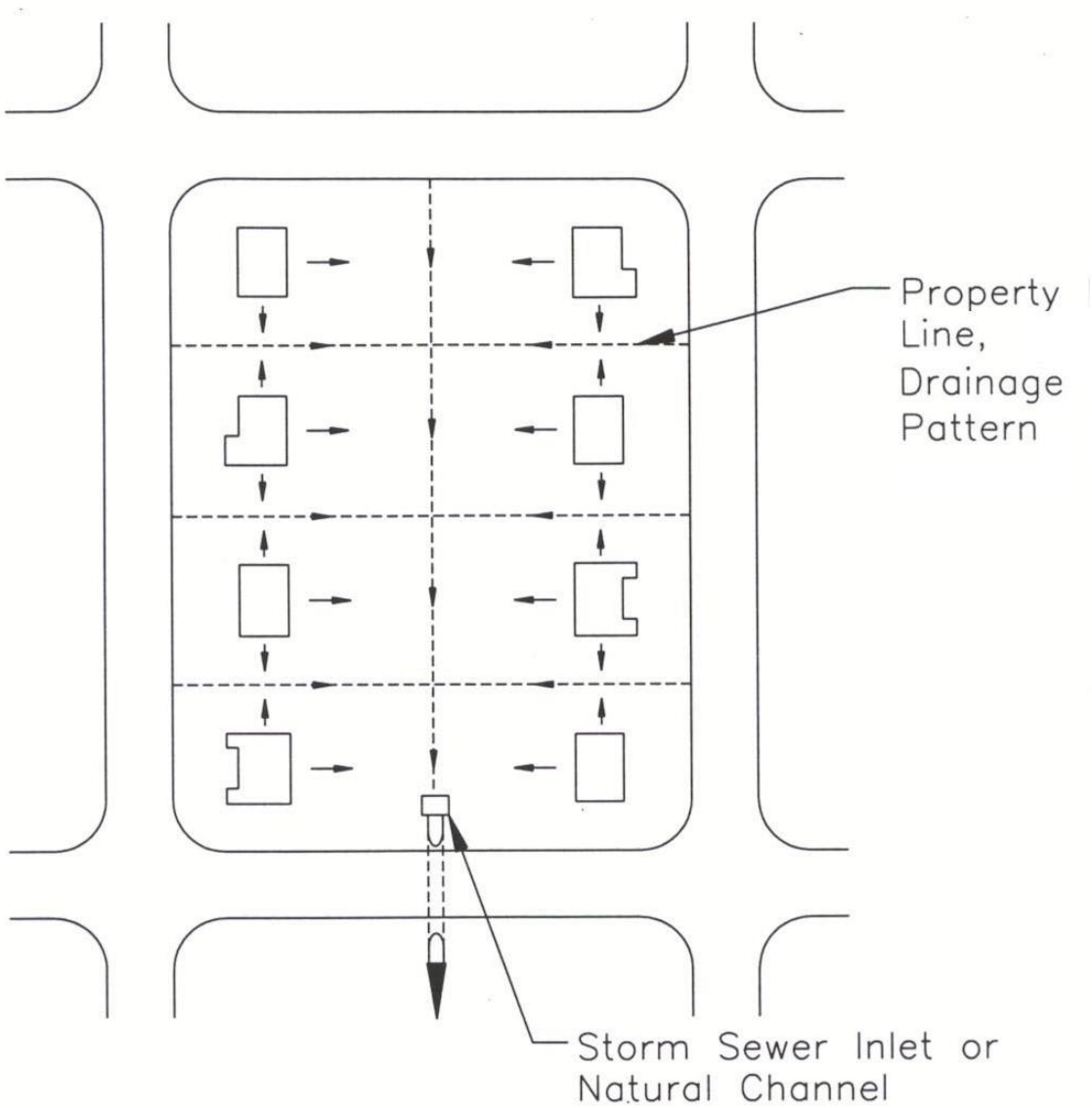


Source: HDR Engineering, Inc.

Figure 6-2 Parking Lot Detention Schematic

6.3.4 Road Embankments

The use of road embankments for temporary storage is an efficient method of attenuating the peak flows from a drainage basin. The design criteria to be used for the temporary detention of water behind road embankments shall include the effects of the major storm event (i.e. 100-year). Planning for the usage of embankments must be done with consideration to avoid damage (flooding, scouring, or washing out) to the embankment, the structure, and adjacent property. Designs utilizing road embankments shall include slope protection measures as outlined in the SCDOT design manual.



Source: HDR Engineering, Inc.

Figure 6-3 Property Line Swale

6.3.5 Onsite Ponds

The construction of onsite ponds provides significant detention. Although often much smaller, the same design criteria presented in Section 6.2 for detention basins shall apply to onsite ponds.

6.3.6 Pipes, Tanks, and Vaults

Underground facilities such as pipes, tanks, French drains, infiltration trenches and vaults may be utilized for runoff detention for storage of flow generated by the design storm event. The outflow from these detention facilities is controlled by orifices and/or weirs. Overflow weirs within the facility shall pass the fully developed design storm. Site grading designs shall include overflow provisions for major storms.

6.3.7 Combinations

In many instances, one onsite detention method cannot conveniently or economically satisfy the required amount of stormwater storage. Limitations in storage capacity, site development conditions, soils limitations, and other constraints may require that more than one method be utilized. For example, parking lot and surface pond storage might both be required to compensate for increases in runoff due to development of a particular site. The combination of suitable storage facilities should be incorporated into the site development plan.

6.3.8 Parking Lot, Cul-de-sac, and Traffic Islands

Areas traditionally reserved as raised or curbed, vegetative or impervious islands can provide both partial storage and treatment for stormwater runoff. These islands should be depressed or un-curbed to receive appropriate volumes of stormwater runoff for storage and treatment by filtration and biological uptake. Depressed islands that are planned to provide storage and treatment should be developed in similar design accordance as bioretention areas or sand filters. Consult Chapter 7 Structural Best Management Practices for guidance on designing bioretention areas and sand filters. They can be designed to have underdrain systems that connect into existing or proposed conveyance systems that drain to a more regional site BMP.

6.4 HYDRAULIC DESIGN METHODS

6.4.1 Hydrograph Procedure for Storage Analysis

The unit hydrograph procedure develops a hydrograph which provides a more reliable solution for detention storage effects by providing the design professional greater flexibility for the representation of actual conditions to be modeled. This procedure can be used for any size drainage area. For detention basin design, 24-hour design storm duration should be used.

The development of the storm runoff hydrograph is discussed in Chapter 2 Hydrology and Runoff Determination. The storm runoff hydrograph presented in Figure 6-4 represents inflow to a reservoir by routing the peak flow over a side channel spillway from the main channel into an adjacent ponding area. The analysis for the reservoir storage must take into consideration the characteristics of the outlet structure discharge which is shown in Figure 6-4 as a solid line. The shape of the solid line reflects the carrying capacity of the outlet works with various headwater elevations. The higher the elevation of the water surface in the reservoir, the greater the discharge through the outlet. In this example, the area between the dashed line and the hydrograph of storm outflow represents the

volume of storage required to reduce channel flow from 200 cubic feet per second to 100 cubic feet per second.

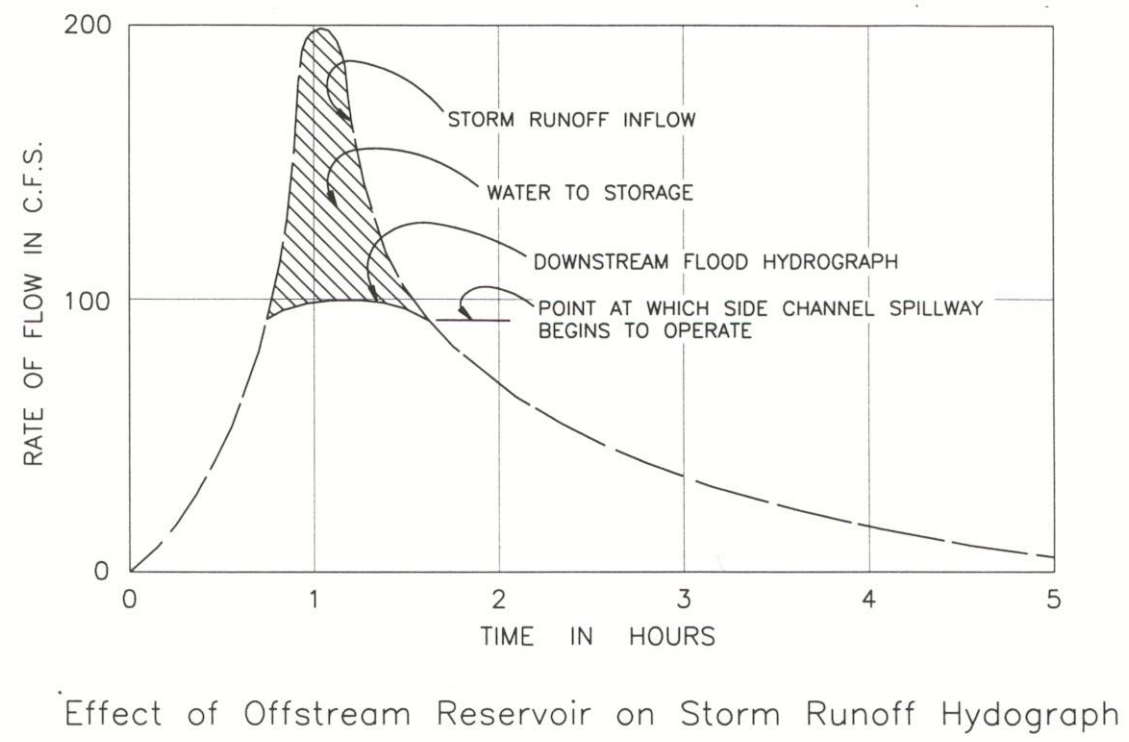


Figure 6-4 Effect of Offstream Reservoir on Storm Runoff Hydrograph

6.4.3 Modified Puls Routing Procedure

A flood routing procedure may be used to determine the required volume of the detention basin. Several flood routing procedures are available in published texts. One commonly used method is the Modified Puls Routing Procedure. The data needed for this routing procedure are:

- Inflow hydrograph,
- Physical dimensions of the storage basin,
- Maximum outflow allowed, and
- Hydraulic characteristics of the outlet structure or spillway.

To perform the Modified Puls procedure, the inflow hydrograph, depth-storage relationship, and depth-outflow relationship must be determined and further combined in a routing routine. The results of the routing are the ordinate of the outflow hydrograph, the depth of storage, and the volume of storage at each point in time of the flood duration.

The routing period, or time interval, Δt, should be small enough so that there is a good definition of the hydrograph and the change in the hydrograph during the period Δt is approximately linear. This may be accomplished by setting Δt = 5 or 10 minutes, depending on size of watershed and hydrograph time to peak.

Several assumptions are made in this procedure and include the following:

- The entire inflow hydrograph is known.
- The storage volume is known at the beginning of the routing.
- The outflow rate is known at the beginning of the routing.
- The outlet structures are such that the outflow is uncontrolled and the outflow rate is dependent only on the structure's hydraulic characteristics.

The derivation of the routing equation begins with the conservation of mass which states that the difference between the average inflow and average outflow during some time period Δt is equal to the change in storage during that time period. This can be written in equation form as:

$$\bar{I} - \bar{O} = \Delta S / \Delta t \qquad (6\text{-}5)$$

Where:

$\bar{I}$	=	average inflow rate
$\bar{O}$	=	average outflow rate
ΔS	=	change in storage volume
Δt	=	routing period

If inflow during the period is greater than outflow, then ΔS is positive and the pond's water surface elevation increases. If inflow is less than outflow during the period, then ΔS is negative and the water depth in the pond gets shallower. Based on the previous assumptions, this equation can be rewritten as:

$$\left[\frac{I_1 + I_2}{2}\right] - \left[\frac{O_1 + O_2}{2}\right] = \left[\frac{S_2 - S_1}{\Delta t}\right] \qquad (6\text{-}6)$$

Where:

I_1	=	inflow rate at time interval 1
I_2	=	inflow rate at time interval 2
O_1	=	outflow rate at time interval 1
O_2	=	outflow rate at time interval 2
S_1	=	storage volume at time interval 1
S_2	=	storage volume at time interval 2
Δt	=	routing period

Multiplying both sides by two and separating the right-hand side yields:

$$(I_1 + I_2) - (O_1 + O_2) = \left[2\frac{S_2}{\Delta t} - 2\frac{S_1}{\Delta t} \right] \tag{6-7}$$

Rearranging so that all the known terms are on the left-hand side and all the unknown terms are on the right-hand side yields the final routing equation:

$$(I_1 + I_2) + \left[2\frac{S_1}{\Delta t} - O_1 \right] = \left[2\frac{S_2}{\Delta t} + O_2 \right] \tag{6-8}$$

Equation 6-8 has two unknowns, S_2 and O_2. A second equation is needed which relates storage and outflow to solve those two unknowns. If outflow is a direct function of reservoir depth (as it is with uncontrolled outflow), there is a direct relationship that exists between reservoir elevation, reservoir storage, and outflow. Therefore, for a particular elevation, there is an answer for storage and outflow (S and O). A relationship between O and $(2S/\Delta t) + O$ is determined for several elevations and plotted on logarithmic graph paper. The routing equation is solved by adding all the known terms on the left side of the Equation 6-10. This yields a value for $(2S_2/\Delta t) + O_2$. This value is found on the log-log plot of $(2S/\Delta t) + O$ versus O and a value of O_2 can be determined.

6.5 DEBRIS AND SEDIMENTATION

The performance and reliability of detention facilities can be reduced by natural and man-made debris. Naturally occurring sedimentation can, over a period of time, reduce the storage capacity of a detention basin and thereby reduce the degree of flood protection provided. The obstruction of low flow conduits by debris can reduce outlet capacity and cause the premature filling of the detention basin with stormwater, again reducing the flood protection provided by the structure. Consequently, adequate care must be exercised in design to provide for protection of the outlet works from debris and for the control and removal of sedimentation in the basin.

6.5.1 Trash Racks

All outlet works and low flow pipes shall be provided with a trash rack for debris control. The trash rack shall provide a maximum bar spacing not to exceed two-thirds of the outlet opening or diameter. The total area of the trash rack shall allow for passage of the design flow with 50 percent of the trash rack blocked. Calculations for head losses through a trash rack shall be included in the hydraulic evaluation of the outlet. The trash rack should have an area equal to 10 times the area of the outlet to maintain low velocities through the trash rack.

6.5.2 Sedimentation

Sediment removal within a detention facility may be facilitated by the use of a sediment trap at the inlet, which concentrates the majority of the incoming sediment bed load to a small portion of the facility. Sediment traps should be provided in conjunction with all detention facilities. Additional information regarding sedimentation and water quality can be found in Chapters 7 and 8. The following list provides guidelines for the design of efficient sediment traps:

1. Sedimentation volume should be provided at an elevation below the invert of the inflow channel.

2. The length/width ratio of the sediment trap should be a minimum of 2:1, with the length measured along a line between the inlet and outlet.

3. The basin shape should be configured to prevent flow short-circuiting from the inlet to the outlet. This can be accomplished by placing the inlet at the opposite end or installing flow baffles. This is to allow the finer sediments the maximum residence time to settle out.

4. Provisions for total drainage and accumulated sediment removal of the sediment trap shall be provided. Maintenance access should also be provided and designed to accommodate dump trucks and other equipment necessary for removal of accumulated sediment.

7.0 BEST MANAGEMENT PRACTICES - STRUCTURAL CONTROL

One of the goals of implementing Best Management Practices (BMPs) is to control nonpoint source pollution by focusing attention and design to potential contributing sources. Structural BMPs, to be further referred in this Chapter as simply BMPs, accomplish this by collecting, concentrating, and/or treating the runoff. The controls incorporated into BMPs are designed to prevent surface runoff from carrying pollutant loads exceeding pre-development levels into surface waters.

Structural BMP types are generally categorized by their primary pollutant removal mechanism as:

- Detention
- Filtration
- Infiltration

Detention processes control stormwater on site either in a permanent or temporary pool with an outlet structure designed to later release the volume at a predetermined rate. The period of detention allows for a dissipation of velocity and the settlement of solids from the discharge. Filtration type BMPs use biological, physical, and chemical processes that provide a reduction in the concentration of contaminants in storm water. Plant uptake, chemical fixation, and physical impedance are processes that assist in filtration. Infiltration controls provide for a reduction in the quantity of storm water generated by enhancing the rate of infiltration of storm water into the soil surface. Some methods for controlling storm water quantity may have the secondary benefit of improving water quality. Once storm water enters the vegetation and soil matrix natural processes work to treat storm water for quality control. Other infiltration techniques require pretreatment, especially when used in coarse soil environments. Selected BMPs can incorporate several of the aforementioned techniques. These BMPs are highly effective in treating a range of pollutants for a range of efficiencies.

All projects shall have in series BMPs and all stormwater management system designs shall contain at a minimum one wet detention BMP, one vegetative BMP and one filter or infiltration based BMP.

Projects shall be designed to include a minimum of three BMPs in series to meet the requirements set forth in the Stormwater Management Ordinance. The BMPs shall be selected based on site conditions to maximize their effectiveness.

7.1 GENERAL DESIGN CRITERIA

South Carolina Coastal Zone Management Program provides regulations for the operation and performance of certain structural BMPs. The BMPs specifically include wet (detention) pond types, extended detention pond types, and infiltration trenches. These regulations can be extrapolated to other BMPs that operate on the same principals of detention and infiltration. These BMPs include constructed wetland systems, bioretention areas, vegetated swales, sand filters, filter strips, and permeable pavements. The performance control for these BMPs is based on their type and proximity to environmentally sensitive areas, such as receiving waters in the coastal zone and shellfish beds. The controls are the volumetric control of runoff and release (drawdown) durations.

In many cases, it may be necessary to incorporate additional storage beyond the water quality volume into a BMP for peak runoff discharge control and release. The design professional should review Chapter 6 Storage Facilities for more guidance on meeting the peak runoff discharge requirements. Additional guidance on the water quality volumetric control is provided in the individual BMP design section.

The Town will require the use of multiple BMPs in series to treat and control water quantity and quality. For example, stormwater runoff from a roadway system in a neighborhood can be collected in vegetated swales, then routed to a bioretention area, which is then discharged to a wet pond. This series of BMPs filters, infiltrates, and detains pollutants as they move through the system.

