Knox County, Tennessee
Stormwater Management Manual

VOLUME 2
TECHNICAL GUIDANCE

Prepared for:

KNOX COUNTY, TENNESSEE
DEPARTMENT OF ENGINEERING AND PUBLIC WORKS
County Mayor: Mike Ragsdale
Director of Engineering and Public Works: Bruce Wuethrich
Stormwater Management Coordinator: Chris Granju P.E.

Prepared by:

amec

AMEC EARTH & ENVIRONMENTAL, INC.
10239 Technology Drive
Knoxville, Tennessee 37932

JANUARY 2008

Chapter 4 Design and Maintenance of Structural BMPs

Chapter 5 Stormwater Credits and Better Site Design Practices

Chapter 9

Construction Site Stormwater Management

Chapter 10

Pollution Prevention After Construction

Appendices

This page left intentionally blank

Table of Contents

CHAPTER 1 — ADDRESSING STORMWATER MANAGEMENT NEEDS

Chapter 1 – List of Figures

Chapter 1 – List of Tables

CHAPTER

1

ADDRESSING STORMWATER MANAGEMENT NEEDS

1.1 Impacts of Development and Stormwater Runoff

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

1.1.1 Development Changes Land and Runoff

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

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

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

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

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

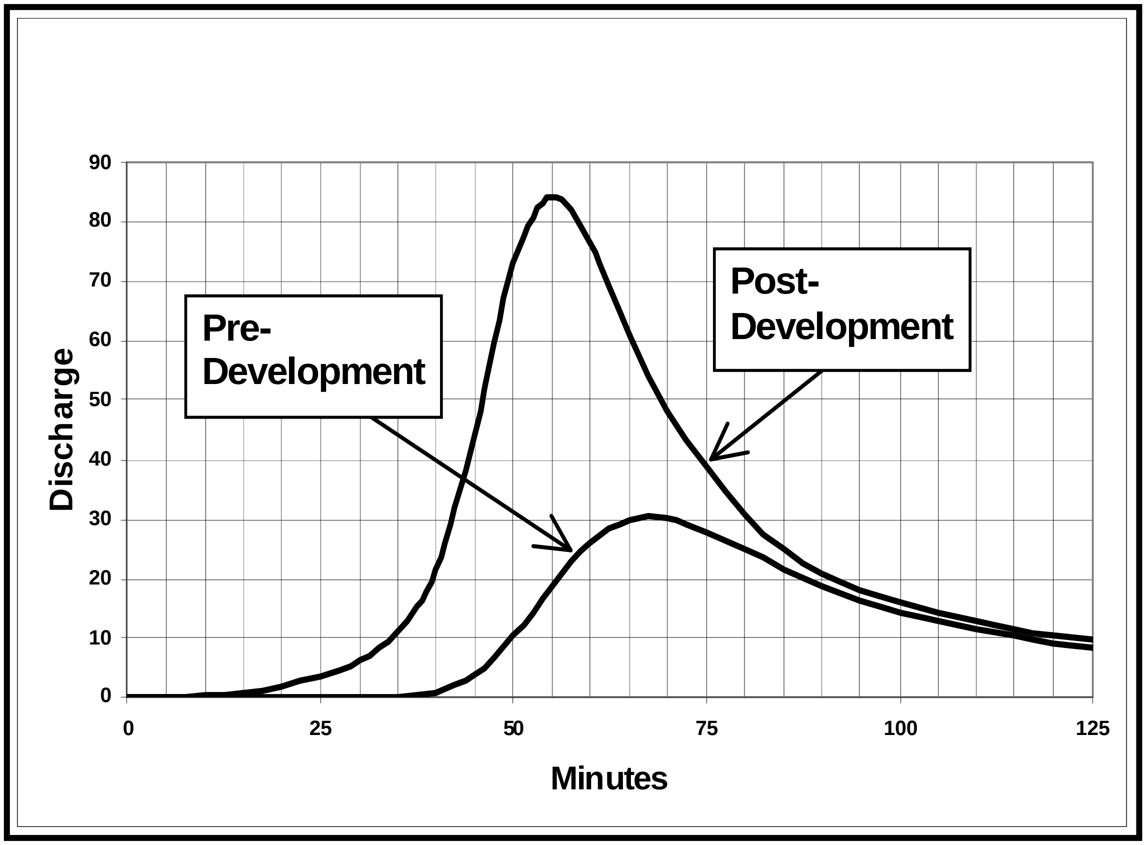
- Changes to stream flow;
- Changes to stream geometry;
- Degradation of aquatic habitat; and,
- Water quality impacts.

1.1.2 Changes to Stream Flow

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

- Increased Runoff Volumes – Land surface changes can dramatically increase the total volume of runoff generated in a developed watershed.
- Increased Peak Runoff Discharges – Increased peak discharges for a developed watershed can be two to five times higher than those for a watershed prior to development. This is depicted in Figure 1-1.
- Greater Runoff Velocities – Impervious surfaces and compacted soils, as well as improvements to the drainage system such as storm drains, pipes and ditches, increase the speed at which rainfall runs off land surfaces within a watershed.
- Timing – As runoff velocities increase, it takes less time for water to run off the land and reach a stream or other waterbody.
- Increased Frequency of Bankfull and Near Bankfull Events – Increased runoff volumes and peak flows increase the frequency and duration of smaller bankfull and near bankfull events which are the primary channel forming events.
- Increased Flooding – Increased runoff volumes and peaks also increase the frequency, duration and severity of out-of-bank flooding.
- Lower Dry Weather Flows (Baseflow) – Reduced infiltration of stormwater runoff causes streams to have less baseflow during dry weather periods and reduces the amount of rainfall recharging groundwater aquifers.

Figure 1-1. Runoff Hydrograph under Pre- and Post-Development Conditions

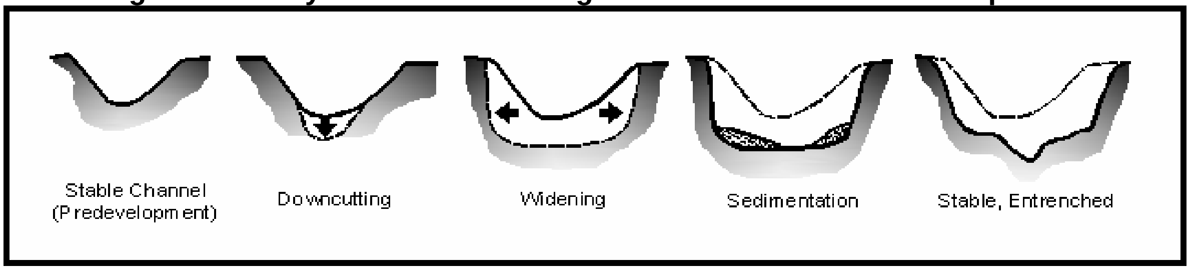


1.1.3 Changes to Stream Geometry

The changes in the rates and amounts of runoff from developed watersheds directly affect the morphology, or physical shape and character, of Tennessee's creeks and streams. This is depicted graphically in Figure 1-2. Some of the impacts due to urban development include:

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

Figure 1-2. Physical Stream Changes Due to Watershed Development



1.1.4 Impacts to Aquatic Habitat

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

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

- Decline of Abundance and Biodiversity – When there is a reduction in various habitats and habitat quality, both the number and the variety, or diversity, of organisms (wetland plants, fish, macroinvertebrates, etc.) are also reduced. Sensitive fish species and other life forms disappear and are replaced by those organisms that are better adapted to the poorer conditions. The diversity and composition of the benthic, or streambed, community have frequently been used to evaluate the quality of urban streams. Aquatic insects are a useful environmental indicator as they form the base of the stream food chain.

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

1.1.5 Water Quality Impacts

Nonpoint source pollution, which is the primary cause of polluted stormwater runoff and water quality impairment, comes from many diffuse or scattered sources, many of which are the result of human activities within a watershed. Development concentrates and increases the amount of these nonpoint source pollutants. As stormwater runoff moves across the land surface, it picks up and carries away both natural and human-made pollutants, depositing them into Knox County's streams, rivers, lakes, wetlands, and groundwater. Nonpoint source pollution is the leading source of water quality degradation in Knox County. According to the State of Tennessee's list of impaired waters, sediment and habitat alteration are considered two major pollutants for streams in Knox County.

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

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

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

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

- Microbial Contamination – The level of bacteria, viruses and other microbes found in urban stormwater runoff often exceeds public health standards for water contact recreation such as swimming and wading. Microbes can also contaminate shellfish beds, preventing their harvesting and consumption and increasing the cost of treating drinking water. The main sources of these contaminants are sewer overflows, septic tanks, pet waste, and urban wildlife such as pigeons, waterfowl, squirrels, and raccoons.

Table 1-1. Major Stormwater Pollutants and Their Potential Effects

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

- Nutrient Enrichment – Runoff from urban watersheds contains increased nutrients such as nitrogen or phosphorus compounds. Increased nutrient levels are a problem as they promote weed and algae growth in lakes, streams and estuaries. Algae blooms block sunlight from reaching underwater grasses and deplete oxygen in bottom waters. In addition, nitrification of ammonia by microorganisms can consume dissolved oxygen, while nitrates can contaminate groundwater supplies. Sources of nutrients in the urban environment include washoff of fertilizers and vegetative litter, animal wastes, sewer overflows and leaks, septic tank seepage, detergents, and the dry and wet fallout of materials in the atmosphere.

- Hydrocarbons – Oils, greases and gasoline contain a wide array of hydrocarbon compounds, some of which have shown to be carcinogenic, tumorigenic and mutagenic in certain species of fish. In addition, in large quantities, oil can impact drinking water supplies and affect recreational use of waters. Oils and other hydrocarbons are washed off roads and parking lots, primarily due to leakage from vehicle engines. Other sources include the improper disposal of motor oil in storm drains and streams, spills at fueling stations and restaurant grease traps.

- Toxic Materials – Besides oils and greases, urban stormwater runoff can contain a wide variety of other toxicants and compounds including heavy metals such as lead, zinc, copper, and cadmium, and organic pollutants such as pesticides, PCBs, and phenols. These contaminants are of concern because they are toxic to aquatic organisms and can bioaccumulate in the food chain. In addition, they also impair drinking water sources and human health. Many of these toxicants accumulate in the sediments of streams and lakes. Sources of these contaminants include industrial and commercial sites, urban surfaces such as rooftops and painted areas, vehicles and other machinery, improperly disposed household chemicals, landfills, hazardous waste sites and atmospheric deposition.

- Sedimentation – Eroded soils are a common component of urban stormwater and a pollutant in their own right. Excessive sediment can be detrimental to aquatic life by interfering with photosynthesis, respiration, growth and reproduction. Sediment particles transport other pollutants that are attached to their surfaces including nutrients, trace metals and hydrocarbons. High turbidity due to sediment increases the cost of treating drinking water and reduces the value of surface waters for industrial and recreational use. Sediment also fills ditches and small streams and clogs storm sewers and pipes, causing flooding and property damage. Sedimentation can reduce the capacity of reservoirs and lakes, block navigation channels, fill harbors and silt estuaries. Erosion from construction sites, exposed soils, street runoff, and streambank erosion are the primary sources of sediment in urban runoff.

- Higher Water Temperatures – As runoff flows over impervious surfaces such as asphalt and concrete, it increases in temperature before reaching a stream or pond. Water temperatures are also increased due to shallow ponds and impoundments along a watercourse as well as fewer trees along streams to shade the water. Since warm water can hold less dissolved oxygen than cold water, this "thermal pollution" further reduces oxygen levels in urban streams. Temperature changes can severely disrupt certain aquatic species, such as trout and stoneflies, which can survive only within a narrow temperature range.

- Trash and Debris – Considerable quantities of trash and other debris are washed through storm drain systems and into streams, lakes and bays. The primary impact is the creation of an aesthetic "eyesore" in waterways and a reduction in recreational value. In smaller streams, debris can cause blockage of the channel, which can result in localized flooding and erosion.

1.1.6 Stormwater Hotspots

Stormwater hotspots are areas of the urban landscape that often produce higher concentrations of certain pollutants, such as hydrocarbons or heavy metals, than are normally found in urban runoff. These areas merit special management and the use of specific pollution prevention activities and/or structural stormwater controls. The Director of the Knox County Department of Engineering and Public Works has the authority to require additional measures for developments and redevelopments that propose such hotspot land uses. Examples of stormwater hotspots include, but are not limited to:

- Gas/fueling stations
- Vehicle maintenance areas
- Vehicle washing / steam cleaning
- Auto recycling facilities
- Outdoor material storage areas
- Plant nurseries, agricultural areas
- Kennels, feed lots, etc.
- Loading and transfer areas
- Landfills
- Construction sites
- Industrial sites
- Industrial rooftops

1.1.7 Effects on Ponds, Lakes and Reservoirs

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

1.2 Addressing Stormwater Impacts

The focus of the Knox County stormwater management program is effective and comprehensive stormwater management. Stormwater management involves both the prevention and mitigation of stormwater runoff quantity and quality impacts as described in this chapter through a variety of methods and mechanisms.

Volume 2 of this manual provides requirements, policies, and guidance for developers in Knox County to effectively implement stormwater management controls on-site to address the potential impacts of new development and redevelopment, and both prevent and mitigate problems associated with stormwater runoff. This is accomplished by:

- Developing land in a way that minimizes its impact on a watershed by reducing both the amount of runoff and the pollutants generated;
- Using the most current and effective erosion and sedimentation control practices during the *construction* phase of development;
- Controlling stormwater runoff peaks, volumes and velocities to prevent both downstream streambank channel erosion and flooding;
- Treating *post-construction* stormwater runoff before it is discharged to a waterway; and
- Implementing pollution prevention practices to prevent stormwater from becoming contaminated in the first place.

The remainder of Chapter 1 outlines the minimum stormwater management standards that Knox County uses to guide the requirements, policies and incentives of the stormwater management program.

1.3 Comprehensive Stormwater Management Planning

This section presents a comprehensive and integrated set of stormwater management standards for new development and redevelopment projects in Knox County. Minimum standards and performance requirements for controlling runoff from development are critical to addressing both the water quantity and quality impacts of post-construction urban stormwater and are required of Knox County in order to comply with National Pollutant Discharge Elimination System (NPDES) stormwater regulations. Minimum stormwater management standards must also be supported by a set of design and management tools and an integrated design approach for implementing both structural and nonstructural stormwater controls. The major elements of Knox County's stormwater management program are:

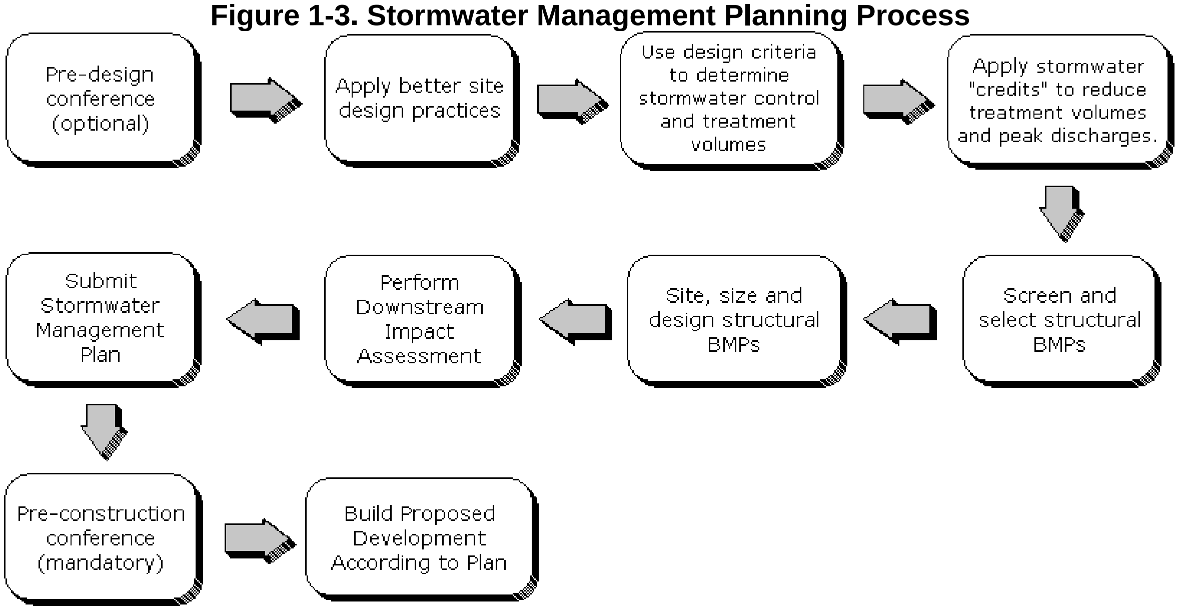
- <u>Incentives for Stormwater Better Site Design</u> – The first step in addressing stormwater management begins with the site planning and design process. The goals of better site development design are to reduce the amount of runoff and pollutants that are generated from a development site and provide for some nonstructural on-site treatment and control of runoff by implementing a combination of approaches collectively known as *stormwater better site design practices*. These include maximizing the protection of natural features and resources on a site, developing a site design that minimizes impact, reducing overall site imperviousness, and utilizing natural systems for stormwater management. General guidance on the types and application of better site design practices is provided in Volume 2, Chapter 5 of this manual.
- <u>Stormwater Credits for Better Site Design</u> – Knox County has developed a set of stormwater "credits" that can be used to provide developers and site designers' incentives to implement better site design practices that can reduce the volume of stormwater runoff and minimize the pollutant loads from a site. While reducing stormwater impacts, the credit system can also translate directly into cost savings to the developer by reducing the size of structural stormwater control and conveyance facilities. Specific technical guidance on the stormwater credits offered by Knox County is presented in Volume 2, Chapter 5 of this manual.
- <u>Integrated Stormwater Design Criteria</u> – The Integrated Stormwater Design (ISD) criteria is a combination of design criteria for stormwater quantity and quality management which addresses the entire range of hydrologic events. These criteria allow the site engineer to calculate the stormwater control volumes required for water quality, downstream channel

protection, and overbank and extreme flood protection. Specific technical guidance on stormwater design criteria is presented in Volume 2, Chapter 2 of this manual.

- Downstream Impact Analysis – Downstream peak discharge analyses are required to ensure that a proposed development is not adversely impacting downstream properties after the on-site stormwater management requirements have been addressed. These analyses can potentially be used to modify the requirement for overbank and extreme flood control, should the analysis reveal that such stormwater control measures would cause a negative flood impact on downstream properties. Downstream impact analysis requirements are presented in the Knox County Stormwater Management Ordinance and in Volume 2, Chapter 2 of this manual.
- Guidance on Structural Stormwater Controls – This manual provides requirements and specifications for a set of structural stormwater controls that can be used to meet Knox County's stormwater management water quantity and quality goals. Specific technical guidance on how to select, size, design, construct and maintain structural controls is provided in Volume 2, Chapter 4 of this manual.
- Stormwater Management Plan – Knox County requires the preparation of a Stormwater Management Plan for development and redevelopment activities. The plan must be approved by Knox County Engineering, prior to obtaining a grading or building permit. The purpose, requirements, and contents for this plan are discussed in Volume 1 of this manual.

Figure 1-3 illustrates how these design tools will be used in the development process to address Knox County's stormwater management requirements.

Figure 1-3. Stormwater Management Planning Process



1.4 Minimum Stormwater Management Standards

This section presents a set of minimum performance standards for stormwater management for development activities in Knox County. These performance standards provide an integrated approach to address both the water quality and quantity problems associated with stormwater runoff due to urban development. They are designed to assist Knox County government in complying with regulatory and programmatic requirements for various State and Federal programs including the National Pollutant Discharge Elimination System (NPDES), Municipal Separate Storm Sewer System (MS4) permit program and the National Flood Insurance Program (NFIP) under the Federal Emergency Management Agency (FEMA).

These minimum standards are provisions of Knox County's Stormwater Management Ordinance and are implemented and supported by the Stormwater Management Manual and Knox County's plans review process. The goal of these stormwater management requirements for areas of new development and redevelopment is to reduce the impact of post-construction stormwater runoff quality and quantity on the watershed. This can be achieved by:

- Maximizing the use of better site design and nonstructural methods to reduce the generation of runoff and pollutants;
- Managing and treating stormwater runoff through the use of structural stormwater controls; and,
- Implementing pollution prevention practices to limit potential stormwater contaminants.

The minimum stormwater management standards presented here incorporate these concepts and cover the entire cycle of development from site planning through long-term maintenance of stormwater management facilities after construction has been completed.

The following standards are the minimum stormwater management performance requirements for new development or redevelopment sites that must submit a stormwater management plan. A detailed explanation of each minimum standard is presented elsewhere in this manual, as noted in italics at the end of the standard title.

Minimum Standard #1 – Stormwater Quality Treatment *(Volume 2, Chapter 2)*

Stormwater runoff from development and redevelopment shall be treated to remove pollutants prior to discharge from the development or redevelopment site. Stormwater management systems shall be designed to remove 80% of the post-development total suspended solids (TSS) load, based on the 85^{th} percentile storm event, and be able to meet any other additional watershed or site-specific water quality requirements, as determined by Knox County Engineering. The justification for this standard is presented later in this chapter. Design criteria and equations are presented in Chapter 2. It is presumed that a stormwater management system complies with this performance standard if:

- Appropriate structural stormwater controls are selected, designed, constructed, and maintained according to the specific criteria in this manual; and
- Runoff from hotspot land uses and activities is adequately treated and addressed through the use of appropriate structural stormwater controls and pollution prevention practices.

Minimum Standard #2 – Use of Better Site Design Practices *(Volume 2, Chapter 5)*

Knox County will give stormwater "credits" for new developments and redevelopments to encourage the use and *Better Site Design* practices, promote the maximum preservation of natural and connected open space within and between developments, and provide additional opportunities for stormwater treatment.

Minimum Standard #3 – Stream Channel Protection *(Volume 2, Chapter 2)*

Stream channel protection shall be provided through the capture and extended detention of the runoff volume from the 1-year return frequency, 24-hour duration storm event.

Minimum Standard #4 – Overbank Flood Protection *(Volume 2, Chapter 2)*

Overbank flood protection shall be provided such that the calculated peak discharge of stormwater runoff resulting from the 2-year, 10-year, and 25-year return frequency, 24-hour duration storm events shall be no greater after development or redevelopment of the site than that which would result from the same 2-year, 10-year, and 25-year return frequency, 24-hour duration storms on the same site prior to development or redevelopment. Minimum standard

#4 can be modified in the event that a downstream impact analysis indicates that negative downstream flood impacts will result from on-site peak discharge controls.

Minimum Standard #5 – Extreme Flood Protection *(Volume 2, Chapter 2)*

Extreme flood protection shall be provided such that the calculated peak discharge of stormwater runoff resulting from a 100-year frequency, 24-hour duration storm shall be no greater after development or redevelopment of the site than that which would result from a 100-year frequency, 24-hour duration storm on the same site prior to development or redevelopment. Minimum standard #5 can be modified in the event that a downstream impact analysis indicates that negative downstream flood impacts will result from on-site peak discharge controls.

Minimum Standard #6 – Downstream Impact Analysis *(Volume 2, Chapter 2)*

A downstream hydrologic analysis shall be performed to determine if the proposed development or redevelopment causes an increase in peak discharge while meeting Minimum Standards #4 and #5. This analysis shall be performed at the outlet(s) of the site, and downstream at each tributary junction to the point(s) in the conveyance system where the area of the portion of the site draining into the system is less than or equal to 10% of the total drainage area above that point. Downstream flood protection shall be provided where the downstream impact analysis indicates peak discharge increases.

Minimum Standard #7 – Erosion Prevention and Sediment Control *(Volume 2, Chapter 9)*

Erosion prevention and sedimentation control (EPSC) practices shall be utilized during the construction phase or during any land disturbing activities. In general, Knox County requires that owners and operators of construction sites implement EPSC practices in accordance with the regulations and design and maintenance standards stated in the State of Tennessee General NPDES Permit for Discharges of Stormwater Associated with Construction Activities (also called the NPDES Construction General Permit), and the *Tennessee Erosion and Sediment Control Handbook*.

Minimum Standard #8 – BMP Operation and Maintenance *(Volume 1, Chapter 4)*

The owner(s) of stormwater management facilities and/or best management practices shall at all times properly operate and maintain all facilities and systems of stormwater treatment and control (and related appurtenances), and all best management practices in such a manner as to maintain the full function of the facilities or best management practices which are installed or used by the property owner(s) to achieve compliance with this ordinance.

Minimum Standard #9 – Pollution Prevention *(Volume 2, Chapter 10)*

New developments and redevelopments shall implement pollution prevention practices during and after development of the site. A Special Pollution Abatement Permit shall be required for land uses that are typically known to have a higher than normal potential for discharge of pollutants.

Minimum Standard #10 – Development In and Around Sinkholes *(Volume 2, Chapter 8)*

Developments and redevelopments that have sinkholes located entirely or partially on-site shall satisfy the design requirements for Minimum Standards #4 and #5. Developments and redevelopments that drain to sinkholes shall implement retention or detention of stormwater runoff such that the post-development peak discharges do not exceed pre-development discharges.

Minimum Standard #11 – Protection Using Water Quality Buffers *(Volume 2, Chapter 6)*

Water quality buffers shall be established, protected and maintained along all community waters, which includes streams, ponds, lakes and wetlands in Knox County.

Minimum Standard #12 – Stormwater Management Plan *(Volume 1, Chapter 4)*

The development project shall prepare a stormwater management plan for review by Knox County Engineering that presents the methods used to address Minimum Standards #1 through #11 in the proposed development or redevelopment.

1.5 Stormwater Quality Treatment Rationale

This section provides an explanation of Minimum Standard #1, which requires 80% removal of total suspended solids (TSS) from post-construction stormwater runoff based on capture of the 85th percentile storm event.

1.5.1 Regulatory Overview

The NPDES Phase II regulation requires that Knox County (and other Phase II regulated communities) develop, implement, and enforce a stormwater management program that reduces the discharge of pollutants from the regulated jurisdiction "to the maximum extent practicable (MEP)". MEP is a technology-based discharge standard that was designed for the reduction of pollutant discharges and established in the Clean Water Act. Using guidance provided by the Environmental Protection Agency (EPA), Knox County can achieve the MEP standard by instituting a stormwater management program that implements and requires best management practices (BMPs) that are designed to protect water quality. No further guidance on MEP is provided by EPA or by the Tennessee Department of Environment and Conservation (TDEC).

Control measure 5 of the National Pollutant Discharge Elimination System (NPDES) Phase II Permit presents the requirements for the control of post-construction (i.e., after development) stormwater runoff. Quoting directly from the NPDES Permit for the State of Tennessee, regulated cities and counties (including Knox County) must:

> *"Develop, implement, and enforce a program to address storm water runoff from new development and redevelopment projects that disturb greater than or equal to one acre, including projects less than one acre that are part of a larger common plan of development or sale, that discharge into your small MS4. Your program must ensure that controls are in place that would prevent or minimize water quality impacts;*
>
> *Develop and implement strategies which include a combination of structural and/or non-structural best management practices appropriate for your community; and*
>
> *Develop and implement a set of requirements to establish, protect and maintain water quality buffers in areas of new development and redevelopment.*
>
> *Use an ordinance or other regulatory mechanism to address post-construction runoff from new development and redevelopment projects to the extent allowable under State or local law."*

As a result of these requirements, Knox County must implement a requirement for new developments and redevelopments to control stormwater quality using both structural (i.e., constructed) and non-structural (i.e., site planning) best management practices (BMPs). This requirement must be fully implemented no later than 2008.

The NPDES Phase II regulation also requires that Knox County focus stormwater management on controlling discharges of pollutants of concern to local impaired streams. Based on the State of Tennessee's 303(d) list of "impaired" streams, the largest pollutant in Knox County is sedimentation. In 2004, over 225 stream miles were considered impaired due to excessive sedimentation.

1.5.2 Attaining the Water Quality Standard

The basic goal of the NPDES Phase II regulation is to reduce the water quality impacts of development. The preferred approach to meet this goal and comply with the NPDES permit is called the “Water Quality Volume method” or “WQv method”. The WQv method is based on a minimum water quality control goal of 80% removal of TSS for the 85th percentile storm event from post-construction stormwater runoff (i.e., after construction of a site is completed. TSS is a commonly used representative stormwater pollutant for measuring sedimentation.

There are a number of factors that support the use of an 80% TSS removal standard as a minimum level water quality goal in Knox County.

1. The Tennessee 303(d) list indicates that sedimentation (i.e., sediment) is a significant pollutant of concern in local streams. This fact alone requires that Knox County implement a stormwater management program that, at least in part, focuses on the removal of sediment from stormwater discharges in order to achieve compliance with the NPDES Phase II regulations to the maximum extent practicable.
2. The use of TSS as an “indicator” pollutant for sediment is well-established.
3. The control of TSS leads to indirect control of other pollutants of concern that can adhere to suspended solids in stormwater runoff. In fact, some research shows that a large fraction of many other pollutants of concern are either reduced along with TSS, or at rates proportional to the TSS reduction.
4. A treatment standard of 80% is not a numeric standard, but a “best available technology” standard. In other words, the 80% TSS removal level is reasonably attainable using properly designed, constructed and maintained structural stormwater BMPs (for typical ranges of TSS concentration found in stormwater runoff). This standard is supported with research data from numerous research projects and compiled by the International Stormwater Best Management Practices (BMP) Database evaluation project, titled Determining Urban Stormwater Best Management Practices Removal Efficiencies, June, 2000.

The WQv method can meet the goal of 80% TSS removal using a two-pronged approach. First, it encourages the reduction of imperviousness (and therefore pollution) from developed sites through incentives for non-structural BMPs, such as natural conservation areas and water quality buffers. Second, it requires treatment of any remaining stormwater runoff with structural controls. This method allows Knox County to meet its water quality goals and regulatory requirements, yet still allows developers flexibility in their site designs.

There are a number of advantages with the WQv method when compared to the County’s current requirement for first flush treatment. These advantages are as follows:

- The WQv method provides a measure of flexibility in site design. The new development or redevelopment site will be required to meet the 80% reduction goal using one or more of a number of locally-acceptable structural BMPs.
- If desired, the developer can also utilize non-structural controls to reduce imperviousness. The WQv method will provide incentives for the reduction of impervious surfaces and the use of non-structural BMPs, such as buffers, natural space preservation, and impervious area disconnection. When utilized, these practices will reduce the amount of stormwater runoff that will require treatment by structural practices, thereby reducing the structural BMP maintenance burden.
- WQv is not a prescriptive approach in that it does not mandate the use of one specific treatment BMP, such as a first flush pond. Instead, the developer can choose from a menu of BMPs, each of which is assigned a % TSS removal efficiency. When constructed alone, or in combination with other structural and/or non-structural BMPs, the minimum percent TSS removal standard can be attained.

- Research shows that extended release "first flush" ponds, which are often called dry extended detention (ED) ponds and are commonly used in East Tennessee, cannot attain a TSS removal standard of 80%. Such ponds have a high propensity for sediment resuspension and subsequent discharges, especially during large storm events. Recent studies of the BMP give it an average TSS reduction somewhere between 50% and 70% (Schueler and Holland, 2000). Of course, pollutant removal ability does depend upon geographic location, overall sediment characteristics, hydrology, and storm event size.

- WQv is a performance based approach. If the BMP(s) are designed, constructed and maintained in accordance with guidance and requirements set by Knox County, then the BMP(s) will be considered "in compliance" with the minimum 80% water quality standard.

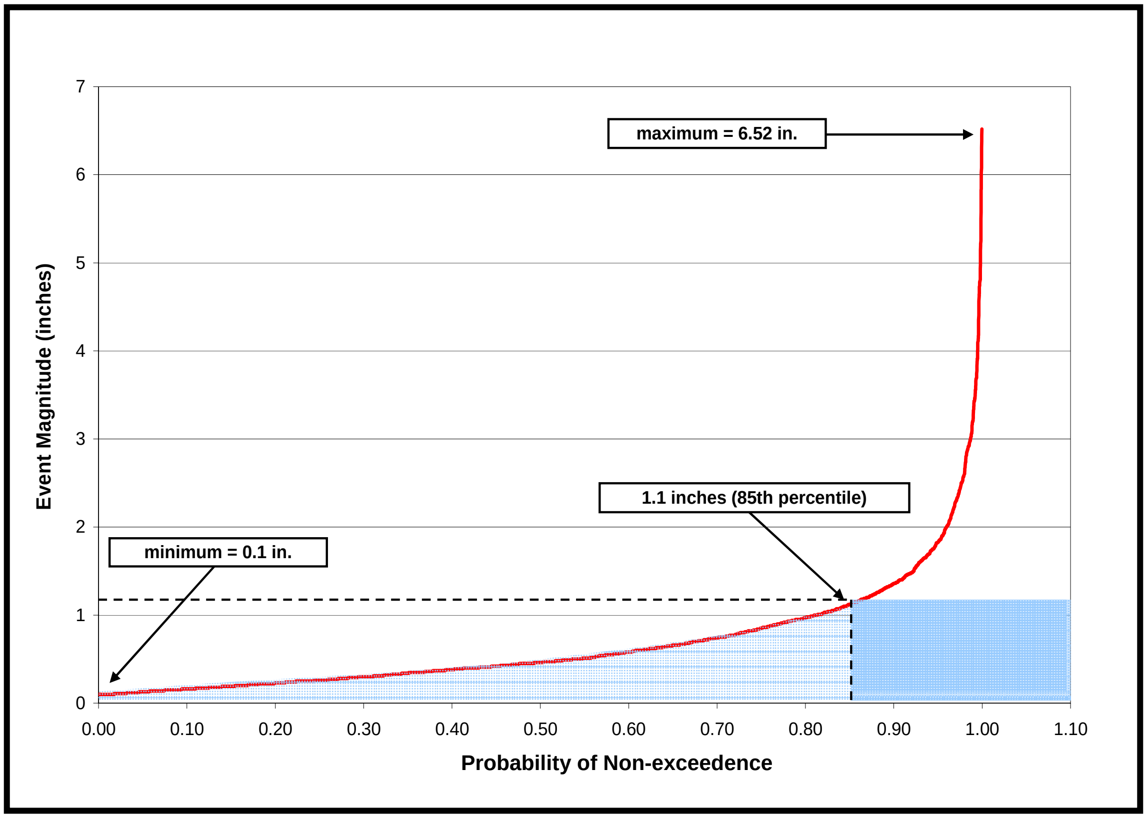
- The WQv method allows a consistent, "apples-to-apples" application of water quality treatment practices on every development site. Each site will be required to design, construct and maintain in accordance with the 80% TSS removal goal.

The WQv is calculated for the 85^{th} percentile storm event using a value of 1.1 inches of rainfall. Thus, a stormwater management system designed for the WQv will treat the runoff from all storm events of 1.1 inches or less, as well as the first 1.1 inches of runoff for all larger storm events. The 85^{th} percentile was chosen because it represents the "knee in the curve" volume that captures a significant number of storms (normally in the 80-90% of all storms range) without attempting to treat the small percentage of much larger storms that result in large volumes of runoff. Such storms would be expensive to treat, are rare in occurrence, and typically diluted in pollution concentration. Figure 1-4 presents a graphical representation of how the 85^{th} percentile rainfall depth was determined, using a "knee-in-the-curve" approach. The value of 1.1 inches for the 85^{th} percentile storm was determined for Knox County based on analysis of rainfall data collected in south Knox County and in Oak Ridge dating back to 1943.

Detailed information on the calculation of the WQv and % TSS removal for a development or redevelopment site are presented in Volume 2, Chapter 2 of this manual.

It is important to note that Knox County is not alone in implementing an 80% TSS removal standard, or the WQv method. Many states, including Maryland, Massachusetts, North Carolina, Georgia, and Florida have set similar statewide TSS goals and have research data to support BMPs meeting this reduction goal. Further, a number of communities in Tennessee, the State of Georgia and the Commonwealth of Virginia have implemented a WQv type of method as the statewide water quality control approach. The BMP design and maintenance guidance from these states can be used and modeled as appropriate to implement a water quality control program that is appropriate to meet Knox County's needs.

Figure 1-4: Knox County 85th Percentile Rainfall Analysis



References

GeoSyntec Consultants, URS, et al. *Determining Urban Stormwater Best Management Practices Removal Efficiencies*. June, 2000.

Schueler T., and Holland, H. *The Practice of Watershed Protection*. Center for Watershed Protection (CWP), 2000.

Suggested Reading

North Carolina Department of Environment and Natural Resources, *Stormwater Management Site Planning*. 1998

This page left intentionally blank

Table of Contents

CHAPTER 2 — CRITERIA FOR STORMWATER DESIGN

Chapter 2 – List of Figures

Chapter 2 – List of Tables

CHAPTER

2

CRITERIA FOR STORMWATER DESIGN

2.1 Integrated Site Design Approach

2.1.1 Introduction

This chapter represents the requirements, policies and other guidance for stormwater management design in Knox County. Included in this chapter are detailed design criteria to support Knox County's overall stormwater runoff quality and quantity management standards which are presented in Volume 2, Chapter 1. The design criteria presented herein address the key adverse impacts of stormwater runoff from a development site in Knox County. The purpose of the design criteria is to provide a framework for design of the site's stormwater management system in order to:

- remove stormwater runoff pollutants and improve water quality;
- prevent downstream streambank and channel erosion;
- reduce downstream overbank flooding; and
- safely pass or reduce the runoff from extreme storm events.

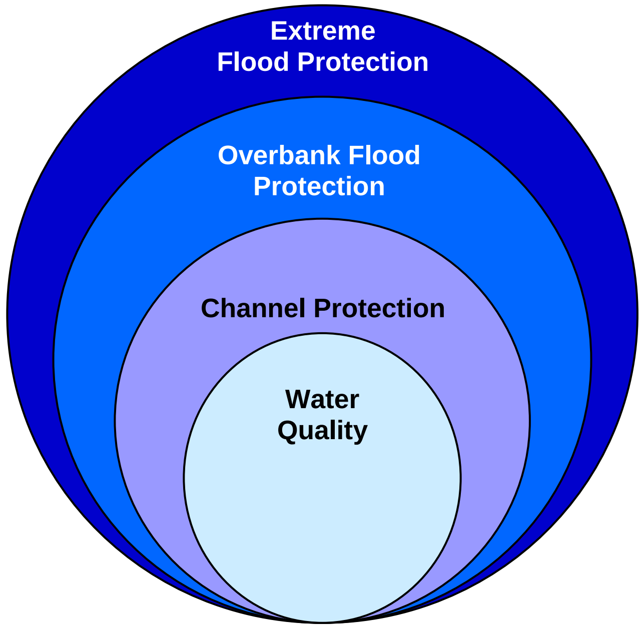
The Integrated Site Design (ISD) approach utilizes a set of design criteria that can be blended together, enabling the site engineer to size and design structural stormwater controls to address all of these objectives to achieve water quality and quantity goals. There are four criteria, one for each of the goals above, which are summarized in Table 2-1 below.

Table 2-1. Summary of Stormwater Criteria

Sizing Criteria	Description
Water Quality (WQv)	Treat the runoff from 85% of the rainfall events that occur in an average year to a load reduction goal of 80% average annual post-development total suspended solids (TSS).
Channel Protection (CPv)	The runoff volume from the 1-year frequency, 24-hour storm must be detained for no less than a 24-hour period in order to provide channel protection for channels and streambanks downstream from the new development.
Overbank Flood Protection (Qp_{25})	Provide peak discharge control of the 2, 10 and 25-year return frequency, 24-hour duration storm events such that the post-development peak rate does not exceed the pre-development rate.
Extreme Flood Protection (Qp_{100})	Provide peak discharge control of the 100-year return frequency, 24-hour duration storm event such that the post-development peak rate does not exceed the pre-development rate.

Each of the stormwater sizing criteria is intended to be designed in conjunction with the others to address the overall stormwater impacts from a development site. When used as a set, the criteria control the range of design events, from the smallest runoff producing rainfalls to the 100-year storm. Figure 2-1 graphically illustrates the relative volume requirements of each of the design criteria and demonstrates that the criteria are "stacked" within one another (i.e., the extreme flood protection volume requirement also contains the overbank flood protection volume, the channel protection volume and the water quality treatment volume).

Figure 2-1. Design Volume “Nesting” of Stormwater Criteria



2.1.2 General Design Policies and Guidance

The following general requirements and policies shall apply to the design of all stormwater control and conveyance facilities located in Knox County, including stormwater quality treatment facilities, stormwater quantity control facilities, pipe systems, culverts, open channels, bridges, and other stormwater conveyance system components.

1. Hydrologic and hydraulic computations shall be performed in accordance with NRCS (formerly known as the Soil Conservation Service or SCS) unit hydrograph procedures using AMC II curve numbers and Type II rainfall distribution, or other criteria that the Director of Engineering and Public Works or his designee (henceforth called “the Director”) shall establish based on scientific and engineering information.
2. All post-development conditions must be routed at appropriately small time intervals through stormwater storage and drainage facilities using either hand calculations or computer models that are widely accepted among engineering professionals. Detailed information on accepted hydrologic and hydraulic calculation methods and procedures is presented in Volume 2, Chapter 3 of this manual.
3. Other hydrologic and hydraulic computation methods may be approved by the Director in the design of curb inlets and small pipe systems when the final result is verified by a SCS method analysis.
4. All hydrologic and hydraulic computations utilized in the design of stormwater facilities must be prepared by a registered engineer proficient in the field of hydrology and hydraulics and licensed to practice engineering in the State of Tennessee.

5. The Director has the authority to require additional water quantity control standards, including restrictions on peak velocity and/or runoff volumes or less frequent design events, in areas where the Director has determined, through stormwater master plans, engineering studies, other regulatory water quality requirements, a history of existing or documented flooding or erosion problems, or engineering judgment, that additional restrictions are needed to limit adverse impacts of the proposed development downstream or upstream of the site.
6. The Director may waive or modify any of the stormwater system design criteria if adequate water quality treatment, and/or channel protection, and/or overbank flooding, and/or extreme flooding is suitably provided by a downstream or shared off-site stormwater facility, or if engineering studies determine that installing the required stormwater facilities would not be in the best interest of Knox County.

2.2 Stormwater Quality Management

2.2.1 General Requirements and Policies

The Knox County Stormwater Management Ordinance requires that stormwater runoff discharging from new development or redevelopment sites be treated to remove pollutants prior to discharge from the site. Minimum Standard #1 (Chapter 1) states that stormwater management systems shall be designed to remove 80% of the 85th percentile storm total suspended solids (TSS) load and be able to meet any other additional watershed or site-specific water quality requirements. This standard is often referred to as “the 80% TSS removal standard”.

The Director has the authority to require additional treatment in specific areas of Knox County to conform to state and/or Federal regulatory requirements, and/or if (in the opinion of the Director) the development or redevelopment has the potential to generate higher than normal pollutant discharges.

It is presumed that a stormwater management system complies with the 80% TSS removal standard if appropriate structural best management practices (BMPs) are selected, designed, constructed and maintained in accordance with the design criteria specified in this manual. Only those BMPs that are published in the Knox County Stormwater Management Manual are permitted for use as a water quality treatment practice in Knox County. Other BMPs are prohibited, unless approved by the Director.

2.2.2 Percent TSS Removal Values for Structural BMPs

The structural stormwater BMPs deemed acceptable for use in Knox County to attain the 80% TSS removal standard are listed in Table 2-2. The table presents the % TSS removal value assigned to each BMP. This value must be used to calculate the total weighted % TSS removal for the development site. The % TSS removal values assigned to each BMP are conservative median pollutant reduction percentages for design purposes that have been derived from existing sampling data, modeling and professional judgment. A structural BMP design may be capable of exceeding these performances; however, the values in the table are median values that can be achieved over time when the structural BMP is sized, designed, constructed and maintained in accordance with required specifications in this manual. The actual % TSS removal value for a BMP in any single storm event may be higher or lower, depending upon a number of factors, including the inflow pollutant concentration, type of storm event, and maintenance condition of the BMP.

The structural BMPs listed in Table 2-2 fall into two categories (general application or limited application) based upon the BMP’s ability to meet stormwater management goals and/or its maintenance requirements. Further discussion of these categories and detailed guidance on BMP selection, design, construction, and maintenance are presented in Volume 2, Chapter 4 for each BMP listed in Table 2-2.

2.2.2.1 Policies for New or Proprietary BMPs

There are many commercially available BMPs that provide water quality or quantity treatment (e.g., Stormceptor, Vortechnics, etc.). Typically, such "proprietary" controls have high installation and maintenance costs and requirements. Therefore, they are best suited for non-residential developments or redevelopments that have limited space for water quality or quantity treatment using standard BMPs. This section describes the requirements that must be met for proprietary BMPs to be approved for a given site.

Table 2-2. TSS Removal % for Structural BMPs

Structural BMP	TSS Removal %
General Application BMPs	
Wet Pond	80
Wet Extended Detention	80
Micropool Extended Detention Pond	80
Multiple Pond System	80
Dry Extended Detention Pond	60
Conventional Dry Detention Basins	10
Shallow Wetland	75
Extended Detention Shallow Wetland	75
Pond/Wetland System	75
Pocket Wetland	75
Bioretention Area	85
Sand Filters (Surface and Perimeter)	80
Infiltration Trench	90
WQ Dry Swales	90
Wet Swales	75
Filter Strip	50
Grass Channel[1]	30
Gravity (oil-grit) Separator	30
Modular Porous Paver Systems[2]	*
Porous Pavement/Concrete[2]	*
Limited Application BMPs	
Organic Filter	80
Underground Sand Filter	80
Submerged Gravel Wetland	75
Alum Treatment System	90
Proprietary Treatment Controls	10[3]
Underground Detention	10

1 – Refers to open channel practice not designed for water quality.
2 – These practices are not treatment BMPs but are source control BMPs, so they are not assigned a pollutant removal.
3 – Provisional % TSS Removal value pending third party information.

Proprietary BMPs, new BMPs, or technologies that are not included in this manual may be approved by the Director on a case-by-case basis for the treatment of stormwater quality. A TSS removal percentage of 10% shall be used for such BMPs (if approved), or a higher percentage may be approved provided that the three conditions listed below are met. Judgments of the below three conditions shall be made by the Director after review of applicable information submitted by the site designer. A poor performance record, high failure rate or unacceptably high maintenance requirements are all valid justifications for not allowing the use of a proprietary system or device. It is the responsibility of the developer to provide the Director with sufficient information to allow modification of the % TSS removal value for any proprietary BMPs. TSS removal rates greater than 10% can be granted when the following conditions are met.

1. The BMP or technology, as applied to the development or redevelopment on which it will be used, meets the minimum standards and design criteria published in the Knox County Stormwater Management Manual. The performance ability of the BMP must be verified by an independent third party, in accordance with the monitoring criteria presented below. If the performance ability of the BMP cannot be verified, the BMP will not be approved for purposes of stormwater quality treatment.
2. The BMP or technology must have a proven record of operational longevity under hydrologic conditions similar to what would be encountered in Knox County (rainfall, slope, soil types, etc.).
3. The BMP or technology must have documented procedures for inspection and maintenance, including the collection and removal of pollutants or debris. BMPs that have unacceptably high maintenance requirements may not be installed within public rights-of way or on public property.

The following monitoring criteria should be met for research/studies to support the three acceptance conditions stated above:

- Water quality treatment performance must be monitored for a minimum of fifteen (15) storm events.
- Water quality treatment performance research/monitoring must be conducted in the field, as opposed to laboratory testing.
- Field monitoring must be conducted using standard protocols which require proportional sampling both upstream and downstream of the device. For guidance on testing protocols, see "Stormwater Best Management Practices Demonstration Tier II Protocol for Interstate Reciprocity" as developed under the Environmental Council of States (ECOS) and Technology Acceptance and Reciprocity Partnership (TARP, 2001).
- Pollutant concentrations reported in the study must be flow-weighted.
- The proprietary system or device must have been utilized in place for at least one year prior to the time of monitoring.
- Although local data is preferred, data from other areas can be accepted as long as the design accounts for the local hydrologic conditions.

2.2.3 Calculation of WQv and % TSS Removal

Compliance with the % TSS removal standard requires the calculation of the WQv and % TSS removed for the entire development site. To obtain the lowest WQv for the site, this calculation should be performed *after* better site design practices that may be envisioned for the site have been considered and are included in design plans.

The WQv shall be calculated using Equation 2-1, as follows:

Equation 2-1

$$WQv = \frac{P \times Rv \times A}{12}$$

where:

WQv = water quality volume of the site (acre-feet);
P = rainfall depth for the 85% storm event (1.1 inches);
Rv = runoff coefficient; and,
A = site area (acres).

The runoff coefficient (Rv) shall be calculated using Equation 2-2.

Equation 2-2

$$Rv = 0.015 + 0.0092 \times I$$

where:

I = percent impervious area of the site (see Equation 2-5 below).

2.2.4 Calculation of % TSS Removal for BMPs in Parallel

The percent TSS removal (%TSS) that is achieved on a site can be calculated using Equation 2-3. Equation 2-3 is an area-weighted TSS reduction equation which accounts for the TSS reduction that is contributed from BMPs that area treating separate areas and not being used in series.

Equation 2-3

$$TSS_{Site} = \frac{\sum_1^n (TSS_1 A_1 + TSS_2 A_2 + \ldots + TSS_n A_n)}{\sum_1^n (A_1 + A_2 + \ldots + A_n)}$$

where:

TSS_n = TSS removal for each structural BMP located on-site (%);
A_n = the area draining to each BMP (acres).

Where BMPs are used in series, the total % TSS removal for the combination of two or more BMPs shall be used for TSS_n in Equation 2-3. Calculation of this value is discussed in the next section.

2.2.5 Calculation of % TSS Removal for BMPs in Series

It will often be the case that the site designer will want to use two or more BMPs (structural and/or non-structural) in series, where stormwater treated in one BMP is discharged into another BMP for further treatment. Such BMP combinations are also called treatment trains. How and why BMPs might be used in treatment trains is discussed in Volume 2, Chapter 4 of this manual. This section presents the calculation of the total % TSS removal for treatment trains.

To calculate the total % TSS removal for a treatment train comprised of two or more structural BMPs, Equation 2-4 shall be used.

Equation 2-4

$$TSS_{train} = TSS_A + TSS_B - \frac{(TSS_A \times TSS_B)}{100}$$

where:

TSS_{train} = total TSS removal for treatment train (%);
TSS_A = % TSS removal of the first (upstream) BMP, from Table 2-2 (%)
TSS_B = % TSS removal of the second (downstream) BMP, from Table 2-2 (%).

For development sites where the treatment train provides the only stormwater treatment on the site, TSS_{train} must be greater than or equal to 80%. For development sites that have other structural BMPs for stormwater treatment that are not included in the treatment train, TSS_{train} must be included in Equation 2-3 in the calculation of the overall % TSS removal for the site. An example application of the latter situation is presented below.

Example 2-1. Calculation of %TSS when Treatment Trains are Used

A stormwater management system located on a 30 acre development site consists of a dry extended detention pond, a water quality dry swale, and a shallow wetland. The extended detention pond and dry swale are located in series, with the pond as the upstream control. The treatment train treats stormwater runoff from 20 acres of the site. The shallow wetland treats 10 acres. All three facilities are designed in accordance with the Knox County Stormwater Management Manual. What is the % TSS removal rate for the site?

The % TSS removal value for each BMP located on the site is determined from Table 2-2, as follows:

Control A (dry extended detention pond) = 60% TSS removal
Control B (water quality dry swale) = 90% TSS removal
Control C (shallow wetland) = 75% TSS removal

Step 1: Calculate TSS_{train}:

$TSS_{train} = TSS_A + TSS_B – (TSS_A \times TSS_B)/100 = 60 + 90 - (60 \times 90)/100 = 96\%$ removal

Step 2: Calculate % TSS removal for the site:

$\%TSS = ((TSS_{train} \times 20 \text{ acres}) + (\%TSS_{wetland} \times 10 \text{ acres})) \div 30 \text{ acres}$

$\%TSS = ((96\% \times 20 \text{ acres}) + (75\% \times 10 \text{ acres})) \div 30 \text{ acres} = 89\%$

Therefore, the % TSS removal for the site is 89%, which exceeds the minimum standard of 80% TSS removal. No other BMPs need to be constructed at the site.

2.2.6 Calculation of % TSS Removal for Flow-through Situations

BMPs within a treatment train may sometimes be separated by a contributing drainage area. In this case, equation 2-4 cannot be used, since some of the flow entering the downstream BMP has not been treated by the upstream BMP. This section presents the calculation of the total % TSS removal for flow-through situations.

To calculate the total % TSS removal for a treatment train separated by a contributing drainage area, Equation 2-5 shall be used.

Equation 2-5

$$TSS_{train} = \frac{TSS_A A_A + TSS_B A_B + \dfrac{TSS_B A_A (100 - TSS_A)}{100}}{A_A + A_B}$$

where:

TSS_{train} = total TSS removal for treatment train (%);
TSS_A = % TSS removal of the first (upstream) BMP, from Table 2-2 (%)
TSS_B = % TSS removal of the second (downstream) BMP, from Table 2-2 (%)
A_A = Area draining to BMP A
A_B = Area draining to BMP B.

For development sites where the treatment train provides the only stormwater treatment on the site, TSS_{train} must be greater than or equal to 80%. An example application of Equation 2-5 is shown below.

<u>Example 2-2. Calculation of %TSS in a Flow-through Situation</u>

A stormwater management system located on a 9 acre development site consists of a dry extended detention pond, and a bioretention cell. Five acres drain to the bioretention cell, which then drains to a pipe system. The pipe system also drains an additional 4 acres that have not been treated for water quality. The pipe system leads to a dry extended detention pond, that is used for final treatment. Both facilities are designed in accordance with the Knox County Stormwater Management Manual. What is the % TSS removal rate for the site?

The % TSS removal value for each BMP located on the site is determined from Table 2-2, as follows:

Control A (bioretention cell) = 85% TSS removal
Control B (dry extended detention pond) = 60% TSS removal

<u>Step 1:</u> Calculate TSS_{train}:

$$TSS_{train} = \frac{TSS_A A_A + TSS_B A_B + \dfrac{TSS_B A_A (100 - TSS_A)}{100}}{A_A + A_B}$$

$$TSS_{train} = \frac{85 * 5 + 60 * 4 + \dfrac{60 * 5(100 - 85)}{100}}{5 + 4}$$

$$TSS_{train} = 78.9\%$$

The % TSS removal for the site is 78.9%, which is below the minimum standard of 80% TSS removal. The conversion of the stormwater pipe system to a grass swale would add additional pollutant removal and help the site meet the 80% criteria.

2.2.7 The Measurement and Calculation of Percent Imperviousness

The percent impervious area (I) that is used to determine WQv is calculated using Equation 2-5.

Equation 2-5

$$I = \frac{I_A}{A} \times 100\%$$

where:

I_A = cumulative area of all impervious surfaces on the site (acres);
A = site area (acres).

Impervious areas are defined in the Knox County Stormwater Management Ordinance as impermeable surfaces which prevent the percolation of water into the soil including, but not limited to, paved surfaces such as walkways, sidewalks, patios, parking areas and driveways, packed gravel or soil, and rooftops. Other examples of impervious areas are paved recreation areas including pool houses and pool decks intended for use as a private (multi-family) or public recreation area, paved athletic courts (e.g., basketball, tennis), and storage buildings.

The determination of the impervious area (I_A) in order to calculate WQv shall be performed in the following manner:

1. For residential subdivisions that will be served by one or more shared stormwater facilities, I_A may be determined using percent (%) impervious values that were developed by the Soil Conservation Service (SCS)[1]. Where the average lot size of a subdivision or a drainage area within the subdivision falls between the lot size categories shown in Table 2-3, the site designer may interpolate the % impervious value based on Table 2-3.

   The values shown in Table 2-3 shall be utilized only for the portion of the subdivision that is covered by individual residential lots and streets. Other areas, such as common areas for recreation or meeting facilities, shall be added separately in the calculation of I_A. The calculation of the % impervious value for a residential subdivision having a common area is presented in Example 2-2.

   If lot sizes within a single subdivision fall into more than one of the lot size ranges listed in Table 2-3, the site designer shall consider the total amount of imperviousness in each lot range separately in the determination of the percent impervious value. Example 2-2 includes the calculation of the % impervious value for a residential subdivision having variable lot sizes.

Table 2-3. % Impervious Area Values for Subdivisions

Residential Lot Size Range[1]	% Impervious
⅛ acre or less	65
¼ acre	38
⅓ acre	30
½ acre	25
¾ acre	22.5[2]
1 acre	20
2 acres and greater	15

1 – Includes lots and streets. Common areas must be measured separately.
2 – The % impervious value is interpolated from SCS data.

2. For planned unit developments where the building and paving footprints are known, as well as all nonresidential developments, I_A shall be determined from the measured impervious footprints for all impervious areas as defined above. It is required that the footprint for all impervious surfaces in the proposed development and the calculation of I_A be shown in the stormwater management plan.

After the development and/or redevelopment of the property is complete, property improvement activities that do not require the submittal of a stormwater management plan will not require recalculation of the impervious percentage and WQv.

<u>Example 2-2. Calculation of Percent Impervious Area (I)</u>

A site design engineer is preparing a stormwater management plan for a proposed residential development in Knox County. The subdivision has a total area of 31 acres, and will include 52 residential lots ranging in area from approximately ¼ acre to no greater than 1 acre (as shown in the

[1] The Soil Conservation Service is now known as the Natural Resource Conservation Service.

table below). Three (3) acres will be preserved as an undisturbed forested water quality buffer located along a stream that crosses the property, and therefore, there is no impervious coverage within these three acres. Another three (3) acres will be utilized for a recreational common area which includes a community pool, tennis courts and an associated parking lot. Due to local topography on the site, the subdivision drains to two separate stormwater facilities, herein called Facility A and Facility B, both of which provide stormwater quality treatment. Twelve acres, including the 3 acre water quality buffer and 3 acre common area, drain to Facility A. The other 19 acres drain to Facility B. The table below provides lot size, area and impervious data for the proposed subdivision. What is the % impervious area for the site?

A	B	C	D
Lot Size	**Number of Lots in Size Range**	**Sub-total Area of Lots in Size Range**	**% Impervious (from Table 2-3)**
DRAINAGE AREA A (AREA DRAINING TO FACILITY A)			
approx. ⅓ acre	0	0 acres	30
approx. ½ acre	0	0 acres	25
approx. ¾ acre	2	1.3 acres	22.5
approx. 1 acre	5	4.7 acres	20
Area A Totals	7 lots	6.0 acres	--
DRAINAGE AREA B (AREA DRAINING TO FACILITY B)			
approx. ⅓ acre	21	6.6 acres	30
approx. ½ acre	16	7.3 acres	25
approx. ¾ acre	7	4.3 acres	22.5
approx. 1 acre	1	0.8 acres	20
Area B Totals	45 lots	19.0 acres	--

Since the site will be served by two separate detention facilities, it is best to determine the impervious area for each drainage area, rather than the overall impervious area for the site.

Step 1: Determine the total impervious area for the portion of each drainage area that is covered by residential lots and associated subdivision roads ($I_{\text{residential areas}}$):

This is calculated by multiplying the sub-total area of each lot size range (column C from the above table) by the corresponding % impervious in that lot size range (column D from the above table). Results of this calculation are shown in the table below.

A	B	C	D
Lot Size	**Sub-total Area of Lots in Size Range**	**% Impervious (from Table 2-3)**	**Sub-total Impervious Area**
DRAINAGE AREA A (AREA DRAINING TO FACILITY A)			
approx. ⅓ acre	0 acres	30	0 x 0.30 = 0 ac
approx. ½ acre	0 acres	25	0 x 0.25 = 0 ac
approx. ¾ acre	1.3 acres	22.5	1.3 x 0.225 = 0.29 ac
approx. 1 acre	4.7 acres	20	4.7 x 0.20 = 0.94 ac
Area A Totals	6.0 acres	--	1.23 acres
DRAINAGE AREA B (AREA DRAINING TO FACILITY B)			
approx. ⅓ acre	6.6 acres	30	6.6 x 0.30 = 1.93 ac
approx. ½ acre	7.3 acres	25	7.5 x 0.25 = 1.88 ac
approx. ¾ acre	4.3 acres	22.5	4.3 x 0.225 = 0.97 ac

A Lot Size	B Sub-total Area of Lots in Size Range	C % Impervious (from Table 2-3)	D Sub-total Impervious Area
approx. 1 acre	0.8 acres	20	0.8 x 0.20 = 0.16 ac
Area B Totals	19.0 acres	--	4.94 acres

Thus, the portions of the site where residential lots are located are covered by impervious surfaces as follows:
$I_{A\ \text{residential areas}}$ = 1.23 acres
$I_{B\ \text{residential areas}}$ = 4.94 acres

Step 2: Measure the area of impervious footprints in the common areas that are located in Area A ($I_{A\ \text{common areas}}$):

The following table presents the measurements of the impervious areas located in the common area.

Area Description	Impervious Area
Community pool (includes pool deck, maintenance building and sidewalk from parking lot)	0.8 acres
Tennis court (includes two courts, surrounding paved areas, and sidewalk from parking lot)	1.2 acres
Common area driveway and parking lot	0.7 acres
Total impervious areas	2.7 acres

Thus, 2.7 acres of the 3 acre common area, located in Area A, is covered by impervious surfaces. $I_{A\ \text{common areas}}$ = 2.7 acres

Step 3: Calculate the % impervious area (I) for each drainage area of the site using Equation 2-5. Because the water quality buffer is entirely undisturbed, and therefore entirely pervious, it is not considered in the calculation.

For Area A:

$I_A = ((I_{A\ \text{residential areas}} + I_{A\ \text{common areas}}) \div 12 \text{ acres}) \text{ X } 100\%$
$I_A = ((1.23 \text{ acres} + 2.7 \text{ acres}) \div 12 \text{ acres}) \text{ X } 100\%$
$I_A = (3.9 \text{ acres} \div 12 \text{ acres}) \text{ X } 100\%$
$I_A = 32.8\%$

For Area B:

$I_B = (I_{B\ \text{residential areas}} \div 19 \text{ acres}) \text{ X } 100\%$
$I_B = (4.94 \text{ acres} \div 19 \text{ acres}) \text{ X } 100\%$
$I_B = 26.0\%$

Therefore, the % impervious area for Area A (I_A) for the site is 32.8%. The % impervious area for Area B (I_B) is 26.0%. These values are utilized in Equation 2-2 to determine the values of Rv which will then be used in Equation 2-1 to find the WQv for each stormwater quality treatment BMP on the site.

2.2.8 Reducing the WQv

One of the key points to remember when calculating WQv is that it is proportional to impervious area, such that the amount of stormwater runoff required for treatment increases as impervious area increases. In other words, the more you pave, the more you treat. Therefore, to reduce the amount of stormwater runoff that must be treated, the developer must find ways to reduce site imperviousness. Reductions in imperviousness are beneficial from a stormwater management standpoint. Decreases in impervious area equate to less runoff, lower post-development peak discharges, and typically lower pollutant discharges. This can result in lower stormwater management costs, as structural BMPs, channel protection, and flooding protection controls can be smaller in size.

In order to reduce the WQv for a development site, Knox County encourages the use of better site design practices. Better site design can be defined as a combination of non-structural design approaches intended to reduce the impacts of stormwater runoff from development through the conservation of natural areas, reduction of impervious areas, and integration of stormwater treatment BMPs. Such practices are often referred to as “non-structural practices or BMPs”. By implementing a combination of these non-structural approaches, it is possible to reduce the amount of runoff and pollutants that are generated from a site and provide for some non-structural on-site treatment and control of runoff.

Knox County does not require the use of better site design practices on a development or redevelopment site to attain the 80% TSS removal standard. However, in accordance with Minimum Standard #2 (Chapter 1), a strong incentive for the use of such practices is provided via the WQv method and through WQv credits. The basic premise of the credit system is to recognize the water quality benefits of certain site design practices by allowing for a reduction in the WQv. The WQv credits that are available in Knox County are listed in Table 2-4. Detailed policies and design requirements for credits and better site design practices are presented in Volume 2, Chapter 5 of this manual.

Table 2-4. Summary of WQv Credits

Credit	Description
Credit 1: Natural area preservation	Undisturbed natural areas are conserved, thereby retaining the pre-development hydrologic and water quality characteristics.
Credit 2: Managed area preservation	Managed areas of open space are preserved, reducing total site runoff and retaining near pre-development hydrologic and water quality characteristics.
Credit 3: Stream and vegetated buffers	Stormwater runoff is treated by directing runoff through a naturally vegetated or forested buffer as overland flow.
Credit 4: Vegetated channels	Vegetated channels are used to provide stormwater treatment.
Credit 5: Impervious area disconnection	Overland flow filtration/infiltration zones are incorporated into the site design to receive runoff from rooftops and other small impervious areas.
Credit 6: Environmentally sensitive large lot neighborhood	A group of site design techniques are applied to low and very low density residential development.

2.3 Channel Protection (CPv)

2.3.1 Background

The increase in the frequency, velocity, and duration of bankfull flow conditions in stream channels after a rainfall event is the primary cause of streambank erosion. Such erosion is common in Knox County, usually in channels and streams where the cumulative effect of development has caused lengthy, increased post-rainfall discharges. The sediment released as a result of streambank erosion is a likely major source of sediment pollutant loads in Knox County streams. Excessive sediment can impact a stream's ability to remain ecologically viable and provide a healthy habitat for aquatic species.

Streambank erosion can cause damaging hydraulic changes in a stream, including excessive widening, deepening, and undercutting. Figure 2-2 presents an example of this problem located on a tributary to Ten Mile Creek in west Knox County. Such changes can be detrimental to the ability of the stream to remain hydraulically stable in the long-term. Moreover, streambank erosion is a common source of complaints from citizens that experience property damage due to fallen trees or outbuildings, or property loss due to widening streams.

Figure 2-2. Example of Significant Streambank Erosion

2.3.2 Design Criteria and Policies

Knox County requires all developments and redevelopments to adhere to channel protection criteria (Minimum Standard #3), herein called the channel protection volume (CPv). This standard requires that the runoff volume from the 1-year frequency, 24-hour storm be detained for no less than a 24-hour period. In the design of the channel protection control, the 24-hour detention period shall be measured from the approximate center-of-mass of inflow to the approximate center-of-mass of outflow.

Downstream channel protection provided by an alternative approach may be considered in lieu of controlling the CPv, provided that sufficient hydrologic and hydraulic analysis shows that the alternative approach will offer adequate channel protection from erosion. Downstream channel protection provided by an alternative approach must be approved by the Director.

2.4 Stormwater Quantity Management

This section addresses the design criteria and policies associated with Knox County's requirements for overbank protection and extreme flood protection. This section also presents justification, policies and requirements for the downstream impact analysis.

2.4.1 Overbank Flood Protection Criteria (Qp_{25})

Minimum Stormwater Management Standard #4 establishes overbank flood protection design criteria (Qp_{25}). The purpose of Qp_{25} is to prevent an increase in the frequency and magnitude of damaging out-of-bank flooding (i.e., flow events that exceed the capacity of the channel and enter the floodplain). It is intended to protect downstream properties from flooding during and after middle-frequency storm events.

The Qp_{25} criteria requires that the calculated peak discharge of stormwater runoff resulting from the 2-year, 10-year, and 25-year return frequency, 24-hour duration storm events be no greater after development or redevelopment of the site than that which would result from the same 2-year, 10-year, and 25-year return frequency, 24-hour duration storms on the same site prior to development or redevelopment. Hydrologic calculation methods for Qp_{25} are provided in Volume 2, Chapter 3.