7.2 SITE CONSIDERATION AND GUIDANCE

There are four screening criteria that can be used to identify the suitable structural BMP for a given site. The first category is the physical suitability of a site for constructing and maintaining the BMP. This includes the technical feasibility criteria related to physical conditions such as slope, soils, geology, groundwater, and area requirements. The second category is related to mitigation of adverse changes in hydrologic conditions such as increases in peak flow and volume of runoff caused by development. The third category is the pollutant removal capability of the structural BMPs, and the final category deals with the environmental and aesthetic amenities provided by the structural BMPs. Table 7-2 lists the criteria for each of the four screening categories.

Figures 7-1 through 7-6 were developed for comparison of screening criteria for several of the main types of BMPs. The data in these Figures has been compiled from various sources and adapted to best represent each BMP type for performance or suitability to meet site considerations or criteria. The prominent sources for the data came from A Practice for Planning and Designing Urban BMPs (Schueler, 1987) and A Community Framework for Ranking Storm water Strategies (Echols, 2002).

TABLE 7-2
SCREENING CRITERIA FOR SELECTING STRUCTURAL BMPs

Screening Category	Criteria
Physical suitability	Drainage area Soil types Slope Depth to water table Proximity to foundations and wells Land-area requirement Maximum depth Suitability with other land uses Ability to handle high sediment inputs Thermal impacts on receiving waters
Hydrologic conditions	Peak discharge control Volume control Groundwater recharge Streambank protection Erosion control
Pollutant removal capability	Sediment Nutrients Oxygen demand Heavy metals Pathogens Temperature
Environmental and aesthetic amenities	Maintenance of low flows for aquatic life Creation of habitat for wildlife and aquatic plants Impact on receiving water temperature Landscape enhancement and aesthetic value Recreational benefits Safety hazard Community acceptance
Source: Effective Watershed Management for Surface Water Supplies	

7.2.1 Physical Suitability Criteria

The screening process of structural BMPs should begin with their physical suitability to local conditions. This is the first and most important step in the selection process. Two of the main physical factors to be considered are the total contributing drainage area and the infiltration rate of the soils. The suitability of BMPs with respect to drainage area and type of soils is presented in Figures 7-1 and 7-2. Experience has shown that both wet and extended detention ponds require a contributing area of at least 10 acres to operate properly. Wet ponds are defined as facilities with permanent pools of water, which require a drainage area and suitable climatic conditions to maintain a permanent flow through the facility during drier periods. The land requirements for extended detention ponds are governed by the size of the discharge structure (usually an orifice). Filter strips be refer to as vegetated areas on new construction or revegetated areas on existing disturbed, graded slopes and are designed to accept only overland sheet flow. This area limitation does not apply to vegetative buffers along streams and water bodies where an

area limitation would not be applicable. Infiltration techniques are better suited for coarse-textured soils while wet ponds are not advised as an option because these soils are highly permeable, making it difficult to maintain a permanent pool of water except where groundwater tables are high enough to maintain the permanent pool elevation.

Other physical restrictions include the slope of the site, proximity of nearby wells and building foundations, contributing area requirements, maximum depth limits, applicability to certain land uses, ability of the structural BMPs to handle large sediment loads without clogging, and thermal impact on receiving waters. Common restrictions for various structural BMPs are presented in Figure 7-3. A high water table that impedes downward movement of water limits use of infiltration BMPs. Care must be taken to avoid locating infiltration BMPs close to buildings or water supply (wells) so cross contamination of drinking water is avoided. Because extended detention ponds often require large areas, placement in existing developments may not be feasible due to space requirements. In addition, many structural BMPs (ponds, trenches, etc.) have depth limitations. When the depth of ponding exceeds 8 feet, stratification during warm weather can occur resulting in anoxic conditions along the bottom of the pond. Several structural BMPs are limited to certain type of land uses or development densities. Vegetated swales are usually limited to low-density residential development and road rights-of-way. Use of sand filters are often appropriate for impervious areas that generate high quantities of hydrocarbons, such as parking lots, gas stations, and commercial developments.

7.2.2 Hydrologic Criteria

Hydrologic criteria focus on the ability of the structural BMPs to reduce the runoff from an area to pre-development levels or some other defined condition. Criteria include peak discharge control, volume control, groundwater recharge, erosion control, and streambank protection. Figure 7-4 compares the ability of various structural BMPs to meet these conditions. As was the case in the physical suitability criteria, there is no single BMP that can mitigate all hydrologic modifications caused by urban development.

The Office of Ocean and Coastal Resource Management (OCRM) requires water quality BMPs to be designed to control and treat in variety of performance levels based on the proximity to sensitive environmental areas and their treatment process. Pond BMPs accomplish this by temporarily storing a portion of the runoff up to desired volume. This is accomplished by using an outlet structure that allows only a certain flow to pass while detaining the additional volume until the peak has passed. When the inflow to the pond falls below the maximum allowable outflow, the volume stored in the structure will begin to decrease. Due to the typically low storage potential of infiltration BMPs, they often have limited capacity to reduce peak flow from higher volume storm events, but may be appropriate for peak control of 2- and 5-year storms. Likewise, vegetative BMPs usually have almost no peak discharge control.

Volume control is the decrease in the total volume of runoff to downstream areas. In this respect, detention ponds are ineffective; since they just provide temporary storage of runoff until the peak passes, and then release the remainder of the volume. On the other hand, infiltration BMPs are designed to divert water back into the soil, thus they are effective in reducing the total volume of runoff from the upstream area and have inherent water quality benefits. These structures also provide an excellent means of providing groundwater recharge by directing runoff into the soil.

As a general rule of thumb, most natural channels with banks flowing full can contain a 2-year storm. Therefore, BMPs that release flows of a 2-year storm or greater event magnitude create erosive conditions. It is desirable to control the discharge so that it is below the 2-year flow as well as the frequency in which these events occur. Extended detention ponds and some infiltration devices, if designed and maintained properly, can do this effectively.

7.2.3 Pollutant Removal Criteria

Pollutant removal is a function of three interrelated factors:

- Removal mechanisms (physical, chemical, and biological processes)
- Fraction of runoff to be treated by the BMP
- Pollutant(s) targeted for removal

Figure 7-5 outlines the types of pollutants that each BMP may be effective in controlling. The BMPs that use settling and filtering processes are effective in removing sediment and pollutants (both solid and soluble) that adhere to sediment particles. Case studies have shown that ponds can remove 75 or more percent of sediment. However, infiltration BMPs are not recommended where there are high loadings of sediment due to the potential for clogging. An alternative is to provide a filter strip to remove sediment prior to the discharge to the infiltration BMP. It has been shown that providing shallow littoral shelves along the perimeter of wet and extended detention ponds can have a moderate to high capability for removing both particulate and soluble pollutants because of settling and biological uptake. The use of vegetated swales and filter strips often provides moderate removal rates in comparison to detention type BMPs.

The pollutant removal efficiencies for structural BMPs are presented in Table 7-3. The removal efficiencies are based on field monitoring, laboratory experiments, modeling, and theoretical considerations. These standards are included to give the designer specific requirements in choosing the appropriate structural BMP based on the pollutants targeted for removal.

TABLE 7-3
POLLUTANT REMOVAL EFFICIENCIES OF STRUCTURAL BMPs

BMP	Suspended Sediment (%)	Total P (%)	Total N (%)	Trace Metals (%)	Overall Removal Efficiency
Wet Pond	75	50	40	45	Moderate-High
Extended Detention Pond	60	30	25	30	Moderate-High
Constructed Wetland System	70	40	35	50	Moderate
Infiltration Trench	75	60	45	60	Moderate-High
Bioretention Area	80	60	50	70	Moderate-High
Filter Strip	50	20	30	50	Low-Moderate
Vegetated Swale	40	35	30	20	Low-Moderate
Sand Filter	80	45	25	45	Moderate
Permeable Pavement	*	40	60	55	Moderate-High
Exfiltration Trench	70	50	40	60	Moderate

* Permeable Pavements should not be used as a primary BMP for controlling sediments.

7.2.4 Environmental and Aesthetic Amenity Criteria

The selection of structural BMPs is often determined by the environmental benefits that can be achieved and by the community's willingness to accept the facility. Figure 7-6 lists the environmental and aesthetic amenities that certain BMPs may contribute. Environmental and aesthetic amenities may benefit the community by the creation of aquatic and wildlife habitat, landscape and aesthetic enhancement, or creation of recreational facilities. However, the environmental and aesthetic benefits may not be realized unless community acceptance of the structure is obtained.

	1 ACRE	5 ACRES	10 ACRES	25 ACRES	50 ACRES	>100 ACRES
WET (RETENTION) POND	○	○	◐	●	●	●
EXTENDED DETENTION POND	○	○	◐	●	●	●
WETLAND SYSTEM	○	◐	◐	●	●	●
INFILTRATION TRENCH	●	●	◐	○	○	○
BIORETENTION AREA	●	●	◐	○	○	○
FILTER STRIP	●	●	◐	○	○	○
VEGETATED SWALE	●	●	◐	○	○	○
SAND FILTER	●	●	◐	○	○	○
PERMEABLE PAVEMENTS	●	◐	○	○	○	○

○ MAY PRECLUDE THE USE OF THIS BMP
◐ CAN BE OVERCOME BY GOOD SITE DESIGN
● GENERALLY NOT A RESTRICTION

Figure 7-1 Drainage Area Restrictions of Structural BMPs

	SAND	SANDY LOAM & LOAMY SAND	LOAM	SILT LOAM	SANDY CLAY LOAM	CLAY LOAM	SILTY CLAY LOAM	SILTY CLAY & SANDY CLAY	CLAY
WET (RETENTION) POND	○	◐	●	●	●	●	●	●	●
EXTENDED DETENTION POND	●	●	●	●	●	●	●	◐	○
WETLAND SYSTEM	○	◐	●	●	●	●	●	◐	○
INFILTRATION TRENCH	●	●	●	◐	○	○	○	○	○
BIORETENTION AREA	●	●	●	◐	○	○	○	○	○
FILTER STRIP	◐	●	●	●	●	●	●	●	◐
VEGETATED SWALE	◐	●	●	●	●	●	●	●	◐
SAND FILTER	●	●	●	◐	○	○	○	○	○
PERMEABLE PAVEMENTS	●	●	●	◐	◐	◐	◐	◐	◐

○ MAY PRECLUDE THE USE OF THIS BMP
◐ CAN BE OVERCOME BY GOOD SITE DESIGN
● GENERALLY NOT A RESTRICTION

Figure 7-2 Soil Restrictions of Structural BMPs

	SLOPE	HIGH WATER TABLE	HIGH BEDROCK	PROXIMITY TO FOUNDATIONS	SPACE CONSUMPTION	MAXIMUM DEPTH	LAND USE RESTRICTIONS	HIGH SEDIMENT LOADS	THERMAL LOADING
WET (RETENTION) POND	●	●	◐	◐	○	○	●	◐	◐
EXTENDED DETENTION POND	●	●	◐	◐	○	○	●	◐	●
WETLAND SYSTEM	◐	◐	◐	◐	◐	○	◐	◐	○
INFILTRATION TRENCH	○	○	○	○	◐	○	●	○	●
BIORETENTION AREA	◐	○	○	○	●	○	◐	○	◐
FILTER STRIP	◐	◐	●	●	◐	●	●	○	○
VEGETATED SWALE	◐	◐	◐	●	●	◐	●	◐	○
SAND FILTER	○	○	○	○	◐	○	◐	○	●
PERMEABLE PAVEMENTS	○	○	◐	◐	◐	●	◐	○	●

○ MAY PRECLUDE THE USE OF THIS BMP
◐ CAN BE OVERCOME BY GOOD SITE DESIGN
● GENERALLY NOT A RESTRICTION

Figure 7-3 Site Restrictions of Structural BMPs

	2-YEAR STORM	10-YEAR STORM	100-YEAR STORM	VOLUME CONTROL	GROUNDWATER RECHARGE	EROSION CONTROL	STREAMBANK PROTECTION
WET (RETENTION) POND	●	●	●	●	○	◐	●
EXTENDED DETENTION POND	●	●	●	●	○	◐	●
WETLAND SYSTEM	●	◐	○	◐	◐	●	●
INFILTRATION TRENCH	◐	◐	○	◐	●	◐	◐
BIORETENTION AREA	◐	○	○	◐	●	◐	○
FILTER STRIP	◐	○	○	○	◐	●	◐
VEGETATED SWALE	●	◐	○	○	◐	●	◐
SAND FILTER	◐	○	○	●	◐	◐	◐
PERMEABLE PAVEMENTS	◐	◐	○	◐	◐	◐	○

○ WILL NOT CONTRIBUTE
◐ CAN CONTRIBUTE WITH GOOD SITE DESIGN
● GENERALLY DOES NOT CONTRIBUTE

Figure 7-4 Hydrologic Benefits of Structural BMPS

	SEDIMENT	PHOSPHORUS	NITROGEN	OXYGEN DEMAND	HEAVY METALS	PATHOGENS	TEMPERATURE
WET (RETENTION) POND	●	●	◐	◐	●	○	○
EXTENDED DETENTION POND	●	◐	◐	◐	◐	○	○
WETLAND SYSTEM	●	◐	○	●	●	○	●
INFILTRATION TRENCH	●	●	●	○	●	●	●
BIORETENTION AREA	●	◐	◐	●	●	◐	◐
FILTER STRIP	◐	◐	◐	○	◐	○	○
VEGETATED SWALE	○	◐	◐	○	○	○	○
SAND FILTER	●	◐	○	○	◐	◐	◐
PERMEABLE PAVEMENTS	◐	◐	●	◐	●	○	●

○ NOT EFFECTIVE
◐ MODERATELY EFFECTIVE
● HIGHLY EFFECTIVE

Figure 7-5 Pollutant Control of Structural BMPs

	STREAM LOW FLOWS	AQUATIC HABITAT	WILDLIFE HABITAT	LANDSCAPING	RECREATION	AESTHETICS
WET (RETENTION) POND	◐	●	◐	◐	●	●
EXTENDED DETENTION POND	○	○	○	◐	●	◐
WETLAND SYSTEM	●	●	◐	●	●	●
INFILTRATION TRENCH	●	○	○	◐	○	○
BIORETENTION AREA	●	○	○	●	○	●
FILTER STRIP	◐	○	◐	●	◐	●
VEGETATED SWALE	●	○	○	○	○	◐
SAND FILTER	◐	○	○	◐	○	○
PERMEABLE PAVEMENTS	◐	○	○	◐	◐	◐

○ WILL NOT CONTRIBUTE
◐ CAN CONTRIBUTE WITH GOOD SITE DESIGN
● GENERALLY DOES CONTRIBUTE

Figure 7-6 Environmental and Aesthetic Amenity Contribution of Structural BMPs

7.3 BMP PLACEMENT STRATEGY

There are many benefits to be gained by strategic placement and location of differing BMP types. The benefits include water quality control, space utilization, facility protection, permitting ease, maintenance effort reduction, and maximization of operation. All BMPs operate on a variety of performance levels when it comes to pollutant treatment and quantity control. Configuring systems of BMPs to meet different objectives may enhance the overall efficiency provided by a single BMP and at a lower cost.

7.3.1 Online versus Offline

In certain instances, a BMP may be sited in the direct conveyance of runoff. This location of a BMP is considered online, since it is directly receiving the concentrated discharge. While online placement may present fewer challenges in directing and collecting storm water, it may not be necessary. Remember that the first flush is considered to have the highest density of pollutant loading during a runoff event. In many instances, treating only the first flush, the early portion of the runoff event exhibiting the highest concentrations of pollutants, is necessary to meet a storm water quality objective. This is achieved through the use of a diversion that is sized only to handle the treatable flow or water quality volume. As the runoff event proceeds and the discharge increases, an online BMP must also be designed to handle the higher peak flows and volumes. This may require extensive land requirement to handle the extra volume and expansive outlet structures to control the peak. The complexity of an offline BMP designed to handle only the first flush may be minimized since it will handle a smaller flow volume. It will likely also operate more efficiently. Larger runoff events will be bypassed to a facility that has been designed only to control large volumes. This approach minimizes the impact to the stability of the offline facility.

Other situations exist where an existing hydrological feature is already classified as a regulated stream or jurisdictional wetland. Siting BMPs offline from these features will limit the occurrence of

required permitting to work in these areas or modify these features. This is typical of BMP retrofits and situations where a man-made conveyance channel has become regulated as stream. Offline BMPs may also present fewer difficulties in BMP maintenance.

However, in any of these circumstances, offline BMP siting does not revoke the requirement to control post construction peak flow rates to levels at or below pre-construction.

7.3.2 Treatment Train

The treatment train concept of BMP design and operation is based on the location of several BMPs in series or in line to receive another BMP's discharge. This concept is advantageous for several reasons. BMPs operate at varying levels of performance and treatment efficiency. The use of BMPs in series to maximize the treatment of a variety of pollutants is common. A wet pond may be used to settle out particulate pollution and then discharged to vegetative filter for nutrient uptake or infiltration. This type of system creates redundancy in treatment processes and configures the storm water for the optimal treatment. For example, overland discharge from wet pond through a filter strip has more opportunity for treatment than shallow concentrated flows. In many cases, a pretreatment of storm water is required. The pretreatment approach is similar to a treatment train. For further information and examples of the use of a "treatment train" concept, refer to the State of Georgia BMP Manual (see Section 7.5 for reference on obtaining Manual).

Other advantages in treatment train design include the reduction in maintenance effort and minimization of space. Certain BMPs located at the head of a treatment train can handle the bulk of a pollutant load therefore reducing the volumetric requirement for additional downstream BMPs. These facilities, when properly located, may be easier to access and maintain then other BMPs. A typical example of this benefit includes the forebay at the head of a sand filter. The forebay acts to collect and concentrate the heaviest pollutant loads for frequent removal and prevent clogging of the downstream sand filter. This concept is termed pretreatment and is necessary for a variety of BMP types. Wet ponds can be limited by their size when it comes to treatment. However, reducing the wet pond to an efficient size and discharging its effluent to storm water wetland will increase pollutant removal efficiency and decrease the needed space of either of the two BMPs.

7.4 BMP MAINTENANCE

Maintenance of BMP structures and surrounding grounds is instrumental to operation and long-term stability of the facilities. Addressing maintenance problems on a regular basis may prevent significant failure of a facility that could result in downstream flood damage or erosion due to uncontrolled velocities. All levels of maintenance shall be performed at their prescribed time. The use of BMPs requires the inclusion of easements for inspection and maintenance.

<u>Levels of Maintenance</u>

There are three levels of maintenance: routine, remedial, and emergency. Routine maintenance includes the prescribed activities listed below. These activities should be performed on the suggested schedule to meet the objective of the maintenance activity. The next level of maintenance is remedial and is performed when issues or problems arise concerning the stability and operation of the BMP. These maintenance activities may require the help of an engineer or qualified professional for diagnostic evaluation and an engineered solution. Additionally, an annual inspection may trigger the need for a remedial maintenance activity. The final level of maintenance is emergency maintenance. Emergency maintenance may be required as a result of a major runoff event or may be required to prevent the potential for hazardous impacts that could occur during a major runoff event. Failure to address

emergency maintenance could result in down stream flood damage and erosion that may incur liability to the owner.

General Maintenance Requirements for BMPs

Typical required maintenance activities for most BMPs are as follows.

- Mowing and maintenance of vegetation on swales and side slopes.
- Debris and litter removal from easements, facility operation areas, and outlet/inlet structures.
- Removal and proper disposal of sediment from forebays, deep pools, ponds, etc.
- Clearance and repair of any structural problems with spillways, embankments, outlet and inlet structures.
- Repair of dissipation structures (riprap apron) or placement of new material.
- Remediation of areas of standing water other than wet ponds and wetlands.

Vegetative Maintenance Requirements

Typical required maintenance activities for most BMPs that include vegetative components are as follows.

- Harvesting of overgrown wetlands, wetland vegetation should cover no more than the prescribed percentage indicated in the design.
- Reestablishment of vegetation where disease of other acts of nature have reduced the population or diversity.
- Reseeded or replanting of barren areas around facilities and easements should be performed.
- Removal of invasive species that threaten to jeopardize the function of the facility.

Infiltration Maintenance Requirements

Typical required maintenance activities for most BMPs that include infiltration components are as follows.

- Sediment removal from filter media, if possible.
- Replacement of poorly operating filter media.
- Maintenance and repair of any poorly operating mechanical equipment, if necessary.
- Flushing or backwashing of filter media and underdrain system.

Maintenance Access

Adequate access must be available for inspections, maintenance personnel, and equipment. The location and configuration of easements must be established during the design phase, built to the design standards, and maintained regularly. Areas requiring access include dam embankments, emergency spillways, side slopes, inlets, forebays, riser structures, outlets, monitoring wells, level spreaders, and filter media. Consideration on width, aerial clearance, and transport surface should be evaluated and designed depending on the required types of maintenance and associated equipment to be used to perform the maintenance activity.

7.5 DESIGN GUIDELINES AND BMP FACT SHEETS

Design guidelines have been established for the aforementioned BMPs. They are located in the following consecutive sections. The design guidelines do not preclude the use of good engineering judgment or design features or specifications that may better serve the stability and function of the BMP. Design guidelines and procedures from previous chapters shall be referenced for BMP component design as

necessary. This includes chapters on Hydrology and Runoff Determination, Culverts and Bridges, Open Channel, and Storage Facilities.

The science and understanding of structural BMPs is an ever changing technology. The design guidelines and information contained in the Fact Sheet series provide a sound yet introductory view for BMP design and implementation. Those who will be designing, constructing, or operating BMPs are encouraged to explore more information on these existing and potentially new BMPs. Both the Town of Bluffton and designer stand to benefit from this exploration as future BMP design and operation continues to protect water quality with hopefully less expense and effort expended by the owner. Several excellent references are provided below for additional understanding and design guidance. Several of these references are the primary basis for design guidelines and the Fact Sheet series of this Manual.

- Georgia Storm Water Management Manual Sediment, Atlanta Regional Commission and Georgia Department of Natural Resources-Environmental Protection Division, 2001
  http://www.georgiastormwater.com/

- Manual of Stormwater Best Management Practices, North Carolina Department of Environment and Natural Resources-Division of Water Quality, 2005
  http://h2o.enr.state.nc.us/su/bmp_updates.htm

- Bioretention Manual, Prince George's Town Programs and Planning Division-Department of Environmental Resources, 2001
  http://www.co.pg.md.us/Government/AgencyIndex/DER/ESD/Bioretention/bioretention.asp?nivel=foldmenu(7)

- Pervious Concrete Pavements, Paul D. Tennis, Michael L. Leming, and David J. Akers, 2004
  http://www.concrete.org/pubs/newpubs/pcp.htm

Additionally, a series of Structural BMP Fact Sheets have been developed and located at the end of this Chapter. The Fact Sheets serve several purposes. They condense a wealth of knowledge about individual BMP types into a concise, organized document. The Fact Sheets include general description of the BMP type, suitability criteria, simple design recommendations, maintenance requirements, site considerations, benefits, implementation challenges, and pollutant removal capability. This series of compact documents provide an efficient and manageable means of communicating BMP design and function to developers, designers, and the general public. They can be copied and distributed with ease. Due to the singularly creation of these Fact Sheets, the series can be updated and expanded as new technology or methodology about storm water control is developed. Finally, the Fact Sheets are a stepping stone for peaking interest on further design and use of BMPs. The documents are not stand-alone design procedures.

The following is a list of BMP types currently included in the Fact Sheet series:

- Constructed Wetland System
- Bioretention Area
- Permeable Pavement System
- Infiltration Trench
- Filter Strip
- Vegetated Swale
- Sand Filter
- Exfiltration Trench
- Wet Pond
- Extended Detention Pond

7.6 WETLAND SYSTEM

7.6.1 Design Components

- Sediment Forebay
- Wetland System
- Inlets/Outlets/Spillways
- Benching/Shelving
- Embankment
- Buffer/Setbacks
- Vegetation/Soils
- Maintenance Access

7.6.2 Design Guidelines

Sediment Forebay

- Forebay should be constructed as a separate cell with a stable barrier (berm) and properly sized outlet between it and the main pool area
- Forebay depths should range from 3 to 6 feet and surface area should be 15 to 25 percent of total wetland area
- Exit velocities from forebay should be non-erosive (generally less than 2 feet per second) and distributed laterally if possible to the wetland system

Wetland System

- A water balance/budget shall be performed to ensure base flow or groundwater interaction maintains wetland depths to support vegetation and permanent pool level.
- Impervious runoff calculations shall be designed for the full proposed upstream development
- Wetland geometry should have a minimum length-to-width ratio of 2:1 to prohibit short circuiting of flow, mid-pool berms or braided channels may also be used to promote suitable detention times and flow paths, where possible
- All variety of wetland systems exist as a combination of some or all of these zones and associated hydrologic depths: deep water/pool (2 to 8 feet), high marsh (0.5 to 1.5 feet), shallow marsh (0 to 0.5 feet), and extended detention (ED) areas (temporarily inundated and drained areas)
- Approximate wetland zone distributions for various types of storm water wetlands are outlined in the following table:

TABLE 7-4
WETLAND ZONE DISTRIBUTIONS FOR STORM WATER WETLAND VARIETIES

Wetland Quantity	Shallow Wetland	ED Shallow Wetland	Pond/ Wetland	Pocket Wetland
Water Quality Volume Pool/Marsh/ED	25/75/0	25/25/50	70/30/0	25/75/0
Surface Area Pool/High Marsh/ Shallow Marsh/ED	20/35/40/5	10/35/45/10	45/25/25/5	10/45/40/5

Source: Massachusetts DEP, 1997 and Schueler, 1992

- Deep water/pool depths shall average from 2 feet to a maximum of 8 feet, with deeper sections should be located near the outlet to prevent sediment re-suspension near outlet
- Permanent pool depths shall not exceed 12 feet
- Deep water/pool bottom should have mild, positive grade for full drainage, not less than 1 percent.
- Additional storage volume may be incorporated into extended detention areas for flood control
- Extended detention volume should not rise more than 3 feet above the normal pool elevation
- Side slopes other than those specified shall not be steeper than 3:1 (H:V)

Benching/Shelving

- Minimum of 10 foot safety bench shall be incorporated around areas of deep water/pool with slopes no steeper than 6:1 (H:V) as measured from normal pool elevation
- High/shallow marsh zones may be incorporated into aquatic bench areas around deep water/pool

Embankment

- Limit embankment height to a maximum of 10 feet and locate top of embankment at least 1 foot above 100-year design storm pool elevation
- Embankment side slopes shall not be steeper than 3:1 (H:V)

Inlets/Outlets/Spillways

- Inlets for forebay and permanent pool should be stabilized by riprap or equivalent feature, reduce incoming velocities to non-erosive conditions, and provide lateral distribution where possible
- If the extended detention pond incorporates added detention for flood control, those design flow requirements must also be met
- Emergency/bypass spillway sized to pass the 100-year design storm
- Deep water/pool should have a bottom drain with a control valve for draining entire pool within a 24-hour period for maintenance purposes where topography allows otherwise a pumping alternative must be addressed
- Anti-seep collars, filter diaphragms, o-rings gaskets (ASTM C361) should be used on principal barrel outlets to prevent pipe failure
- Anti-flotation and anti-vortex trash racks should be installed on principal outlets
- Outlet protection in the form of riprap or other energy dissipation devices shall be provided at the end of the outfall for the deep water/pool

Buffer/Setbacks

- Wetland system should have a minimum of a 20-foot vegetated buffer with a 10 foot setback from buffer for structures, buffer shall be measured from normal pool elevation

Vegetation/Soils

- Soil types (Hydrologic Group C and D) conducive to growing wetland vegetation should be used during construction, placed to minimum depths of 0.5 feet, and loosely compacted
- A planting plan should be developed and submitted as part of the stormwater wetland design. Plant selection should be consistent with wetland zones and associated hydrologic depths
- Wetland vegetation should be planted within 7 days of the wetland grading to prevent the establishment of undesirable vegetation.
- Consult Appendix D-Wetland Vegetation Specifications of SC DHEC's Stormwater Management BMP Handbook for vegetation guidance.

Maintenance Access

- Provide a minimum 15-foot access path to sediment forebay for cleaning, additional access considerations should be made for inlet/outlet/spillway structures
- Access paths should not exceed slopes of 10 percent and need to be stabilized for vehicle traffic

7.6.3 Maintenance and Monitoring

Monthly

- Clear and remove debris from inlets, outlets, and spillways
- Inspect entire facility and surrounding area for erosion and correct immediately

Seasonal

- Mow side slopes and embankment
- Remove vegetation that is dead, diseased, or invasive
- Remove vegetation that is not part of the planned landscape, particularly woody vegetation in the embankment

Annual

- Inspect and test mechanical components and valves for proper operation
- Inspect inlet/outlet/spillway/embankment for stability, potential failure, and seepage

As Needed

- Monitor sediment accumulation in forebay with fixed vertical sediment marker and remove sediment when 50 percent of the storage volume has been lost
- Remove sediment accumulation in temporary pool when 25 percent of the volume has been lost
- Replant wetland vegetation in marsh areas if 50 percent of the surface area coverage is not achieved by the second growing season

7.6.4 General Plan and Profile

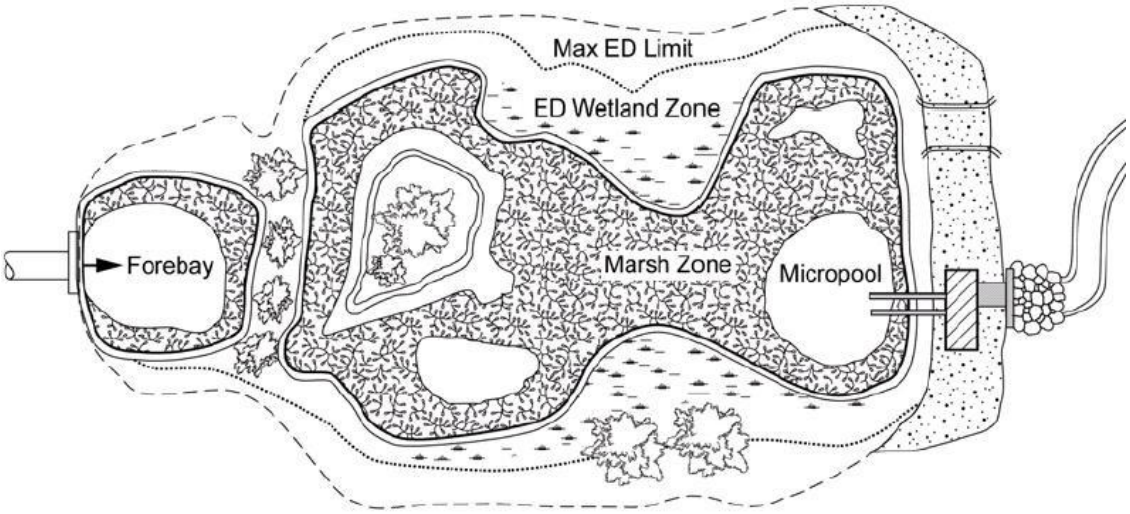


Figure 7-7 Wetland System General Plan

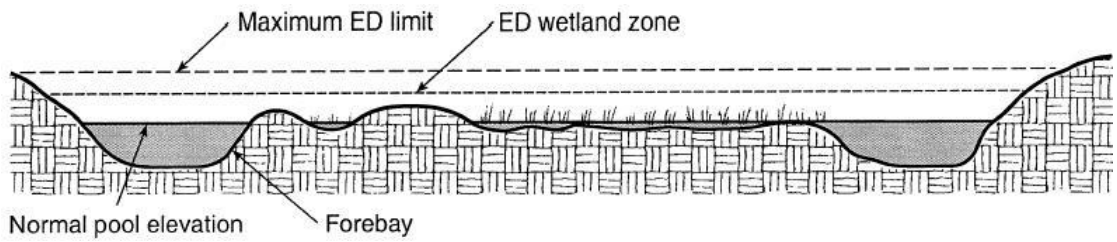


Figure 7-8 Wetland System Profile

Source: Design of Storm water Wetland Systems, Schueler 1992

7.7 BIORETENTION AREAS

7.7.1 Design Components

- Bioretention Area
- Filter Media
- Pretreatment
- Diversion/Bypass
- Underdrain System
- Vegetation/Buffer
- Maintenance Access

7.7.2 Design Guidelines

Bioretention Area

- Bioretention areas that do not infiltrate more than 75 percent of their OCRM water quality volume may not be an appropriate BMP within 1,000 feet of shellfish beds
- Bioretention area depths shall range between 3 to 5 feet, surface area is unlimited but shall have an approximate length to width ratio of 2:1 for draining purposes
- Bioretention area bottom should be nearly level to promote even flow distribution and conveyance
- Bioretention areas shall be located 25 feet from foundations and fills, 150 feet from private water supply wells and surface waters, and 500 feet from public water supplies
- Clearance between bottom of trench and groundwater should be a minimum of 2 feet
- Ponding depth above bioretention area shall not exceed 2 feet
- Surface area (square feet) for the bioretention area can be approximated from the following equation:

$$SA = \frac{Vd_{bio}}{kt(d_{bio} + d_{pond})} \qquad (7\text{-}1)$$

Where:

V	=	water quality volume, in cubic feet
d_{bio}	=	average depth of bioretention area, in feet
d_{pond}	=	average depth of ponding above bioretention area, in feet, max. 1 foot
k	=	coefficient of filter media permeability, feet per day, use 0.5 feet per day
t	=	design drain time for filter media, in days, use 2 days

Filter Media

- Soils used as filter media should have an infiltration rate higher than 0.5 inches per hour, clay content less than 20 percent or silt/clay content less than 40 percent, and pH levels between 5.5 and 6.5
- Planting soils shall have organic content of 2 to 5 percent
- A top layer of sand may be used to help distribute flow, a maximum depth of 1 foot shall not be exceeded
- A top layer of mulch may be used to help retain moisture, prevent erosion, and promote microbiological activity
- Mulch layer should consist of coarse shredded mulch, minimum-aged 6 months
- Permeable geotextile filter fabric shall be used between soil/mulch and sand layers

Pretreatment

- A pretreatment device (forebay, filter strip, pea gravel diaphragm) is necessary for the bioretention area to prevent clogging of filter media and reduce energy
- Pretreatment device shall be sized appropriately, and pretreatment volume may be counted towards total water quality volume
- Exit velocities from pretreatment device should be non-erosive (generally less than 2 feet per second) and distributed laterally if possible to the bioretention area

Diversion/Bypass

- Bioretention areas constructed offline shall have a properly sized diversion to permit only the design volume and associated flows
- Diversion may consist of deflector weir, slotted curb inlet, or similar flow diversion structure
- Bioretention areas constructed online shall have an overflow outlet to pass flows and volumes exceeding the treatment capacity

Underdrain System

- An underdrain system placed at the bottom of the bioretention area is necessary to facilitate drainage between rainfall events
- The system shall be composed of 4 to 6-inch PVC pipe (AASHTO M252) with 3/8 inch perforations spaced at 6 inch centers with a minimum of 4 holes per row
- Underdrain system shall be located in gravel layer with a minimum of 2 inches of cover (8 inch minimum layer depth)
- Gravel for backfill should have a porosity of 40 percent, sized 1.5 to 2.5 inches in diameter (AASHTO #1), washed and cleaned
- Underdrain laterals shall be placed on 10 foot centers and have a minimum of 0.5 percent slope.
- Gravel layer shall be separated from filter media with permeable geotextile filter fabric
- Single monitoring/clean out wells shall consist of similar PVC pipes extending vertically out of filter media and located for every 1,000 square feet of surface area
- Outlet protection in the form of riprap or other energy dissipation devices shall be provided at the end of the outfall for the underdrain system, as needed

Vegetation/Buffer

- A planting plan should be developed and submitted as part of the bioretention design. Plant selection should favor native, hydric specified, and aesthetic species and include a combination of woody and herbaceous vegetation is recommended with natural, random planting
- Bioretention areas sited with overland drainage shall have stable dense vegetation stands on all sides

- Bioretention vegetation should be planted within 7 days of the filter media filling to prevent the establishment of undesirable vegetation.
- Consult Appendix D-Wetland Vegetation Specifications of SC DHEC's Stormwater Management BMP Handbook for vegetation guidance.