Typically, peak discharge control is achieved through detention of runoff for the design events; however, Knox County does not mandate the use of detention facilities. It should be noted that the smaller design events (e.g., 2-year and 10-year) are often effectively controlled through the combination of the required channel protection (CPv) control (i.e., extended detention of the 1-year event) and the control of the 25-year frequency event.

2.4.2 Extreme Flood Protection (Qp_{100})

The intents of Minimum Standard #5, also called the extreme flood protection design criteria (Qp_{100}), are to:

- prevent flood damage from infrequent, but large, storm events;
- maintain the boundaries of the mapped 100-year floodplain; and,
- protect the physical integrity of the structural stormwater BMPs as well as downstream stormwater and flood control facilities.

The Qp_{100} criteria requires that the calculated peak discharge of stormwater runoff resulting from a 100-year frequency, 24-hour duration storm be no greater after development or redevelopment of the site than that which would result from a 100-year frequency, 24-hour duration storm on the same site prior to development or redevelopment. In addition, all drainage systems shall be designed to insure that no habitable finished floor elevations are flooded for the 100-year frequency storm. Pipes and culverts designed for a 100-year storm shall be constructed of reinforced concrete if such pipes or culverts lie in public lands or easements.

Design of stormwater systems that will include extreme flood protection controls must route the Qp_{100} through the drainage system and stormwater management facilities to determine the effects on the facilities, adjacent property, and downstream areas. Emergency spillways of structural BMPs must be designed appropriately to safely pass the Qp_{100}.

Further guidance on hydrologic analysis and design for the Qp_{100} criteria is provided in Volume 2, Chapter 3.

2.4.3 Downstream Impact Analysis

2.4.3.1 Background

The Qp_{25} and Qp_{100} flood protection criteria require the design to control peak discharges at the outlet of a site, such that the post-development peak discharge does not exceed the pre-development peak discharge. Typically, this peak discharge control is achieved through construction of one or more on-site detention facilities. However, stormwater master plans developed for a number of Knox County watersheds indicate that peak discharge control does not always provide effective water quantity control from the site, and may actually exacerbate flooding problems downstream of the site. Moreover, master plans have shown that a development site's location within a watershed may preclude the requirement for overbank flood control from a particular site.

A major reason for negative impacts due to detention involves the timing of the peak discharge from the site in relation to the peak discharges in the receiving stream and/or its tributaries. If detention structures are indiscriminately placed in a watershed without consideration of the relative timing of downstream peak discharges, the structural control may actually increase the peak discharge downstream. An example of this situation is presented in Figure 2-3, which shows a comparison of the total downstream flow on a receiving stream (after development) with and without detention controls. In Figure 2-3, the smaller dashed-dot and solid lines denote the runoff hydrograph for a development site with and without detention, respectively. These runoff hydrographs will combine with a larger runoff hydrograph of the receiving stream (not shown). The combined discharges from the site and receiving stream are shown in the larger solid and dashed lines.

Figure 2-3 conveys a possible consequence of detention. The post-development flow from the site is reduced as required by flood protection design criteria to result in the detained flow (the smaller dashed-dot hydrograph). However, the timing of the peak discharge for the detained post-development flow, while reduced in magnitude, corresponds more closely with the timing of the peak discharge of the receiving stream (not shown) than the peak discharge of the post-development flow that was not detained. Therefore, the combination of the detained flow with the flow in the receiving stream is actually higher than would occur if no detention were required, as shown in the larger dashed hydrograph. Hence, there is a peak flow increase that is caused by detention.

Poor peak discharge timing can have an even greater impact when one considers all the developments located in a watershed and the cumulative effects of increases in runoff volume and the duration of high volume runoff in the channel, as well as peak discharge timing. Even if peak discharges are handled effectively at the site level and immediately downstream, the longer duration of higher flows due to the increased volume from many developments located on or near a stream may combine with downstream tributaries and receiving streams to dramatically increase the downstream peak flows.

Figure 2-4 illustrates this concept. The figure shows the pre- and post-development hydrographs at the confluence of two tributaries. Development occurs, meets the local flood protection criteria (i.e., the post-development peak flow is equal to the pre-development peak flow at the outlet from the site), and discharges to Tributary 1. When the post-development detained flow from Tributary 1 combines with the first downstream tributary (Tributary 2), it causes a peak flow increase when compared to the pre-development combined flow. This is due to the increased volume and timing of runoff from Tributary 1, relative to the peak flow and timing in Tributary 2. In this case, the detention volumes on Tributary 1 would have to have been increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

Figure 2-3. Potential Effect of On-Site Detention on Receiving Streams

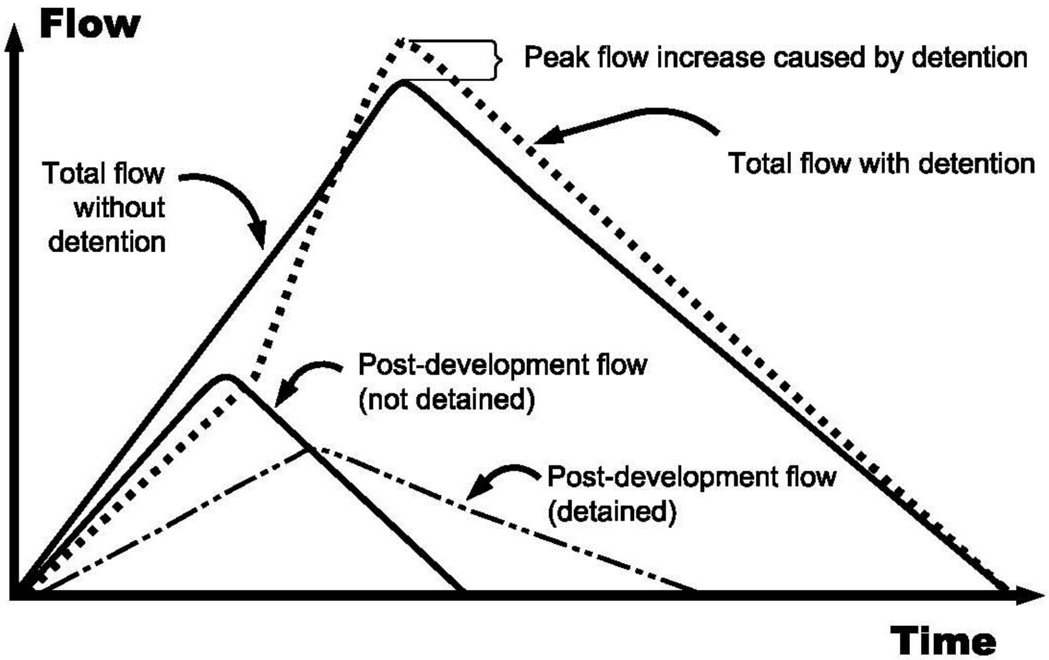
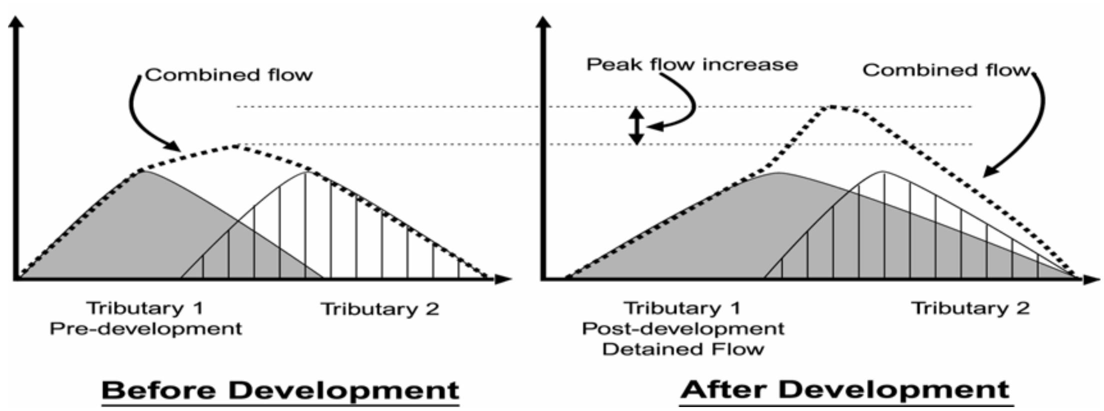


Figure 2-4. Potential Effect of Cumulative Detention Ponds



Potential problems such as those described above are quite common, but can be avoided through the use of a stormwater master plan and/or downstream analysis of the effects of a planned development. Studies have shown that if a developer is required to assess the impacts of a development downstream to the point where the developed property is 10% of the total drainage area, and there are no adverse impacts (i.e., stream peak discharge increases), then it is very unlikely that there will be significant increases in flooding problems further downstream. For example, for a 10-acre site, the assessment would have to take place down to a point where the total accumulated drainage area is 100 acres.

While this assessment does require some additional labor on the part of the design engineer, it allows smart stormwater management within a watershed. The assessment provides the developer, Knox County and downstream property owners with a better understanding (and corresponding documentation) of the potential downstream impacts of development. In turn, this information identifies those developments for which waivers or reductions in the flood protection requirements may prove beneficial.

2.4.3.2 Regulations and Policies

Regulations and policies pertaining to the downstream impact analysis are listed below.

1. In accordance with Minimum Standard #6 (Chapter 1), downstream impact analysis shall be required for all developments and redevelopments for which a stormwater management plan is required. The analysis shall determine if the proposed development or redevelopment causes an increase in peak discharge as compared to pre-development runoff rates for the same site, or has the potential to cause downstream channel and streambank erosion. This analysis must be done for the 2-year, 10-year, 25-year and 100-year return frequency, 24-hour duration storm events, at the outfall(s) of the site, and at each downstream tributary junction and each public or major private downstream stormwater conveyance structure to the point(s) in the stormwater system where the area of the portion of the site draining into the system is less than or equal to 10% of the total drainage area above that point.
2. If the downstream impact analysis shows that the development or redevelopment causes an increase in peak discharges, downstream flood protection shall be provided such that the calculated peak discharges for the 2-year, 10-year, 25-year and 100-year return frequency, 24-hour duration storm events after development of the site are not greater than that which would result from the same duration storms in the same downstream analysis area prior to development or redevelopment. These criteria must be applied throughout the 10% downstream analysis area
3. Downstream flood protection can be provided by downstream conveyance improvements and/or purchase of flow easements in lieu of peak discharge controls subject to prior approval by the Director and satisfaction of the following requirements:

   (1) Sufficient hydrologic and hydraulic analysis must be presented that shows that the alternative approach will offer adequate protection from downstream flooding for all potentially affected downstream property owners.

   (2) The applicant is responsible for providing to the County sufficient technical material for submittal and approval of any necessary CLOMR prior to construction, and a LOMR upon completion of construction. The applicant is also responsible for payment of any expenses or fees associated with preparation of such technical material and with submittal and approval of a CLOMR and/or LOMR.

   (3) The applicant is responsible for all State and Federal permits that may be applicable to the site including TDEC NPDES and ARAP permits, US Army Corps of Engineers Section 404 permits, and TVA Section 26A permits.
4. Developments and redevelopments that do not cause an increase in peak discharges are not exempt from conformance with the minimum standards for water quality treatment (WQv) and channel protection (CPv), presented earlier in this chapter.
5. The downstream analysis should be performed after any WQv credits for better site design practices have been taken into consideration in the calculation of peak discharges leaving the site. While there are no credits for flood protection criteria, the use of better site design practices will inherently reduce runoff volumes and potentially reduce post-development peak discharges, both on-site and downstream of the site.
6. The data and results of the downstream analysis must be presented to Knox County Engineering as part of the stormwater management plan.

2.5 Stormwater Conveyance Design

2.5.1 General Criteria

The components of the stormwater conveyance system, excluding the treatment and flood control facilities discussed in the previous sections, include pipe systems, culverts, open channels, and bridges. Design criteria that are specific to each of these components are presented in the following sections. General design criteria that are applicable to all of these components are presented below.

1. The design of the stormwater system, excluding stormwater management facilities for water quality treatment, channel protection, and overbank, extreme, and downstream flood protection shall be based on the 25-year frequency storm event unless otherwise specified by the Director. This criterion shall be applied to both closed conduit and open channel components. Minor systems that discharge to sinkholes must be designed to safely carry the 100-year frequency storm event.
2. All drainage systems shall be designed to insure that no habitable finished floor elevations are flooded for the 100-year frequency storm, and that no structures are located within the vertical projection of the 10-year floodplain line (i.e. located within the 10-year floodplain). Pipes and culverts designed for a 100-year storm shall be constructed of reinforced concrete if such pipes or culverts lie in public lands or easements.
3. Off-site runoff must be taken into account in the design of stormwater components if such runoff could affect the area that the stormwater system is serving.
4. Pipes or culverts that carry public water under pavement surfaces, and any pipe, culvert, or drainage system dedicated to Knox County, a private individual or a Homeowners' Association, whether inside or outside the right-of-way, can be constructed of reinforced concrete, high-density polyethylene (HDPE) or corrugated metal, subject to the approval of the Director. It shall be the responsibility of the property owner to provide all necessary design, data, and installation details for construction to ensure failure will not occur, and prevent flooding or potential property damage on adjacent properties or rights-of-way.

2.5.2 Pipe Systems

Stormwater pipe systems, also called storm drains, are pipe conveyances that are designed to collect and transport surface stormwater through drainage inlets and convey that water through closed conduits to outfalls at structural stormwater BMPs and receiving waters. The conduit system is comprised of different lengths, material types, shapes, and sizes of storm drain pipes which are connected by appurtenant structures such as manholes, junction boxes, or other miscellaneous structures. Stormwater pipe systems are sometimes referred to as storm sewers. To some people, such terminology implies that the stormwater system is the same as the wastewater (i.e., sewage) system. It is important in Knox County not to confuse the two systems: the stormwater system collects and transports stormwater drainage only, while sewage is carried via a different closed conduit system.

To the degree feasible, Knox County encourages the use of natural drainageways and/or properly vegetated open channels for stormwater runoff conveyance. Prior to design of a new development or redevelopment site, the use of the better site design practices (and corresponding site design credits) that are discussed in Chapter 5 of this manual should be considered to reduce the overall length of a piped stormwater conveyance system. However, pipe drain systems are necessary in many areas to ensure the safe collection and conveyance of stormwater away from habitable structures and streets. Pipe systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainage ways and/or vegetated open channels is not feasible.

Piped stormwater systems should be designed to ensure that storms in excess of pipe design flows can be safely conveyed without damaging structures or flooding major roadways near to the system and downstream. This is often done through design of both a major and minor drainage system. The minor (piped) system carries the mid-frequency design flows while larger runoff events may flow across lots and along streets as long as it will not cause property damage or impact public safety.

Pipe systems for stormwater runoff must be designed and constructed in accordance with the criteria listed below. Guidance on the design of storm drain pipe systems is given in Volume 2, Chapter 7.

1. Pipe systems serving local, collector and arterial streets must keep one ten (10) foot lane of traffic open in each direction for the 25-year design storm, and the 100-year storm shall be contained within the right-of-way. In pipe systems serving local roads, a ten (10) foot lane is allowed at inlets.
2. The minimum easement width for public piped stormwater systems that are located less than twelve (12) feet below the ground surface shall be twenty (20) feet. For piped stormwater systems that are located twelve (12) feet or more below the ground surface, the minimum easement width shall be thirty (30) feet.
3. The minimum acceptable diameter for any public storm drain is fifteen (15) inches or equivalent arch pipe.
4. When connecting into an existing storm drain system, the existing storm drain system shall be analyzed to determine available capacity.
5. New storm drains and manholes shall not be located under existing or future curb and gutter or sidewalk, whenever possible.
6. For ordinary conditions, storm drain pipes should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The Manning Formula is recommended for capacity calculations.
7. The minimum desirable physical slope should be 0.5% for concrete or smooth wall plastic (HDPE) and 1.0% for corrugated metal pipe (CMP), or the slope that will produce a velocity of 3.0 feet per second when the storm sewer is flowing full, whichever is greater.
8. The hydraulic grade line for the 25-year design storm for any piped stormwater system shall remain below the elevation of the ground surface.

2.5.3 Roadway Culverts

A culvert, sometimes called a cross drain, is a short, closed (covered) conduit that conveys stormwater runoff under an embankment, usually a roadway. The primary purpose of a culvert is to convey surface water. Culverts may also be designed to restrict flow and reduce downstream peak flows. On a development site, culverts are typically aligned with ditches, swales, and open channels which serve as primary drainageways that carry stormwater to more regional stormwater collection systems. In addition to the hydraulic function, a culvert must also support the embankment, roadway, or other structure under which it lies, and protect traffic and adjacent property owners from flood hazards.

Culvert design is influenced by purpose, hydraulic efficiency, site topography, effects on adjacent property, and cost. The most critical aspect of culvert design is the determination of stable and predictable performance of the culvert during all possible flows. This can be best determined when the type of flow (i.e., weir, orifice, or pipe) is known. The relationship between head and discharge can be determined using equations for weir flow, orifice flow or pipe flow.

Culverts must be designed and constructed in accordance with the criteria listed below. Further guidance on culvert design is contained in Volume 2, Chapter 7.

1. All culverts shall be designed for the 25-year design storm. The design engineer shall ensure that the culvert does not cause flooding of nearby structures in the 100-year design storm.
2. All culverts shall be hydraulically designed to determine whether inlet and outlet control conditions govern for the design storm discharge(s).
3. Culverts shall be located and designed to present a minimum hazard to traffic, persons, and property. Projecting ends shall not be permitted for culverts intended to become public.
4. Survey and resource information should include topographic features, channel characteristics, aquatic life, riparian habitat, high-water information, existing structures, and other related site specific information, as applicable.
5. Roadway culverts shall be designed to accommodate debris or proper provisions shall be made for debris maintenance. Where practicable, some means shall be provided for personnel and equipment access to facilitate maintenance.
6. Material selection shall include consideration of service life, hydraulic efficiency, and maintenance and shall not be made using initial cost as the sole criteria.
7. Low water or at-grade dip crossings of FEMA designated/mapped washes or other riverines are not permitted for public or private roadways which serve as the primary access to a development or single family residence.
8. Culvert or bridge crossings of FEMA designated/mapped washes shall be analyzed with HEC-2 Water Surface Profiles, HEC-RAS, or a pre-approved equal model. It must be demonstrated and certified by the engineer that there will be no increases on the base flood elevations(s) and/or limits upstream or downstream of the crossing.
9. Performance curves shall be developed for all public culverts for evaluating hydraulic capacity versus various headwater depths, outlet velocities, and scour depths.
10. The culvert length and slope shall be chosen to approximate existing topography, and to the degree practicable, the culvert shall be aligned with the channel bottom and the skew angle of the watercourse. Multiple barrel culvert crossings should fit onto the natural channel cross-section with minimal widening of the channel so as to avoid conveyance loss and sediment deposition.
11. Multiple barrel culverts shall be avoided, if practical, where the approach velocity is high, particularly supercritical, to avoid adverse hydraulic jump effects.
12. The minimum velocity through a culvert should be three (3) feet per second for the 1-year storm.

2.5.4 Open Channels

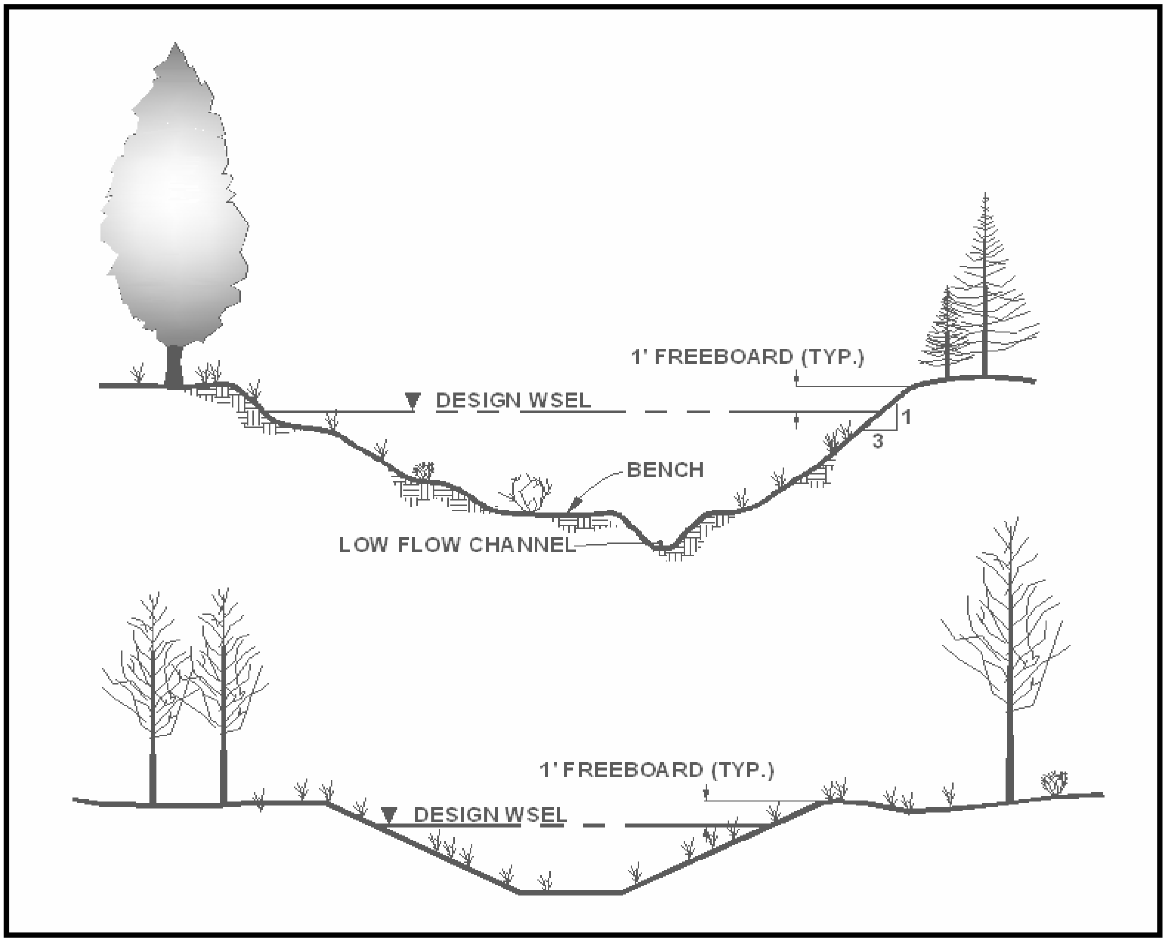
The design of open channel systems, particularly for development sites utilizing better site design, is an integral part of an overall drainage plan. An open channel is defined as a conveyance in which water flows with a free surface. Open channels can be either natural or artificial. Natural channels will typically consist of a compound cross section comprised of a low flow channel and the adjacent overbank floodplain. Artificial channels typically include roadside channels, irrigation ditches, and swales which have a general geometric cross section and can be either lined or unlined. An example of a typical compound cross-section channel and a typical trapezoidal channel is presented in Figure 2-5.

The three main classifications of open channel types according to channel linings are vegetated, flexible and rigid.

2.5.4.1 Vegetated Linings

Vegetated linings include grass with mulch, sod and lapped sod, and wetland channels. Vegetation, where practical, is the most desirable lining for an artificial channel because it can provide stability for the channel body, consolidate the soil mass of the bed, prevent erosion from the channel surface, and provide habitat and water quality benefits. Chapters 4 and 5 of Volume 2 of this manual provide guidance for the design of enhanced swales and grass channels for water quality treatment purposes. This section provides guidance solely for the purpose of stormwater conveyance.

Figure 2-5. Typical Channel Sections
(top - compound x-section, bottom- trapezoidal x-section)



Conditions under which the use of vegetated linings may not be acceptable include, but are not limited to areas where:

- high flow velocities are anticipated;
- a permanent or semi-permanent standing pool of water is anticipated;
- water will flow continuously (e.g., a conveyance channel that also serves as a landscaped waterway for a continuously flowing waterfall or pond);
- regular, necessary maintenance to prevent the growth of undesirable vegetation will not be available;

- there is a lack of nutrients and/or inadequate topsoil to properly sustain the vegetated lining; or
- there is excessive shade.

Proper seeding, mulching and soil preparation are required during construction to assure establishment of healthy vegetation. Long-term regular maintenance is necessary to ensure the long-term proper operation and stability of the channel.

2.5.4.2 Flexible Linings

Rock riprap, including rubble, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance. However, they may require the use of a filter fabric depending on the underlying soils, and the growth of grass and weeds may present maintenance problems.

2.5.4.3 Rigid Linings

Rigid linings are generally constructed of concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and channel headcutting.

2.5.4.4 Design Criteria

The principles of open channel flow are the same regardless of the channel type. Flow classifications are generally categorized as steady or unsteady, uniform or varied, and subcritical or supercritical. Open channels must be designed and constructed in accordance with the criteria listed below. Further guidance on open channel design is contained in Volume 2, Chapter 7.

1. Open channels shall be designed to follow natural drainage alignments whenever possible.
2. All channels which are to be maintained by Knox County must be dedicated to the County either in fee title or granted as a drainage easement. Vegetated channels that are eligible to gain water quality volume (WQv) credits for stormwater treatment must be granted to the County as a water quality easement.
3. Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 12 to 1, or with compound cross sections.
4. Channel side slopes shall be physically stable throughout the entire length and side slope shall depend on the channel material. A maximum of 2:1 should be used for channel side slopes, unless otherwise justified by calculations. Roadside ditches should have a maximum side slope of 3:1.
5. Trapezoidal cross sections are preferred over triangular shapes for artificial channel designs.
6. The design of artificial channels shall consider the frequency and type of maintenance required and allow for access of maintenance equipment.
7. For vegetative channels, flow velocities within the channel should not exceed the maximum permissible velocities stated in Chapter 7.
8. Channel banks shall be left in a stabilized condition upon completion of the project. No actively eroding, bare or unstable banks shall remain unless the Director has determined there is no better alternative.
9. If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions to the extent practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated. Unless proper authorization is obtained from Knox County and the adjacent property owner(s), open channels must enter and exit a site where the

channel historically flows. Knox County is not responsible for obtaining any State and/or Federal permits that may be applicable to channel relocation on a development or redevelopment site.

2.5.5 Outlet Protection

Storm system conveyance outlets, whether open channels or pipe systems, are critical locations of erosion potential. High exit velocities and flow expansion turbulence often result in local scour, channel degradation, and conduit failure. Often, the stormwater transported by man-made conveyances reaches velocities that exceed the capacity of the receiving channel or area to resist erosion. In order to prevent scour at stormwater outlets, protect the outlet structure and minimize the potential for downstream erosion, a flow transition structure (energy dissipater) is needed to absorb the initial impact of flow and reduce the speed of the flow to a non-erosive velocity. Often, such dissipaters are relatively inexpensive to install, such as a rip rap apron, a stilling basin or a baffled outlet.

Design guidance on energy dissipaters is presented in Volume 2, Chapter 7. Design criteria for outlet protection are as follows:

1. Energy dissipaters shall be utilized wherever the velocity of flows leaving a stormwater management facility exceed the erosion velocity of the downstream channel system. When utilized, such devices shall provide uniform redistribution (spreading out) of the flow without creating excessive turbulence in order to protect downstream areas from erosion.
2. Riprap basins, stilling basins, or concrete energy dissipaters can be utilized to reduce high velocity outlet flows to within acceptable limits.

2.5.6 Gutters & Inlets

The design of the roadway drainage system is essential to traffic safety and roadway level of service. Excess water on the roadway can be hazardous to not only vehicular traffic but pedestrians as well. Poor roadway drainage can increase the potential for hydroplaning, limit visibility due to excessive splashing and spray, and cause loss of steering control when puddles are encountered.

Street drainage requires consideration of surface drainage, gutter flow, and drainage inlet capacity. The design of these components is dependent upon the design frequency and the allowable spread of stormwater on the pavement surface. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity.

General design criteria for gutters and inlets are provided below. Design specifications are provided in greater detail in Volume 2, Chapter 7.

1. Street drainage and roadways shall be designed so as to maintain the natural drainage patterns existing prior to development, whenever possible.
2. The street section shall be designed to convey local runoff only and shall not be used as major stormwater carriers for contributing watersheds.
3. Drainage facilities shall be installed to convey runoff under streets or street grades shall be set so diversion of runoff or ponding will not occur on adjacent properties.
4. Street slopes (longitudinal and transverse) and curb heights shall not be increased to create more carrying capacity for runoff. Curb overtopping is not permitted for the specified design storm.

5. Drainage facilities shall be placed to intercept runoff from sources outside the street section to avoid significant concentrated flows onto and over sidewalks or curb and gutter.
6. In all cases, street drainage shall be confined to the public right-of-way. Runoff which leaves the right-of-way shall do so in a controlled manner and shall be contained in appropriate right-of-way or drainage easement.

2.6 Bridge Requirements

A bridge is defined as a structure that transports vehicular traffic over a watercourse or other obstruction, including the approach roadway over the floodplain and the relief openings. A bridge typically has a minimum span length of twenty (20) feet. Bridge hydraulics is very important in determining water surface profiles for use in flood studies, stream design, stream stability and scour evaluations. General design criteria for bridges are provided below. Design specifications are provided in greater detail in Volume 2, Chapter 7.

1. Bridge analysis, design and construction shall conform to the pertinent floodplain development regulations that are contained in the Knox County Stormwater Management Ordinance and the Knox County Flood Protection Ordinance.
2. The final design selection for any bridge shall consider the maximum backwater allowed by the Knox County floodplain regulations unless exceeding the limit can be justified by special hydraulic conditions. Backwater shall not increase flood damage to upstream property.
3. The final design shall not significantly alter the flow distribution in the floodplain and whenever possible, bridge structures should be designed so that there is little or no disturbance to the flow. Velocities through the structure shall not damage either the roadway facility or increase damages to adjacent property.
4. Degradation or aggregation of the watercourse as well as contraction and local scour shall be estimated, and appropriate positioning of the foundation, below the total scour depth if practicable, shall be included as part of the final design.
5. Bridges should be designed to minimize disruption of ecosystems and values unique to the floodplain and channel being crossed.

References

ARC. *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

Technology Acceptance and Reciprocity Partnership (TARP). *Protocol for Stormwater Best Management Practice Demonstrations*. 2001.

This page left intentionally blank

Table of Contents

CHAPTER 3 — STORMWATER HYDROLOGY

Chapter 3 - List of Figures

Chapter 3 - List of Tables

CHAPTER

3

STORMWATER HYDROLOGY

3.1 Introduction to Hydrologic Methods

Hydrology is the science dealing with the characteristics, distribution, and movement of water on and below the earth's surface and in the atmosphere. Hydrology in this manual shall be limited to estimating flow peaks, volumes, and time distributions of stormwater runoff. The analysis of these parameters is fundamental to the design of stormwater management facilities, such as storm drainage systems and structural stormwater BMPs. In the hydrologic analysis of a development site, there are a number of variable factors that affect the nature of stormwater runoff from the site. Some of the factors that need to be considered include:

- rainfall amount and storm distribution;
- drainage area size, shape and orientation;
- ground cover and soil type;
- slopes of terrain and stream channel(s);
- antecedent moisture condition;
- storage potential (floodplains, ponds, wetlands, reservoirs, channels, etc.);
- watershed development potential; and
- characteristics of the local drainage system.

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

- Rational Method;
- United States Geological Survey (USGS) and Tennessee Valley Authority (TVA) Regression Equations;
- Soil Conservation Service (SCS) Unit Hydrograph Method;
- Clark Unit Hydrograph;
- Water Quality Volume (WQv) Calculation; and
- Water Balance Calculations.

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

Table 3-1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications, relevant to regulations and policies in Knox County. Table 3-2 provides some limitations on the use of several methods.

Table 3-1. Design Applications for Recommended Hydrologic Methods

Analysis or Design Application	Manual Section	Rational Method	USGS Equations	SCS Method	Clark Unit Hydrograph	Water Quality Volume	TVA Equations
Water Quality Volume (WQv)	2.2.3					✓	
Channel Protection Volume(CPv)	2.3			✓			
Overbank Flood Protection(Qp_2, Qp_{10},Qp_{25})	2.4.1			✓			
Extreme Flood Protection (Qp_{100})	2.4.2			✓			
Storage Facilities	3.2			✓			
Outlet Structures	3.3			✓			
Gutter Flow and Inlets	7.6	✓					
Storm Drain Pipes	7.2	✓	✓	✓			✓
Culverts	7.3	✓	✓	✓			✓
Small Ditches	7.4	✓	✓	✓			✓
Open Channels	7.4		✓	✓			✓
Energy Dissipation	7.5		✓	✓			✓
Flood Studies	8.4.3		✓	✓	✓		✓

95

Table 3-2. Constraints on Using Recommended Hydrologic Methods

Method	Size Limitations[1]	Comments
Rational	0 – 5 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems. <u>Not to be used for storage design.</u>
USGS Rural	0.36 mi^2 to 21,400 mi^2	Method can be used for estimating peak flows for all design applications in rural areas.
USGS Urban	0.21 mi^2 to 24.3 mi^2	Method can be used for estimating hydrographs for all design applications in urban areas.
TVA	> 0.36 mi^2	Method can be used for estimating peak flows for storm system design applications such as culverts, channels, etc.
SCS[2,3]	0 – 2000 acres	Method can be used for estimating peak flows and hydrographs for all design applications.
Clark[2]	See Comments	Method may not be applicable to very large drainage basins. Large drainage basins may need to be subdivided to overcome limitations of this method.
Water Quality	Limits set for each Structural Control	Method used for calculating the WQv

1 - Size limitation refers to the drainage basin for the stormwater management facility (e.g., culvert, inlet).
2 - There are many readily available programs (such as HEC-1) that utilize this methodology.
3 - 2,000-acre upper size limit applies to single basin simplified peak flow only.

In general:

- the Rational Method is recommended for small, highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters; and
- the USGS regression equations are recommended for drainage areas with characteristics within the ranges given for the equations. The USGS equations should be used with caution when there are significant storage areas within the drainage basin or where other drainage characteristics indicate that general regression equations might not be appropriate; and
- the TVA regression equations are used for stormwater system design (discussed in Chapter 7), choosing the more conservative solution from between the results of the applicable USGS regression equation and the TVA regression equation.

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

3.1.1 Symbols and Definitions

To provide consistency within this section, the symbols listed in Table 3-3 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this manual, the symbol will be defined where it occurs in the text or equations.

Table 3-3. Symbols and Definitions for Stormwater Runoff

Symbol	Definition	Units
A	Drainage area	acres (or mi^2)
B_f	Baseflow	cfs
C	Runoff coefficient	-
C_f	Frequency factor	-
CN	SCS-runoff curve number	-
CPv	Channel protection volume	acre-feet
d	Time interval	hours
D_{wq}	Water quality runoff depth	in
E	Evaporation	ft
E_t	Evapotranspiration	ft
F_p	Pond and swamp adjustment factor	-
G_h	Hydraulic gradient	ft/ft
I or i	Runoff intensity	in/hr
IA	Percent of impervious cover	%
I	Infiltration	ft
I	Inflows	cfs
I_a	Initial abstraction from total rainfall	in
k_h	Infiltration rate	ft/day
L	Flow length	ft
n	Manning roughness coefficient (Manning's "n")	-
O_f	Overflow	acre-feet
O	Outflows	cfs
P	Accumulated rainfall	in
P_2	2-year, 24-hour rainfall	in
P_w	Wetted perimeter	ft

Symbol	Definition	Units
PF	Peaking factor	-
Q	Rate of runoff or depth of runoff	cfs or inches
Qp	Peak rate of discharge	cfs
Qp_2	2-year event peak discharge	cfs
Qp_{10}	10-year event peak discharge	cfs
Qp_{25}	25-year event peak discharge	cfs
Qp_{100}	100-year event peak discharge	cfs
Q_{wq}	Water quality peak discharge	cfs
Q_{wv}	Water quality runoff peak volume	in
q	Storm runoff during a time interval	in
q_u	Unit peak discharge	cfs (or cfs/mi^2/inch)
R	Clark watershed storage constant	hours
R	Hydraulic radius	ft
R_o	Runoff	acre-feet
Rv	Runoff coefficient	-
S	Ground slope	ft/ft or %
S	Potential maximum retention	in
S	Slope of hydraulic grade line	ft/ft
T_L	Lag time	hours
T_p	Time to peak	hours
T_t or t_t	Travel time	min or hours
t	Time	min
Tc	Time of concentration	min
TIA	Total impervious area	%
V	Velocity	ft/s
V	Pond volume	acre-feet
WQv	Water quality volume	acre-feet
W_s	Average ground surface slope as a percentage	%

3.1.2 Rainfall Estimation

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

Duration (hours) – Length of time over which rainfall (storm event) occurs;

Depth (inches) – Total amount of rainfall occurring during the storm duration; and

Intensity (inches per hour) – Rate of rainfall or depth divided by the duration

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

Exceedance Probability – Probability that a storm event having the specified duration and volume will be exceeded in one given time period, typically 1-year.

Return Period – Average length of time between events that have the same duration and volume.

Thus, if a storm event with a specified duration and volume has a 1% chance of occurring in any given year, then it has an exceedance probability of 0.01 and a return period of 100-years. A design storm event over 24-hours with a 1% chance of occurring in any given year is often referred to as the 100-year, 24-hour storm. This design storm would be developed based on assumptions regarding intensity and distribution of the storm over the specified timeframe (24-hours for this scenario). Therefore, a design storm event is used to estimate actual storm events even though it would be very unlikely that an actual storm event would match up with all of the design storm event assumptions.

Rainfall intensities for Knox County are provided in Table 3-4 and should be used for all hydrologic analysis. The sources of the values in this table are the Weather Bureau Technical Papers TP-25 and TP-40 (Hershfield, 1961) and National Weather Service publication Hydro-35 (NOAA, 1977). The intensity values have been adjusted to produce smooth intensity-duration-frequency (IDF) curves and cumulative rainfall distributions. Table 3-5 shows the rainfall depths for hypothetical storm events.

Figure 3-1 shows the IDF curves for Knox County for the 1, 2, 5, 10, 25, and 100-year, 24-hour storms. These curves are plots of the tabular values. No values are given for times less than 5 minutes.

Table 3-4. Intensity-Duration-Frequency Curve Data

(Sources: Hershfield, 1961; NOAA, 1977)

ARI[1] (years)		24-Hour Precipitation Frequency Estimates (inches/hour) by Return Periods					
Hours	**Minutes**	**2-year**	**5-year**	**10-year**	**25-year**	**50-year**	**100-year**
0.083	5	4.60	5.55	6.25	7.30	7.90	8.60
0.170	10	3.70	4.60	5.25	6.20	6.80	7.49
0.250	15	3.19	3.98	4.60	5.45	6.00	6.60
0.330	20	2.82	3.50	4.10	4.90	5.45	6.02
0.420	25	2.48	3.12	3.70	4.45	4.95	5.50
0.500	30	2.22	2.80	3.34	4.03	4.53	5.03
0.580	35	2.02	2.55	3.06	3.67	4.14	4.62
0.670	40	1.86	2.35	2.82	3.38	3.80	4.24
0.750	45	1.73	2.18	2.62	3.14	3.53	3.93
0.830	50	1.62	2.04	2.46	2.94	3.30	3.67
0.920	55	1.53	1.92	2.32	2.77	3.10	3.45
1.000	60	1.45	1.82	2.20	2.62	2.93	3.26
1.500	90	1.06	1.36	1.64	1.95	2.18	2.45
2.000	120	0.86	1.09	1.31	1.55	1.71	1.95
3.000	180	0.66	0.80	0.97	1.13	1.23	1.38
6.000	360	0.41	0.50	0.58	0.66	0.75	0.83
12.000	720	0.24	0.30	0.34	0.39	0.43	0.48
24.000	1440	0.14	0.17	0.20	0.23	0.25	0.27

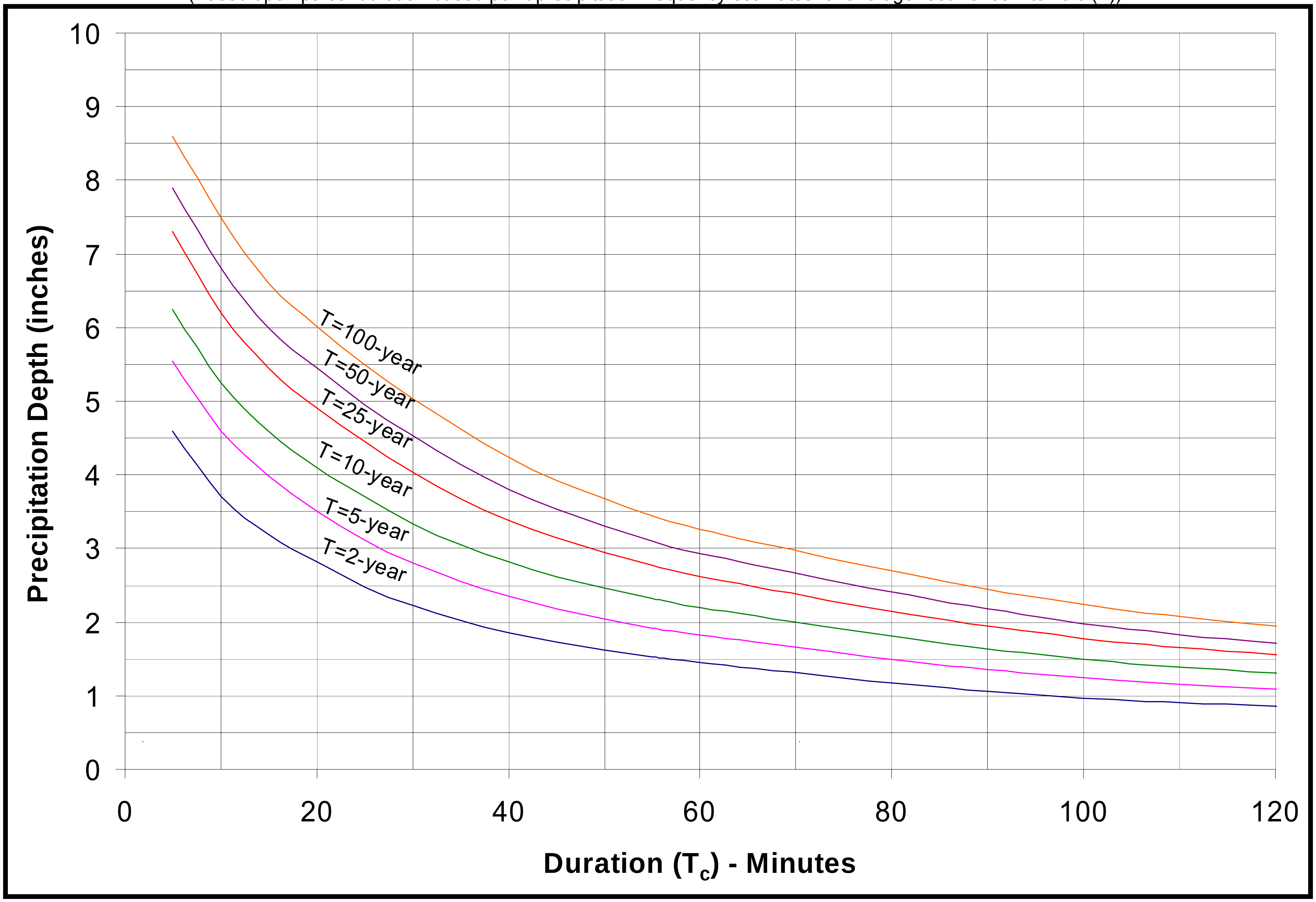
1 - ARI= Average Recurrence Interval

Table 3-5. Rainfall Depths for Hypothetical Storm Events

Rainfall Depths for Hypothetical Storm Events	
Storm Event	**24-Hr Depth (in)**
1-year	2.5
2-year	3.3
5-year	4.1
10-year	4.8
25-year	5.5
100-year	6.5

Figure 3-1. Intensity-Duration-Frequency-(IDF) Curves for Knox County 24-hour Storms

(Based upon partial duration-based point precipitation frequency estimates for average recurrence intervals (T))



3.1.3 Rational Method

A popular approach for determining the peak runoff rate is the Rational Formula. The Rational Method considers the entire drainage area as a single unit and estimates the peak discharge at the most downstream point of that area.

The Rational Formula follows the assumptions that:

- the rainfall is uniformly distributed over the entire drainage area and is constant over time;
- the predicted peak discharge has the same probability of occurrence (return period) as the used rainfall intensity (I);
- peak runoff rate can be represented by the rainfall intensity averaged over the same time period as the drainage area's time of concentration (Tc); and
- the runoff coefficient (C) is constant during the storm event.

When using the Rational Method some precautions should be considered:

- in determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility;
- if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), the basin should be divided into sub-drainage basins. The single equation used for the Rational Method uses one composite C and one Tc value for the entire drainage area; and,
- the charts, graphs, and tables included in this section are given to assist the engineer in applying the Rational Method. The engineer shall use sound engineering judgment in applying these design aids and shall make appropriate adjustments when specific site characteristics dictate that these adjustments are appropriate.

3.1.3.1 Application

The Rational Method can be used to estimate stormwater runoff peak flows for the design of gutter flows, drainage inlets, storm drain pipe, culverts and small ditches. It is most applicable to small, highly impervious areas. Knox County policies regarding the use of the Rational Method are as follows:

- In Knox County, the Rational Method shall not be utilized for drainage areas greater than five (5) acres.
- The Rational Method shall not be used for storage design or any other application where a more detailed routing procedure is required.
- The Rational Method shall not be used for calculating peak flows downstream of bridges, culverts or storm sewers that may act as restrictions and impact the peak rate of discharge.

3.1.3.2 Equations

The Rational Method estimates the peak rate of runoff at a specific watershed location as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration, Tc. The Tc is the time required for water to flow from the most remote point of the basin to the location being analyzed.

The Rational Method is expressed in Equation 3-1. Further explanation of each variable in the Rational Method equation is presented in Sections 3.1.3.3 and 3.1.3.4.

Equation 3-1 $$Q = CIA$$

where:

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

3.1.3.3 Runoff Coefficient

The runoff coefficient (C) is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 3-6 gives the recommended runoff coefficients for the Rational Method.

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 3-6 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as rooftops, asphalt, and concrete. The composite procedure can be applied to an entire drainage area or to typical “sample" blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

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

Using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) can in some cases yield a higher peak runoff value than by using the whole site. Peak flow calculations can be underestimated due to areas where the overland portion of flow is grassy (yielding a longer Tc).

Note that the coefficients given in Table 3-6 are applicable for storms of 5 to 10-year frequencies. Less frequent, higher intensity storms may require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright - McLaughlin Engineers, 1969). The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor Cf. The Rational Formula for major storm events now becomes:

Equation 3-2 $$Q = C_f CIA$$

C_f values are listed in Table 3-6. The product of C_f times C shall not exceed 1.0.

Table 3-6. Frequency Factors for Rational Formula

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

Table 3-7. Recommended Runoff Coefficient Values for Rational Method

Land Use	Runoff Coefficient (C) by Hydrologic Soil Group and Ground Slope											
	A			B			C			D		
	<2%	2 - 6%	>6%	<2%	2 - 6%	>6%	<2%	2 - 6%	>6%	<2%	2 - 6%	>6%
Forest	0.08	0.11	0.14	0.10	0.14	0.18	0.12	0.16	0.20	0.15	0.20	0.25
Meadow	0.14	0.22	0.30	0.20	0.28	0.37	0.26	0.35	0.44	0.30	0.40	0.50
Pasture	0.15	0.25	0.37	0.23	0.34	0.45	0.30	0.42	0.52	0.37	0.50	0.62
Farmland	0.14	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Res. 1 acre	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Res. 1/2 acre	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.46
Res. 1/3 acre	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.36	0.40	0.50
Res. 1/4 acre	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Res. 1/8 acre	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Industrial	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
Streets: ROW	0.76	0.77	0.79	0.80	0.82	0.84	0.84	0.85	0.89	0.89	0.91	0.95
Parking	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97
Disturbed Area	0.65	0.67	0.69	0.66	0.68	0.70	0.68	0.70	0.72	0.69	0.72	0.75

3.1.3.4 Rainfall Intensity (I)

The rainfall intensity (I) is the average rainfall rate in in/hr for a selected return period that is based on a duration equal to the time of concentration (Tc). Once a particular return period has been selected for design and a time of concentration has been calculated for the drainage area, the rainfall intensity can be determined from rainfall-intensity-duration data given in Table 3-4 or Figure 3-1. Calculation of Tc is discussed in detail in the next section.

3.1.3.5 Time of Concentration

Use of the Rational Method requires calculating the time of concentration (Tc) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). The basin time of concentration is defined as the time required for water to flow from the most remote part of the drainage area to the point of interest for discharge calculations. The time of concentration is computed as a summation of travel times within each flow path as follows:

Equation 3-3

$$T_C = t_{t1} + t_{t2} + t_{tm}$$

where:

Tc = time of concentration (hours)
t_t = travel time of segment (hours)
m = number of flow segments

Knox County policies regarding the calculation of Tc are as follows:

- The Tc shall be the longest sub-basin travel time when all flow paths are considered.
- The minimum Tc for all computations shall be five (5) minutes.

Time of concentration calculations are subject to the following limitations:

1. the equations presented in this section should not be used for overland (i.e., sheet) flow on impervious land uses where the flow length is longer than 50 feet; and
2. in watersheds with storm sewers, use care to identify the appropriate hydraulic flow path to estimate Tc.

Two common errors should be avoided when calculating time of concentration. First, in some cases runoff from a highly impervious portion of a drainage area may result in a greater peak discharge than the calculated peak discharge for the entire area. Second, the designer should consider that the overland flow path does not necessarily remain the same when comparing pre-development and post-development areas. Grading operations and development can alter the overland flow path and length. Selecting overland flow paths for impervious areas that are greater than 50 feet should be done only after careful consideration. For typical urban areas, the time of concentration consists of multiple flow paths including overland flow, shallow concentrated flow and the travel time in the storm drain, paved gutter, roadside ditch, or drainage channel.

Overland Flow:
Overland flow in urbanized basins occurs from the backs of lots to the street, across and within parking lots and grass belts, and within park areas, and is characterized as shallow, steady and uniform flow with minor infiltration effects. The travel time (T_t) for sheet flow over plane surfaces for distances of less than 100 lineal feet for unpaved surfaces (50 feet for paved surfaces) can be calculated using Manning's kinematic solution (Overton and Meadows, 1976), shown in Equation 3-4. Following the equation, Table 3-8 presents Manning's "n" roughness coefficients for use in Equation 3-4.

Equation 3-4

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}}$$

where:

- T_t = travel time (hours)
- n = Manning's roughness coefficient (see Table 3-8)
- L = flow length (ft)
- P_2 = 2-year 24-hour rainfall (inches)
- S = ground slope, (ft/ft)

Table 3-8. Roughness coefficients for Overland (Sheet) Flow (Manning's "n")[1]

(Source: Soil Conservation Service, 1986)

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

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

Shallow Concentrated Flow:

After a maximum of 100 feet (50 feet for paved areas), overland flow will normally become shallow concentrated flow. The average velocity of this flow can be determined from Figure 3-2, in which average velocity is a function of watercourse slope and type of channel. Equations 3-5 and 3-6 can be used to determine the average flow velocity on paved and unpaved surfaces for slopes less than the minimum slope in Figure 3-2 (0.005 ft/ft):

Equation 3-5 Unpaved $V = 16.13(S)^{0.5}$

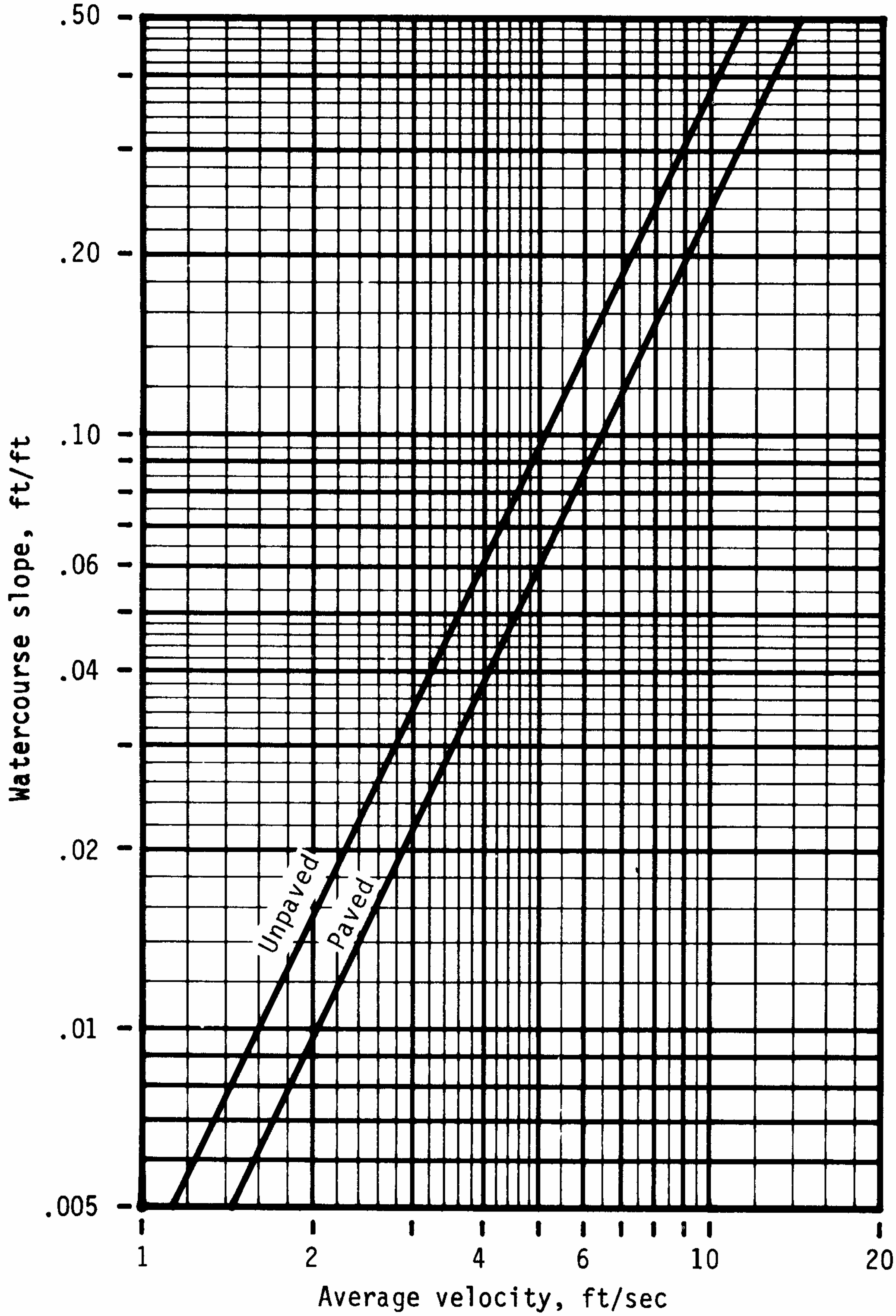
Equation 3-6 Paved $V = 20.33(S)^{0.5}$

where:

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

After determining average velocity, use Equation 3-7 to estimate travel time for the shallow concentrated flow segment.

Figure 3-2. Average Velocities - Shallow Concentrated Flow
(Source: Soil Conservation Service, 1986)



Equation 3-7 $$T_t = \frac{L}{60V}$$

where:

T_t = travel time (min)
L = reach length (ft)
V = velocity in reach (ft/sec)

Paved Gutter and Open Channel Flow:
The travel time within the storm drain, gutter, swale, ditch, or other drainage way can be determined through an analysis of the hydraulic properties of these conveyance systems using Manning's equation (Equation 3-8).

Equation 3-8 $$V = \frac{1.49(R)^{2/3}(S)^{1/2}}{n}$$

where:

V = average velocity (ft/s)
R = hydraulic radius (feet) and equals A/P_w
A = cross sectional flow area (sq.ft.)
P_w = wetted perimeter (feet)
S = slope of energy grade line (may be estimated as channel slope, ft/ft), and
n = Manning's roughness coefficient for open channel flow

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, where channels have been identified by TDEC or Knox County, where blue lines (indicating streams) appear on USGS quadrangle sheets, or where defined channels are identified by field inspection or topographic map indications. Equation 3-8 or water surface profile information can be used to estimate average flow velocity. Average flow velocity for travel time calculations is usually determined for bankfull elevation assuming low vegetation winter conditions.

Values of Manning's "n" for use in Equation 3-8 may be obtained from standard design textbooks such as Chow (1959) and Linsley et al. (1949). These values are also included as a part of the discussion of Manning's equation within Chapter 7 of this Manual, *Stormwater Drainage System Design*.

SCS Lag equation:
Additionally, the SCS lag equation is an acceptable method for calculating the time of concentration (Tc) based on watershed lag time (T_L). The SCS lag equation shall be used only for predeveloped rural areas with less than 10% imperviousness. It shall never be used in postdeveloped time of concentration calculations. T_L is defined as the time between the center of mass of excess rainfall to the time of peak runoff (similar to an average flow time for a small homogeneous area). The following equations can be used to determine Tc:

Equation 3-9 $$T_c = 1.67T_L$$

where:

T_C = time of concentration of overland flow portion of flow path (hours)
T_L = NRCS lag time (hours)

Equation 3-10 $$T_L = \frac{L^{0.8}(S+1)^{0.7}}{1900W_s^{\ 0.5}}$$

where:

T_L = SCS lag time (hours)
L = flow length for overland flow over the surface (feet)
S = potential maximum soil retention (inches) = 1000/CN-10
W_s = average ground surface slope as a percentage (%)

Example 3-1. Calculation of Peak Discharge Using Rational Method

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

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

- Average overland slope = 2.0% = 0.02 ft/ft
- Length of overland flow = 50 ft
- Length of main basin channel = 2,250 ft
- Hydraulic Radius R taken from channel dimensions = 1.62
- Slope of channel = 0.018 ft/ft = 1.8%
- Roughness coefficient (n) of channel was estimated to be 0.040
- Roughness coefficient (n) of overland flow area was estimated to be 0.090
- From existing land use maps, land use for the drainage basin was estimated to be:
  - Residential ($^1/_2$ acre) - 80%
  - Pasture - sandy soil, 3% slope - 20%
- From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be: lawn – silty clay soil, 2% slope
- 2-year, 24-hour rainfall = 3.3 inches

Step 1: The overland flow time can be calculated using Equation 3-4:

$T_t = 0.007[(0.090)(50)]^{0.8}/(3.30)^{0.5}(0.02)^{0.4}$
$= 0.061$ hrs $= 3.7$ minutes

Step 2: Calculate the channel flow time by first calculating the main channel velocity using Equation 3-10:

$V = 1.49(1.62)^{2/3}(0.018)^{1/2}/(0.040)$
$= 6.9$ ft/s

The flow time is calculated using Equation 3-9:

$T_t = 2250/[(6.9)(60)]$
$= 5.4$ minutes

Step 3: Calculate Tc.

$Tc = 3.7 + 5.4 = 9.1$ min (use 9 min)

Step 4: From Table 3-4, use interpolation to calculate the intensity for a duration equal to 9 minutes,

$I_{25} = 6.42$ in/hr

Step 5: A weighted runoff coefficient (C) for the total drainage area is determined below by utilizing the C values from Table 3-6. Assume the silty clay soil specified is classified in hydrologic soil group C. See table on next page.

Step 6: The Rational Method estimate of peak runoff for a 25-yr design storm for the given basin is:

$Q_{25} = C_f CIA = (1.10)(0.364)(6.42)(23)$
$= 59.1$ cfs

1	2	3	4
Land Use	**Percent of Total Land Area**	**Runoff Coefficient**	**Weighted Runoff Coefficient**[1]
Residential ($^1/_2$ acre)	80	0.35	0.280
Pasture	20	0.42	0.084
Total Weighted Runoff Coefficient = 0.364			

1 - Column 4 equals Column 2 multiplied by Column 3.

3.1.4 Regression Methods

3.1.4.1 USGS Regressions Equations

Two sets of USGS Regression Equations are presented in this section. Table 3-9 presents urban equations intended for use in the preliminary design of culverts across streams that are depicted as blue lines (i.e., waters of the state) on USGS quadrangle maps.

Table 3-9. USGS Urban Peak Flow Regression Equations
(Source: United States Geological Survey, 1984)

Frequency[1]	Equations[2, 3]
2-year	$Q_2 = 1.76A^{0.74} IA^{0.48} P^{3.01}$
5-year	$Q_5 = 5.55A^{0.75} IA^{0.44} P^{2.53}$
10-year	$Q_{10} = 11.8A^{0.75} IA^{0.43} P^{2.12}$
25-year	$Q_{25} = 21.9A^{0.75} IA^{0.39} P^{1.89}$
50-year	$Q_{50} = 44.9A^{0.75} IA^{0.40} P^{1.42}$
100-year	$Q_{100} = 77.0A^{0.75} IA^{0.40} P^{1.10}$

A = drainage area, mi^2
IA = total impervious area, % (e.g., 30% would be input as 30 not 0.30)
P = 2-year, 24-hour rainfall (inches) = 3.30 inches for Knox County
1 - Extrapolation is required to determine the 500-year peak flow.
2 - These equations are applicable for drainage areas between 0.21 mi^2 and 24.3 mi^2.
3 - These equations are applicable for impervious areas between 4.7% and 74%.

Table 3-10 presents USGS rural equations and USGS urban “three parameter” estimating equations (USGS, 1983). These equations were utilized by TVA to calculate peak discharges for the 2006 Flood Insurance Study of Knox County, Tennessee (FEMA, not yet dated). The equations presented in Table 3-10 must be used for preparation of new, and/or updating of existing, flood elevation studies in Knox County. *Note: the designer may be required to utilize an existing HEC-1 model as opposed to using the equations presented in Table 3-10 to prepare or modify a flood elevation study in Knox County. Consult Knox County Engineering prior to beginning a flood elevation study to determine the appropriate peak discharge calculation method.*

The three parameter equations require the determination of the basin development factor (BDF) and equivalent rural discharge (RQ_x) prior to use of the equations. These parameters are discussed in the following paragraphs.

Basin Development Factor (BDF): The BDF is a somewhat subjective parameter that is intended to account for the effects of urbanization in a watershed (USGS, 1984). The BDF index range from a minimum value of zero for a drainage area with very little development, to a maximum value of 12 for a drainage area with a high level of development. Four urbanization factors that are considered in the development of a BDF are channel improvements, channel linings, storm drains and curbed streets. For drainage areas that have BDF values of zero, the rural regression equations should be used to determine peak discharges for flood elevation studies. The urban three parameter equations should be used for drainage areas that have a BDF that is greater than zero.

Table 3-10. USGS Rural and Urban Three Parameter Equations

(Source: United States Geological Survey, 1983)

Frequency	Rural Equations[1]	Three Parameter Equations[1, 2, 3]
2-year	$RQ_2 = 118A^{0.753}$	$Q_2 = 13.2A^{.21}(13\text{-BDF})^{-.43}RQ_2^{.73}$
5-year	$RQ_5 = 198A^{0.736}$	$Q_5 = 10.6A^{.17}(13\text{-BDF})^{-.39}RQ_5^{.78}$
10-year	$RQ_{10} = 259A^{0.727}$	$Q_{10} = 9.51A^{.16}(13\text{-BDF})^{-.36}RQ_{10}^{.79}$
25-year	$RQ_{25} = 344A^{0.717}$	$Q_{25} = 8.68A^{.15}(13\text{-BDF})^{-.34}RQ_{25}^{.80}$
50-year	$RQ_{50} = 413A^{0.711}$	$Q_{50} = 8.04A^{.15}(13\text{-BDF})^{-.32}RQ_{50}^{.81}$
100-year	$RQ_{100} = 493A^{0.703}$	$Q_{100} = 7.70A^{.15}(13\text{-BDF})^{-.32}RQ_{100}^{.82}$
500-year	$RQ_{500} = 670A^{0.694}$	extrapolation required

1 - A = drainage area, mi^2
2 - BDF = basin development factor (see discussion below)
3 - RQ_x = equivalent rural discharge for an X-year event (cfs)

When using the USGS three parameter estimating equations to update an existing flood elevation study, the nature and size of the development will determine if the BDF that was determined for the existing flood elevation study should be increased to reflect the increased urbanization of the drainage areas to the stream. Knox County Engineering should be consulted prior to peak discharge calculation to determine if existing BDF's should be increased. Consult the USGS reference document (USGS, 1984) for more information on the determination of the BDF for any one basin.

Equivalent Rural Discharge (RQ_x): The RQ_x parameter is determined using the USGS rural regression equations presented in Table 3-10.

3.1.4.2 TVA Regression Equations

TVA developed a set of regression equations in the 1970s that can be used to calculate peak discharges in Knox County. These equations, shown in Table 3-11, can be used for the preliminary design of culverts across streams that are depicted as blue lines (waters of the state) on USGS quadrangle maps.

Table 3-11. TVA Regional Regressions Relationships for Natural Streams

(Source: City of Knoxville, 2003)

Frequency[1]	Equations[2, 3]
2-year	$Q_2 = 107\ A^{.804}\ I^{0.30}$
10-year	$Q_{10} = 217\ A^{.802}\ I^{0.26}$
50-year	$Q_{50} = 344\ A^{.796}\ I^{0.22}$
100-year	$Q_{100} = 402\ A^{.796}\ I^{0.20}$
500-year	$Q_{500} = 556\ A^{.795}\ I^{0.16}$

A = drainage area, mi^2
I = percent of contributing drainage area that is impervious, %

3.1.5 SCS Hydrologic Method

The SCS* hydrologic method requires basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. However, the SCS approach is more sophisticated in that it also considers the time distribution of the rainfall and the initial rainfall losses due to interception and depression storage. A typical application of the SCS method includes the following basic steps:

1. determination of curve numbers that represent different land uses within the drainage area;
2. calculation of time of concentration to the study point;
3. use of the SCS Type II rainfall distribution in this area; and
4. use of the unit hydrograph approach to develop the hydrograph of direct runoff from the drainage basin.

The SCS method can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows. The SCS method can be used for most design applications, including storage facilities and outlet structures, storm drain systems, culverts, small drainage ditches and open channels, and energy dissipators.

3.1.5.1 Equations and Concepts

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

The following discussion outlines the equations and basin concepts used in the SCS method.

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

<u>Rainfall</u> - The SCS method applicable to Knox County is based on a storm event that has a Type II time distribution. This distribution is used to distribute the 24-hour volume of rainfall for the different storm frequencies.

<u>Rainfall-Runoff Equation</u> - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The SCS runoff equation (Equation 3-12) is used to estimate direct runoff from 24-hour or 1-day storm rainfall.

---

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

Equation 3-12

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

where:

- Q = accumulated direct runoff (in)
- P = accumulated rainfall or potential maximum runoff (in)
- I_a = initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (in)
- S = potential maximum soil retention (in) = (1000/CN)-10

An empirical relationship used in the SCS method for estimating I_a is presented in Equation 3-13. This is an average value that could be adjusted for flatter areas with more depressions if there are calibration data to substantiate the adjustment.

Equation 3-13

$$I_a = 0.2S$$

Substituting 0.2S for I_a in Equation 3-12, the SCS rainfall-runoff equation becomes Equation 3-14.