Maintenance Access

- Provide a minimum 15-foot access path to bioretention area, additional access considerations should be made for diversion, underdrain system, pretreatment device, and monitoring well

7.7.3 Maintenance and Monitoring

Monthly

- Clear and remove debris from outlets, diversion, bypass, and pretreatment device,
- Inspect entire facility and surrounding area for erosion and correct immediately

Seasonal

- Remove vegetation that is dead, diseased, or invasive
- Inspect monitoring wells after heavy runoff events to ensure acceptable infiltration rates

Annual

- Test filter media pH and correct to 5.5 to 6.5 range with limestone to raise the pH or iron sulfate plus sulfur to lower the pH
- Inspect mulch for voids or damage from erosive flows, replace voids

As Needed

- Replace mulch every 2 to 3 years as needed

7.7.4 General Plan and Profile

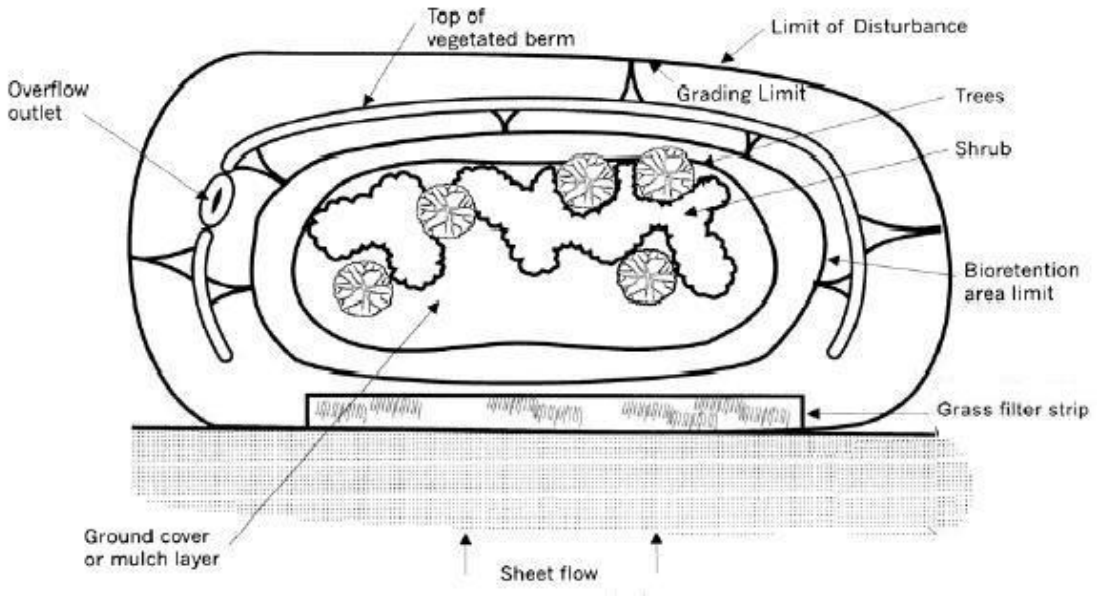


Figure 7-9 Bioretention Area General Plan

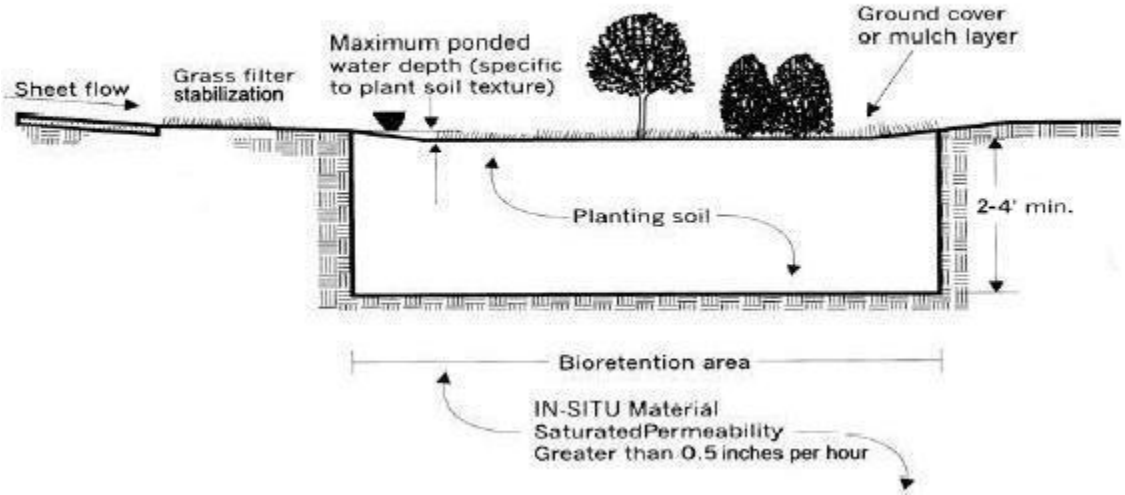


Figure 7-10 Bioretention Area Profile

Source: An Integrated Design Approach, Prince Georges Town 2000

7.8 PERMEABLE PAVEMENT SYSTEMS

7.8.1 Design Components

- General Design Considerations
- Pretreatment
- Subgrade
- Permeable Surface (Permeable Pavers)
- Permeable Surface (Porous Concrete)
- Underdrain System/Outlet
- Overflow
- Monitoring Well
- Maintenance Access

7.8.2 Design Guidelines

Imperviious Requirements

- One hundred percent (100%) of all parking spaces above the required amount identified in the Unified Development Ordinance shall be constructed of permeable surfaces; where soil conditions allow (soils comprised primarily of Hydrologic Soil Group A and B exhibit higher infiltration rates, and therefore are suitable for pervious paving)
- Infill development of single family lots, not part of a larger common plan of development, shall be required to implement the General Requirements of this manual. In addition they shall use permeable surfaces for driveways, sidewalks, and patios.

Structural Design Considerations

- Design/installation should be based on consideration of both hydrologic and structural properties of the permeable cover and underlying soils (subgrade)
- High loading areas, such as entrances, main travel lanes, exits, laoding zones, or access lanes shall be constructed with traditional paving construction (i.e. concrete, asphalt)
- Uses of permeable pavement systems shall be limited to parking lots, driveways, and access lanes for pedestrian vehicles, light delivery vans and emergency vehicles.
- Other uses can include recreational vehicle (i.e. golf carts, bicycles) and pedestrian paths and residential driveways.
- Permeable pavement systems and subgrade shall be designed to adequately support anticipated traffic loads.
- Determine in-situ subgrade strength value as an equivalent CBR or soil support value.
- Design permeable pavement layer structural capacity in accordance with AASHTO Guide for Design of Pavement Structures.
- Estimation of traffic weight and load analysis to include both weight of vehicles, and the sum of the Equivalent Single Axel Loads (ESALs) for the design life of the pavement.

Design Considerations

- Permeable pavement systems shall be located 25 feet from foundations and fills, 150 feet from private water supply wells and surface waters, and 500 feet from public water supplies.
- Permeable pavement systems should not be used where runoff has the potential to be contaminated with hazardous material that may contaminate groundwater during infiltration
- Permeable pavement systems should not be sited to receive offsite drainage that has the potential to convey sediment. The conveyance of sediments to permeable pavement systems can present clogging of the pavement and filter media.
- Properly designed, installed, and maintained porous paving technologies, including permeable concrete and pavers, will be considered 100 percent pervious and will not count against any total allowable impervious percentage on site, nor will it be considered impervious in determining the hydrologic runoff properties up to and including the 25-year design storm.

Subgrade

- Perform one in-situ infiltration capacity test per 5,000 square feet of permeable pavement system footprint
- Subgrade should have an infiltration rate higher than 0.5 inches per hour, clay content less than 30 percent or silt/clay content less than 40 percent.
- Subgrade should be graded to the slope not more than 2 percent.
- Avoid compaction of subgrade during construction activity or account for reduction of infiltration capacity due to compaction during construction activities.
- Clearance between top of subgrade and seasonal high groundwater should be a minimum of 2 feet.

Permeable Pavers

- Permeable paver systems include structural units made of concrete block, bricks, or reinforced plastic grids with regularly intermittent void spaces where a permeable substrate such as turf, gravel, and sand can be placed.
- Permeable pavers should be used where they receive drainage from impervious surfaces not to exceed three times the surface area occupied by the permeable pavers.
- Surface area of the selected permeable pavers should have a minimum void ratio of 40%.
- For permeable paver systems with surface voids not intended for planting, the paver fill material should consist of clean washed medium sand meeting ASTM C-33 concrete sand requirements.

- For permeable paver systems with surface voids intended for planting, the paver fill material should consist of a sandy loam with a minimum infiltration rate of 1 inch/hour.
- The permeable paver layer should be separated from the aggregate subbase with an 1 inch layer of clean washed medium sand meeting ASTM C-33 concrete sand requirements.
- Aggregate for backfill should have a porosity of 40 percent, sized 1.5 to 2.5 inches in diameter (AASHTO #1), washed and cleaned.
- Minimum depth for the aggregate subbase shall be 9 inches except where additional depth is needed. Aggregate subbase depth should detain total design volume using 1 inch of aggregate subbase per 0.32 inches of water.
- Permeable geotextile filter fabric should be lined on all sides of the permeable paver system and placed above and below the aggregate subbase.

Porous Concrete

- Porous concrete systems consist of a combination of portland cement, uniform, open-graded course aggregate and water that once installed promotes the high infiltration of runoff.
- Porous concrete should be used where they receive drainage from impervious surfaces not to exceed three times the surface area occupied by the permeable pavers.
- Porous concrete systems typically consist of four layers: porous concrete, top filter layer, aggregate reservoir, and bottom filter layer.

TABLE 7-5
POROUS PAVEMENT LAYERS DESIGN CRITERIA

Layer	Layer Depth (inches)	Layer Material (inch)	Standard
Porous Concrete	2-4	3/8 inch (maximum) course aggregate	ASTM C33
Top Filter	1-2	1/2 inch crushed stone	N/A
Aggregate Reservoir	24-48	1 ½ - 2 ½ inch	AASHTO #1
Bottom Filter	6	Concrete sand	ASTM C33
Source: Georgia Storm Water Management Manual (ARC, 2001)			

- Porous concrete and aggregate reservoir layer depths should be sized within acceptable ranges to detain the water quality volume using 1 inch of layer depth per 0.18 inches and 0.32 inches of volume for porous concrete layer and aggregate reservoir layer, respectively.
- All layer material should be washed and cleaned.
- Permeable geotextile filter fabric should be lined on all sides and bottom of the porous pavement system.
- Overflow structures should be incorporated into the porous concrete system to accommodate larger storm volumes.
- An underdrain system may be used with porous concrete systems where the subgrade infiltration rate is below specified criteria.
- Warnings prohibiting resurfacing the porous concrete systems should be posted in conspicuous places or referenced in maintenance plan.

Underdrain System/Outlet

- An underdrain system may be used with permeable pavement systems where the subgrade infiltration rate is below specified criteria.
- The system shall be composed of 4 to 6-inch PVC pipe (AASHTO M252) with 3/8 inch perforations spaced at 6-inch centers with a minimum of 4 holes per row.

- Underdrain system should be located in gravel layer with a minimum of 2 inches of cover (8 inch minimum layer depth).
- Gravel for backfill should have a porosity of 40 percent, sized 1.5 to 2.5 inches in diameter (AASHTO #1), washed and cleaned.
- Underdrain laterals shall be placed on 10 foot centers and have a minimum slope of 0.5 percent.
- Outlet protection in the form of riprap or other energy dissipation devices shall be provided at the end of the outfall for the underdrain system, as needed.

Overflow

- Storm drains located slightly above porous pavement elevation can provide for overflow while providing some ponding.
- Outlet protection in the form of riprap or other energy dissipation devices shall be provided at the end of the outfall for the overflow system.

Monitoring Well

- A 4- to 6-inch perforated PVC pipe shall be placed vertically extending to the bottom of the trench to monitor infiltration rates
- Monitoring wells shall be located every 5,000 square feet of permeable pavement system surface area and strategically as not to interfere with the function of the system.

Maintenance Access

- Provide access to the permeable pavement for the periodic scheduled maintenance of the infiltration surface. Access for a mechanical sweeper, vacuum sweeper, and or manual operations is to be planned.

7.8.3 Maintenance and Monitoring

Monthly

- Clear and remove debris from permeable pavement system surface, underdrain, and bypass.
- Inspect entire facility and surrounding area where the permeable pavement system surface appears to not quickly absorb rainfall or drainage.
- Inspect observation well after heavy runoff events to ensure acceptable infiltration rates or that leakage or bypassing is not occurring

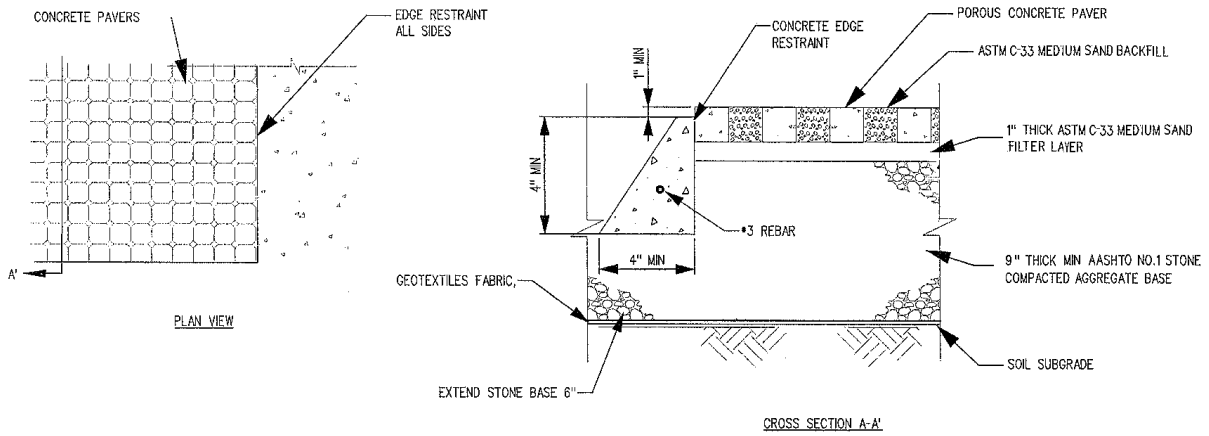
Annual

- Clean by appropriate methods of vacuum and or sweeping operations for collected sediment in voids of pavement infiltration surface.
- Inspect pavement structure for unacceptable cracks, spalling, settlement, visible clogging, or other defects

As Needed

- Replace top layer of permeable pavement system if infiltration rate has decreased even after surface cleaning.

7.8.4 General Plan and Profile



Source: HDR Engineering Inc of the Carolinas

Figure 7-11 Typical Permeable Pavement Plan

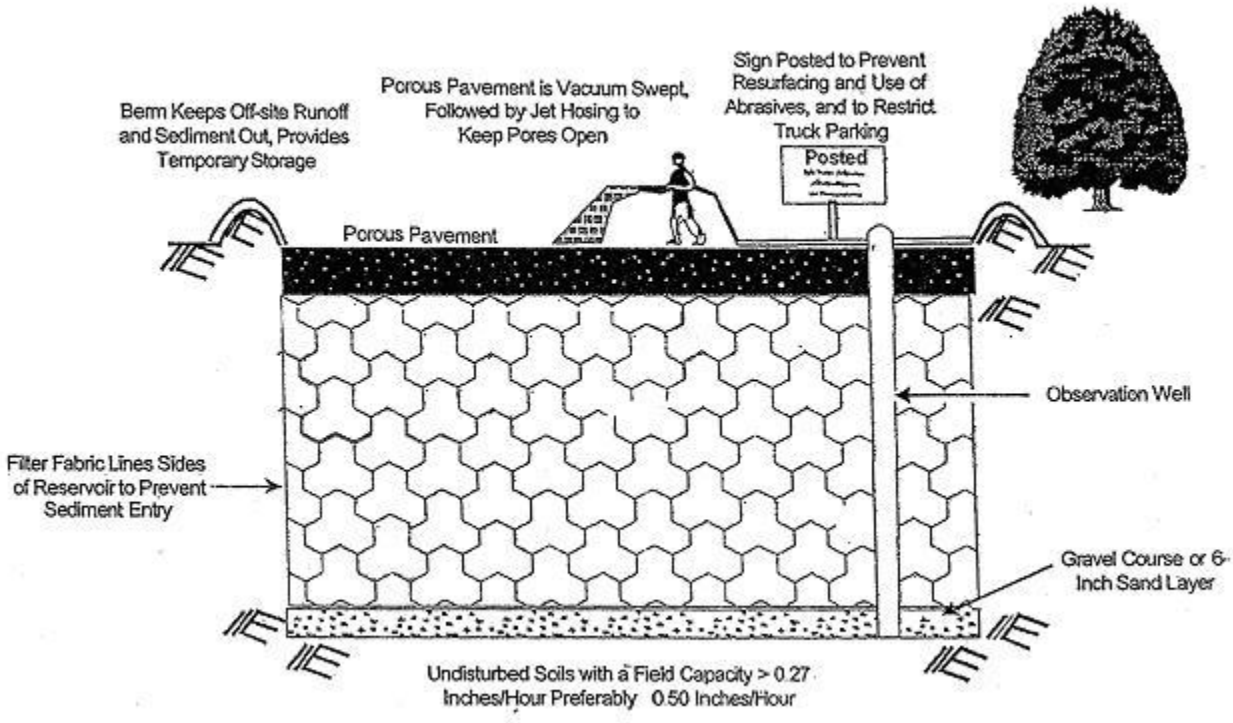


Figure 7-12 Typical Permeable Pavement Profile

7.9 INFILTRATION TRENCH

7.9.1 Design Components

- Trench
- Infiltration Media
- Pretreatment
- Buffer/Setbacks
- Monitoring Well
- Maintenance Access

7.9.2 Design Guidelines

Trench

- Trenches shall have a maximum width of 25 feet and depths should range from 3-8 feet, trench depths greater than 4 feet require shoring during construction for worker protection.
- Trench bottom should be nearly level, not to exceed 5 percent, to promote even flow distribution and conveyance.
- Trench bottom shall be at least 0.5 feet above the seasonal high water table, which is representative of the maximum height of the water table on an annual basis during years of normal precipitation.
- Underlying soils should have an infiltration rate higher than 0.5 inches per hour, clay content less than 20 percent or silt/clay content less than 40 percent.
- Surface area (square feet) for the infiltration trench can be approximated from the following equation:

$$SA = \frac{V}{0.32d + \frac{kT}{12}} \qquad (7\text{-}2)$$

Where:

V	=	water quality volume, in cubic feet
d	=	depth, in feet
k	=	infiltration, in inches per hour, use 0.5 inches per hour
T	=	time for volume to fill, in hours, use 2 hours in absence of other data

Infiltration Media

- Aggregate for backfill should have a porosity of 40 percent, sized 1.5 to 2.5 inches in diameter (AASHTO #1), washed and cleaned.
- A 0.5 foot minimum layer of sand shall be placed at the bottom the trench to prevent compaction of in-situ surfaces and promote infiltration.
- Line trench bottom and sides within a 0.5 foot of the top with permeable geotextile filter fabric to prevent soil from entering aggregate layer.
- A finishing layer of pea gravel may be used on the top of the trench.
- Permeable geotextile filter fabric shall be used between soil and aggregate/sand layers.
- If an outlet is used, outlet protection in the form of riprap or other energy dissipation devices shall be provided at the end of the outfall for the infiltration trench.