Equation 3-14

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

where:

- S = (1000/CN) - 10
- CN = SCS curve number

Figure 3-3 presents a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurs on a watershed with a curve number of 85.

Equation 3-14 can be rearranged so that the curve number can be estimated if the rainfall and runoff volume are known, as shown in Equation 3-15 (Pitt, 1994).

Equation 3-15

$$CN = \frac{1000}{10 + 5P + 10Q - 10\left(Q^2 + 1.25QP\right)^{1/2}}$$

where:

- CN = SCS curve number
- P = accumulated rainfall or potential maximum runoff (in)
- Q = accumulated direct runoff (in). Can be Q_{wv}, Q_2, Q_{10}, etc…

3.1.5.2 Runoff Factor/Curve Numbers

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The SCS method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the SCS has divided soils into four hydrologic soil groups (HSG).

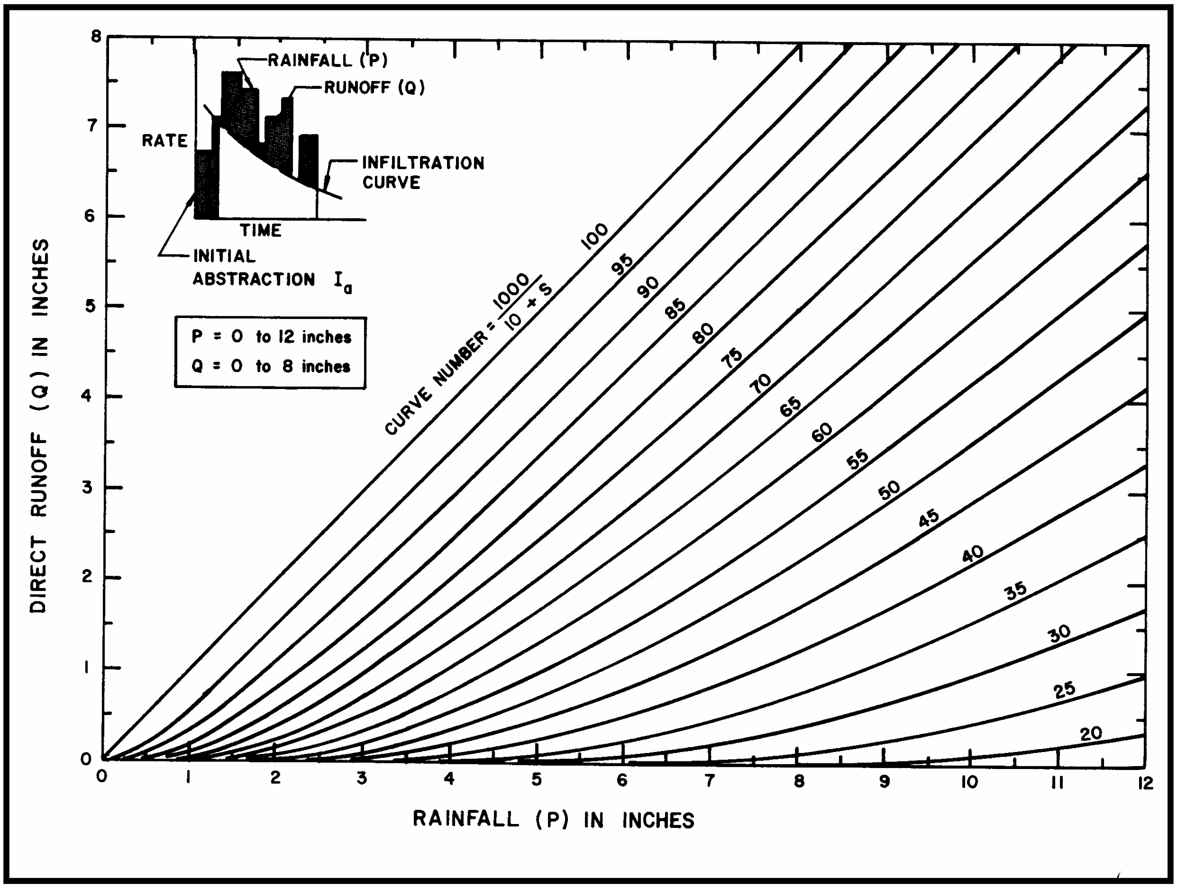
<u>Group A</u> Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

<u>Group B</u> Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils

with moderately fine to moderately coarse textures.

Figure 3-3. SCS Solution of the Runoff Equation

(Source: Soil Conservation Service, 1986)



<u>Group C</u> Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

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

A list of soils throughout Knox County and their hydrologic classification can be found in the reference SCS, 1986. Soil survey maps can be obtained from the local Natural Resources Conservation Service or the Knox County Soil Conservation office for use in estimating soil type.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions. Antecedent runoff conditions (ARC II) are recommended for most hydrologic analyses, except in the design of developments in sinkhole drainage areas where ARC III may be allowed. Areas with high water table conditions may want to consider using ARC III antecedent soil moisture conditions. This should be considered a calibration parameter for modeling against real calibration data. Table 3-12 gives recommended curve number values for a range of different land uses assuming ARC II.

This space left intentionally blank.

Table 3-12. SCS Method Runoff Curve Numbers[1]

Cover Description	Cover Type and Hydrologic Condition	Average Percent Impervious Area[2]	Curve numbers for Hydrologic Soil Groups			
			A	B	C	D
Cultivated land:	without conservation treatment		72	81	88	91
	with conservation treatment		62	71	78	81
Pasture or range land:	poor condition		68	79	86	89
	good condition		39	61	74	80
Meadow	Generally mowed for hay		30	58	71	78
Wood or forest land:	thin stand, poor cover		45	66	77	83
	good cover		25	55	70	77
Open space (lawns, parks, golf course, cemeteries, etc.)[3]	poor condition (grass cover <50%)		68	79	86	89
	fair condition (grass cover 50% to 75%)		49	69	79	84
	good condition (grass cover > 75%)		39	61	74	80
Impervious areas:	paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:	paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
	paved; open ditches (including right-of-way)		83	89	92	93
	gravel (including right-of-way)		76	85	89	91
	dirt (including right-of-way)		72	82	87	89
Urban districts:	commercial and business	85%	89	92	94	95
	industrial	72%	81	88	91	93
Residential districts:	1/8 acre or less (town houses)	65%	77	85	90	92
	1/4 acre	38%	61	75	83	87
	1/3 acre	30%	57	72	81	86
	1/2 acre	25%	54	70	80	85
	1 acre	20%	51	68	79	84
	2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (pervious areas only, no vegetation)			77	86	91	94

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

Curve numbers are related to initial abstraction through equations 3-13, 3-14 and 3-15. This relationship is shown in Table 3-13.

Table 3-13. Initial Abstraction (I_a) for Runoff Curve Numbers

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

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses, but sees the drainage area as a uniform land use represented by the composite curve number. Composite curve numbers for a drainage area can be calculated by using the weighted method as presented in Example 3-2.

Example 3-2. Composite Curve Number Calculation

Calculate the composite SCS curve number for a variable watershed. A watershed contains two primary land uses: 80% high density residential with HSG B soils and 20% meadow with HSG C soils. The watershed can be assumed to be divided into two sub-areas as shown in the table:

Step 1. Determine the curve number values for the given land uses and HSGs using Table 3-12.

Step 2. Calculate the weighted curve number for each sub-area of the watershed, and combine to obtain the composite curve number.

Land Use	% of Total Land Area	CN	Weighted CN (% area x CN)
Residential 1/8 acre Soil group B	0.8	85	68
Meadow Good condition Soil group C	0.2	71	14

The composite curve number =68 + 14 = 82.

Any number of land uses can be included. However, if the land use spatial distribution is important to the hydrologic analysis, then sub-basins should be developed and separate hydrographs developed and routed to the study point.

3.1.5.3 Urban Modifications of the SCS Method

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for developed areas. For example, consider whether the impervious areas connect directly to the drainage system or outlet onto lawns or other pervious areas where infiltration can occur. The curve number values given in Table 3-12 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system. It is possible to reduce curve number values from urban areas by not directly connecting impervious surfaces to the drainage system, but instead allowing runoff to flow as sheet flow over significant pervious areas. Chapter 5 (in Volume 2 of this manual) explains the benefits of using better site design techniques such as disconnected areas impervious area.

The following discussion will give some guidance for adjusting curve numbers for different percentages and types of impervious areas. The curve numbers provided in Table 3-12 for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

1. pervious urban areas are equivalent to pasture in good hydrologic condition, and
2. impervious areas have a CN of 98 and are directly connected to the drainage system.

If the typical values presented in Table 3-12 for impervious area percentages or the pervious land use assumptions are not applicable (e.g. the impervious areas are not directly connected), the following figures should be used to compute the composite CN value.

<u>For Site Impervious Area > 30%:</u>

- Figure 3-4 should be used to compute the composite CN value.

<u>For Site Impervious Area < 30%:</u>

- If all of the impervious area is directly connected to the drainage system: use Figure 3-4 to compute the composite CN value.
- If any of the impervious area is NOT directly connected (i.e., "disconnected") to the drainage system: use Figure 3-5 to compute the composite CN value. (Enter the right half of Figure 3-5 with the percentage of total impervious area and the ratio of total disconnected impervious area to total impervious area.)

Figure 3-4. Composite CN for Total Impervious Area > 30%
And Direct Connected Impervious Areas < 30%
(Source: Soil Conservation Service, 1986)

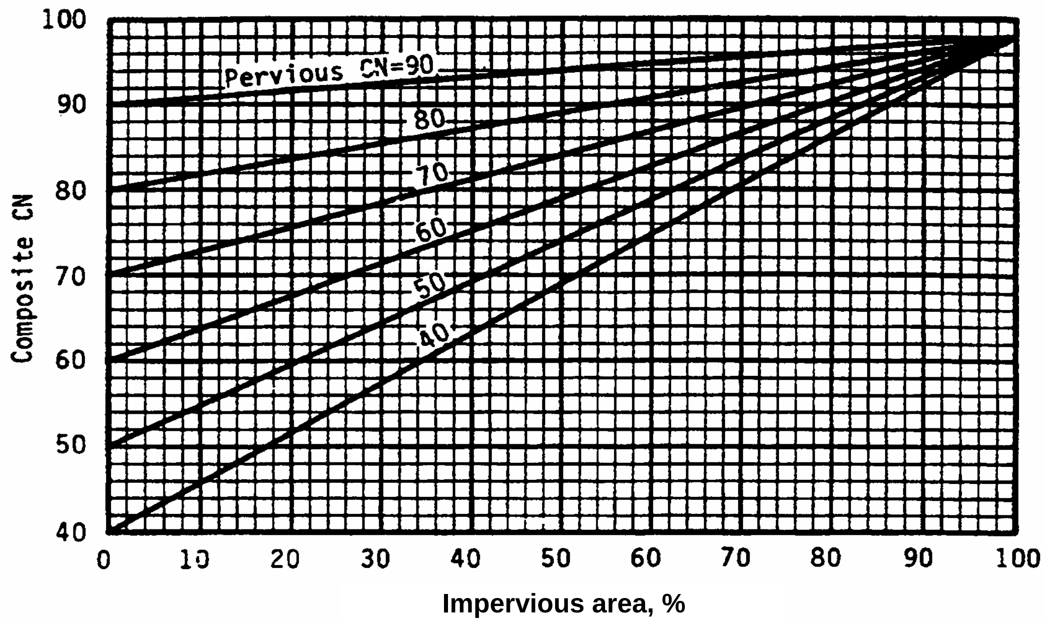
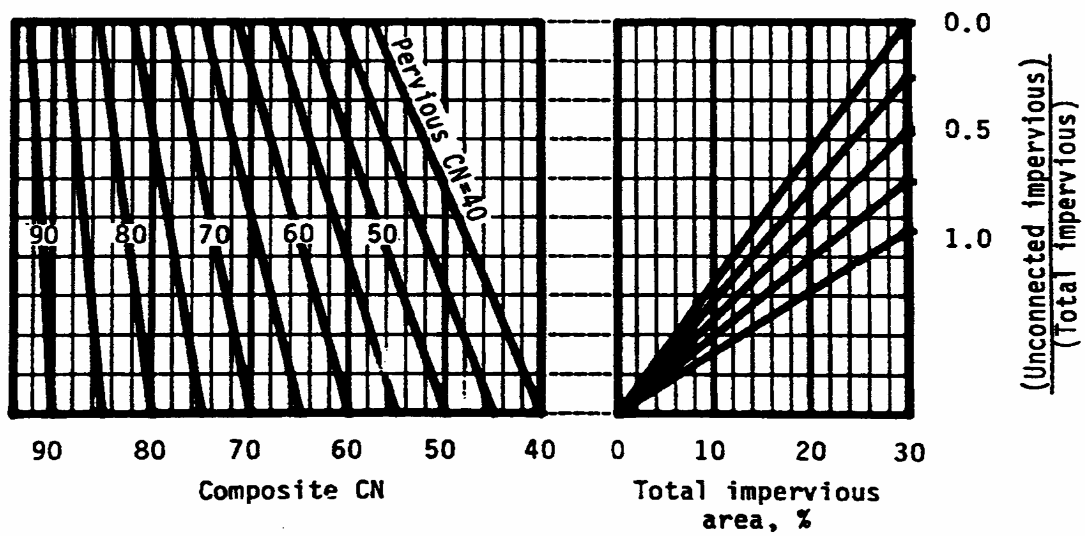


Figure 3-5. Composite CN with Disconnected Impervious Areas < 30%
(Source: Soil Conservation Service, 1986)



Examples 3-3 and 3-4 present the calculation of composite curve numbers for directly connected and disconnected impervious areas, respectively. (For additional information see TR-55, Chapter 2.)

Example 3-3. Curve Number Calculation for a Directly Connected Impervious Area Example
Assume a residential ½ acre lot with HSG B soils and an actual impervious area of 20%. Calculate the curve number for the directly connected area.

Step 1. Read the curve number of 70 for the given land use and HSG from Table 3-12. Note that this curve number is based on assumed impervious area of 25%.

Step 2. Adjust the curve number from the table to reflect less impervious area by using the connected impervious area of 20% and the pervious CN of 61 by using Figure 3-5. Enter Figure 3-5 along the bottom @ 20%, go vertically until the 61 CN line is met, then go to the left vertical axis to read the composite CN. The composite CN obtained from Figure 3-5 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Example 3-4. Curve Number Calculation for an Unconnected Impervious Area Example
Assume a residential ½ acre lot with a pervious CN of 61 and 20% total impervious area (75% of which is unconnected). Calculate the composite curve number for the lot.

Step 1. Enter the right half of Figure 3-5 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. The ratio of unconnected impervious area to total impervious area is 0.75.

Step 2. Then, move left to the appropriate pervious CN and read down to find the composite CN. The composite CN is 66.

Step 3. If all of the impervious area is connected, the resulting CN (from Figure 3-4) would be 68.

3.1.5.4 Simplified SCS Peak Runoff Rate Calculation

The calculation presented in this section is applicable to drainage areas less than 2,000 acres that have homogeneous land uses that can be described by a single CN value (SCS, 1986). The SCS peak discharge equation is presented as Equation 3-16.

Equation 3-16

$$Q_p = q_u A Q F_p$$

where:

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

The computation sequence for the peak discharge method is presented in steps 1 through 6 below.

1. The 24-hour rainfall depth is determined from rainfall Table 3-5 for the selected location and return frequency.
2. The runoff curve number, CN, is estimated from Table 3-12 and direct runoff, Q, is calculated using Equation 3-14.

3. The CN value is used to determine the initial abstraction, I_a, from Table 3-13, and the ratio I_a/P is then computed (P = accumulated 24-hour rainfall).
4. The watershed time of concentration is computed using the procedures in Section 3.1.3.5 and is used with the ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 3-6 for the Type II rainfall distribution. If the ratio I_a/P lies outside the range shown in the figure, either use the limiting values or use another peak discharge method. Note: Figure 3-6 is based on the Knox County standard peaking factor of 484. See Section 3.1.5.5 for additional information about peaking factor.
5. If pond and swamp areas are spread throughout the watershed and are not considered in the Tc computation, an adjustment is needed. The pond and swamp adjustment factor, F_p, is estimated from Table 3-14.

Table 3-14. Adjustment Factors for Ponds and Swamps

Pond and Swamp Areas (%[1])	F_p
0	1.00
0.2	0.97
1	0.87
3	0.75
5 or greater	0.72

[1] Percent of entire drainage basin

6. The peak runoff rate is computed using Equation 3-16.

<u>**Example 3-5. Calculate the 100-year peak discharge using the SCS Peak Discharge Equation.**</u>

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

1. Forest land - good cover (hydrologic soil group B) = 10 ac
2. Forest land - good cover (hydrologic soil group C) = 10 ac
3. Residential - 1/3 acre lots (hydrologic soil group B) = 20 ac
4. Industrial development (hydrological soil group C) = 10 ac

Other data include the following: Total impervious area = 13.2 acres, % of pond/swamp area = 0

<u>Step 1.</u> Calculate the rainfall excess:

The 100-year, 24-hour rainfall is 6.5 inches – From Table 3-5.
The calculation of the composite runoff coefficient for the watershed is shown in the following table.

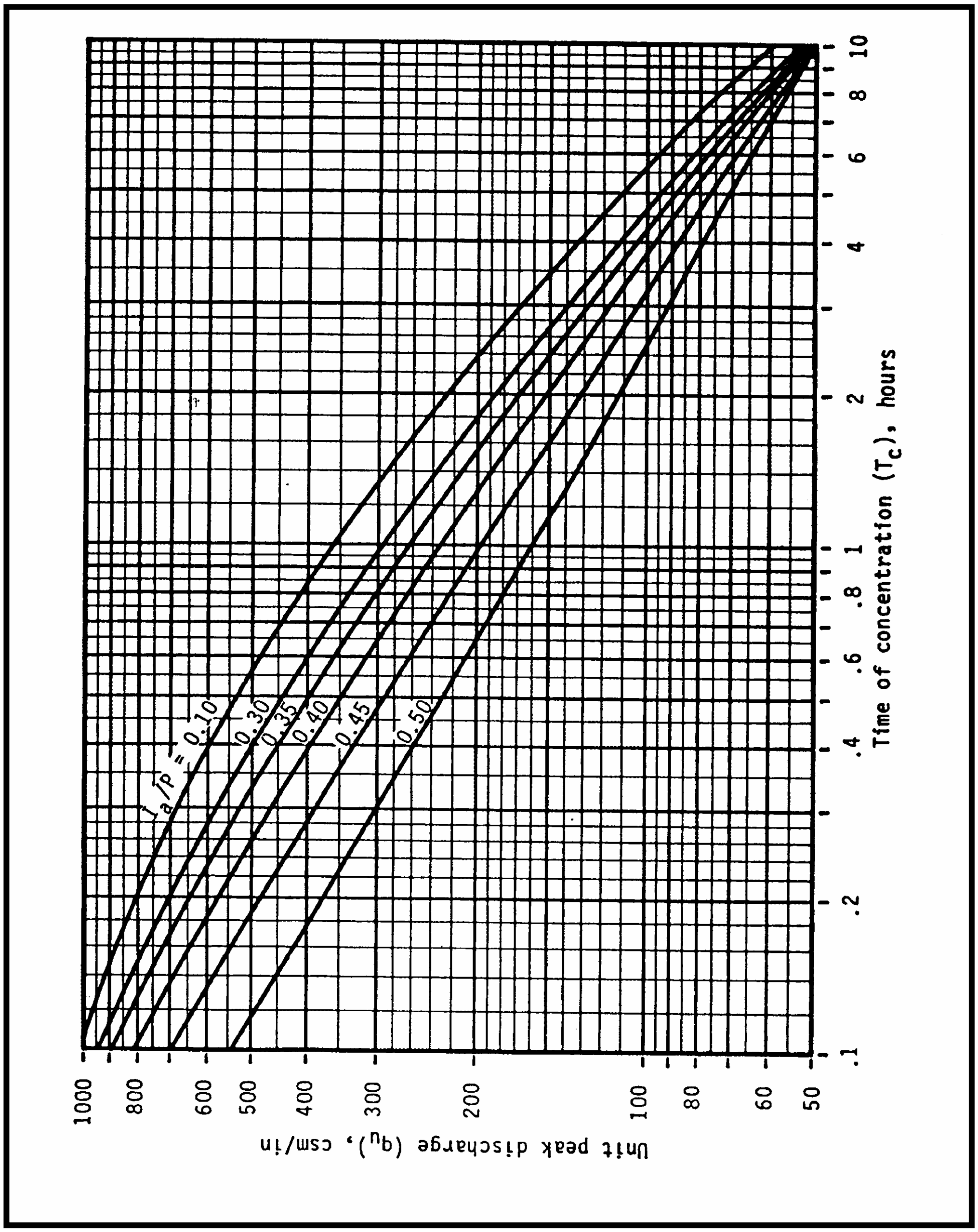
Dev. #	Area	% Total	CN[1]	Composite CN[2]
1	10 ac.	20	55	11.0
2	10 ac.	20	70	14.0
3	20 ac.	40	72	28.8
4	10 ac.	20	91	18.2
Total	50 ac.	100	-	72

[1] CN from Table 3-12
[2] Composite CN = % Total * CN

From Equation 3-14, Q (100-year) = 3.39 inches

Figure 3-6. SCS Type II Unit Peak Discharge Graph

(Source: Soil Conservation Service, 1986)



Step 2. Calculate time of concentration. The hydrologic flow path for this watershed = 1,890 ft

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

[1] For the main channel, n = 0.06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

Segment 1 - Travel time from Equation 3-4 with P_2 = 3.36 inches (0.14 x 24 – Table 3-4)
T_t = $0.007[(0.24)(40)]^{0.8}/(3.36)^{0.5}(0.02)^{0.4}$
= 0.112 hrs = 6.68 minutes

Segment 2 - Travel time from Figure 3-2 or Equation 3-7
V = 2.1 ft/s (from Equation 3-7)
T_t = 750/[(60)(2.1)] = 5.95 min

Segment 3 - Using Equation 3-10
V = $[(1.49)(1.43)^{0.67}(0.005)^{0.5}]/0.06$
= 2.23 ft/s
T_t = 1100/60(2.23) = 8.22 min

Therefore, using Equation 3-3
Tc = 6.68 + 5.95 + 8.22 = 20.85 min = 0.35 hrs

Step 3. Calculate I_a/P (for CN = 72), I_a = 0.778 (Table 3-13)

I_a/P = (0.778/6.48) = 0.12 (Note: Use I_a/P = 0.10 to facilitate use of Figure 3-6. Straight line interpolation could also be used.)

Step 4. Unit discharge q_u (100-year) from Figure 3-6= 650 csm/in,

Step 5. Calculate peak discharge with F_p = 1 using Equation 3-16

Q_{100} = 650(50/640)(3.39)(1) = 172 cfs

3.1.5.5 Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph from a drainage area. The SCS has developed a Tabular Hydrograph procedure that can be used to generate the hydrograph for drainage areas less than 2,000 acres. The Tabular Hydrograph procedure uses unit discharge hydrographs that have been generated for a series of time of concentrations. In addition, SCS has developed hydrograph procedures to be used to generate composite flood hydrographs (SCS, 1986).

The unit hydrograph equations used in the SCS method for generating hydrographs include a constant to account for the general land slope in the drainage area. This constant, called a peaking factor, can be adjusted when using the method. A default value of 484 for the peaking factor represents rolling hills – a medium level of relief. SCS indicates that for mountainous terrain the peaking factor can go as high as 600, and as low as 300 for flat (coastal) areas. In Knox County, the default value of 484 must be used for the peaking factor.

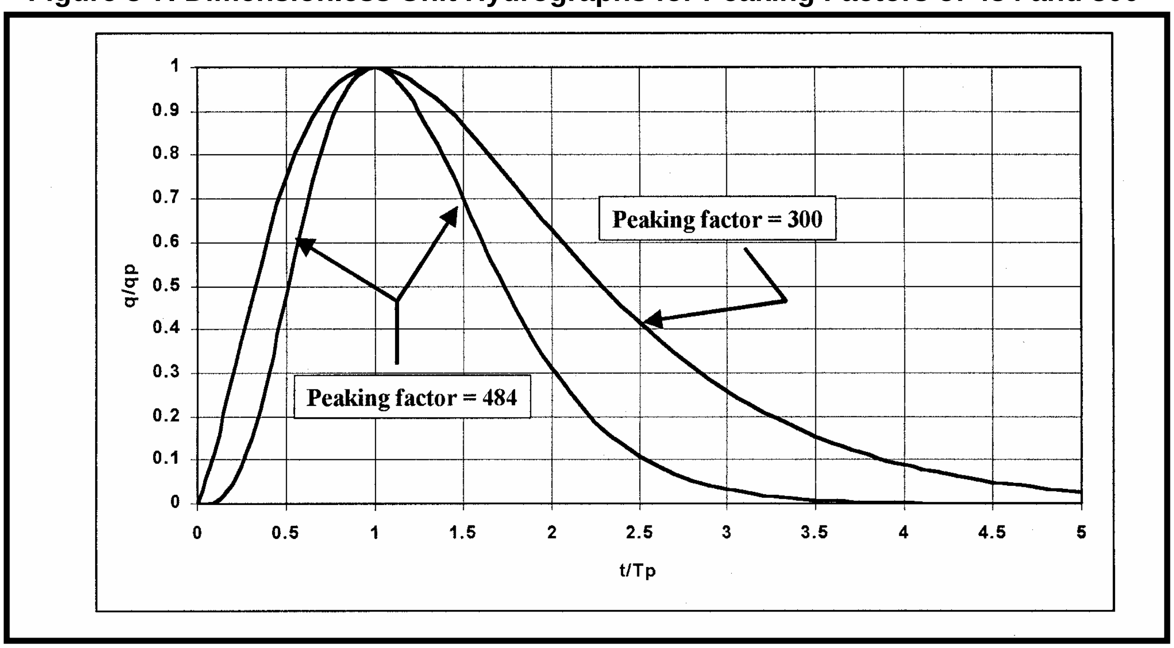
The development of a runoff hydrograph from a watershed is a laborious process not normally done by hand. For that reason this discussion is limited to an overview of the process and is given here to assist the designer in reviewing and understanding the input and output from a typical computer program.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

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

1. Development or selection of a design storm hyetograph (a graph of the time distribution of rainfall over a watershed). Often, the SCS 24-hour storm described in Section 3.1.5.3 is used.
2. Development of curve numbers and lag times for the watershed using the methods described in Sections 3.1.5.4, 3.1.5.5, and 3.1.5.6.
3. Development of a unit hydrograph from the standard (peaking factor of 484) dimensionless unit hydrographs. See discussion below.
4. Step-wise computation of the initial and infiltration rainfall losses and, thus, the excess rainfall hyetograph using a derivative form of the SCS rainfall-runoff equation (Equation 3-12).
5. Application of each increment of excess rainfall to the unit hydrograph to develop a series of runoff hydrographs, one for each increment of excess rainfall (this is called "convolution").
6. Summation of the flows from each of the small incremental hydrographs (keeping proper track of time steps) to form a runoff hydrograph for that watershed or sub-basin.

Figure 3-7 and Table 3-16 can be used along with Equations 3-17 and 3-18 to assist the designer in using the SCS unit hydrograph in Knox County. The unit hydrograph with a peaking factor of 300 is shown in the figure for comparison purposes, but should not be used for areas in Knox County.

Figure 3-7. Dimensionless Unit Hydrographs for Peaking Factors of 484 and 300



Equation 3-17 is used to find the peak discharge, given the peaking factor, area, and time to peak. The peak discharge is then multiplied by the q/q_u values in Table 3-15 for each time step to give the flow at that time.

Equation 3-17

$$q_u = \frac{(PF)A}{T_p}$$

where:

q_u = unit hydrograph peak rate of discharge (cfs)
PF = peaking factor
A = area (mi^2)
T_p = time to peak = d/2 + 0.6 Tc (hours)
d = rainfall time increment (hours)

Table 3-15. Dimensionless Unit Hydrograph 484

t/T_p	484	
	q/q_u	Q/Qp
0.0	0.000	0.000
0.1	0.005	0.000
0.2	0.046	0.004
0.3	0.148	0.015
0.4	0.301	0.038
0.5	0.481	0.075
0.6	0.657	0.125
0.7	0.807	0.186
0.8	0.916	0.255
0.9	0.980	0.330
1.0	1.000	0.406
1.1	0.982	0.481
1.2	0.935	0.552
1.3	0.867	0.618
1.4	0.786	0.677
1.5	0.699	0.730
1.6	0.611	0.777
1.7	0.526	0.817
1.8	0.447	0.851
1.9	0.376	0.879
2.0	0.312	0.903
2.1	0.257	0.923
2.2	0.210	0.939
2.3	0.170	0.951
2.4	0.137	0.962
2.5	0.109	0.970
2.6	0.087	0.977
2.7	0.069	0.982
2.8	0.054	0.986
2.9	0.042	0.989
3.0	0.033	0.992
3.1	0.025	0.994
3.2	0.020	0.995
3.3	0.015	0.996
3.4	0.012	0.997
3.5	0.009	0.998
3.6	0.007	0.998
3.7	0.005	0.999
3.8	0.004	0.999
3.9	0.003	0.999
4.0	0.002	1.000

For ease of spreadsheet calculations, the dimensionless unit hydrograph using a peaking factor of 484 can be approximated using Equation 3-18.

Equation 3-18

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

where:

X = 3.79 for the PF = 484 unit hydrograph.

Example 3-6. Calculation of Unit Hydrograph

Compute the unit hydrograph for the 50-acre wooded watershed in Example 3-5.

Computations

Step 1. Calculate T_p (time to peak) and time increment

The time of concentration (Tc) is calculated to be 21 minutes for this watershed. If we assume a computer calculation time increment (d), duration of excess rainfall, of 3 minutes then:

T_p = d/2 + 0.6 Tc = 3/2 + 0.6(21) = 14.1 min = 0.235 hours

Step 2. Calculate q_u

q_u = (484)(50/640)/(0.235) = 161 cfs

Step 3. Calculate the hydrograph using unit hydrograph 484. The table below was derived based on spreadsheet calculations using Equations 3-17 and 3-18.

Time		484	
t/Tp	time (min)	q/q_u	Q
0.00	0	0.00	0.00
0.21	3	0.06	9.15
0.43	6	0.35	56.32
0.64	9	0.72	116.58
0.85	12	0.96	154.41
1.00	14	1.00	160.90
1.07	15	0.99	160.14
1.28	18	0.88	142.28
1.49	21	0.70	113.61
1.71	24	0.52	83.90
1.92	27	0.36	58.37
2.14	30	0.24	38.74
2.35	33	0.15	24.75
2.56	36	0.09	15.32
2.78	39	0.06	9.24
2.99	42	0.03	5.45
3.20	45	0.02	3.15
3.42	48	0.01	1.79
3.63	51	0.01	1.00
3.84	54	0.00	0.55

Time		484	
t/Tp	time (min)	q/q_u	Q
4.06	57	0.00	0.30
4.27	60	0.00	0.16
4.48	63	0.00	0.09
4.70	66	0.00	0.05
4.91	69	0.00	0.02

3.1.6 Clark Unit Hydrograph

In Knox County, use of the Clark Unit Hydrograph method is acceptable only for hydrologic calculations that are prepared for flood studies and flood elevation calculations. See Volume 2, Chapter 8 for more information on flood study preparation.

The Clark method defines a unit hydrograph for a given basin using the concept of the instantaneous unit hydrograph (IUH). An IUH is a theoretical hydrograph that would result when a single unit of rainfall excess was spread out evenly over an entire basin and allowed to run off. The IUH can be converted to a unit hydrograph of a desired duration by conventional techniques for developing unit hydrographs (Hoggan, 1997).

The Clark method is based on the effects of translation and attenuation as the primary forces involved in the flow of water through a watershed. Translation is defined as the 'downhill' flow of water as a result of the force of gravity. Attenuation is defined as the resistance of flow that is caused by either friction in the channel or water storage. According to Clark, translation in a watershed can be described with a time-area curve. This curve displays the portion of watershed area that is contributing runoff as a function of time. The curve should start at the point in which effective precipitation begins. Effective precipitation is any precipitation that does not infiltrate into the soil or is retained in a ponding area. Equation 3-19 presents these concepts.

Equation 3-19

$$S = RO$$

where:

S = Storage
R = Attenuation (Watershed Storage) Constant
O = Outflow

A synthetic hydrograph could be produced by proportionally routing an inch of direct runoff to the channel in accordance with the time-area curve. The runoff entering the channel would then be routed through a linear reservoir. More recent studies have indicated that it is not necessary to produce detailed time-area curves in order to produce accurate synthetic hydrographs. The dimensionless time-area curve included in HEC-1 and HEC-HMS hydrologic models (developed by the United States Army Corps of Engineers) have produced accurate synthetic hydrographs. In order to apply the Clark method in a HEC-1 or HEC-HMS model, the time of concentration (Tc) and a watershed storage constant (R) are required as inputs. In stormwater master plans prepared for Knox County in the late 1990's and early 2000's, research indicated that Equation 3-20, which equates to setting R = Tc, produced accurate estimates of peak discharges for small drainage areas. However, the engineer performing the flood study should determine the most appropriate equation to determine the value of R.

Equation 3-20
$$\frac{R}{T_c + R} = 0.5$$

where:

R = Attenuation (Watershed Storage) Constant
Tc = Time of concentration

3.1.7 Water Quality Calculations

3.1.7.1 Water Quality Volume Calculation

In Knox County, the Water Quality Volume (WQv) is the treatment volume required to remove 80% of the average annual, post-development total suspended solids (TSS) load. This is achieved by intercepting and treating a portion of the runoff from all storms and all the runoff from 85% of the storms that occur on average during the course of a year. The water quality treatment volume is calculated using Equation 3-21.