Pretreatment

- A pretreatment device (forebay, filter strip) is necessary for the infiltration trench to prevent clogging of filter media and reduce energy.
- Pretreatment device shall be sized appropriately; pretreatment volume may be counted towards total water quality volume.
- Exit velocities from pretreatment device should be non-erosive (generally less than 2 feet per second) and distributed laterally if possible to the infiltration trench
- Infiltration trench constructed offline shall have a properly sized diversion to permit only the design volume and associated flows
- Infiltration trenches constructed online shall have an overflow outlet to pass flows and volumes exceeding the treatment capacity.

Buffer/Setbacks

- Infiltration trenches sited with overland drainage shall have stable dense vegetation stands on all sides.
- Infiltration trenches shall be located 25 feet from foundations and fills, 150 feet from private water supply wells and surface waters, and 500 feet from public water supplies.

Monitoring Well

- A 4- to 6-inch perforated PVC pipe shall be placed vertically extending to the bottom of the trench to monitor infiltration rates
- Monitoring wells shall be located for every 50 feet of infiltration trench length

Maintenance Access

- Provide a minimum 15-foot access path to infiltration trench, additional access considerations should be made for diversion, outlet, pretreatment device, and monitoring well

7.9.3 Maintenance and Monitoring

Monthly

- Clear and remove debris from inlets, diversion, and bypass
- Inspect entire facility and surrounding area for erosion and correct immediately

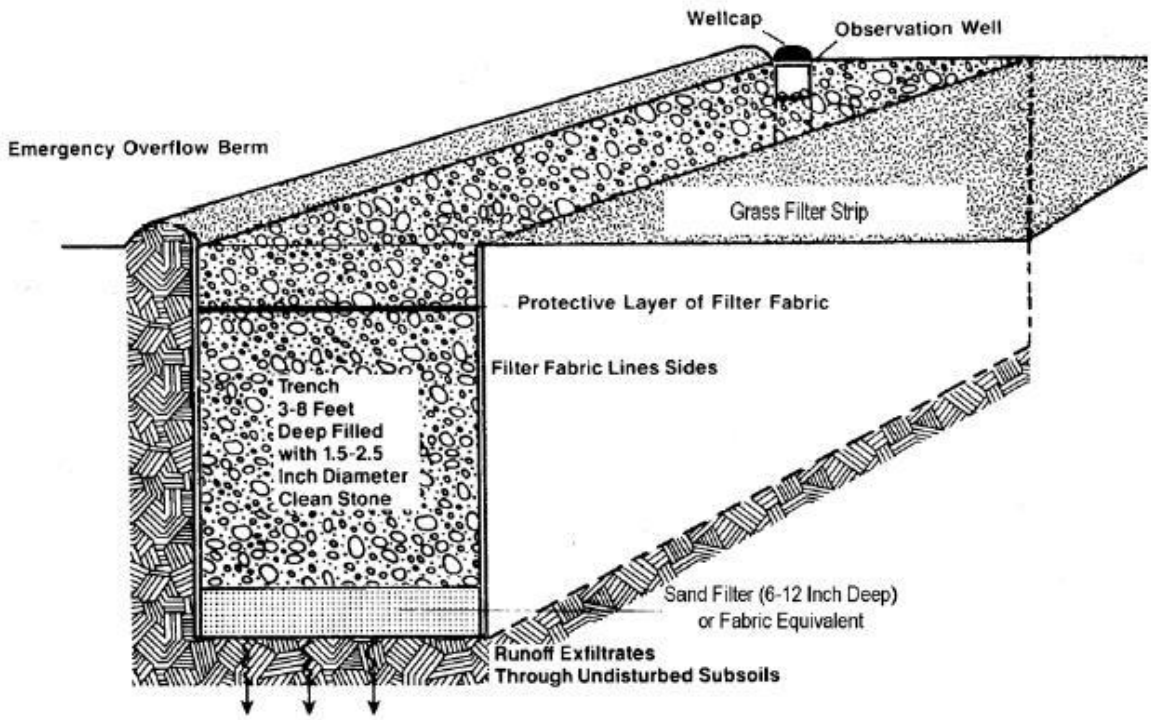
Seasonal

- Mow side slopes around infiltration trench
- Remove vegetation that is not part of the planned landscape
- Inspect monitoring wells after heavy runoff events to ensure acceptable infiltration rates

As Needed

- Replace top layer of filter media when infiltration rates begin to decrease
- Replace entire infiltration trench system if failure occurs

7.9.4 General Plan and Profile



Source: Design of Storm water Wetland Systems, Schueler 1992

Figure 7-13 Infiltration Trench General Plan and Profile

7.10 FILTER STRIP

7.10.1 Design Components

- Length/Width
- Grading/Slope
- Vegetation
- Level Spreader
- Maintenance Access

7.10.2 Design Guidelines

Length/Width

- Filter strip shall have a minimum length parallel to flow of 20 feet.
- Maximum discharge per unit width of filter strip can be calculated from Manning's equation below:

$$q = \frac{0.00236}{n} d^{\frac{5}{3}} S^{\frac{1}{2}} \tag{7-3}$$

Where:

q	=	discharge per one foot width of filter strip, in cubic feet per second per foot
d	=	allowable depth, in inches, preferably 1 to 3 inches
S	=	slope of filter strip, in percent
n	=	Manning's roughness coefficient, use 0.15 for medium grass, 0.25 for dense grass and 0.35 very dense grass

- The minimum filter strip width perpendicular to flow can be calculated from the equation below:

$$w = \frac{Q}{q} \tag{7-4}$$

Where:

w	=	minimum total width of filter strip, in feet
Q	=	2-year design storm runoff, in cubic feet per second
q	=	discharge per one foot width of filter strip, in cubic feet per second per foot

Grading/Slope

- Design flow through the filter strip shall range in depth from 1 to 3 inches.
- Velocity of flow through the filter strip shall not exceed 2 feet per second.
- Filter strip slope shall range from 2 to 6 percent.
- The head and toe of the filter strip shall have near level slopes and graded for smooth transitions into the surrounding grade.

Vegetation

- Vegetation shall be planted within 7 days of filter strip grading and maintained at 2 to 4 inches
- Vegetation shall be deep rooted, have well branched top growth, and resistant to drought and inundation
- Consult Appendix C-Seeding Rates of SC DHEC's Stormwater Management BMP Handbook for vegetation guidance

Level Spreader

- Level spreaders (natural berms, structural devices, aggregate diaphragms) shall be used to create sheet flow into the filter strip when flow is concentrated
- Berm type, level spreaders shall be designed to have a uniform, minimum height of 0.5 feet

Maintenance Access

- Access should be provided to mow filter strip and repair level spreader, if necessary

7.10.3 Maintenance and Monitoring

Monthly

- Inspect entire facility and surrounding area for erosion and correct immediately

Seasonal

- Mow or maintain vegetation in the filter strips
- Remove vegetation that is not part of the planned landscape

Annual

- Inspect level spreader for wear and evenness

As Needed

- Re-grade and re-seed if erosion rills or gullies have formed

7.10.4 General Plan and Profile

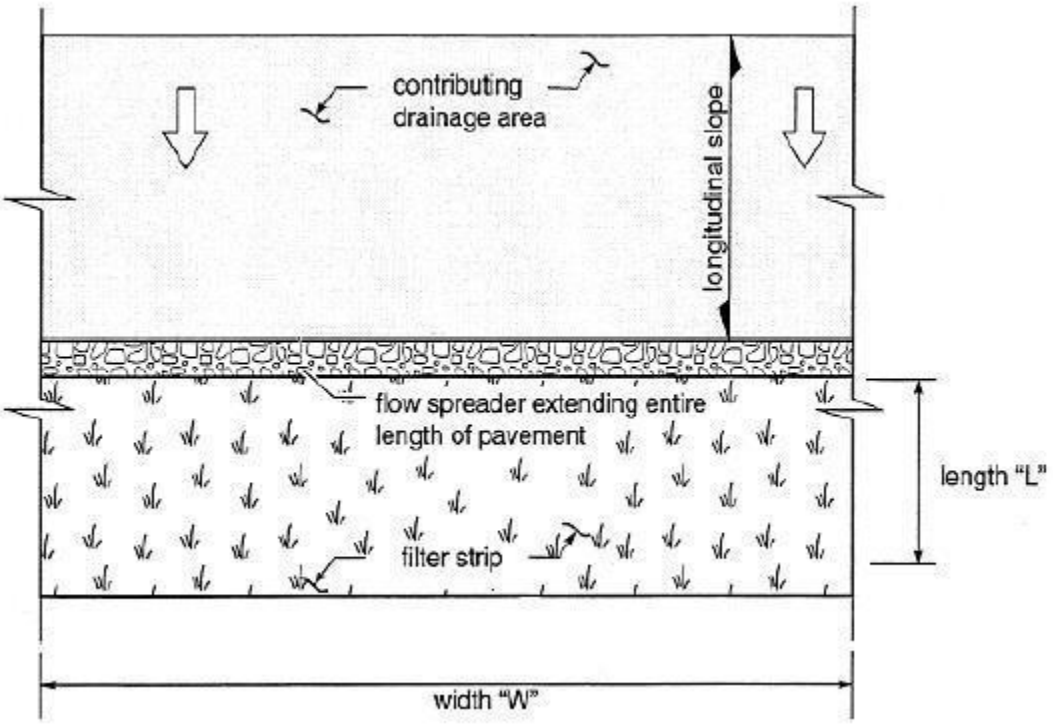


Figure 7-14 Filter Strip General Plan

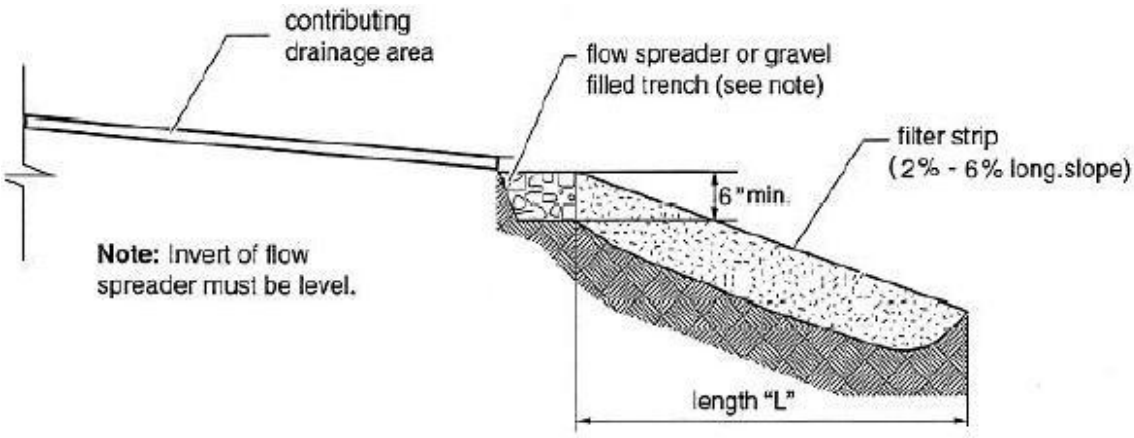


Figure 7-15 Filter Strip Profile

Source: Hydraulics Manual, WSDOT 2005

7.11 VEGETATED SWALES

7.11.1 Design Components

- Swale
- Dry Swale
- Pretreatment
- Outlet/Bypass
- Vegetation
- Maintenance Access

7.11.2 Design Guidelines

Swale

- Lot swales graded for drainage can be developed for water quality treatment and serve as an initial BMP, when properly designed.
- Longitudinal slopes shall not exceed 2 percent.
- Slopes greater than 2 percent should incorporate 6 to 12-inch drop-down structures in series at a minimum of 50 feet for conveyance control.
- Drop-down structures may consist of aggregate check dams or weirs with weep holes or notches for flow control.
- Swale bottom widths shall range from 2 to 8 feet, larger widths should incorporate berms, walls, or multi-level cross sections to promote uniform conveyance and prevent channel braiding.
- Swale geometry shall be parabolic or trapezoidal with side slopes not steeper than 3:1 (H:V).
- A maximum flow depth of 1.5 feet in the swale for the design volume shall occur at the end of the swale, depth elsewhere should average 1 foot.
- Velocity shall be limited to 2 feet per second.
- Online swales should be designed to pass 25-year, 24-hour design storm with a minimum of 0.5 foot of freeboard.
- Swales shall have a minimum length of 100 feet to effectively treat for water quality however; where obtaining the minimum length is a constraint the requirement may be halved and directed down multiple swales each with a minimum length of 50 feet.

Dry Swale

- Dry swales incorporate a base layer of permeable soil for infiltration of total water quality volume within 24 to 48 hours.
- Permeable soil layer shall have an infiltration rate of 0.5 inches per hour and minimum depth of 2.5 feet.
- An underdrain system may be used with dry swales where the subgrade infiltration rate is below specified criteria An underdrain system placed at the bottom of the dry swale is necessary to facilitate drainage between rainfall events.

Pretreatment

- A pretreatment device (forebay, filter strip, and pea gravel diaphragm) may be necessary for the swale to prevent clogging of filter media where fine particles are present and reduce energy.
- Pretreatment device shall be sized appropriately; pretreatment volume may be counted towards total water quality volume.
- Exit velocities from pretreatment device should be non-erosive (less than 2 feet per second) and distributed laterally if possible to the swale.

Outlet/Bypass

- Outlet protection in the form of riprap or other energy dissipation devices shall be provided at the end of wet swales and by the outfall of the underdrain for dry swales.
- Swales constructed offline shall have a properly sized diversion to permit only the design volume and associated flows.
- Diversion may consist of deflector weir, slotted curb inlet, or similar flow diversion structure.
- Swales constructed online shall be designed to pass flows and volumes exceeding the treatment capacity.

Underdrain System

- The system shall be composed of 4 to 6-inch PVC pipe (AASHTO M252) with 3/8 inch perforations spaced at 6 inch centers with a minimum of 4 holes per row.
- Underdrain system shall be located in gravel layer with a minimum of 2 inches of cover (8 inch minimum layer depth).
- Gravel for backfill should have a porosity of 40 percent, sized 1.5 to 2.5 inches in diameter (AASHTO #1), washed and cleaned.
- Underdrain laterals shall be placed on 10 foot centers and have a minimum of 0.5 percent.
- Gravel layer shall be separated from filter media with permeable geotextile filter fabric.
- Single monitoring/clean out wells shall consist of similar PVC pipes extending vertically out of filter media and located for every 1,000 square feet of surface area..

Vegetation

- Dry swale vegetation shall consist of turf grass and be maintained to heights of 2 to 4 inches.
- Wet swale vegetation can consist of a mixture of wetland vegetation and turf grass
- A water balance should be performed for wet swales incorporating wetland vegetation to ensure proper hydrologic conditions
- All vegetation should be resistant to periods of drought and inundation and stress associated with maximum velocities of 2 feet per second
- Swale vegetation should be planted within 7 days of the grading to prevent the establishment of undesirable vegetation.
- Consult Appendix C-Seeding Rates of SC DHEC's Stormwater Management BMP Handbook for vegetation guidance.

Maintenance Access

- Provide access path to swale for cleaning, additional access considerations should be made for pretreatment, outlet, bypass, and underdrain system .

7.11.3 Maintenance and Monitoring

Monthly

- Clear and remove debris from outlets, diversion, bypass, and pretreatment device.
- Inspect entire facility and surrounding area for erosion and correct immediately.

Seasonal

- Mow side slopes and swale.
- Remove vegetation that is dead, diseased, or invasive.

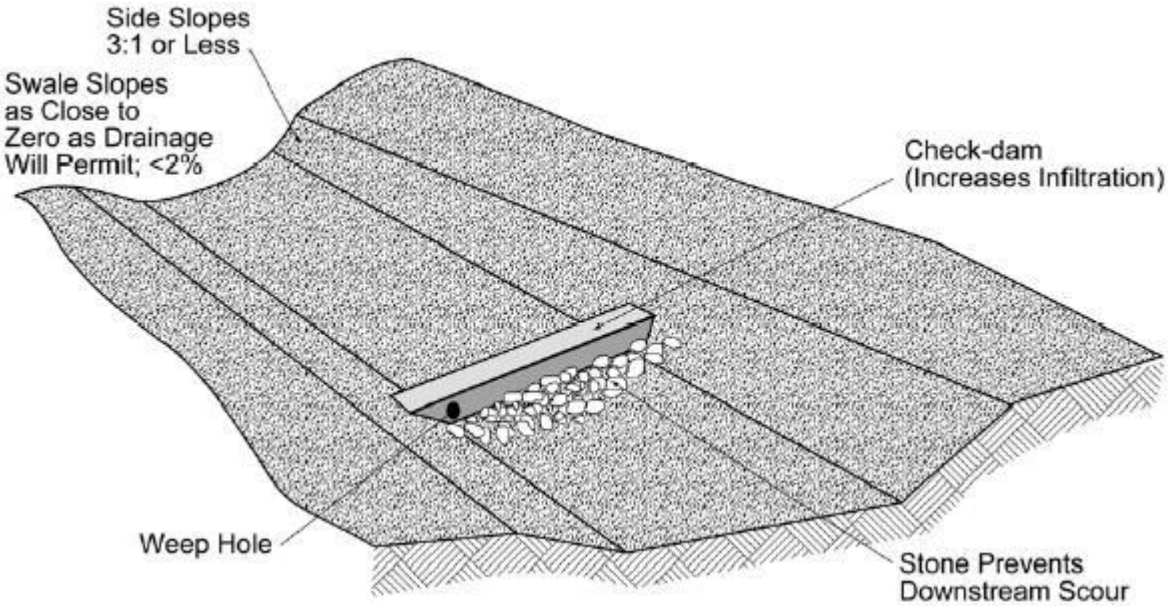
Annual

- Re-grade and re-seed when erosion rills or gullies have formed.