Equation 3-21
$$WQv = \frac{1.1 R_v A}{12}$$

where:

WQv = water quality volume (acre-feet)
1.1 = the 85th percentile rainfall depth in Knox County (inches)
Rv = volumetric runoff coefficient (see Equation 3-22)
A = total drainage area (acres)

The volumetric runoff coefficient (Rv) is directly proportional to the percent impervious cover of the development or drainage area. Rv is calculated using Equation 3-22.

Equation 3-22
$$Rv = 0.015 + 0.0092(I)$$

where:

I = percent of impervious cover (%)

3.1.7.2 Water Quality Peak Discharge Calculation

The peak rate of discharge for the water quality design storm (Q_{wq}, also called the water quality peak discharge) is needed to size off-line diversion structures, such as for sand filters and infiltration trenches. However, traditional peak discharge calculation methods are not appropriate for this application. For example, the use of the Rational Method for sizing water quality controls would require the choosing of an arbitrary storm event. Further, conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events of less than two inches. This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff bypasses the structural control due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

The method employed to size water quality diversions uses the runoff coefficient to find the depth of runoff for the water quality storm of 1.1”. The SCS method is then used to find a unit peak discharge that is combined with the runoff depth to find a peak runoff rate.

The following procedure can be used to calculate Q_{wq}. This procedure relies on the Rv and the simplified peak discharge calculation:

1. The runoff coefficient (Rv) is determined using Equation 3-22.
2. The depth of runoff that must be treated for the water quality storm is determined.

$$D_{wq} = 1.1Rv$$

where:

D_{wq} = water quality runoff depth (inches)
Rv = runoff coefficient

3. A CN reflecting the runoff characteristics during the water quality design storm is calculated utilizing a form of Equation 3-15:

$$CN = 1000/[10 + 5P + 10D_{wq} - 10(D_{wq}^2 + 1.25\ D_{wq}P)^{0.5]}$$

where:

P = rainfall depth for water quality design storm = (1.1 inches)

4. Initial abstraction (Ia) is found from CN using Table 3-13. Ia/P is then calculated.
5. Concentration time (Tc) is computed per section 3.1.3.5.
6. Unit peak discharge (q_u) is determined from Tc and Ia using Figure 3-6.
7. The water quality peak discharge (Q_{wq}) is computed using drainage area (A), unit peak discharge (q_u) and water quality runoff depth (D_{wq}) using Equation 3-16.

Example 3-7. Calculation of Water Quality Peak Flow

Using the data and information from Example 3-5, calculate the WQv and the Q_{wv}.

Step 1: Compute volumetric runoff coefficient, Rv using Equation 3-22:

Rv = 0.015+(0.0092)(I) = 0.015+(0.00920(18/50)(100) = 0.35

Step 2. Compute depth of runoff that must be treated for water quality, D_{wq}:

D_{wq} = 1.1Rv = 1.1(0.35) = 0.39 inches

Step 3: Compute the synthetic curve number (CN) using Equation 3-15:

$CN = 1000/[10 + 5(1.1) + 10(0.39) – 10[(0.39)^2 +1.25(0.39)(1.1)]^{0.5}= 90$

Step 4: Find Ia from CN with Table 3-13:

Ia = 0.22 inches
Ia/P = 0.22/1.1 = 0.20

Step 5: Compute time of concentration, Tc:

Tc computed as 0.35 hours in Example 3-5.

Step 6: Find q_u, using Tc = 0.35 and I_a/P = 0.20 using Figure 3-6:

q_u = 580 cfs/mi²/in

Step 7: Compute water quality peak flow rate using Equation 3-16.

Q_{wq} = 580(50/640)(0.39)(1) = 17.67 cfs

3.1.8 Water Balance Calculations

Water balance calculations can help to determine if a drainage area is large enough or has the right characteristics to support a permanent pool of water during average or extreme conditions. When in doubt, a water balance calculation may be advisable for retention pond and wetland design.

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

3.1.8.1 Basic Equations

Water balance is defined as the change in volume of the permanent pool resulting from the total inflow minus the total outflow (actual or potential). Equation 3-23 presents this calculation.

Equation 3-23

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

where:

- Δ = delta or "change in"
- V = pond volume (ac-ft)
- Σ = "the sum of"
- I = Inflows (ac-ft)
- O = Outflows (ac-ft)

The inflows consist of rainfall, runoff and baseflow into the pond. The outflows consist of infiltration, evaporation, evapotranspiration, and surface overflow out of the pond or wetland. Equation 3-23 can be expanded to reflect these factors, as shown in Equation 3-24. Key variables in Equation 3-24 are discussed in detail below the equation.

Equation 3-24

$$\Delta V = PA + R_o + Bf - IA - EA - EtA - Of$$

where:

- P = precipitation (ft)
- A = area of pond (ac)
- R_o = runoff (ac-ft)
- B_f = baseflow (ac-ft)
- I = infiltration (ft)
- E = evaporation (ft)
- Et = evapotranspiration (ft)
- Of = overflow (ac-ft)

Rainfall (P) – Monthly rainfall values can be obtained from the National Weather Service climatology at *http://www.srh.noaa.gov/mrx/climat.htm*. Monthly values are commonly used for calculations of values over a season. Rainfall is the direct amount that falls on the pond surface for the period in question. When multiplied by the pond surface area (in acres) it becomes acre-feet of volume. Table 3-16 presents average monthly rainfall values for Knoxville based on a 30-year period of record.

Table 3-16. Average Rainfall Values in Inches for Knoxville, Tennessee

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
P (inches)	4.57	4.01	5.17	3.99	4.68	4.04	4.71	2.89	3.04	2.65	3.98	4.49
Annual Precipitation 48.2 inches												

Source: www.ncdc.noaa.gov/oa/climate/online/ccd/nrmpcp.txt

Runoff (R_o) – Runoff is equivalent to the rainfall for the period times the "efficiency" of the watershed, which is equal to the ratio of runoff to rainfall (Q/P). In lieu of gage information, Q/P can be estimated one of several ways. The best method would be to perform long-term simulation modeling using rainfall records and a watershed model.

Equation 3-21 gives a ratio of runoff to rainfall volume for a particular storm. If it can be assumed that the average storm that produces runoff has a similar ratio, then the Rv value can serve as the ratio of rainfall to runoff. Not all storms produce runoff in an urban setting. Typical initial losses (often called "initial abstractions") are normally taken between 0.1 and 0.2 inches. When compared to the rainfall records in Knox County, this is equivalent to about a 10% runoff volume loss. Thus, in a water balance calculation, a factor of 0.9 should be applied to the calculated Rv value to account for storms that produce no runoff. Equation 3-25 reflects this approach. Total runoff volume is then simply the product of runoff depth (Q) times the drainage area to the pond.

Equation 3-25 $$Q = 0.9PRv$$

where:

Q = runoff volume (in)
P = precipitation (in)
Rv = volumetric runoff coefficient [Equation 3-22]

Baseflow (B_f) – Most stormwater ponds and wetlands have little, if any, baseflow, as they are rarely placed across perennial streams. If so placed, baseflow must be estimated from observation or through theoretical estimates. Methods of estimation and baseflow separation can be found in most hydrology textbooks.

Infiltration (I) – Infiltration is a very complex subject and cannot be covered in detail here. The amount of infiltration depends on soils, water table depth, rock layers, surface disturbance, the presence or absence of a liner in the pond, and other factors. The infiltration rate is governed by the Darcy equation, shown in Equation 3-26.

Equation 3-26 $$I = Ak_hG_h$$

where:

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

G_h can be set equal to 1.0 for pond bottoms and 0.5 for pond sides steeper than about 4:1. Infiltration rate can be established through testing, though not always accurately. Table 3-17 can be used for initial estimation of the saturated hydraulic conductivity.

Table 3-17. Saturated Hydraulic Conductivity
(Source: Ferguson and Debo, 1990)

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

Material	Hydraulic Conductivity	
	in/hr	ft/day
Silt loam	0.27	0.54
Sandy clay loam	0.17	0.34
Clay loam	0.09	0.18
Silty clay loam	0.06	0.12
Sandy clay	0.05	0.10
Silty clay	0.04	0.08
Clay	0.02	0.04

Evaporation (E) – Evaporation is from an open lake water surface. Evaporation rates are dependent on differences in vapor pressure, which, in turn, depend on temperature, wind, atmospheric pressure, water purity, and shape and depth of the pond. It is estimated or measured in a number of ways, which can be found in most hydrology textbooks. Pan evaporation methods are also used, though there are no longer pan evaporation sites active in Knox County. Formerly pan evaporation methods were utilized at the Knoxville Experiment Station.

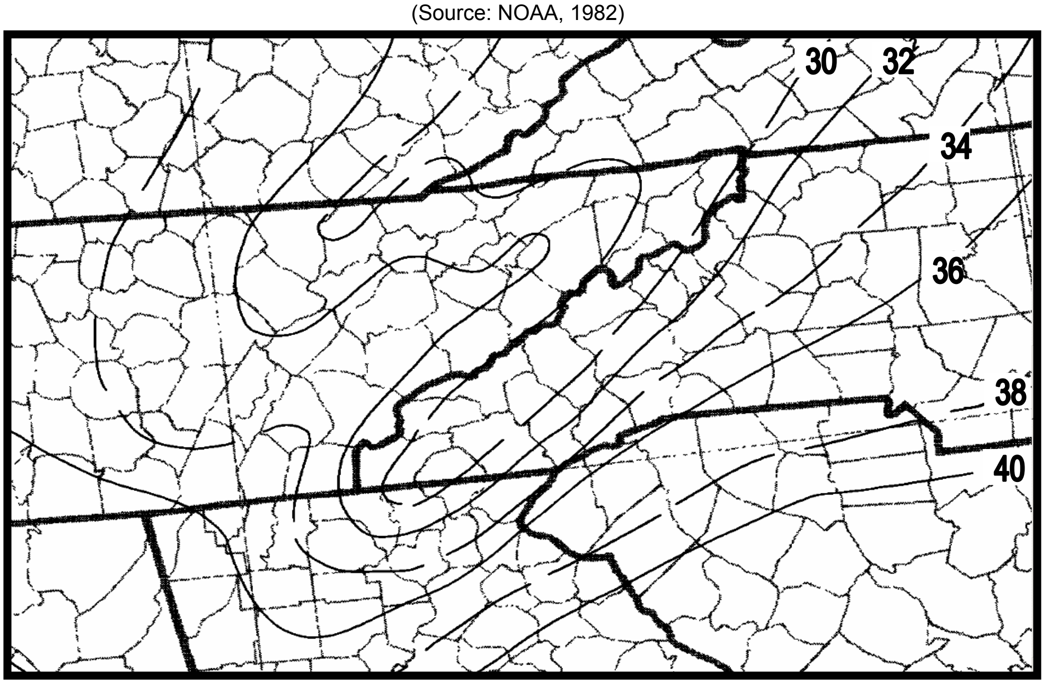
Table 3-18 presents pan evaporation rate distributions for a typical 12-month period based on pan evaporation information from one station in Knox County. Figure 3-8 depicts a map of annual free water surface (FWS) evaporation averages for Tennessee based on a National Oceanic and Atmospheric Administration (NOAA) assessment done in 1982. FWS evaporation differs from lake evaporation for larger and deeper lakes, but can be used as an estimate of it for the type of structural stormwater ponds and wetlands being designed in Knox County. Total annual values can be estimated from this map and distributed in accordance with the percentages presented in Table 3-18.

Table 3-18. Pan Evaporation Rates - Monthly Distribution

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2.9%	3.8%	7.2%	10.6%	13.1%	13.1%	13.2%	12.4%	9.8%	6.7%	4.1%	3.1%

Figure 3-8. Average Annual Free Water Surface Evaporation (in inches)

(Source: NOAA, 1982)



Evapotranspiration (E_t). Evapotranspiration consists of the combination of evaporation and transpiration by plants. The estimation of E_t for crops is well documented and has become standard practice. However, the estimating methods for wetlands are not documented, nor are there consistent studies to assist the designer in estimating the wetland plant demand on water volumes. Literature values for various places in the United States vary around the free water surface lake evaporation values. Estimating E_t only becomes important when wetlands are being designed and emergent vegetation covers a significant portion of the pond surface. In these cases conservative estimates of lake evaporation should be compared to crop-based E_t estimates and a decision made. Crop-based E_t estimates can be obtained from typical hydrology textbooks or from the web sites mentioned above. A value of zero shall be assumed for E_t unless the wetland design dictates otherwise.

Overflow (O_f) – Overflow is considered as excess runoff, and in water balance design is either not considered since the concern is for average precipitation values, or is considered lost for all volumes above the maximum pond storage. Obviously, for long-term simulations of rainfall-runoff, large storms would play an important part in pond design.

Example 3-8. Water Balance Calculation for Pond

Knox Farms, a 26-acre site in Knox County, is being developed along with an estimated 0.5-acre surface area pond. There is no baseflow. The desired pond volume to the overflow point is 2 acre-feet. Will the site be able to support the pond volume? From the basic site data we find that the site is 75% impervious with sandy clay loam soil.

Step 1: From Equation 3-22, Rv = 0.015 + 0.0092(75) = 0.71. With the correction factor of 0.9 the watershed efficiency is 0.64.
The annual lake evaporation from Figure 3-8 is about 32.5 inches.
For a sandy clay loam the infiltration rate is I = 0.34 ft/day (Table 3-17).
From a grading plan, it is known that 10% of the total pond area is sloped greater than 4:1.
Monthly rainfall for Knox County was found from the Web site provided above.

Step 2: The table below shows summary calculations for this site for each month of the year.

	Value	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	Days per Month	31	28	31	30	31	30	31	31	30	31	30	31
2	Precip. (in)	4.57	4.01	5.17	3.99	4.68	4.04	4.71	2.89	3.04	2.65	3.98	4.49
3	Evap. Dist. (%)	2.9	3.8	7.2	10.6	13.1	13.1	13.2	12.4	9.8	6.7	4.1	3.1
4	R_0 (ac-ft)	6.22	5.45	7.03	5.43	6.36	5.49	6.41	3.93	4.13	3.60	5.41	6.11
5	P (ac-ft)	0.19	0.17	0.22	0.17	0.20	0.17	0.20	0.12	0.13	0.11	0.17	0.19
6	E (ac-ft)	0.04	0.05	0.10	0.14	0.18	0.18	0.18	0.17	0.13	0.09	0.06	0.04
7	I (ac-ft)	5.01	4.52	5.01	4.85	5.01	4.85	5.01	5.01	4.85	5.01	4.85	5.01
8	Bal. (ac-ft)	1.36	1.05	2.14	0.60	1.38	0.64	1.42	-1.12	-0.72	-1.38	0.68	1.25
9	Run. Bal. (ac-ft)	1.36	2.00	2.00	2.00	2.00	2.00	2.00	0.88	0.16	0.00	0.68	1.92

Explanation of Table:

1. Days per month
2. Monthly precipitation from web site is shown in Table 3-16.
3. Distribution of evaporation by month from Table 3-18.
4. Ro = Watershed efficiency (0.64) * Rainfall (line 2) * Site Area w/o Pond (26-0.5) / 12 (conversion to acre-feet). From Equation 3-25.
5. Precipitation volume directly into pond = Rainfall (line 2) * Pond Area (0.5) / 12 (conversion to acre-feet).
6. Evaporation = Monthly % (line 3) / 100 (convert to decimal) * Annual Evaporation (32.5) * Pond Area (0.5) / 12 (conversion to acre-feet).
7. Infiltration = **[**infiltration rate (0.34) * 90% of Pond Area (0.90 * 0.5) + infiltration rate (0.34) * 10% of Pond Area (0.1 * 0.5) * 0.5 (Reduction factor for banks greater than 4:1)**]** * # of Days in month (line 1) (conversion from acre-feet/day to acre-feet).
8. Balance is lines (4 + 5) minus lines (6 + 7).
9. Running Balance is accumulated total from line 9 keeping in mind that all volume above 2 acre-feet overflows and is lost in the trial design. (Any value > 2, set = 2. Any value < 0, set = 0)

It can be seen that for this example the pond has potential to go dry in the fall. This can be remedied in a number of ways including compacting the pond bottom, placing a liner of clay of geosynthetics, and changing the pond geometry to decrease the surface area.

3.1.9 Calculating Downstream Impacts (the Ten Percent Rule)

In the Knox County Stormwater Management Manual, the "ten-percent" rule has been adopted as the approach for ensuring that stormwater quantity detention ponds maintain pre-development peak flows through the downstream conveyance system.

The ten-percent rule recognizes the fact that a structural control providing detention has a "zone of influence" downstream where its effectiveness can be observed. Beyond this zone of influence the structural control becomes relatively small and insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, that zone of influence is considered to be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. For example, if the structural control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater.

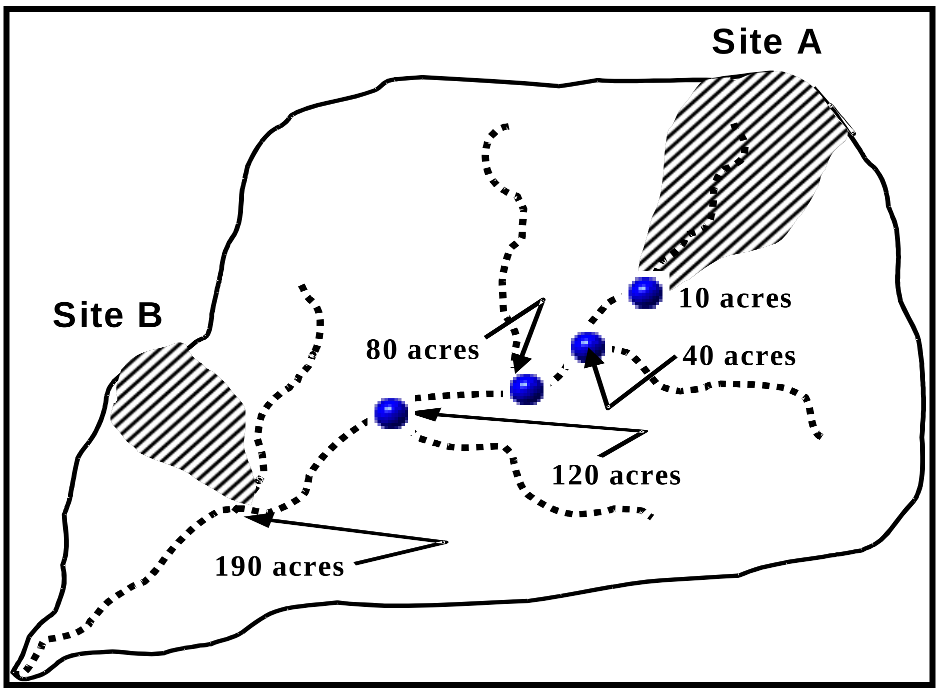
Typical steps in the application of the ten-percent rule are:

1. Using a topographic map determine the lower limit of the "zone of influence" (i.e., the 10% point), and determine all 10% rule comparison points (at the outlet of the site and at all downstream tributary junctions or other points of interest).
2. Using a hydrologic model determine the pre-development peak discharges (pre-Qp_2, pre-Qp_{10}, pre-Qp_{25}, and pre-Qp_{100}) and timing of those peaks at each tributary junction beginning at the pond outlet and ending at the next tributary junction beyond the 10% point.
3. Change the site land use to post-development conditions and determine the post-development peak discharges (post-Qp_2, post-Qp_{10}, post-Qp_{25}, and post-Qp_{100}). Design the structural control facility such that the post-development peak discharges from the site <u>for all storm events</u> do not increase the pre-development peak discharges at the outlet of the site and at each downstream tributary junction and each public or major private downstream stormwater conveyance structure located within the zone of influence.
4. If post-development conditions do increase the peak flow within the zone of influence, the structural control facility must be redesigned or one of the following options must be chosen:

- Control of the Qp_2, Qp_{10}, Qp_{25}, and/or Qp_{100} may be waived by the Director of Engineering and Public Works (the Director) if adequate overbank flood protection and/or extreme flood protection is suitably provided by a downstream or shared off-site stormwater facility, or if engineering studies determine that installing the required stormwater facilities would not be in the best interest of Knox County. However, a waiver of such controls does not eliminate the requirement to comply with the water quality and channel protection standards defined in the Ordinance and in this Stormwater Management Manual.
- The developer can coordinate with Knox County Engineering (and other state/federal agencies as appropriate) to determine other acceptable approaches to reduce the peak discharges (and, therefore the flow elevation) through the channel (e.g., conveyance improvements) for all design storm events.
- The developer can obtain a flow easement from downstream property owners through the zone of influence where the post-development peak discharges are higher than pre-development peak discharges.

Example 3-9. Ten Percent Rule Example

The figure below illustrates the concept of the ten-percent rule for two sites in a watershed.



Site A is a development of 10 acres, all draining to a wet ED stormwater pond. The overbank flooding and extreme flood portions of the design are going to incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked "120 acres." The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The designer constructs a simple HEC-1 (HEC-HMS) model of the 120-acre areas using single existing condition sub-watersheds for each tributary. Key detention structures existing in other tributaries must be modeled. An approximate curve number is used since the *actual* peak flow is not the key for initial analysis; only the increase or decrease is important. The accuracy in curve number determination is not as significant as an accurate estimate of the time of concentration. Since flooding is an issue downstream, the pond is designed (through several iterations) until the peak flow does not increase at junction points downstream to the 120-acre point.

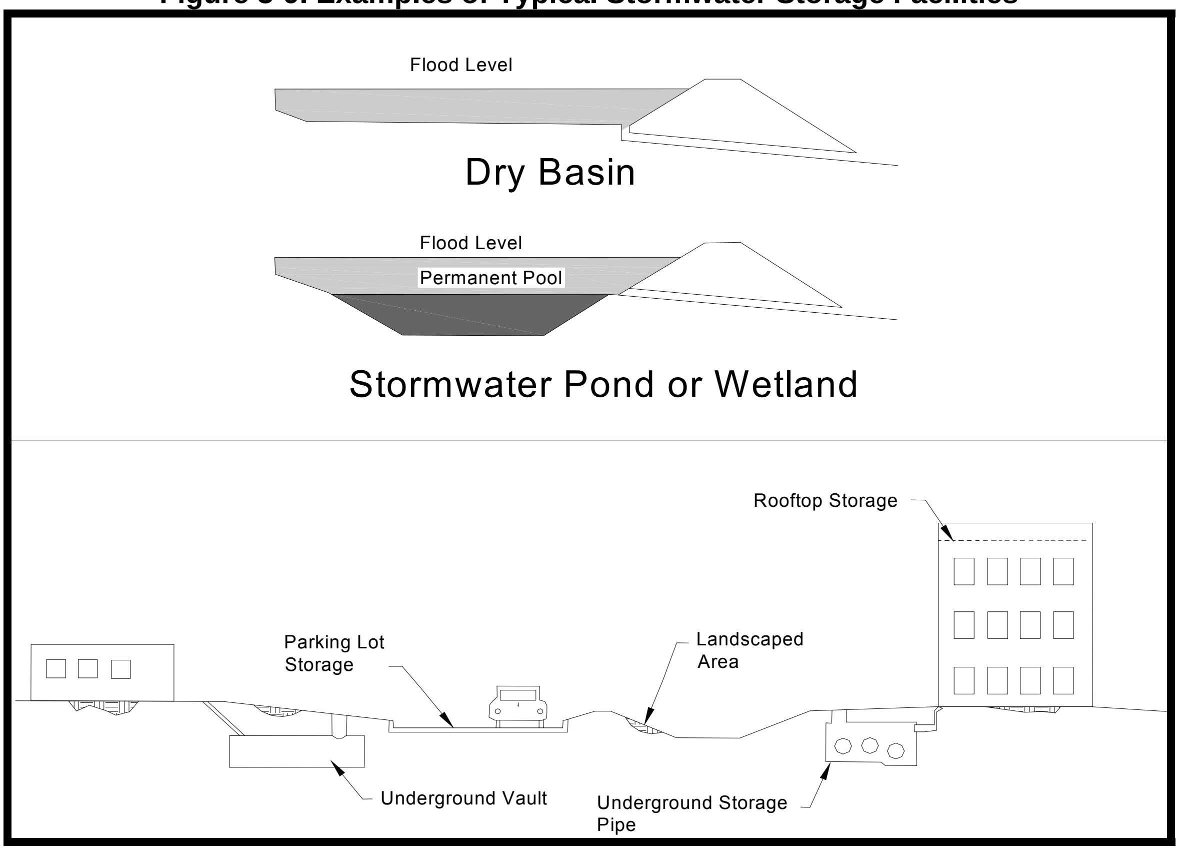
Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin with care taken to estimate the time of concentration for input into the hydrologic model of the watershed. The model shows that a detention facility, in this case, will actually increase the peak flow in the stream.

3.2 Storage Design

3.2.1 General Storage Concepts

This section provides general guidance on stormwater runoff storage for meeting control of the WQv, CPv, Qp_2, Qp_{10}, Qp_{25} and the Qp_{100}. Storage of stormwater runoff within a stormwater management system is critical to providing the extended detention of flows for water quality treatment and downstream channel protection, as well as for peak flow attenuation of the larger overbank and extreme flood protection flows. Runoff storage can be provided within an on-site system through the use of structural stormwater BMPs and/or non-structural features and landscaped areas. Figure 3-9 illustrates various storage facilities that can be considered for a development site.

Figure 3-9. Examples of Typical Stormwater Storage Facilities

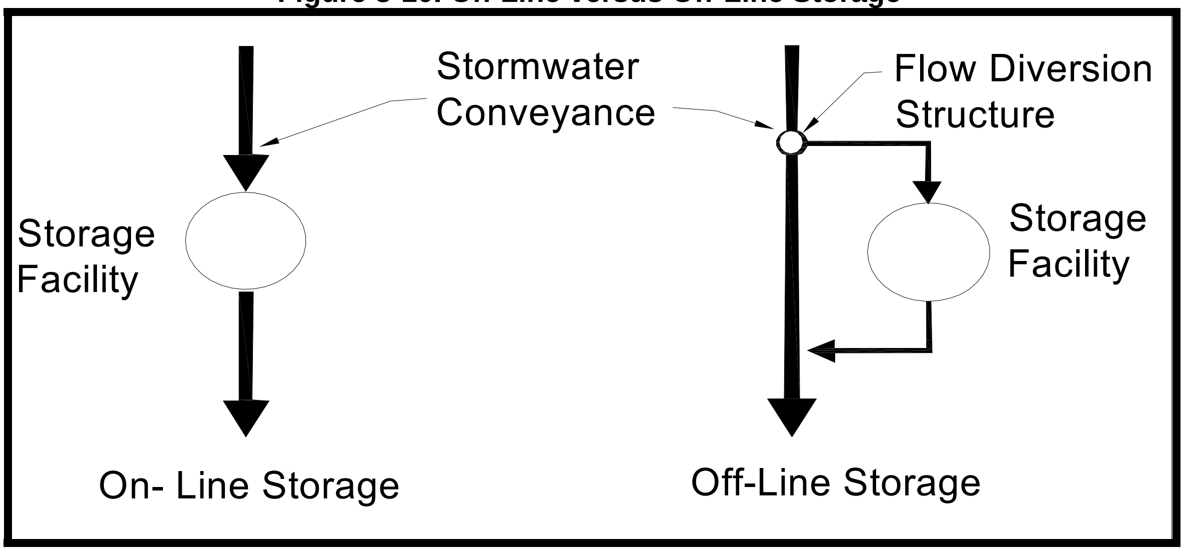


There are three main types of stormwater runoff storage: *detention, extended detention, and retention*. Stormwater *detention* is used to reduce the peak discharge and detain runoff for a specified short period of time. Detention basins are designed to completely drain after the design storm has passed. Detention is used to meet overbank flood protection criteria, and extreme flood criteria where required. *Extended detention* (ED) is used to drain a runoff volume over a specified period of time, typically 24 hours, and is used to meet channel protection criteria. Some structural BMP designs (wet ED pond, micropool ED pond, dry extended pond and shallow ED marsh) also include extended detention storage of a portion of the water quality volume. *Retention* facilities, such as stormwater ponds and wetlands, are designed to contain a permanent pool of water that is used for water quality treatment. Some facilities include one or more types of storage. An example of a combined storage facility is one that is sized to provide extended detention of the WQv as well as detention of the Qp_{100}.

Storage facilities are often classified on the basis of their location and size. *On-site* storage is constructed on individual development sites and most often only provides control of the runoff that discharges that individual site. *Regional* storage facilities are designed to manage stormwater runoff from multiple projects and/or properties, or are constructed at the lower end of a sub-basin within which multiple properties are located. Knox County Engineering will determine if the use of a regional storage facility is applicable on a case-by-case basis.

Storage can also be categorized as *on-line* or *off-line*. On-line storage uses a structural BMP facility that intercepts flows directly within a conveyance system or stream. Off-line storage is a separate storage facility to which flow is diverted from the conveyance system. Figure 3-10 illustrates on-line versus off-line storage.

Figure 3-10. On-Line versus Off-Line Storage



3.2.1.1 Stage-Storage Relationship

A stage-storage curve defines the relationship between the depth of water (stage) and storage volume in a storage facility. An example of a stage-storage curve is presented in Figure 3-11. This curve relationship allows the volume of storage to be calculated by using simple geometric formulas expressed as a function of depth. The storage volume for natural basins may be developed using a topographic map and the double-end area, frustum of a pyramid, prismoidal or circular conic section formulas.

<u>Double-end area method:</u> The double-end area method uses the areas of the planes at two given elevations to calculate the volume between the two area planes. This concept is presented in Figure 3-12. The double-end area equation is presented in Equation 3-27.

Equation 3-27

$$V_{1-2} = \left[\frac{(A_1 + A_2)}{2}\right] d$$

where:

V_{1-2} = storage volume (ft^3) between elevations 1 and 2
A_1, A_2 = surface area at elevation 1 and 2, respectively (ft^2)
d = change in elevation between points 1 and 2 (ft)

Figure 3-11. Stage-Storage Curve

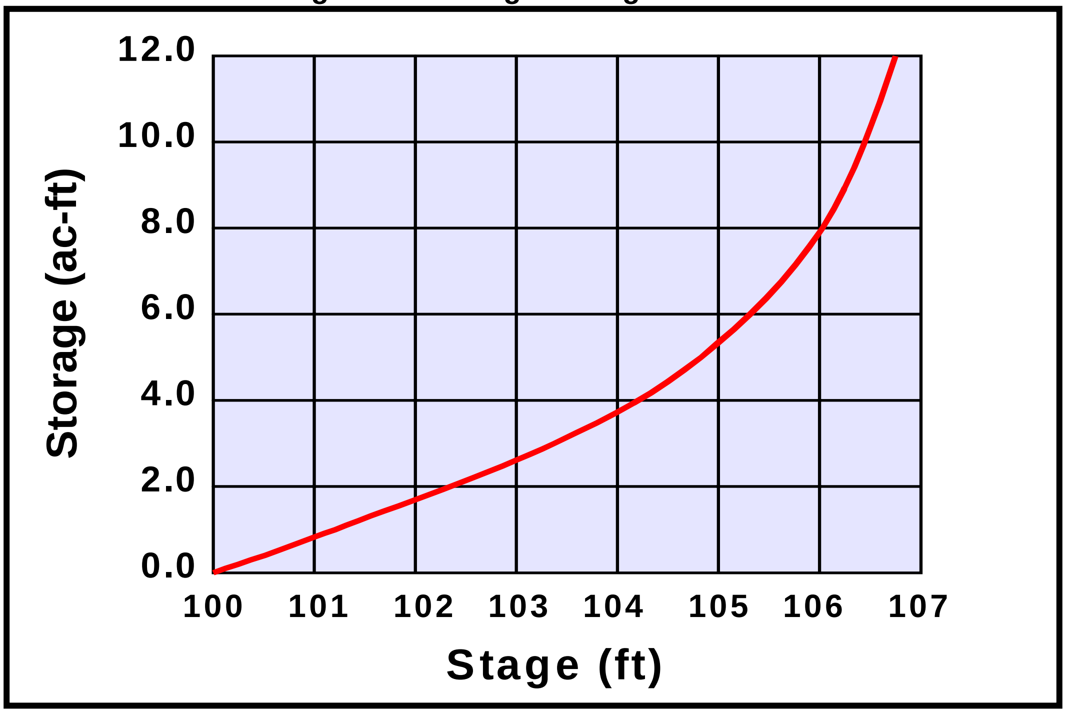
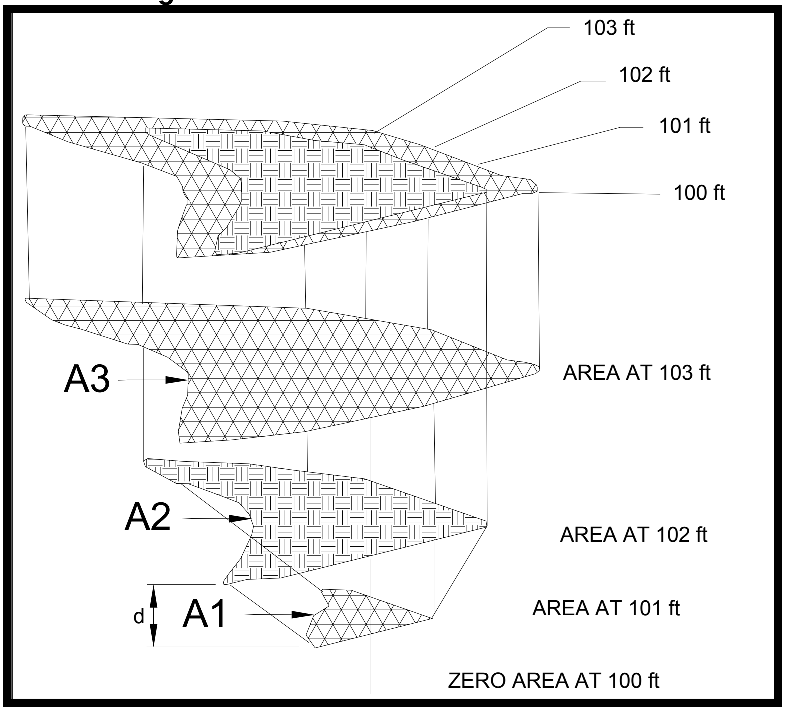


Figure 3-12. Double-End Area Method



<u>Frustrum of a pyramid method:</u> Another calculation involves treating the storage as a pyramid frustum, or a part of a pyramid. The frustum is formed by truncating the pyramid using two planes parallel to the pyramid base. The frustum of a pyramid can be calculated using Equation 3-28.

Equation 3-28

$$V = \frac{d}{3}\left[A_1 + \left(A_1 \times A_2\right)^{0.5} + A_2\right]$$

where:

- V = volume of frustum of a pyramid (ft^3)
- d = change in elevation between points 1 and 2 (ft)
- A_1 = surface area at elevation 1 (ft^2)
- A_2 = surface area at elevation 2 (ft^2)

<u>Frustrum of a prismoid method:</u> A trapezoidal basin can be represented as a prismoid since the volume is formed by the trapezoidal faces. Equation 3-29 presents the prismoidal equation for trapezoidal basins.

Equation 3-29

$$V = LWD + \left(L + W\right)ZD^2 + \frac{4}{3}Z^2D^3$$

where:

- V = volume of trapezoidal basin (ft^3)
- L = length of basin at base (ft)
- W = width of basin at base (ft)
- D = depth of basin (ft)
- Z = side slope factor, ratio of horizontal to vertical

<u>Frustrum of a cone or conic section method:</u> Equations 3-30 and 3-31 present the calculation approach for the basin storage volume represented is as a circular cone.

Equation 3-30 $$V = 1.047D\left(R_1^{\ 2} + R_2^{\ 2} + R_1R_2\right)$$

Equation 3-31 $$V = 1.047D\left(3R_1^{\ 2} + 3ZDR_1 + Z_2D^2\right)$$

where:

V = volume of circular cone basin (ft^3)
R_1, R_2 = bottom and surface radii of the conic section (ft)
D = depth of basin (ft)
Z = side slope factor, ratio of horizontal to vertical

3.2.1.2 Stage-Discharge Relationship

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. Figure 3-13 presents an example stage-discharge curve. A typical storage facility has multiple outlets or spillways: a principal outlet that handles the range of design storms and design criteria and a secondary (or emergency) outlet. The principal outlet is usually designed with a capacity sufficient to convey the design discharges and volumes without allowing flow to enter the emergency spillway. Pipes, culverts, weirs, perforated risers and other appropriate outlets can be used in the principal spillway or outlet.

The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal outlet. This spillway should be designed taking into account the potential threat to downstream areas if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway. For more details, see Section 3.3 of this chapter.

3.2.2 Symbols and Definitions

To provide consistency within this section, the symbols listed in Table 3-19 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Figure 3-13. Example Stage-Discharge Curve

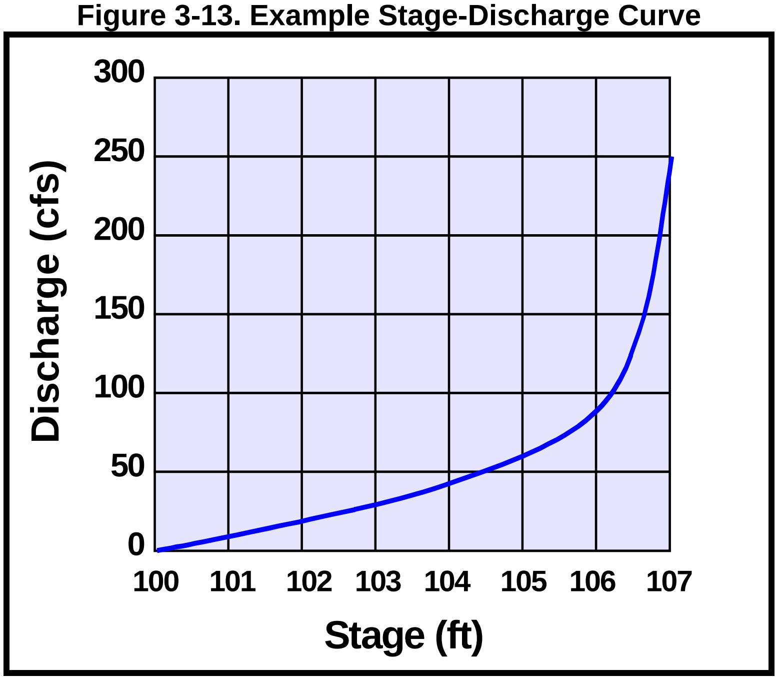


Table 3-19. Symbols and Definitions for Storage Design

Symbol	Definition	Units
a,b	Rainfall factors for Modified Rational Method	-
A	Cross sectional or surface area	ft^2
A	Drainage area	acres (or mi^2)
C_w	Weir coefficient	-
C	Rational Method Runoff Coefficient	-
CN	Curve number	-
CPv	Channel protection volume	acre-ft
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
i	Rainfall intensity	in/hr
L	Length	ft
P_x	Storm depth for x duration storm	in
q_i,q_o	Peak inflow or outflow rate	cfs, in
R	Surface Radii	ft
Rv	Runoff coefficient	-
t	Routing time period	sec
t_b	Time base of hydrograph	sec, hr
Tc	Time of concentration	min
T_d	Critical storm duration	min
T_i	Duration of basin inflow	hr, min, sec
t_P	Time to peak of hydrograph	hr
T_t	Travel time	min
V, V_s	Storage volume	ft^3, acre-ft
V_r	Runoff volume	ft^3, acre-ft
W	Width of basin	ft
WQv	Water quality volume	acre-ft
Z	Side slope factor	-

3.2.3 Flood Protection Storage Design Procedures

This section discusses the general design procedures for designing storage to provide flood protection detention of stormwater runoff for the Qp_2, Qp_{10}, Qp_{25} and Qp_{100}. The design procedures for all storage facilities are the same whether or not they include a permanent pool of water. In the latter case, the permanent pool elevation is taken as the "bottom" of storage and is treated as if it were a solid basin bottom for routing purposes.

The location of a storage facility can have a sizeable impact on the effectiveness of such facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the downstream conveyance system, which could decrease or increase flood peaks in different downstream locations. Therefore, a downstream peak flow analysis (i.e., the 10% rule) should be performed as part of the storage facility design process. In multi-purpose multi-stage facilities such as stormwater ponds, the storage design must be integrated with the overall design for water quality treatment objectives. See Volume 2, Chapter 4 for further guidance and criteria for the design of structural best management practices (BMPs) for water quality control.

3.2.3.1 Design Procedure

The following data are needed for storage design and routing calculations:

- inflow hydrograph for all selected design storms;
- stage-storage curve for proposed storage facility; and
- stage-discharge curve(s) for all outlet control structures.

A general procedure for using the above data in the design of storage facilities is presented below.

1. Compute inflow hydrographs for the 2, 10, 25 and 100-year, 24-hour design storms using the hydrologic methods outlined in Section 3.1. Both existing and post-development conditions hydrographs are required.
2. Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1.
3. Determine the physical basin dimensions necessary to hold the volumes determined in Step 2, including freeboard, which is defined as 1.0 foot above the Qp_{100} water surface elevation to the lowest point in the detention embankment, excluding the emergency spillway. The maximum storage requirement calculated from Step 2 should be used. From the selected basin shape, determine the maximum depth in the pond.
4. Select the type of outlet(s) and size each outlet structure. The outlet type and size will depend on the type of basin (detention, extended detention or retention) as well as the allowable discharge. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure(s) should be sized to convey the allowable discharge at this stage.
5. Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model. If the routed post-development peak discharges (Qp_2, Qp_{10}, Qp_{25} and the Qp_{100}) exceed the existing conditions peak discharges, then revise the available storage volume, outlet device(s), etc., and return to Step 3 until the basin size, basin depth, outlet type and outlet size meet the allowable discharge requirements.
6. Apply the 10% rule (i.e., downstream effects of detention outflows) for the 2-year, 10-year, 25-year and 100-year storms to ensure that the routed hydrograph does not cause peak flow increases, water level increases or downstream flooding problems.
7. Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

Routing hydrographs through storage facilities is critical to the proper facility design and is required in Knox County. Although storage design procedures have been developed that use inflow/outflow analysis without routing, these design procedures have not produced acceptable results in designing detention facilities for many areas of the country, including Knox County.

Although hand calculation procedures are available for routing hydrographs through storage facilities, these procedures are very time consuming, especially when several different designs are evaluated. Many standard hydrology and hydraulics textbooks give examples of hand-routing techniques. For this manual, it is assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given here.

3.2.4 Preliminary Detention Calculations

Procedures for preliminary detention calculations are included here to provide a simple method that can be used to estimate storage needs and also provide a quick check on the results of using different computer programs. Standard routing should be used for actual (final) storage facility calculations and design.

3.2.4.1 Storage Volume Estimation

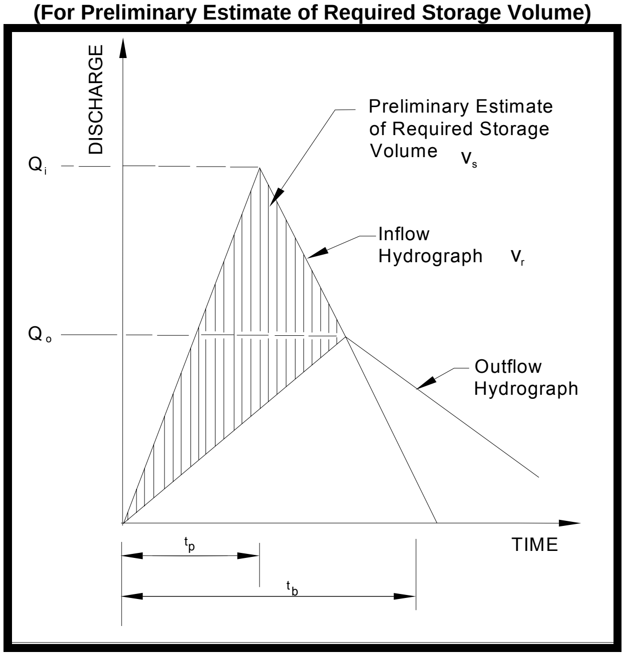
For small drainage areas, a preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular hydrograph shapes shown in Figure 3-14. The required storage volume may be estimated from the hatched area between the inflow and outflow hydrographs. This preliminary storage volume estimate can be calculated using Equation 3-32.

Equation 3-32
$$V_s = 0.5t_b\left(Q_i - Q_o\right)$$

where:

V_s = storage volume estimate (ft^3)
t_b = time base of hydrograph (s)
Q_i = peak inflow rate (cfs)
Q_o = peak outflow rate (cfs)

Figure 3-14. Triangular-Shaped Hydrographs
(For Preliminary Estimate of Required Storage Volume)



3.2.4.2 Alternative Storage Volume Estimation Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained using the following regression equation procedure (Wycoff and Singh, 1976).

1. Determine input data, including the allowable peak outflow rate, Q_O; the peak flow rate of the inflow hydrograph, Q_i; the time base of the inflow hydrograph, t_b; and the time to peak of the inflow hydrograph, t_p.
2. Calculate a preliminary estimate of the ratio V_S/V_r using the input data from Step 1 and Equation 3-33.

Equation 3-33
$$\frac{V_s}{V_r} = \frac{1.291\left(1-\frac{Q_o}{Q_i}\right)^{0.753}}{\left(\frac{t_p}{t_b}\right)^{0.411}}$$

where:

V_s = volume of storage (in)
V_r = volume of runoff (in)
Q_o = outflow peak flow (cfs)
Q_i = inflow peak flow (cfs)
t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak]
t_p = time to peak of the inflow hydrograph (hr)

3. Multiply the volume of runoff, V_r, times the ratio V_s/V_r, calculated in Step 2 to obtain the estimated storage volume V_s.

3.2.4.3 Peak Flow Reduction Estimate

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

1. Determine volume of runoff, V_r; peak flow rate of the inflow hydrograph, Q_i; time base of the inflow hydrograph, t_b; time to peak of the inflow hydrograph, t_p; and storage volume V_s.
2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using Equation 3-34 (Wycoff and Singh, 1976):

Equation 3-34

$$\frac{Q_o}{Q_i} = 1 - 0.712\left(\frac{V_s}{V_r}\right)^{1.328}\left(\frac{t_b}{t_p}\right)^{0.546}$$

where:

Q_o = outflow peak flow (cfs)
Q_i = inflow peak flow (cfs)
V_s = volume of storage (in)
V_r = volume of runoff (in)
t_b = time base of the inflow hydrograph (hr) [Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak]
t_p = time to peak of the inflow hydrograph (hr)

3. Multiply the peak flow rate of the inflow hydrograph, Q_i, by the potential peak flow reduction calculated from Step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume.

3.2.5 Estimation of the Channel Protection Volume

The Simplified SCS Peak Runoff Rate Calculation approach (see Section 3.1.5.4) can be used for estimation of the channel protection volume (CPv) for storage facility design. The calculation procedure is as follows.

Step 1. 2.5 inches is used for the 1-year, 24-hour rainfall depth (P, in inches).

Step 2. A runoff curve number (CN) is then estimated according to the procedure in section 3.1.5.2.

Step 3. The CN value is used to determine the initial abstraction (I_a) from Table 3-13, and the ratio I_a/P is computed.

Step 4. The accumulated runoff (Q_d, inches) can then be calculated using Equation 3-12.

Step 5. Compute the drainage area time of concentration (t_c) for the post-development land use using the method outlined in section 3.1.3.5.

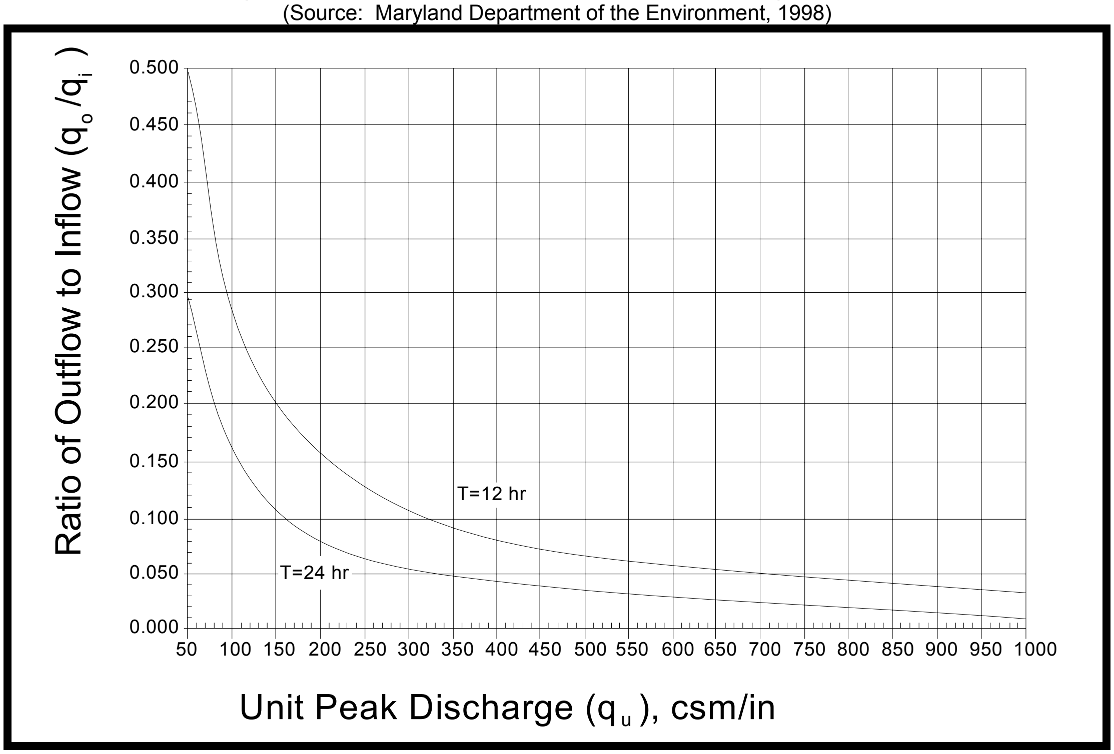
Step 6. Use t_c with the ratio I_a/P to obtain the unit peak discharge, q_u, from Figure 3-6 for the Type

II rainfall distribution. If the ratio I_a/P lies outside the range shown in the figure, either use the limiting values or use another peak discharge method.

Step 7. Knowing q_u and T (extended detention time, minimum of 24 hours and maximum of 72 hours); the q_o/q_i ratio (peak outflow discharge/peak inflow discharge) can be estimated from Figure 3-15.

Figure 3-15. Detention Time vs. Discharge Ratios

(Source: Maryland Department of the Environment, 1998)



Step 8. V_s/V_r is then determined from Figure 3-16 or Equation 3-35, which were developed from the SCS TR-55 hydrologic model using a Type II rainfall distribution.

Equation 3-35

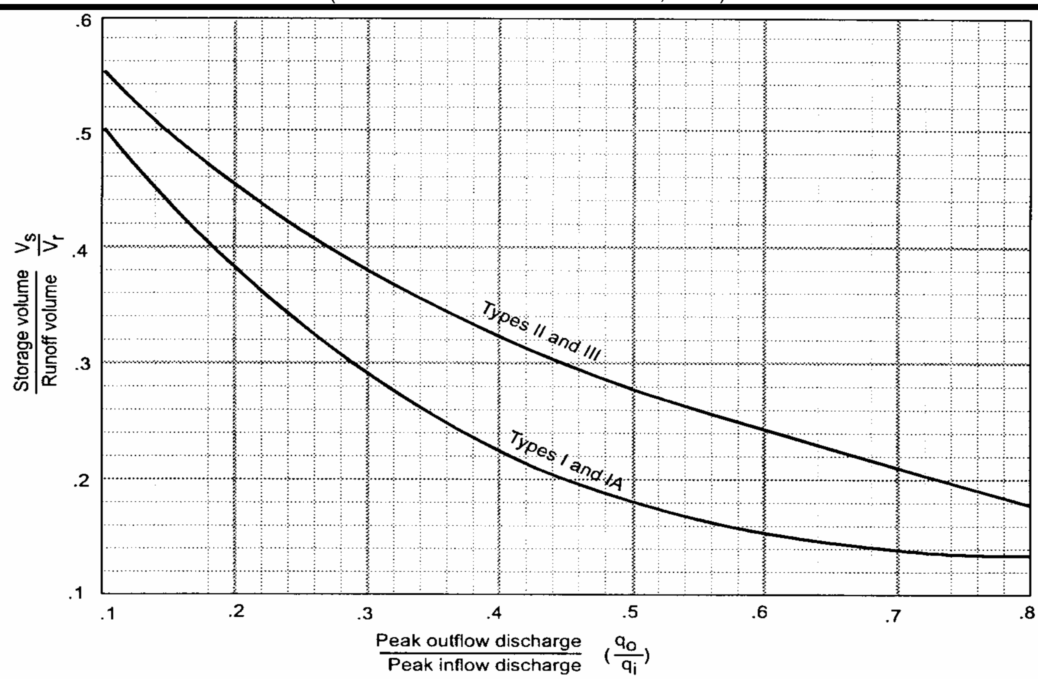
$$\frac{V_s}{V_r} = 0.682 - 1.43\left(\frac{q_o}{q_i}\right) + 1.64\left(\frac{q_o}{q_i}\right)^2 - 0.804\left(\frac{q_o}{q_i}\right)^3$$

where:

V_s = required storage volume (acre-feet) (CPv in this example)
V_r = runoff volume (acre-feet)
q_o = peak outflow discharge (cfs)
q_i = peak inflow discharge (cfs)

This equation is only reliable when $0.1 < q_o/q_i < 0.8$.

Figure 3-16. Approximate Detention Basin Routing for Rainfall Types I, IA, II, and III
(Source: Soil Conservation Service, 1986)



Step 9. The required storage volume (CPv in this case) can then be calculated using Equation 3-36. To check the CPv estimate, the volume must be incorporated into a BMP design and the 1-year 24-hour storm routed through the BMP. The CPv is adequate when the 1-year 24-hour design storm is detained for 24 hours, measured from the centroid of the inflow hydrograph to the centroid of the outflow hydrograph.

Equation 3-36

$$V_s = \frac{\left(\frac{V_s}{V_r}\right) Q_d A}{12}$$

where:

V_s and V_r are defined above
Q_d = the developed runoff for the design storm (inches) (see Figure 3-3)
A = total drainage area (acres)

<u>**Example 3-10. Estimation of CPv**</u>

Estimate the CPv necessary for a 50-acre wooded watershed located in Knox County, which will be developed as follows:

Forest land - good cover (hydrologic soil group B) = 10 ac
Forest land - good cover (hydrologic soil group C) = 10 ac
Residential with 1/3 acre lots (hydrologic soil group B) = 20 ac
Industrial development (hydrological soil group C) = 10 ac
Total impervious area = 18 acres
% of pond and swamp area = 0

<u>Step 1</u> Calculate the rainfall excess.
The 1-year, 24 hour rainfall is 2.5 inches (Table 3-5).

Step 2 Determine the weighted runoff coefficient as in the table below.

Dev. #	Area (ac)	% Total	CN	Composite CN[1]
1	10	20	55	11
2	10	20	70	14
3	20	40	72	28.8
4	10	20	91	18.2
Total	50	100	-	72

1 – Composite CN = $\frac{\%\ Total}{100}$ * CN.

From Equation 3-12 or Figure 3-3, Q_2= 0.53

Step 3 Calculate Ia/P for CN = 72
Ia = 0.778 (Table 3-13)
Ia/P = 0.778/2.5 = 0.31

Step 4 Calculate Qd for 1-year 24-hour storm using Equation 3-12
Qd = (2.5-0.778)2/(2.5-0.778+5*0.778) = 0.53 inches

Step 5 Calculate Tc.
The hydrologic flow path for this watershed = 1,890 ft. It is divided into segments as shown in the table below.

Segment	Type of Flow	Length (ft)	Slope (%)
1	Overland n = 0.24	40	2.00
2	Shallow channel	750	1.70
3	Main channel*	1100	0.50

* For the main channel, n = 0.06 (estimated), width = 10 feet, depth = 2 feet, rectangular channel

Segment 1 – Travel time from Equation 3-4 with P_2 = 3.30 in (0.138 x 24 from Table 3-4)
T_t = $0.007[(0.24)(40)]^{0.8}/(3.30)^{0.5}(0.02)^{0.4}$
= 0.115 hrs = 6.75 minutes

Segment 2 – Travel time from Figure 3-2 or Equation 3-7
V = 2.1 ft/s
T_t = 750/(60)(2.1) = 5.95 min

Segment 3 - Using Equations 3-10 and 3-9
V = $[(1.49)(0.06)(1.43)]^{0.67}/(0.005)^{0.5}$ = 2.23 ft/s
T_t = 1100/60(2.23) = 8.22 min

Therefore, adding the three segments using Equation 3-3:
Tc = 6.75 + 5.95 + 8.22 = 20.92 min = 0.35 hours

Step 6 Calculate unit discharge (q_u) from Figure 3-6 using T_c and Ia from previous steps
Unit discharge q_u (1-year) = 540 csm/in

Step 7 Estimate channel protection volume (CPv = V_S)
Knowing q_u (1-year) = 540 csm/in from Step 6 and T (extended detention time of 24 hours), find q_o/q_i from Figure 3-15.
q_o/q_i = 0.035

Step 8 Estimate storage/runoff using Equation 3-35,

$V_s/V_r = 0.682 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$

$V_s/V_r = 0.682 - 1.43(0.035) + 1.64(0.035)^2 - 0.804(0.035)^3 = 0.63$

Step 9 The necessary channel protection volume is then calculated using Equation 3-36

$CPv = Vx \approx (0.64)(0.53)(50)/12 \approx 1.39$ ac-ft

3.3 Outlet Structures

3.3.1 Symbols and Definitions

To provide consistency within this section as well as throughout this manual, the symbols listed in Table 3-20 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 3-20. Symbols and Definitions for Outlet Structures

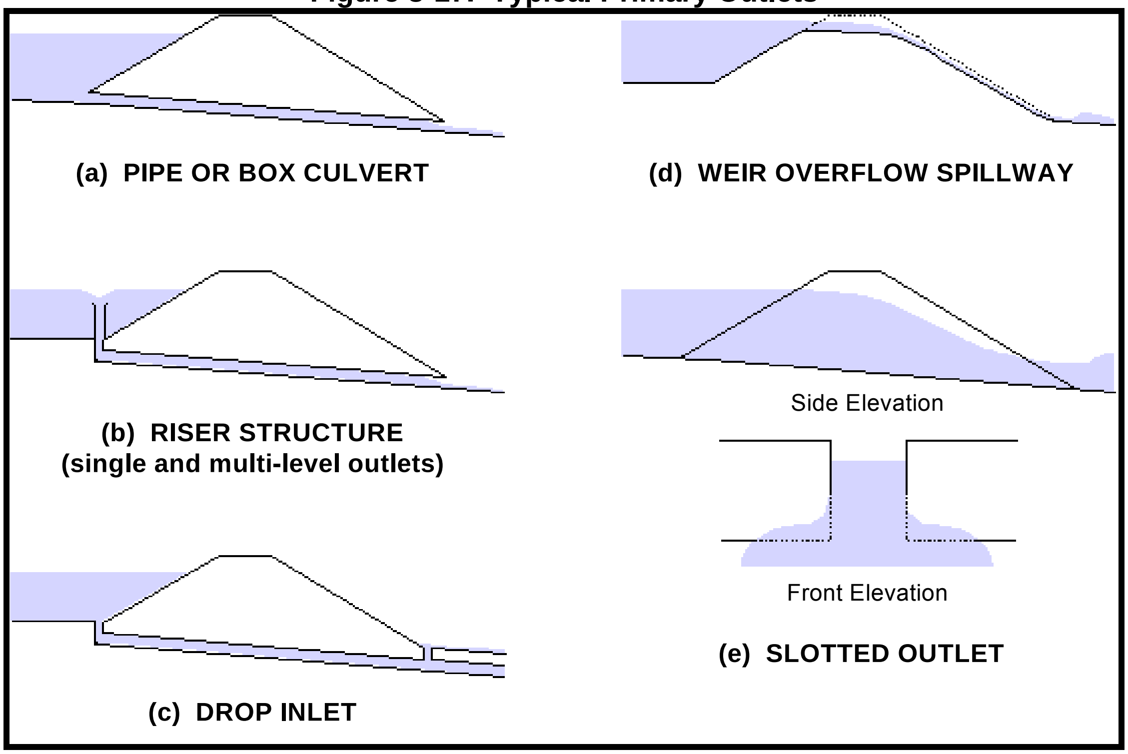
Symbol	Definition	Units
A,a	Cross sectional or surface area	ft^2
A_m	Drainage area	acres (or mi^2)
A_p	Cross sectional area of all holes (perforated riser)	ft^2
B	Breadth of weir	ft
C_w	Weir coefficient or Discharge coefficient	-
C_p	Discharge coefficient for perforations	-
CPv	Channel protection volume	ac-ft
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft/s^2
H	Head on structure	ft
H_C	Height of weir crest above channel bottom	ft
k_m	Coefficient of minor losses (1.0)	-
k_p	Pipe friction coefficient	-
K_g	Bar shape factor	-
L	Length	ft
n	Manning's "n"	-
Q,q	Peak inflow or outflow rate	cfs, in
Q_f	Free flow	cfs
Q_S	Submergence flow	cfs
V_u	Approach velocity	ft/s
WQv	Water quality volume	ac-ft
w	Maximum cross sectional bar width facing the flow	in
x	Minimum clear spacing between bars	in
θ	Angle of v-notch	degrees
θ_γ	Angle of the grate with respect to the horizontal	degrees

3.3.2 Primary Outlets

Primary outlets provide the critical function of the regulation of flow for structural stormwater BMPs. The different types of outlets consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control. Figure 3-17 shows several typical primary outlets. For a single stage system, the stormwater facility can be designed as a simple pipe or culvert. For multi-stage control structures such as an extended detention pond used for control of WQv, CPv, Qp_2, Qp_{10}, Qp_{25}, and Qp_{100}, the inlet is designed considering a range of design flows. A stage-discharge curve is developed for the full range of flows that the structure would experience. The outlets may be housed in a riser or several pipes or culverts in a single location.

This section provides an overview of outlet structure hydraulics and design for stormwater storage facilities. The design engineer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

Figure 3-17. Typical Primary Outlets



3.3.2.1 Outlet Structure Types

There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in stormwater facility design.

- orifice
- perforated riser
- pipe/culvert
- sharp-crested weir
- broad-crested weir
- V-notch weir
- proportional weir
- combination outlet

Each of these outlet types has a different design purpose and application. The control of WQv and CPv flows are normally handled with smaller, more protected outlet structures such as reverse slope pipes, hooded orifices, orifices located within screened pipes or risers, perforated plates or risers, and V-notch weirs. Larger discharges, such as Qp_2, Qp_{10}, Qp_{25} and Qp_{100} flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.

3.3.2.2 Orifices

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate is dependant on the height of the water above the opening and the size and edge treatment of the orifice. For a single orifice, as illustrated in Figure 3-18(a), the orifice discharge can be determined using the standard orifice equation shown in Equation 3-37 below. Figure 3-18(c) shows a perforated riser that has multiple openings.

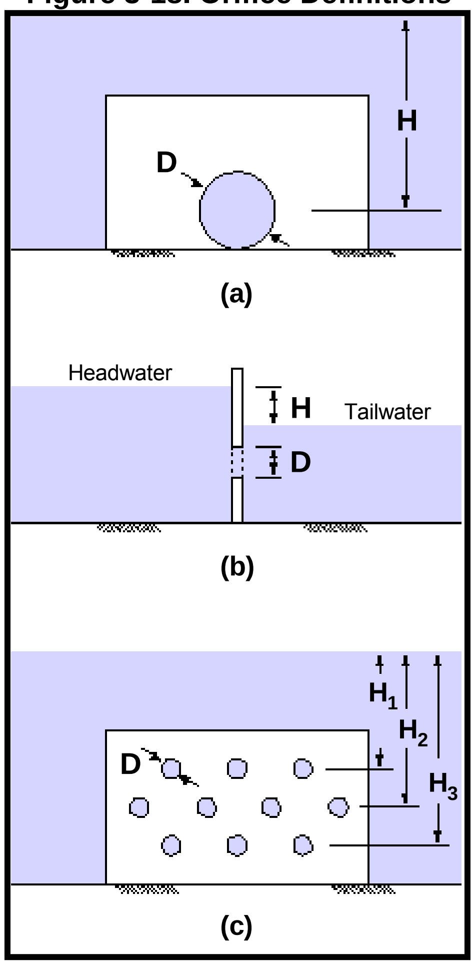
Equation 3-37

$$Q = CA(2gH)^{0.5}$$

where:

Q	= the orifice flow discharge (cfs)
C	= discharge coefficient
A	= cross-sectional area of orifice or pipe (ft^2)
g	= acceleration due to gravity (32.2 ft/s^2)
H	= effective head on the orifice, from the center of orifice to the water surface

Figure 3-18. Orifice Definitions



When the material used for the orifice is thinner than the orifice diameter (i.e., it has sharp edges), a discharge coefficient of 0.6 should be used. When the material is thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges are rounded, a coefficient of 0.92 can be used. Equation 3-38 presents a simplification of the orifice equation that can be used for a round orifice with square-edged entrance conditions:

Equation 3-38 $$Q = 0.6A(2gH)^{0.5} = 3.78D^2H^{0.5}$$

where:

Q = the orifice flow discharge (cfs)
C = discharge coefficient
A = cross-sectional area of orifice or pipe (ft^2)
g = acceleration due to gravity (32.2 ft/s^2)
H = effective head on the orifice, from the center of orifice to the water surface
D = diameter of orifice or pipe (ft)

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces as shown in Figure 3-18(b).

Flow through multiple orifices, such as the perforated plate shown in Figure 3-18(c), can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

Perforated orifice plates for the control of discharge can be of any size and configuration. However, the Denver Urban Drainage and Flood Control District has developed standardized dimensions that have worked well. These are presented in Table 3-21.