As Needed

- Till permeable soil layer of dry swale if drawdown durations are not being met, replace vegetation
- Monitor sediment accumulation in swale and remove sediment when vegetation before vegetation is fully covered.

7.11.4 General Plan and Profile



Source: Controlling Urban Runoff, Schueler 1987

Figure 7-16 Vegetated Swale General Plan and Profile

7.12 SAND FILTER

7.12.1 Design Components

- Filter
- Pretreatment/Diversion
- Filter Media
- Underdrain System/Outlet
- Vegetation
- Maintenance Access

7.12.2 Design Guidelines

Filter

- Sand filters that discharge directly into receiving waters are not a sufficient BMP for use within 1,000 feet of shellfish beds.
- Filter depth shall be a minimum of 1.5 feet for surface sand filters and 1 to 1.5 feet for perimeter sand filters.
- Ponding depths may range from 1 to 6 feet.
- Filter walls should be constructed with an impermeable structure such as concrete, if an excavation in earth is to be used geotextile filter fabric should be used to line the structure
- Surface area (square feet) for the infiltration trench can be approximated from the following equation:

$$SA = \frac{Vd_{sand}}{kt(d_{sand} + d_{pond})} \tag{7-5}$$

Where:

V	=	water quality volume, in cubic feet
d_{sand}	=	average depth of sand filter, in feet
d_{pond}	=	average depth of ponding above sand filter, in feet, maximum of 6 feet
k	=	coefficient of permeability of filter media, in feet per day, use 3.5 feet per day for sand
t	=	design drain time for filter media, in days, use 1 day

Pretreatment/Diversion

- A pretreatment device (sedimentation chamber or other similar device) is necessary for the sand filter to prevent clogging of filter media and reduce energy
- Sedimentation chamber with detention structures shall be appropriately sized to prevent clogging; this volume may be included for the water quality volume requirement.
- Sedimentation chamber shall have a length to width ratio of at least 2:1.
- Exit velocities from the pretreatment device shall be non-erosive, limited to 2 feet per second, and distributed laterally where possible.
- Surface sand filter types discharge runoff from the sedimentation chamber via a perforated standpipe, perimeter sand filter types discharge runoff from the sedimentation chamber via a weir.
- Surface area (square feet) for the sedimentation chamber can be approximated from the following equation, remember to proportion the water quality volume based on the filter type (surface versus perimeter):

$$SA = 0.066V \tag{7-6}$$

Where:

V = water quality volume (cubic feet) for impervious cover < 75 percent

$$SA = 0.0081V \tag{7-7}$$

Where:

V = water quality volume (cubic feet) for impervious cover ··75 percent

Filter Media

- Shall consist of clean washed medium sand meeting ASTM C-33 concrete sand requirements

Underdrain System/Outlet

- An underdrain system placed at the bottom of the sand filter is necessary to facilitate drainage between rainfall events.
- The system shall be composed of 4 to 6-inch PVC pipe (AASHTO M252) with 3/8 inch perforations spaced at 6-inch centers with a minimum of 4 holes per row.
- Underdrain system should be located in gravel layer with a minimum of 2 inches of cover (8 inch minimum layer depth).
- Gravel for backfill should have a porosity of 40 percent, sized 1.5 to 2.5 inches in diameter (AASHTO #1), washed and cleaned.
- Underdrain laterals shall be placed on 10 foot centers and have a minimum slope of 0.5 percent.
- Gravel layer shall be separated from filter media with permeable geotextile filter fabric.
- Outlet protection in the form of riprap or other energy dissipation devices shall be provided at the end of the outfall for the underdrain system.

Vegetation

- Surface sand filters may incorporate a 3 to 5-inch layer of soil with turfgrass as part of the top layer
- Soil layer shall be separated from filter media with permeable geotextile filter fabric
- Turfgrass should be resistant to periods of drought and inundation
- Consult Appendix D-Seeding Rates of SC DHEC's Stormwater Management BMP Handbook for vegetation guidance.

Maintenance Access

- Provide a minimum 15-foot access path to sand filter, additional access considerations shall be made for pretreatment, outlet, and underdrain system

7.12.3 Maintenance and Monitoring

Monthly

- Clear and remove debris from pretreatment device, diversion, and outlet
- Inspect entire facility and surrounding area for erosion and correct immediately

Seasonal

- Mow side slopes for surface sand filter
- Remove vegetation that is not part of the planned landscape
- Inspect filter after heavy runoff events to ensure acceptable infiltration rates or that leakage or bypassing is not occurring

Annual

- Inspect filter structure, particularly if it is made of impermeable material such as concrete, for cracks, spalling, or other structural defects.

As Needed

- Replace top layer of filter media when infiltration rates begin to decrease.
- Monitor sediment accumulation in sedimentation chamber with fixed vertical sediment marker and remove sediment when 50 percent of the storage volume has been lost.

7.12.4 General Plan and Profile

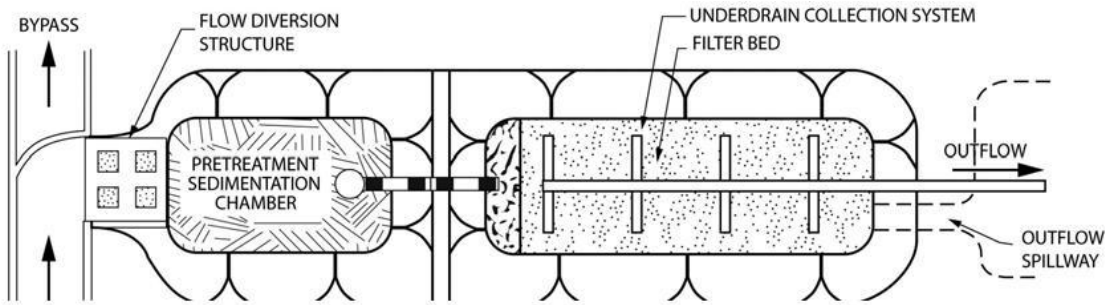


Figure 7-17 Sand Filter General Plan

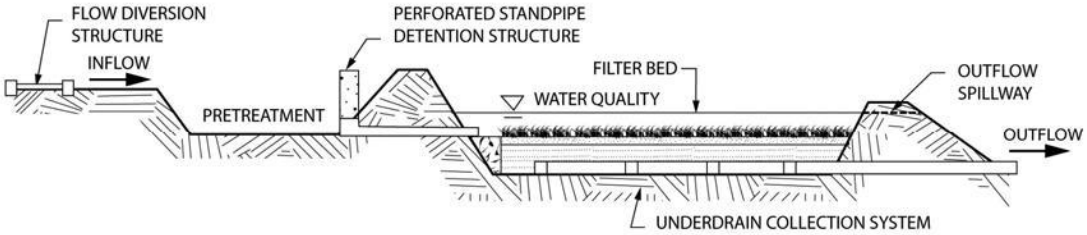


Figure 7-18 Sand Filter Profile

7.13 EXFILTRATION TRENCH

7.13.1 Design Components

- Trench
- Inlets/Sumps
- Filter Fabric
- Perforated Pipe
- Risers/Clean-Outs
- Exfiltration Media

7.13.2 Design Guidelines

Trench

- Trenches within 1,000 feet of shellfish beds shall be sized to infiltrate 1.5 inch of runoff from the drainage area.
- Trenches shall have a maximum width of 10 feet and depths should range from 3-8 feet, trench depths greater than 4 feet require shoring during construction for worker protection.
- Trench bottom should be nearly level, not to exceed 5 percent, to promote even flow distribution and conveyance.
- Trench bottom shall be at least 0.5 feet above the seasonal high water table, which is representative of the maximum height of the water table on an annual basis during years of normal precipitation.
- Underlying soils should have an infiltration rate higher than 0.5 inches per hour, clay content less than 20 percent or silt/clay content less than 40 percent.
- Strict adherence to OSHA's Trench Safety Code or other local regulations relative to acceptable construction practice shall be observed. Depending upon the length and width of trench, either backhoe or wheel or ladder type trencher may be used for excavation. Excavated material should be stored at least 10 feet from the trench to avoid soil compaction and cave-ins.
- Trenches shall have vertical sides in stable soil but may be sloped to 2:1 side slopes in low cohesion soils or sands.
- Minimum trench width is 3 feet.

Inlet/Sumps

- Inlets connected to the trench shall be equipped with sumps.
- Inlets shall be covered during construction to prevent sediment entering the trench.

Filter Fabric

- Permeable geotextile filter fabric shall be used between soil and aggregate/sand layers.
- Line trench bottom and sides within a 0.5 foot of the top with permeable geotextile filter fabric to prevent soil from entering aggregate layer.
- Non-woven geotextiles shall be used for these applications and shall meet AASHTO M288 "Geotextile Specification for Highway Applications, " latest edition.

Perforated Pipe

- Pipes manufactured of plastic, steel, aluminum, concrete, or other materials are available for this application. Perforated metal pipes usually have 3/8-inch diameter perforations uniformly spaced around the full periphery of a pipe. Specifications stipulate not less than 30 perforations per square foot of pipe surface. Other perforations not less than 5/16-inch diameter or slots, are permitted if they provide a total opening area of not less than 3.31 square inches per square foot of pipe surface. A slot opening 3/8-inch wide by 4-inches long running around circumference of pipe may also be used.
- Pipe diameter shall be minimum 12 inches
- Minimum Cover Requirements:
  - Non-traffic installations:
    - up to 48-inch diameter: 12 inches of cover (from top of pipe to top of grade)
    - 60-inch diameter: 24 inches of cover (from top of pipe to top of grade)
  - Traffic installations:
    a.) Flexible Pavement:
      - Up to 36-inch diameter: 12 inches (from top of pipe to bottom of bituminous pavement section)

- 42–60-inch diameter: 24 inches (from top of pipe to bottom of bituminous pavement section)

b.) Rigid Pavement:
- Up to 36-inch diameter: 12 inches (from top of pipe to top of pavement)
- 42–60-inch diameter: 24 inches (from top of pipe to top of pavement)

Risers/Clean-Outs
- A 4- to 6-inch perforated PVC pipe shall be placed vertically extending to the bottom of the trench to monitor exfiltration rates
- Monitoring wells shall be located for every 50 feet of exfiltration trench length
- Provide risers/clean-out ports to the pipe for periodic inspections and cleaning due to siltation.

Exfiltration Media
- Aggregate for backfill should have a porosity of 40 percent, sized 1.5 to 2.5 inches in diameter (AASHTO #1), washed and cleaned.
- Clean, washed stone aggregate shall be placed in the excavated trench in lifts, lightly compacted to form the base. Unwashed stone contains associated sediment posing a clear risk of clogging at the soil/filter cloth interface.
- If an outlet is used, outlet protection in the form of riprap or other energy dissipation devices shall be provided at the end of the outfall for the exfiltration trench.

7.13.3 Maintenance and Monitoring

Monthly
- Clear and remove debris from inlets, sumps, and riser/clean-outs
- Inspect entire facility and surrounding area for erosion and correct immediately
- Inspect after every major storm for the first few months to ensure proper functioning. Drain times should be observed to confirm that designed drain times has been achieved.

Seasonal
- Mow trench area and perimeter.
- Remove vegetation that is not part of the planned landscape.
- Inspect facility for signs of wetness or damage to structures, signs of petroleum hydrocarbon contamination, standing water, trash and debris, sediment accumulation, slope stability, standing water, and material buildup.
- Check for standing water or, if available, check observation wells following 3 days of dry weather to ensure proper drain time.

Annual
- Clean trench when loss of infiltrative capacity is observed. If drawdown time is observed to have increased significantly over the design drawdown time, removal of sediment either manually of by vacuum may be necessary. This is an expensive maintenance activity and the need for it can be minimized through prevention of upstream erosion and incorporation of sumps in inlets.
- Trenches with filter fabric should be inspected for sediment deposits by removing a small section of the top layer. If inspection indicates that the trench is partially or completely clogged, it should be restored to its design condition.

As Needed
- Total rehabilitation of the trench should be conducted to maintain storage capacity within 2/3 of the design water quality volume and 72-hour exfiltration period limit.

- Trench walls should be excavated to expose clean soil.
- All of the stone aggregate and filter fabric or media must be removed. Accumulated sediment should be stripped from the trench bottom. At this point the bottom may be scarified or tilled to help induce infiltration. New fabric and clean stone aggregate should be refilled.

7.13.4 General Profile

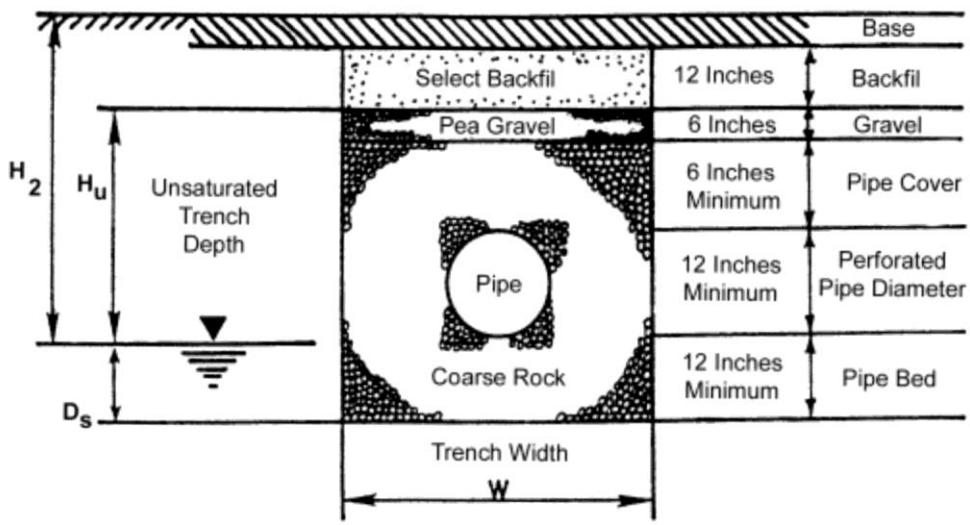


Figure 7-19 Exfiltration Trench Profile

7.14 WET (RETENTION) POND

7.14.1 Design Components

- Sediment Forebay
- Permanent Pool
- Benching/Shelving
- Inlets/Outlets/Spillways
- Embankments/Linear
- Buffer/Setbacks
- Maintenance Access

7.14.2 Design Guidelines

Sediment Forebay

- Forebay should be constructed as a separate cell with a stable barrier (berm) and properly sized outlet between it and the main pool area.
- Size forebay to be appropratly size for anticipated flows form the drainage area, forebay volume may be counted towards total water quality volume.
- Forebay depths should range from 3-6 feet and surface area should be 15 to 25 percent of total wet pond area.
- Exit velocities from forebay should be non-erosive (generally less than 2 feet per second) and distributed laterally if possible to the permanent pool.

Permanent Pool

- Runoff calculations shall be designed for the full-proposed upstream development.
- Permanent pool geometry should have a minimum length-to-width ratio of 3:1 to prohibit short circuiting of flow, mid-pool berms may also be used to promote suitable detention times and flow paths.
- Permanent pool depths shall average from 5 feet to a maximum of 8 feet, deeper sections should be located near the outlet to prevent sediment re-suspension near outlet.
- Permanent pool depths shall not exceed 12 feet.
- Side slopes other than those specified shall not be steeper than 3:1 (H:V).
- Pool bottom should have mild, positive grade for full drainage, not less than 1 percent.
- Additional storage volume may be incorporated into design for flood control.

Benching/Shelving

- Permanent pool construction shall include a two-bench system (safety and aquatic). Each bench (safety and aquatic) shall have a minimum width of 10 feet.
- The safety bench, located above the permanent pool elevation, shall not exceed slopes of 6:1 (H:V) and may be inundated during less frequent design storms
- Aquatic bench, located immediately below permanent pool elevation, shall be permanently inundated with water 1 to 1.5 feet deep and planted with hydrophytic (wetland) vegetation for added pollutant control. Aquatic bench shall be a minimum of 10 feet in width and will be constructed round the entire perimeter of the pond.
- Aquatic vegetation should be planted within 7 days of the aquatic bench construction to prevent the establishment of undesirable vegetation.
- A planting plan should be developed and submitted as part of the stormwater wetland design. Plant selection should be consistent with wetland zones and associated hydrologic depths
- Consult Appendix D-Wetland Vegetation Specifications of SC DHEC's Stormwater Management BMP Handbook.

Inlets/Outlets/Spillways

- Inlets for forebay and permanent pool should be stabilized by riprap or equivalent feature, reduce incoming velocities to non-erosive conditions, and provide lateral distribution where possible.
- If the wet pond incorporates added detention for flood control, post construction discharge rates must be controlled to pre-development conditions.
- If wet ponds are connected to operate as a single interconnected system with the same normal pool elevation, then only the downstream pond is required to have a riser-type structure as the primary outlet.
- Emergency/bypass spillway sized to pass the 100-year, 24-hour design storm.
- Permanent pool should have a bottom drain with a control valve for draining entire pool within a 24-hour period for maintenance purposes where topography allows otherwise a pumping alternative must be addressed.
- Anti-seep collars, filter diaphragms, o-rings gaskets (ASTM C361) should be used on principal barrel outlets to prevent pipe failure.
- Anti-flotation and anti-vortex trash racks should be installed on principal outlets .
- Outlet protection in the form of riprap or other energy dissipation devices shall be provided at the end of the outfall for the permanent pool.

Embankments/Liner

- Limit embankment height to a maximum of 10 feet and locate top of embankment at least 1 foot above 100-year design storm pool elevation
- Embankment side slopes shall not be steeper than 3:1 (H:V)
- Use 0.5 to 1 foot thick low permeable linear (clay) to prevent permanent pool loss, when necessary

Buffer/Setbacks

- Wet pond shall have a minimum of a 20-foot vegetated buffer with a 10 foot setback from buffer for structures, buffer shall be measured from normal pool elevation
- Aquatic bench, located immediately below the permanent pool elevation, shall be permanently inundated with water 1 to 1.5 feet deep and planted with hydrophytic (wetland) vegetation for added pollutant control. Aquatic bench shall be a minimum of 10′ in width and will be constructed round the entire perimeter of the pond.
- A five foot wide vegetated strip shall be provided along the bank of all detention/retention pond systems where such systems are adjacent to public or common ownership areas except behind residential developments. At a minimum 75% of such boundaries shall be vegetated. Vegetation does not include use of turf or other grasses that are normally mowed.

Maintenance Access

- Provide a minimum 15-foot access path to sediment forebay for cleaning, additional access considerations should be made for inlet/outlet/spillway structures
- Access paths should not exceed slopes of 10 percent and need to be stabilized for vehicle traffic

7.14.3 Maintenance and Monitoring

Monthly

- Clear and remove debris from inlets, outlets, and spillways
- Inspect entire facility and surrounding area for erosion and correct immediately

Seasonal

- Mow side slopes and embankment
- Remove vegetation that is dead, diseased, or invasive
- Remove vegetation that is not part of the planned landscape, particularly woody vegetation in the embankment

Annual

- Inspect and test mechanical components and valves for proper operation
- Inspect inlet/outlet/spillway/embankment for stability, potential failure, and seepage

As Needed

- Monitor sediment accumulation in forebay with fixed vertical sediment marker and remove sediment when 50 percent of the storage volume has been lost

7.14.4 General Plan and Profile

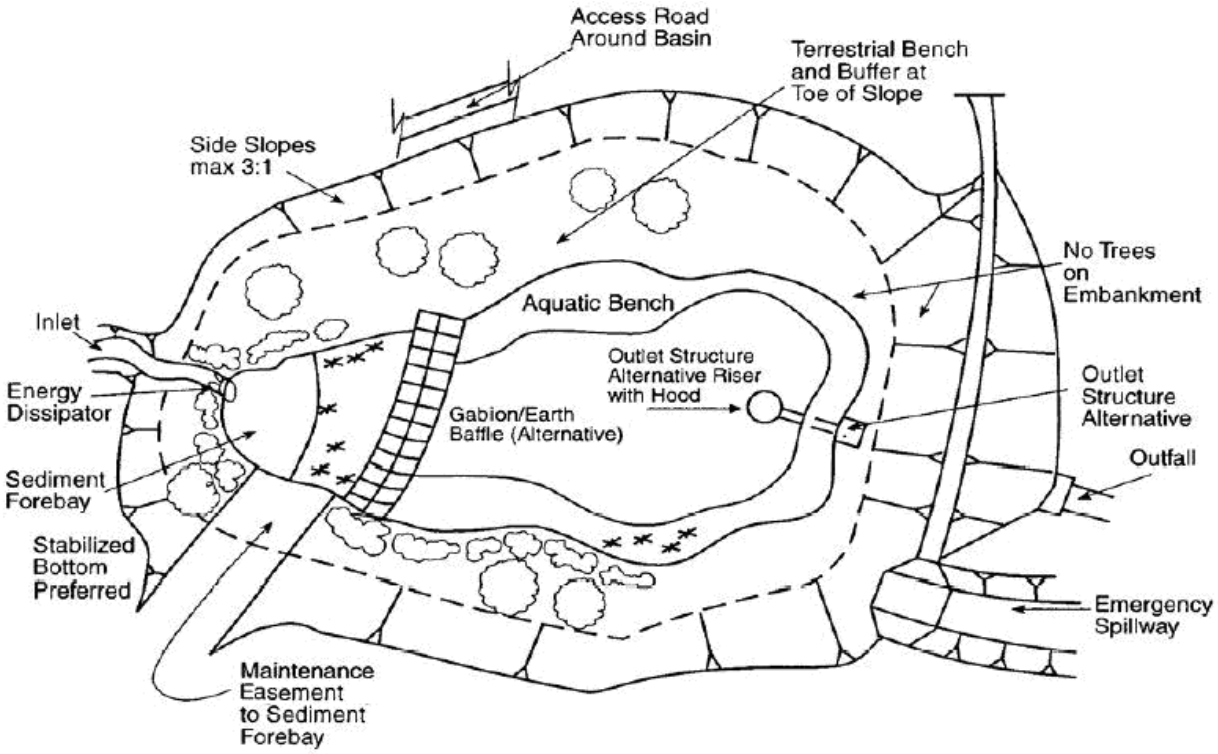


Figure 7-20 Wet Pond General Plan

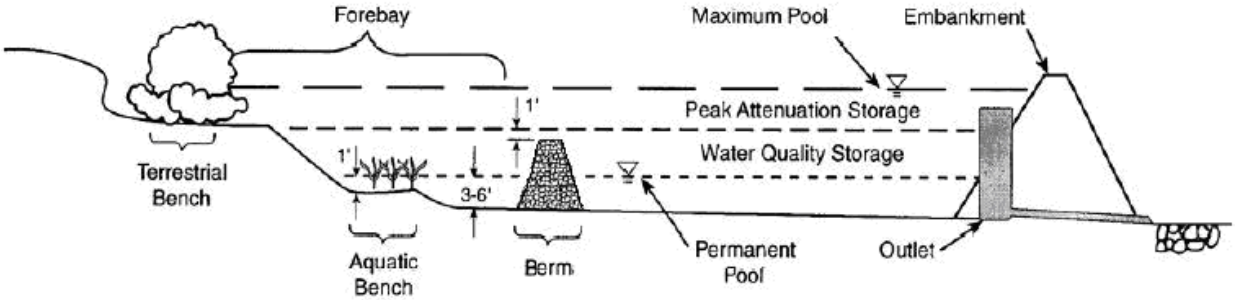


Figure 7-21 Wet Pond Profile

Source: North Carolina Storm water BMP Manual, NC DENR 2005

7.15 EXTENDED DETENTION POND

7.15.1 Design Components

- Sediment Forebay
- Temporary Pool
- Inlets/Outlets/Spillways
- Embankments/Linear
- Buffer/Setbacks
- Maintenance Access

7.15.2 Design Guidelines

Sediment Forebay

- Forebay should be constructed as a separate cell with a stable barrier (berm) and properly sized outlet between it and the main pool area.
- Size forebay to accept anticipated flows from the drainage area, forebay volume may be counted towards total water quality volume.
- Forebay depths should range from 3 to 6 feet and surface area should be 15 to 25 percent of total extended detention pond area.
- Exit velocities from forebay should be non-erosive (generally less than 2 feet per second) and distributed laterally if possible to the temporary pool.

Temporary Pool

- Temporary pool shall be sized to detain the water quality volume from the drainage area.
- Extended detention ponds are not a sufficient BMP for use within 1,000 feet of shellfish beds.
- Temporary pool geometry should have a minimum length-to-width ratio of 1.5:1 to prohibit short circuiting of flow.
- Temporary pool depths shall range from 2 feet to a maximum of 8 feet, deeper sections should be located near the outlet to prevent sediment re-suspension near outlet.
- Side slopes should not be steeper than 3:1 (H:V).
- Low flow channel shall be constructed and stabilized with vegetation to facilitate drainage through the temporary pool during more frequent storms.
- Temporary pool bottom should be graded to the slope not less than 1 percent to fully drain to low flow channel or outlet.
- Additional storage volume may be incorporated into design for flood control.

Inlets/Outlets/Spillways

- Inlets for forebay and permanent pool should be stabilized by riprap or equivalent feature, reduce incoming velocities to non-erosive conditions, and provide lateral distribution where possible.
- If the extended detention pond incorporates added detention for flood control, post construction discharge rates must be controlled to pre-development conditions.
- Emergency/bypass spillway shall be sized to safely pass the 100-year, 24-hour design storm.
- Temporary should have a bottom drain with a control valve for draining entire pool within a 24-hour period for maintenance purposes where topography allows otherwise a pumping alternative must be addressed.
- Anti-seep collars, filter diaphragms, o-rings gaskets (ASTM C361) should be used on principal barrel outlets to prevent pipe failure.
- Anti-flotation and anti-vortex trash racks should be installed on principal outlets.

- Riprap outlet protection for the temporary pool shall be provided at the end of the outfall.
- In areas where A and B type soils exist, the lowest level of discharge from a control structure must be equal to or higher than the existing seasonal ground water level.

Embankments

- Limit embankment height to a maximum of 10 feet and locate top of embankment at least 1 foot above 100-year design storm pool elevation
- Embankment side slopes shall not be steeper than 3:1 (H:V)

Vegetation

- Aquatic vegetation should be planted within 7 days of the aquatic bench construction to prevent the establishment of undesirable vegetation.
- Consult Appendix B-Seeding Rates of SC DHEC's Stormwater Management BMP Handbook.

Buffer/Setbacks

- Extended detention pond shall have a minimum of a 20-foot vegetated buffer with a 10 foot setback from buffer for structures, buffer shall be measured from normal pool elevation
- A 5-foot wide vegetated strip shall be provided along the bank of all detention/retention pond systems where such systems are adjacent to public or common ownership areas except behind residential developments. At a minimum 75% of such boundaries shall be vegetated. Vegetation does not include use of turf or other grasses that are normally mowed.

Maintenance Access

- Provide a minimum 15-foot access path to sediment forebay and temporary pool for cleaning, additional access considerations should be made for inlet/outlet/spillway structures
- Access paths should not exceed slopes of 10 percent and need to be stabilized for vehicle traffic

7.15.3 Maintenance and Monitoring

Monthly

- Clear and remove debris from inlets, outlets, and spillways
- Inspect entire facility and surrounding area for erosion and correct immediately

Seasonal

- Mow side slopes and embankment
- Remove vegetation that is dead, diseased, or invasive
- Remove vegetation that is not part of the planned landscape, particularly woody vegetation in the embankment

Annual

- Inspect and test mechanical components and valves for proper operation
- Inspect inlet/outlet/spillway/embankment for stability, potential failure, and seepage

As Needed

- Monitor sediment accumulation in forebay with fixed vertical sediment marker and remove sediment when 50 percent of the storage volume has been lost
- Remove sediment accumulation in temporary pool when 25 percent of the volume has been lost

7.15.4 General Plan and Profile

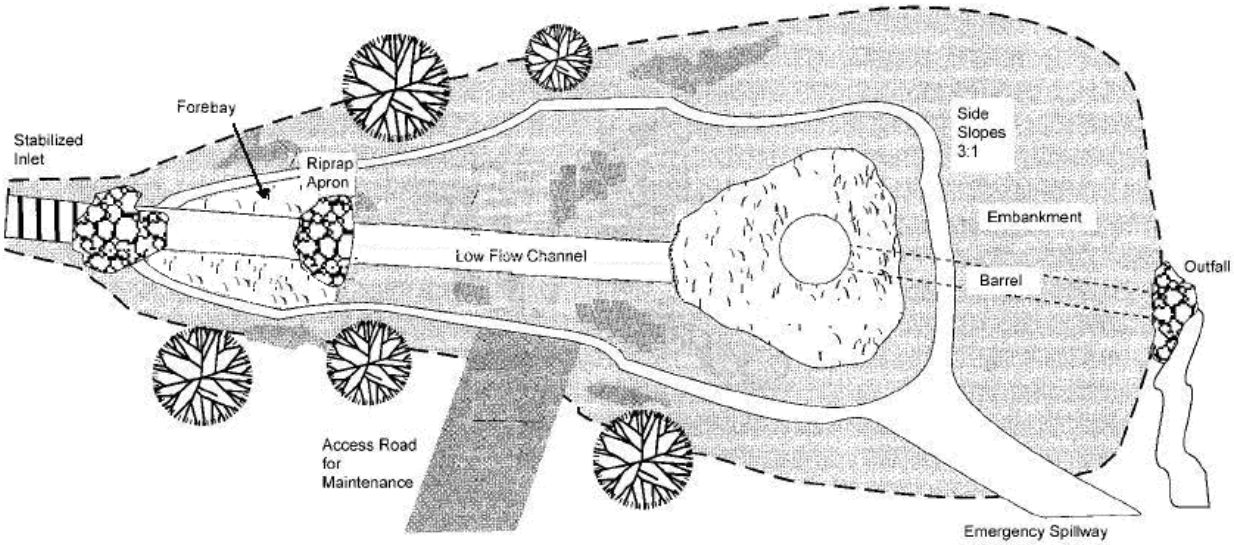
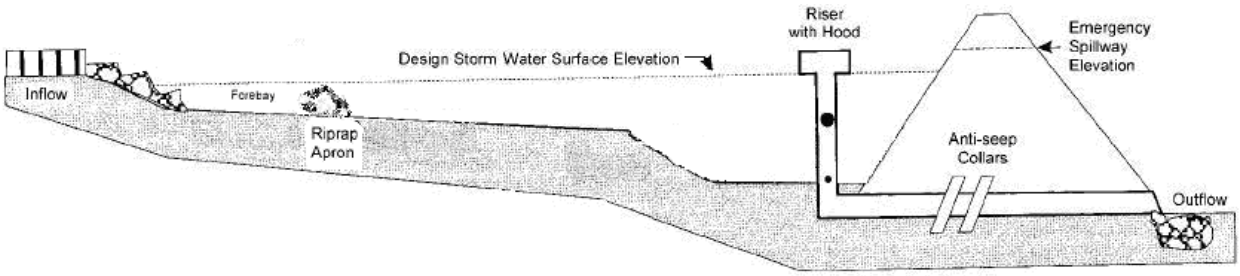


Figure 7-22 Extended Detention Pond General Plan



Source: Controlling Urban Runoff, Schueler 1992

Figure 7-23 Extended Detention Pond Profile

8.0 WATER QUALITY ANALYSIS

Uncontrolled stormwater may have significant, adverse impact on the health, safety and general welfare of the Town and the quality of life of its citizens by transporting pollutants into receiving waters and by causing erosion and/or flooding. Development and redevelopment may alter the hydrologic response of local watersheds and increases stormwater rates and volumes, flooding, soil erosion, stream channel erosion, non-point pollution, and sediment transport and deposition, as well as reducing groundwater recharge. These changes in stormwater may contribute to increased quantities of water-borne pollutants and alterations in hydrology which are harmful to public health, safety, and welfare, as well as to the natural environment.

8.1 GENERAL DESIGN CRITERIA

The design criteria required for all new development to control and reduce water quality degradation within Town will be addressed at two levels. Pursuant to Chapter 1 of this Manual and the Ordinance, the following levels of analysis and protection shall be adhered to for new development:

Method 1: For parcels less than 20 acres

Method 2: For parcels equal to or greater than 20 acres

Redevelopment or expansion of existing development shall follow the same requirements as outlined in this chapter. Water quality analysis is required as determined in the Ordinance, and design requirements are adhered to, as stated in the same.

<u>Method 1 – Water Quality Control</u>

Developments shall install and maintain structural BMPs approved by Town to achieve targeted pollutant removal efficiencies. The targeted pollutant for design analysis and BMP selection only shall be phosphorus. Engineering calculations shall be submitted to evaluate whether or not a proposed BMP plan for a development project will meet the recommended anti-degradation water quality goal. Analysis and evaluation of predicted performance will utilize the expected phosphorus removal efficiencies identified in this chapter. BMPs approved for design and accepted for operation upon construction under this methodology will be deemed to satisfy targeted pollutant removal efficiencies.

This methodology is established to address all potential pollutants (i.e. suspended solids, nitrogen, etc.), and does not attempt to imply that only phosphorus is being removed or that phosphorus is the only pollutant of concern. Phosphorus removal (as well as other pollutant removal) occurs as a result of its attachment to solids. Therefore by focusing on phosphorus, the BMP will indirectly target other pollutants, such as sediment. This methodology has also been chosen to streamline the analysis effort and to be consistent with other stormwater programs with similar water quality concerns.

<u>Method 2 – Water Quality Control</u>

Developments shall install and maintain structural BMPs approved by Town to achieve zero degradation as compared to predevelopment pollutant loads. The permit applicant shall submit a water quality modeling plan to the Town for approval prior to submitting a stormwater management plan. The modeling plan submittal shall include an explanation of the analysis approach, identification of pollutants or indicators and relationships thereof, description of model methodology, expected range of accuracy in result prediction, and sources of all data to be used for modeling.

8.2 POLLUTANT TYPES AND SOURCES

A pollutant is a man-made or naturally occurring constituent that creates an undesirable effect when introduced to a specific environment. Elements such as nutrients, sediment, organic matter, organic compounds, and metals are naturally occurring constituents do not create adverse affects when introduced to an aquatic system in balanced proportions. In fact, many of these constituents are essential for the propagation of aquatic life. However, the introduction of excessive, unbalanced quantities can create an undesirable effect and result in their acting as pollutants. Constituents that provide no beneficial use in an aquatic system are also termed pollutants. Water quality control is the balancing of required constituent masses with the elimination of pollutants to provide a desirable aquatic system. Typical pollutants found in stormwater include but are not limited to the following:

- Sediment (suspended and dissolved) from erosion, exposed ground, and construction activity.
- Nutrients (nitrogen, phosphorus) from fertilizers and other chemicals in the urban environment, including atmospheric deposition.
- Oxygen demanding matter (BOD) as a result of decomposition of organic matter in stormwater.
- Heavy metals (iron, lead, manganese, etc.) from vehicular waste and other transportation related activities.
- Bacteria and other pathogens from domestic pets, birds, and leaking sanitary sewers.
- Oil and grease from illegal dumping, or poorly maintained vehicles.
- Household hazardous waste (insecticide, pesticide, solvents, paints, etc.)
- Polycyclic Aromatic Hydrocarbons (PAH), as a result of outboard exhaust and petroleum and tire residues on paving.

Human activities, or the results of human activities, are the principal source of pollutants that affect water quality. These sources are classified as either point or non-point. Point sources are easily identified as they are usually pipes from sewage treatment plants, stormwater systems, or industries. Non-point sources are far more difficult to assess quantitatively and to control. Non-point pollution sources represent a significant source of pollutants. These sources include soil erosion and land disturbances, animal waste, agri-chemicals, waste from automotive use and maintenance, onsite wastewater systems, resource extraction, atmospheric deposition, and more.

Stormwater, and non-point source pollution in particular, has the potential to significantly impact surface water quality. Stormwater generated from impervious surfaces or saturated soil conditions collects and transports pollutants from the terrestrial landscape to the surface water. When considering surface water impacts, it is helpful to understand the phenomenon called first flush. Studies have shown that the portion of stormwater captured during the first fifteen minutes of a storm event contains high concentrations of pollutants. This is commonly referred to as the first flush. Thus, the pollutant accumulation in intense small runoff events can be more detrimental to water quality than larger flooding events. As a result, upstream management practices that control small volumes of initial runoff can be very effective in enhancing non-point source pollutant removal.

8.3 WATER QUALITY IMPACT

Impacts associated with the discharge of stormwater into surface water depend upon the type of pollutants present. In general, organic compounds lead to a decrease in dissolved oxygen and potentially to loss of aquatic life while suspended solids lead to sedimentation in the water body and poor water quality. Nutrients, such as phosphorous and nitrogen, lead to the eutrophication of surface water, algal blooms, and eventually poor water clarity. Aquatic organisms amass metals such as lead, cadmium, zinc, and other toxic chemicals by the process of bioaccumulation. In fish this process results in concentrations of toxic substances that are far greater than the concentrations present in the water the fish inhabit. This, in turn, may further result in

food chain biomagnification causing increased concentrations of substances as one food chain level is consumed by the next. Trace metals and organic contaminants can result in acute or chronic toxic effects if concentrations become extreme. Pathogens result in surface waters that are hazardous for human recreation, consumption, and fish harvesting.

8.4 POLLUTANT LOADING ANALYSIS

The Town, in order to quantify the impact of stormwater pollution for developments, requires a computation of the expected removal efficiency of the targeted pollutants. Pollutant exports are the quantity of a pollutant, typically expressed as mass of pollutant per year, expected to be generated by a specific land use and transported by runoff off the site. All sites, natural or disturbed, produce some composition and quantity of pollutants.

For purposes of pollutant export control, the Town requires that the excess pollutant quantities be controlled to the maximum extent practicable on all new development and redevelopment, except as defined in the Ordinance. The best way to control pollutant export is by preventing the generation of pollutants in the first place, however, complete prevention of pollutant generation is unrealistic and impractical. Therefore, pollution export must be controlled to the greatest extent possible through the use of BMPs. Some BMPs prevent the interaction of stormwater with the pollutant thus preventing it from leaving the site while most perform some type of control or treatment of the pollutant through an engineered structure. These BMPs are termed structural BMPs and are discussed further in Chapter 7.

8.4.1 Pollutant Removal Analysis

The BMP fact sheets in Chapter 7 provide estimated removal efficiencies of BMPs for certain pollutants. These efficiencies were derived from a range of literature citations and documented laboratory field experiments and represent an anticipated level of removal under normal conditions. BMP efficiency is highly variable and dependant on many environmental and physical factors.

Method 1 – Water Quality Control

For the purpose of BMP selection, evaluation, and approval, the following table lists the assumed total phosphorous removal for approved BMP facilities:

TABLE 8-1
ASSUMED BMP PHOSPHORUS REMOVAL EFFICIENY

BMP	EFFICIENY (%)
Wet (Retention) Pond	50
Extended Detention Pond	30
Constructed Wetland System	40
Infiltration Trench	60
Bioretention Area	60
Filter Strip	20
Vegetated Swale	35
Sand Filter	45
Permeable Pavement System	40
Exfiltration Trench	50

A worksheet in the following section provides a step-by-step process for assessing the pollutant removal efficiency of the planned BMPs, and comparing that with the pollutant removal required by the Town. Using the worksheet, the required removal efficiency (based on imperviousness of the developed portion of the site) is compared to the assumed removal efficiency provided by a series of BMPs inline. The developed area would include roof tops, buildings, sidewalks, patios, paths, streets, and parking areas but would exclude managed open space areas such as lawns, gardens, parks, recreation facilities, wetlands, and ponds. If the BMP series is not sufficient, then suggested alternatives include adding a third BMPs in series, using BMPs with higher removal efficiency, or changing open space or impervious area to reduce the targeted removal efficiency.

Method 2 – Water Quality Control

The larger the development, the greater the risk of detrimental impact to receiving waters due to improper functioning of BMPs. Town requires specific detailed modeling to assess the pre and post development pollutant loading from the proposed development in order to minimize that risk. Model results for large size developments will show through the use of BMPs in multiple locations and/or in series (i.e. treatment train) such that the post development pollutant discharge is equal to or less than the predevelopment pollutant discharge. The permit applicant shall submit a water quality modeling plan to the Town for approval prior to submittal of the stormwater management plan. The modeling plan submittal shall identify the analysis approach, pollutants identified for analysis, model methodology, expected range of accuracy in pollutant removal prediction, and source of data to be used for modeling.

8.4.2 Method 1 – Worksheet

Step 1 - Calculation of Percentage of Impervious Cover for Developed Area

Total Project Site Area (acres)
Includes developed area and area left undisturbed during construction _____ A_{site}

Impervious Cover within Disturbed Area (acres)
Includes developed areas that will be covered with an impermeable surfaces such as roof tops, buildings, sidewalks, patios, paths, streets, parking areas, etc. Compacted gravel surfaces for this calculation are considered impermeable. _____ A_{imp}

Pervious Cover within Disturbed Area (acres)
Includes developed areas that will be stabilized and covered with a natural infiltrative surface such as lawns, gardens, parks, recreation facilities, wetlands, ponds, etc. _____ A_{per}

Open Space Preservation (acres)
Includes areas left undisturbed during construction and intended to remain preserved in this manner. _____ A_{open}

Percentage of Impervious Cover for Developed Area (%)
$A_{imp} / (A_{imp} + A_{per}) \times 100$ _____ I

Step 2 – Determination of Phosphorus Removal Percentage

Required Removal Percentage (%)
Select removal percentage base on percentage of impervious cover (*I*) from list below: _____ R

I	%	*I*	%
0-10	0	51-60	52
11-20	27	61-70	54
21-30	42	71-80	58
31-40	46	81-90	63
41-50	50	91-100	68

Removal Percentage Correction for Open Space Preservation (acres)
$100 - (100 - R) / [(A_{imp} + A_{per}) / A_{site}]$ Use zero for $R_{reqr} < 0$ _____ R_{req}

Step 3 – BMP Series Removal Efficiency Calculation

First BMP Removal Efficiency Percentage (%)
Select an assumed BMP removal percentage from Table 8-1.
_____ E_1

Second BMP Removal Efficiency Percentage (%)
Select an assumed BMP removal percentage from Table 8-1. _____ E_2

Percentage of Developed Area Treated (%)
Indicate the percentage of area to be served by this BMP system. _____ A_{ser}

Calculated BMP Removal Efficiency (%)

$$[(E_1 / 100) + (E_2 \times (100 - E_1)) / 100^2] \times A_{ser}$$

_____ E_{cal}

Use integers for percentages

If $E_{cal} < R_{req}$, then proposed BMP configuration is not meeting required phosphorus removal standards for the site. To meet standards consider the following suggestions and verify through recalculation.

- Increase open space preservation planned for the project site
- Decrease impervious areas planned for the project site
- Use BMPs with higher efficiencies for removal

8.4.3 Method 2 – Example

A developer is building an apartment complex on 12 acres of land that will result in parking lots, sidewalks, and buildings compromising 6.7 acres of disturbed area. The balance of the disturbed area will be managed lawns and landscaping. The developer is required to preserve the 35-foot buffer around the site which has a perimeter of 2,100 feet. The developer is planning on using a single stormwater wetland followed by a vegetated swale to treat the entire post construction runoff from the developed area. Work through the worksheet to determine if a stormwater wetland-vegetated swale series is sufficient in meeting water quality control.

Step 1 - Calculation of Percentage of Impervious Cover for Developed Area

Total Project Site Area (acres) Includes developed area and area left undisturbed during construction that drains to the BMP series.	12	A_{site}
Impervious Cover within Disturbed Area (acres) Includes developed areas that will be covered with an impermeable surfaces such as roof tops, buildings, sidewalks, patios, paths, streets, parking areas, etc. Compacted gravel surfaces for this calculation are considered impermeable.	6.7	A_{imp}
Pervious Cover within Disturbed Area (acres) Includes developed areas that will be stabilized and covered with a natural infiltrative surface such as lawns, gardens, parks, recreation facilities, wetlands, ponds, etc.	3.6	A_{per}
Open Space Preservation (acres) Includes areas left undisturbed during construction and intended to remain preserved in this manner.	1.7	A_{open}
Percentage of Impervious Cover for Developed Area (%) $A_{imp} / (A_{imp} + A_{per}) \times 100$	65	I

For Step 1, the total impervious cover for the disturbed area is calculated. The problem statement provided the total project site acreage (12 acres) and acreage (6.7 acres) to be covered by impermeable surfaces. It can also be derived from the problem statement that the open space preservation will be 1.7 acres by multiplying the buffer width by the site perimeter. Subtracting the open space acreage from the total project site acreage yields the disturbed or developed acreage (10.3 acres). Approximately, 6.7 acres of the total disturbed acreage (10.3 acres) comprises impervious cover; the other 3.6 acres (10.3-6.7) is left as pervious cover.

Step 2 – Determination of Phosphorus Removal Percentage

Required Removal Percentage (%)

Select removal percentage base on percentage of impervious cover (*I*) from list below: **54** *R*

I	%	*I*	%
0-10	0	51-60	52
11-20	27	61-70	54
21-30	42	71-80	58
31-40	46	81-90	63
41-50	50	91-100	68

Removal Percentage Correction for Open Space Preservation (acres)

$100 - (100 - R) / [(A_{imp} + A_{per}) / A_{site}]$ Use zero for $R_{req} < 0$ **46.4** R_{req}

Using the 65 percent impervious cover for the disturbed area from Step 1, 54 percent is the corresponding BMP removal efficiency for phosphorus. This BMP removal efficiency is reduced as a credit for preserving open space by virtue of the buffer requirements. The new targeted BMP removal efficiency is corrected to be 46.4 percent

Step 3 – BMP Series Removal Efficiency Calculation

First BMP Removal Efficiency Percentage (%)

Select an assumed BMP removal percentage from Table 8-1. **40** E_1

Second BMP Removal Efficiency Percentage (%)

Select an assumed BMP removal percentage from Table 8-1. **25** E_2

Percentage of Developed Area Treated (%)

Indicate the percentage of area to be served by this BMP system. **100** A_{ser}

Calculated BMP Removal Efficiency (%)

$[(E_1 / 100) + (E_2 \times (100 - E_1)) / 100^2] \times A_{ser}$ **55** E_{cal}

Use integers for percentages

If $E_{cal} < R_{req}$ then proposed BMP configuration is not meeting required phosphorus removal standards for the site. To meet standards consider the following suggestions and verify through recalculation.

- Increase open space preservation planned for the project site
- Decrease impervious areas planned for the project site
- Use BMPs with higher efficiencies for removal

In Step 3, the developer's plan for stormwater quality control is evaluated against the expected removal derived in Step 2. The developer is planning on using a stormwater wetland in series with a vegetated swale to treat all runoff from the disturbed area. From Table 8-1 for a constructed wetland system and vegetated swale, BMP efficiencies of 40 and 25 percent respectively, are assumed. Efficiency 1 for the stormwater wetland since it receives the stormwater initially and Efficiency 2 for the vegetated swale since it receives the discharge from the stormwater wetland. As previously mentioned, the entire disturbed acreage (100 percent) is being conveyed for treatment to the BMPs. The calculated BMP series removal efficiency for this configuration is 55 percent, which meets the targeted removal efficiency of 46.4 percent.

If the targeted removal efficiency had not been met, then developer could utilize a number of options to meet the targeted BMP removal efficiency. Some suggestions include using a more efficient BMP that still would be able to handle the water quality volumes from an approximately 10 acre disturbed site or an additional BMP in series. Other alternatives would include: reducing the impervious cover by decreasing excessive parking stalls or roadways widths or using permeable pavement systems in order to reduce the targeted BMP removal efficiency, or increasing the buffer widths to receive more credit towards the targeted BMP removal efficiency.

9.0 CONSTRUCTION SITE EROSION AND SEDIMENTATION CONTROL

Sedimentation involves three basic geologic processes: erosion, transportation, and deposition. These are natural geologic phenomena, however, land development activities may initiate severe, highly undesirable and damaging alterations in the natural sedimentation cycle by drastically accelerating the erosion and transportation process. Receiving waters are the final destination for sediment transport and deposition. However, natural streams and lakes are not capable of handling the excessive sediments created by this accelerated cycle. Therefore, excessive sediment loads result in turbid waters and heavy deposition over the substrate. The impact of these events directly affects the propagation of aquatic life which relies on clear substrates and water to feed and reproduce. Sediment laden waters affect human activities through the degradation of waters used for aquatic recreation and sport fishing and complicate water treatment processes. Consequently, minimizing the occurrence of erosion and effective control of sediment transport is imperative to all.

9.0.1 General Criteria

All construction site activities must adhere the SCDHEC General Permit SC0010000 for Large and Small Site Construction Activities. In addition, the Town will require as a minimum, implementation of the following Construction Site BMPs:

Single Family Development, not part of a larger common plan of development:

1. Silt Fencing buried a minimum of 6 inches below disturbed grade, where applicable,
2. In areas where more than two feet of fill material has been placed or in areas adjacent to all wetlands, silt fencing meeting the requirements of SCDOT must be used.
3. Temporary gravel driveways a minimum of 15 feet by 10 feet, where applicable,
4. Sediment barriers surrounding all catch basins or drop inlets on site and sediment socks on all catch basins or drop inlets adjoining to the site

Single Family and Multi-Family Development, part of a larger common plan of development, and Non-residential Development:

1. Silt Fencing buried a minimum of 6 inches below disturbed grade
2. Temporary gravel driveways a minimum of 15 feet by 10 feet
3. Sediment barriers surrounding all catch basins or drop inlets on site and sediment socks on all catch basins or drop inlets adjoining to the site.
4. Flow dissipation devices, such as check dams, in all swales and ditches
5. Temporary seeding shall be placed within 7 days of the end of a land disturbance activity
6. Floating pump suctions for all temporary or permanent ponds or pumping of excavations
7. Discharge velocities shall be reduced to provide non-erosive flows from dewatering for all temporary or permanent ponds or pumping of excavations
8. Continuous nephelometric turbidity monitoring during qualifying events on all receiving water(s) and stormwater outfall discharge(s)
9. No more than 25 nephelometric turbidity units (NTU) difference between upstream and downstream monitoring sites for surface water(s) receiving stormwater discharge(s). Stormwater discharge(s) not directly received by a surface water shall have a value of no more than 25 NTUs.
10. Site inspections must be performed by a CEPSCI individual. Copies of inspection reports shall be provided to the Town within 7 days of inspection
11. Temporary stockpile areas and appropriate BMPs to be identified on plans
12. Two rows of silt fence are required between land disturbing activities and adjacent wetlands.

9.0.2 Sedimentation Cycle

Soil erosion is usually caused by the impact force of raindrops and by the sheer stress of runoff flowing in rills and streams. Raindrops falling on bare or sparsely vegetated soil detach soil particles; runoff, in the form of sheet flow along the ground, picks up and carries these particles to surface waters. As the runoff gains velocity and concentration, it detaches more soil particles, cuts deeper rills and gullies into the surface of the soil, and adds to its own sediment load. Coalescing rivulets produce streams which have a larger volume and usually an increased velocity. These increasing streams have a greater capacity to remove sediment and transport it downstream. The further the runoff runs uncontrolled, the greater its erosive force and the greater the resulting damage. As the distance and volume of uncontrolled flow increase the control becomes increasingly difficult. At some point, the energy in the stream dissipates to level that can no longer support the transport of the sediment. At this time, the sediment falls out of the water column and deposits. Over time the sediment will either be incorporated into the substrate or be re-suspended for further transport.

9.0.3 Factors Influencing Erosion

The erosion potential of a site is principally determined by the soil type, vegetative cover, topography, climate, and season. These factors contribute to the detachment of soil particles and their transport off-site.

- Soil Type – Erodibility, the amount of energy needed to break down soil structure, is dependant on soil composition and texture. Soils with high erodibility require less energy to detach soil particles.
- Vegetative Cover – Vegetation shields soils from the impact energy of raindrops and traps suspended sediment from runoff.
- Topography – Steeper and longer slopes generate runoff with more velocity and energy to erode and transport more sediment.
- Climate – Rainfall frequency and intensity cumulatively contribute energy in the form of raindrop impact and runoff volume to detach and transport soil particles.
- Season – Seasonal variations in wind, temperature, humidity, and rainfall may create more ideal conditions for erosion.

9.0.4 Concepts of Erosion and Sedimentation Control

Principles of erosion and sedimentation control are based on minimizing the effects of the soil and climatologic factors just discussed. None of the following concepts provide a singular solution for controlling those factors, nor can they all be performed at every site. However, the integration of as many concepts as possible provides the most effective erosion and sedimentation control:

A. Compatible Site Planning

- Minimize development within sensitive areas (e.g. highly erosive soils).
- Limit the length and steepness of the designed slopes.
- Maintain natural vegetative cover when possible.

B. Disturbed Areas Reduction

- Minimize the extent of the disturbed area and the duration of exposure.
- Phase or stage development so that only the areas that are actively being developed are disturbed.
- Minimize large or critical area grading during the season of maximum erosion potential.

C. Disturbed Areas Protection

- Complete grading as quickly as possible.
- Establish permanent vegetation as soon as possible on disturbed areas.
- Divert runoff from disturbed areas.

D. Sediment Retention within Site Boundaries

- Filter runoff as it flows from a disturbed area
- Impound sediment-laden runoff temporarily so that the soil particles are deposited onsite.

The NPDES Phase II storm water regulations enacted by the Clean Water Act of 1972 and promulgated by Stormwater Phase II Final Rule (1999) require that any activity disturbing an acre or greater of land, or a smaller project part of a larger common plan for development or sale, obtain NPDES construction permit coverage. This regulation differs somewhat from the South Carolina state regulations relating to areas of disturbance. Any land disturbing activity in the Town that meets the aforementioned criteria of one acre or more of disturbance will need to will comply with the state process for permitting. Application and issuance of an approved permit under the South Carolina state regulations for erosion and sedimentation control will meet the requirements for coverage under NPDES Phase II as well.