Table 3-21. Circular Perforation Sizing

(Source: Urban Drainage and Flood Control District, 1999)

Hole Diameter (in)	Minimum Column Hole Centerline Spacing (in)	Flow Area per Row (in^2)		
		N=1	N=2	N=3
0.25	1	0.05	0.1	0.15
0.3125	2	0.08	0.15	0.23
0.375	2	0.11	0.22	0.33
0.4375	2	0.15	0.3	0.45
0.50	2	0.2	0.4	0.6
0.5625	3	0.25	0.5	0.75
0.625	3	0.31	0.62	0.93
0.6875	3	0.37	0.74	1.11
0.75	3	0.44	0.88	1.32
0.8125	3	0.52	1.04	1.56
0.875	3	0.6	1.2	1.8
0.9375	3	0.69	1.38	2.07
1.0	4	0.79	1.58	2.37
1.0625	4	0.89	1.78	2.67
1.125	4	0.99	1.98	2.97
1.1875	4	1.11	2.22	3.33
1.25	4	1.23	2.46	3.69
1.3125	4	1.35	2.7	4.05
1.375	4	1.48	2.96	4.44
1.4375	4	1.62	3.24	4.86
1.50	4	1.77	3.54	5.31
1.5625	4	1.92	3.84	5.76
1.625	4	2.07	4.14	6.21
1.6875	4	2.24	4.48	6.72
1.75	4	2.41	4.82	7.23
1.8125	4	2.58	5.16	7.74

Hole Diameter (in)	Minimum Column Hole Centerline Spacing (in)	Flow Area per Row (in^2)		
		N=1	N=2	N=3
1.875	4	2.76	5.52	8.28
1.9375	4	2.95	5.9	8.85
2.0	4	3.14	6.28	9.42
N = Number of columns of perforations				

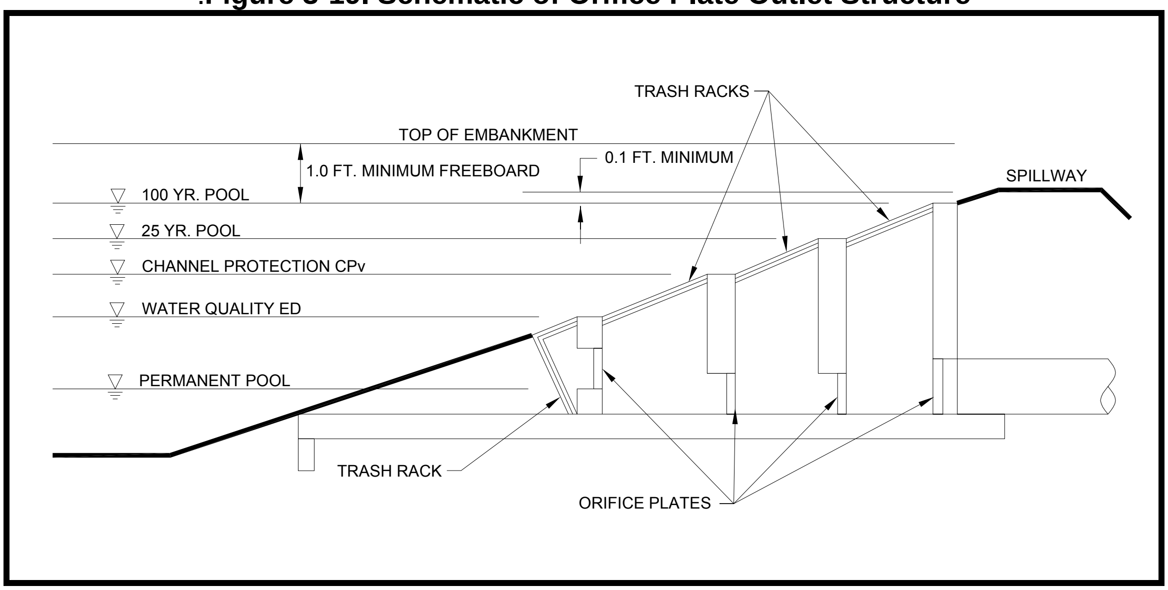
Minimum steel plate thickness	0.25”	0.3125”	0.375”

The vertical spacing between orifice row hole centerlines is always 4-inches.

Only one column of rectangular slots is allowed unless additional columns are hydraulically independent.

Figure 3-19 provides a schematic of an orifice plate outlet structure for a wet ED pond showing the design pool elevations and the flow control mechanisms. For simplicity, the outlets for the 2-year and 10-year pools are not shown.

.Figure 3-19. Schematic of Orifice Plate Outlet Structure



3.3.2.3 Perforated Risers

A special kind of orifice is a perforated riser as illustrated in Figure 3-20. In the perforated riser, an orifice plate at the bottom of the riser, or in the outlet pipe just downstream from the elbow at the bottom of the riser, controls the flow. It is important that the perforations in the riser convey more flow than the orifice plate so that the perforations do not become the control. Referring to Figure 3-20, Equation 3-39 presents a formula that has been developed to estimate the total flow capacity of the perforated section (McEnroe, 1988).

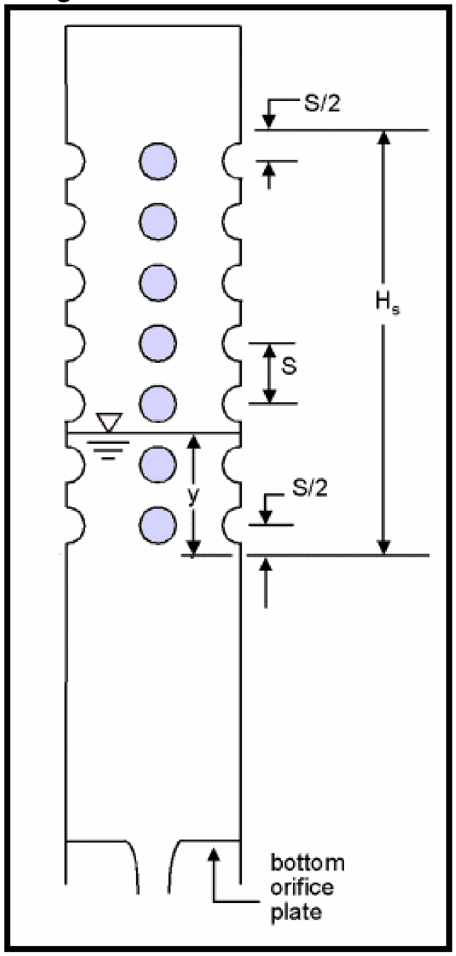
Equation 3-39

$$Q = C_p \frac{2A_p}{3H_s} \sqrt{2g} H^{3/2}$$

where:

- Q = discharge (cfs)
- C_p = discharge coefficient for perforations (normally 0.61)
- A_p = cross-sectional area of all the holes (ft^2)
- g = acceleration due to gravity (32.2 ft/s^2)
- H_s = distance from S/2 below the lowest row of holes to S/2 above the top row (ft)

Figure 3-20. Perforated Riser



3.3.2.4 Pipes and Culverts

Discharge pipes are often used as outlet structures for stormwater control facilities. The design of these pipes can be for either single or multi-stage discharges. A reverse-slope, underwater pipe is often used for water quality or channel protection outlets.

Pipes smaller than 12 inches in diameter may be analyzed as a submerged orifice as long as *H/D* is greater than 1.5. *Note: For low flow conditions, when the flow reaches and begins to overflow the pipe, discharge over the weir is the controlling flow. As the stage increases the controlling flow will transition to the flow through the orifice.* Pipes greater than 12 inches in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet control culvert nomographs and procedures given in Chapter 7 or by using Equation 3-39 (NRCS, 1984). Equation 3-40 is a general pipe flow equation that is derived through the use of the Bernoulli and continuity principles, and should only be used for pipes flowing full. The use of FHWA HDS-5 methodology, and HY-8 or similar software is highly recommended for analysis of culverts. Concrete pipe is required for outflow pipes of all flood protection BMPs.

Equation 3-40

$$Q = a\left[\frac{(2gH)}{(1 + k_m + k_p L)}\right]^{0.5}$$

where:

Q = discharge (cfs)
a = pipe cross sectional area (ft^2)
g = acceleration of gravity (ft/s^2)
H = elevation head differential (ft)
k_m = coefficient of minor losses (use 1.0)
k_p = Manning's "n" friction head loss / unit length = $5087 * n^2/dia(in)^{4/3}$
L = pipe length (ft)

3.3.2.5 Sharp-Crested Weirs

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear of the weir plate as it overflows, the overflow is termed a *sharp-crested* weir. The weir's crest is the edge where the water flows over the weir. If the sides of the weir also cause the flow through the weir to contract, it is termed an *end-contracted* sharp-crested weir. Sharp-crested weirs have stable stage-discharge relations and are often used as a flow measurement device. A sharp-crested weir with no end contractions is illustrated in Figure 3-21(a). The discharge equation for this configuration is presented in Equation 3-41 (Chow, 1959).

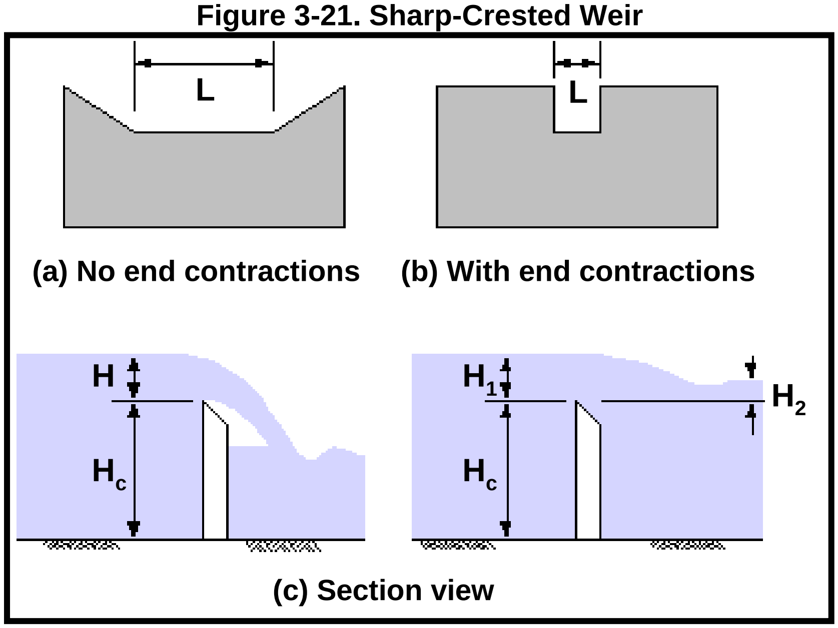
Equation 3-41

$$Q = \left[3.27 + 0.4\left(\frac{H}{H_c}\right)\right] L H^{1.5}$$

where:

Q = discharge (cfs)
H = head above weir crest excluding velocity head (ft)
H_c = height of weir crest above channel bottom (ft)
L = horizontal weir length (ft)

Figure 3-21. Sharp-Crested Weir



A sharp-crested weir with two end contractions is illustrated in Figure 3-21(b). Equation 3-42 presents the discharge equation for this configuration (Chow, 1959).

Equation 3-42

$$Q = \left[3.27 + 0.4\left(\frac{H}{H_c}\right)\right] (L - 0.2H) H^{1.5}$$

where:

Q = discharge (cfs)
H = head above weir crest excluding velocity head (ft)
H_c = height of weir crest above channel bottom (ft)
L = horizontal weir length (ft)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. Equation 3-43 presents the discharge equation for a submerged sharp-crested weir (Brater and King, 1976).

Equation 3-43

$$Q_s = Q_f \left[1 - \left(\frac{H_2}{H_1} \right)^{1.5} \right]^{0.385}$$

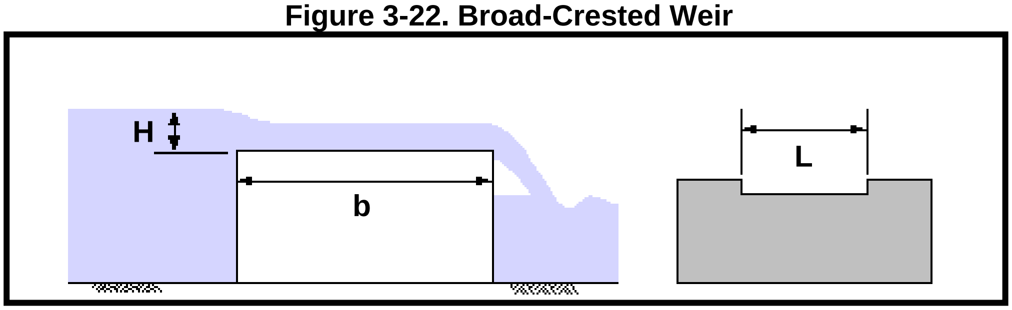
where:

Q_s = submergence flow (cfs)
Q_f = free flow (cfs)
H_1 = upstream head above crest (ft)
H_2 = downstream head above crest (ft)

3.3.2.6 Broad-Crested Weirs

A weir in the form of a relatively long raised channel control crest section is a *broad-crested* weir as shown in Figure 3-22. The flow control section can have different shapes, such as triangular or circular. True broad-crested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. For example, a thick wall or a flat stop log can act like a sharp-crested weir when the approach head is large enough that the flow springs from the upstream corner. If upstream head is small enough relative to the top profile length, the stop log can act like a broad-crested weir (USBR, 1997).

Figure 3-22. Broad-Crested Weir



Equation 3-44 presents the discharge equation for a broad-crested weir (Brater and King, 1976).

Equation 3-44

$$Q = CLH^{1.5}$$

where:

Q = discharge (cfs)
C = broad-crested weir coefficient
L = broad-crested weir length perpendicular to flow (ft)
H = head above weir crest (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent flow contraction and if the slope of the crest is as great as the friction head loss, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.32. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 3-22.

Table 3-22. Broad-Crested Weir Coefficient (C) Values
(Source: Brater and King, 1976)

Measured Head (H)[1] in feet	Weir Crest Breadth (b) in feet										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.27	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

1 - Measured at least 2.5H upstream of the weir.

3.3.2.7 V-Notch Weirs

Equation 3-45 presents the discharge equation for a V-notch weir (Brater and King, 1976). Figure 3-23 presents an example V-notch weir.

Equation 3-45

$$Q = 2.5 \tan\left(\frac{\theta}{2}\right) H^{2.5}$$

where:

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

3.3.2.8 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary non-linearly with head. A typical proportional weir is shown in Figure 3-24.

Figure 3-23. V-Notch Weir

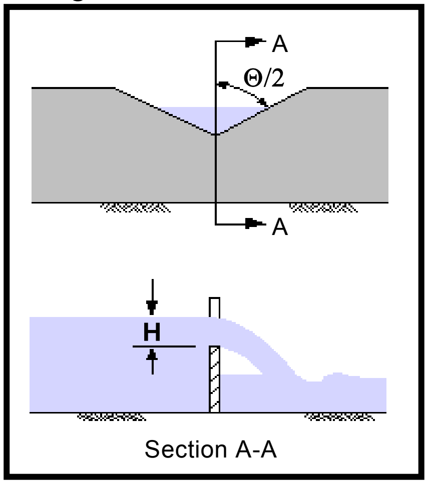
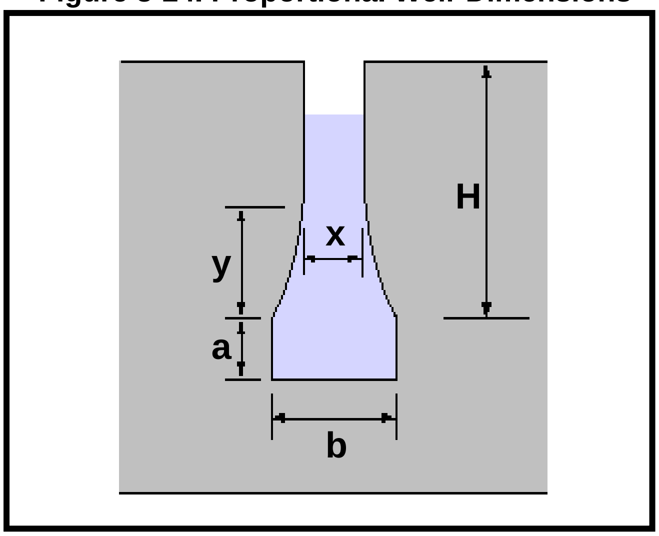


Figure 3-24. Proportional Weir Dimensions



Equations 3-46 and 3-47 present the design equations for proportional weirs (Sandvik, 1985).

Equation 3-46

$$Q = 4.97a^{0.5}b\left(H - \frac{a}{3}\right)$$

Equation 3-47

$$\frac{x}{b} = 1 - \left(\frac{1}{3.17}\right)\left[\arctan\left(\frac{y}{a}\right)^{0.5}\right]$$

where:

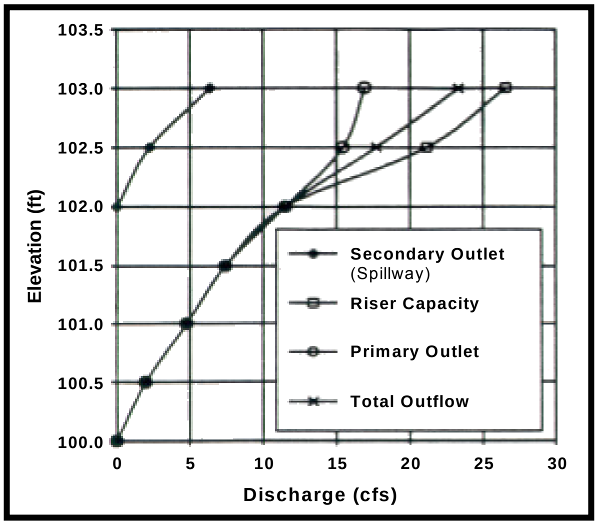
Q = discharge (cfs)

Dimensions a, b, H, x, and y are shown in Figure 3-24

3.3.2.9 Combination Outlets

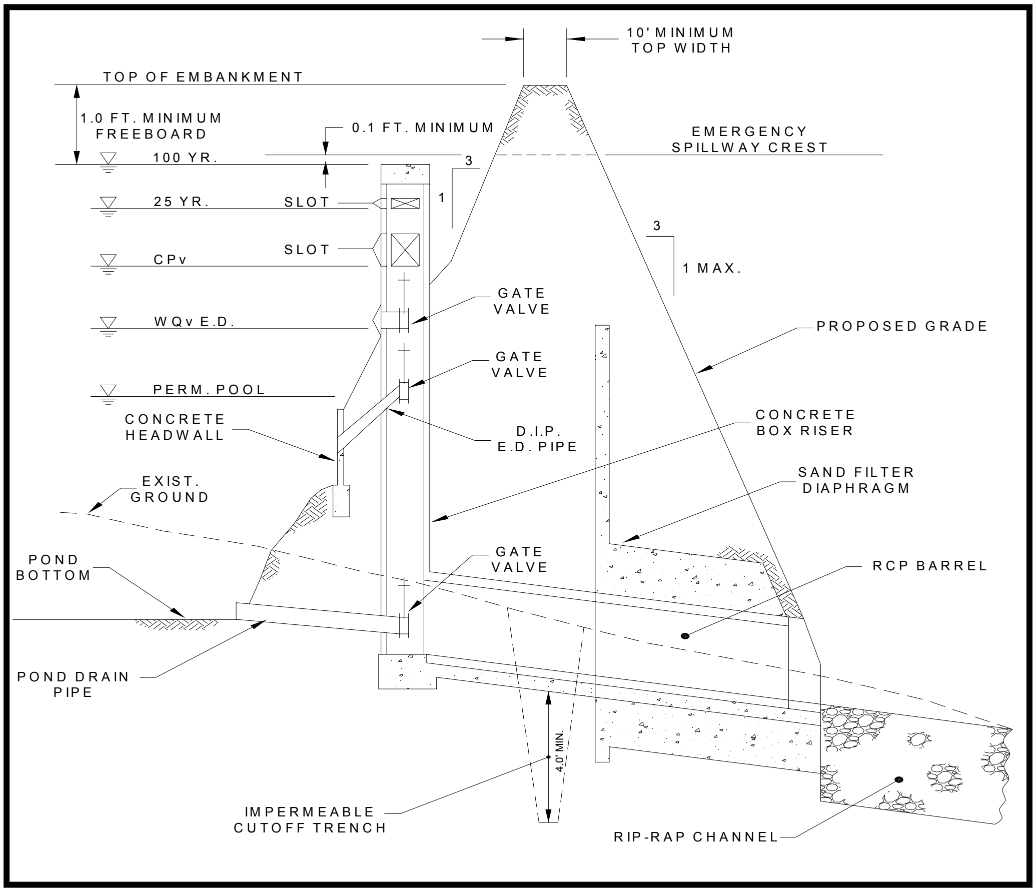
Combinations of orifices, weirs and pipes are typically used to provide multi-stage outlet control for different control volumes/discharges within a storage facility (i.e., WQv, CPv, Qp_2, Qp_{10}, Qp_{25} and Qp_{100}). The use of a combination outlet requires the construction of a composite stage-discharge curve suitable for control of multiple storm flows. An example composite curve is presented in Figure 3-25. The design of multi-stage combination outlets is discussed in detail in Section 3.3.4.

Figure 3-25. Composite Stage-Discharge Curve



There are generally of two types of combination outlets: *shared outlet control structures and separate outlet controls*. *Shared outlet control* is typically a number of individual outlet openings (orifices), weirs or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Figure 3-26 shows an example of a riser designed for a wet pond. The orifice plate outlet structure in Figure 3-20 is another example of a combination outlet.

Figure 3-26. Schematic of Shared Outlet Control Structure



Separate outlet controls are less common and may consist of several pipe or culvert outlets at different levels in the storage facility that are either discharged separately or are combined to discharge at a single location.

3.3.3 The Design of Extended Detention Outlets

3.3.3.1 Outlet Sizing

Extended detention (ED) orifice sizing is required in design applications that provide extended detention for the ED portion of the water quality volume (WQv). In both cases an extended detention orifice or reverse slope pipe can be used for the outlet. For a structural control facility providing both WQv extended detention and CPv control (wet ED pond, micropool ED pond, and shallow ED wetland), there will be a need to design two outlet orifices. The water quality control outlet will be sized using drawdown time principles and the channel protection outlet will be sized through routing to achieve a minimum of 24 hours separation between the centroid of the inflow hydrograph and the outflow hydrograph for the 1-year 24-hour design storm.

(The following procedures are based on the water quality outlet design procedures included in the Virginia Stormwater Management Handbook, 1999)

In an extended detention facility for water quality treatment, the storage volume is detained and released over a specified amount of time (e.g., no less than 24-hours). The release period is a brim drawdown time, with the assumption that the entire WQv is present in the basin at the beginning of drawdown. The entire calculated volume drains out of the basin over no less than 24 hours. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention outlet can be sized using either of the following two methods:

1. Use the maximum hydraulic head associated with the storage volume and maximum flow, and approximate the orifice size needed to achieve the required drawdown time.
2. Use a drawdown analysis to determine the drawdown time.

These two procedures are outlined in the examples below.

Example 3-11. ED Outlet Design Method 1: Maximum Hydraulic Head

A wet ED pond sized for the required water quality volume will be used here to illustrate the sizing procedure for an extended-detention orifice. Given the following information, calculate the required orifice size for water quality design.

- Water Quality Volume (WQv) = 0.76 ac-ft = 33,106 ft^3
- Maximum Hydraulic Head (H_{max}) = 5.0 ft (from stage vs. storage data)

Step 1. Determine the maximum discharge resulting from the 24-hour drawdown requirement. It is calculated by dividing the WQv by the required time to find the average discharge, and then multiplying by two to obtain the maximum discharge.

Q_{avg} = 33,106ft^3/(24hr)(3,600sec/hr) = 0.38 cfs
Q_{max} = $2Q_{avg}$ = 0.76 cfs

Step 2. Determine the required orifice diameter by using the orifice equation (Equation 3-37) and Q_{max} and H_{max}:

$Q = CA(2gH)^{0.5}$, or $A = Q/C(2gh)^{0.5}$
$A = 0.76/0.6[(2)(32.2)(5.0)]^{0.5} = 0.071\ ft^2$

Step 3. Determine pipe diameter
A = $3.14d^2/4$, then $d = (4A/3.14)^{0.5}$
D = $[4(0.071)/3.14]^{0.5}$ = 0.30 ft = 3.61 inches

Therefore, use a 3.6-inch diameter water quality orifice.

Example 3-12. ED Outlet Design Method 2: Drawdown Analysis

Using the data from the previous example (Example 3-11) use Method 2 to calculate the size of the outlet orifice. Use of a spreadsheet is highly recommended.

- Water Quality Volume (WQv) = 0.76 ac-ft = 33,106 ft^3
- Maximum Hydraulic Head (H_{max}) = 5.0 ft (from stage vs storage data)

Step 1. Determine the pond stage-storage curve at increments of 0.1' or less.

Step 2. Choose pond water elevation (first increment at H_{max}, others at end elevation of previous increment).

Step 3. Assume an orifice size:
Orifice diameter = 1"
Orifice area = $(\pi/4)*(\text{Diam}/12)^2$
Orifice area = $(3.14/4)*(1/12)^2 = 0.00545\ ft^2$

Step 4. Calculate flowrate at water surface elevation using orifice equation:
$Q = CA(2gH)^{0.5}$
$Q = 0.6*0.00545*(2*32.2*5)^{0.5}$
Q = 0.0587 cfs

Step 5. Calculate time to drain pond volume increment (keeping track of elapsed time):
Time = Volume/Flowrate (Volume of increment from stage-storage curve)
Time = 200/.0587 = 3407 seconds = 56.8 minutes

Step 6. Repeat steps 1 through 5 for each elevation from WQv elevation to orifice center (keeping track of elapsed time).

Step 7. Check whether total drawdown time is greater than 24-hours:

3.3.3.2 Outlet Protection

Small low flow orifices such as those used for extended detention applications can easily clog, preventing the structural control from meeting its design purpose(s) and potentially causing adverse impacts. Therefore, extended detention orifices need to be adequately protected from clogging. There are a number of different anti-clogging designs, including:

- The use of a *reverse slope pipe* attached to a riser for a stormwater pond or wetland with a permanent pool. This configuration is presented in Figure 3-27. The inlet is submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond.
- The use of a *hooded outlet* for a stormwater pond or wetland with a permanent pool. This configuration is shown in Figures 3-28 and 3-29.

- Internal orifice protection through the use of an *over-perforated vertical stand pipe with ½-inch orifices or slots* that are protected by wirecloth and a stone filtering jacket. This configuration is shown in Figure 3-30.

- Internal orifice protection through the use of adjustable gate valves can achieve an equivalent orifice diameter. This configuration is not shown in a figure.

Figure 3-27. Reverse Slope Pipe Outlet

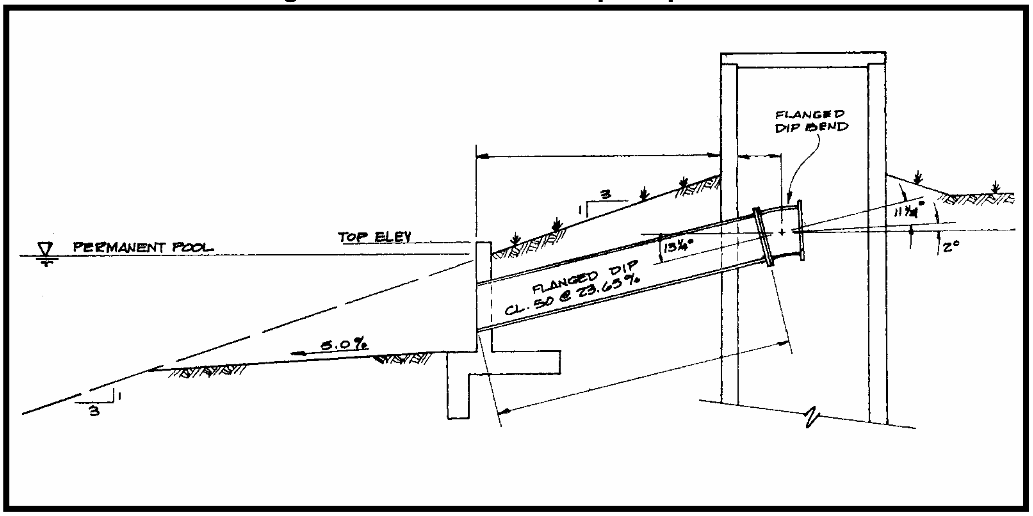


Figure 3-28. Hooded Outlet

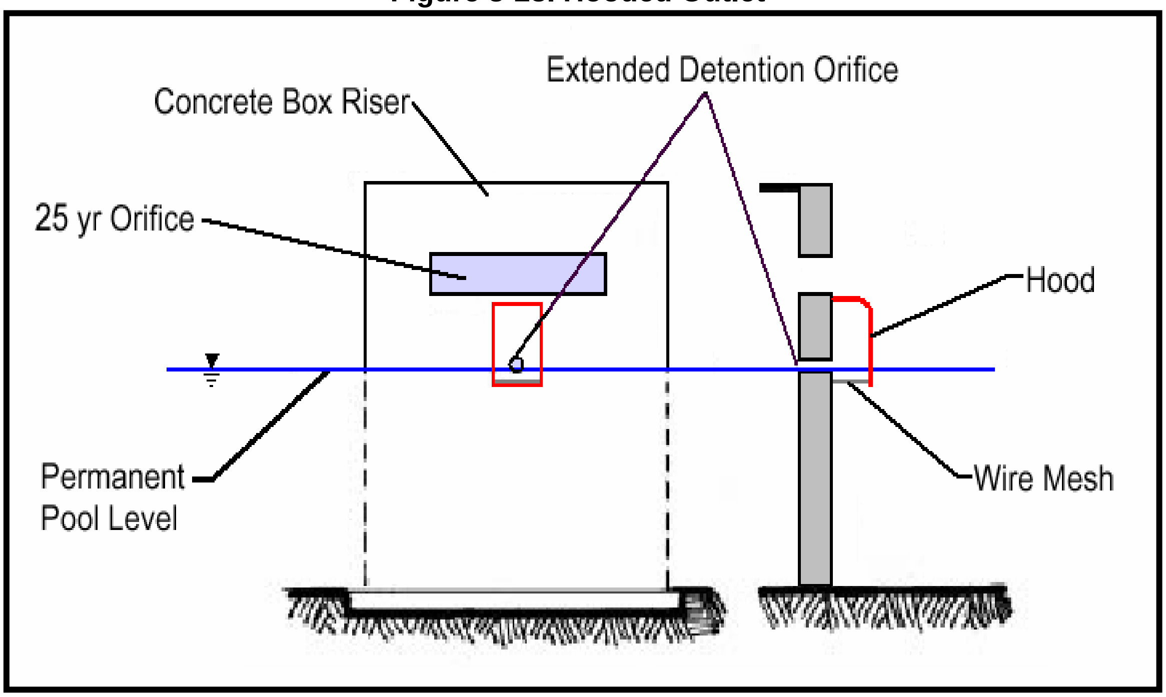


Figure 3-29. Half-Round CMP Orifice Hood

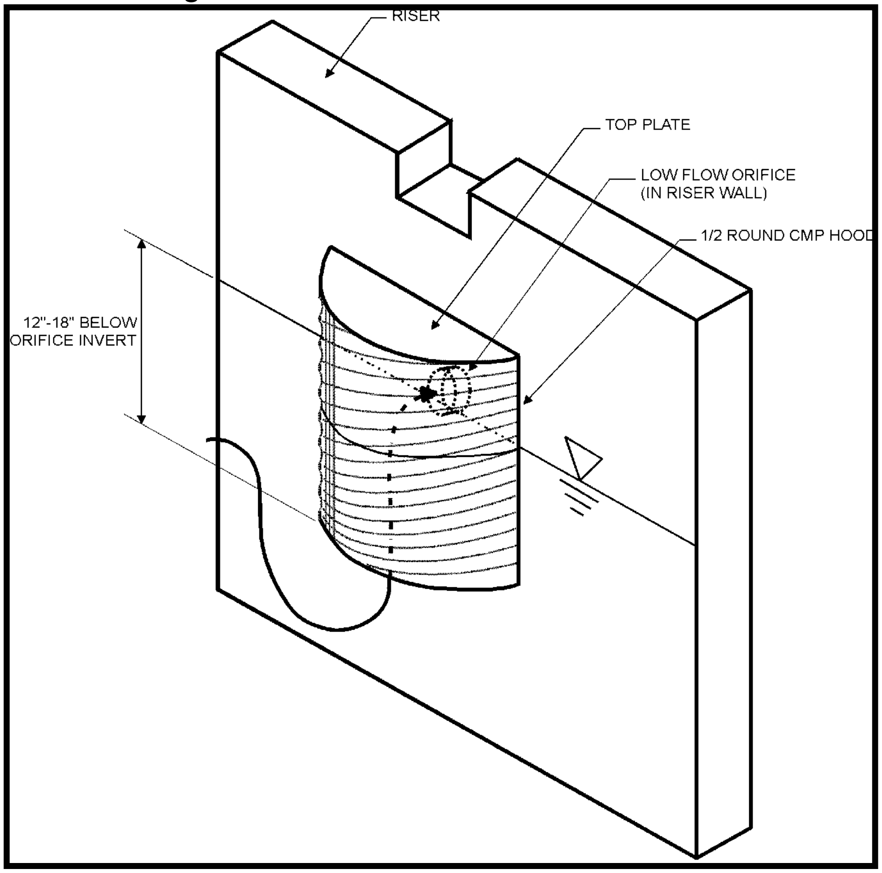
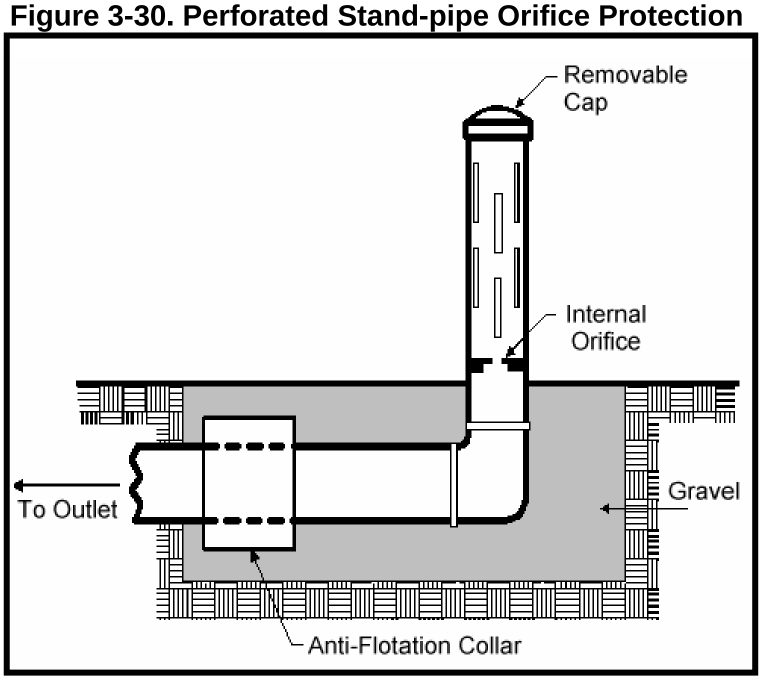


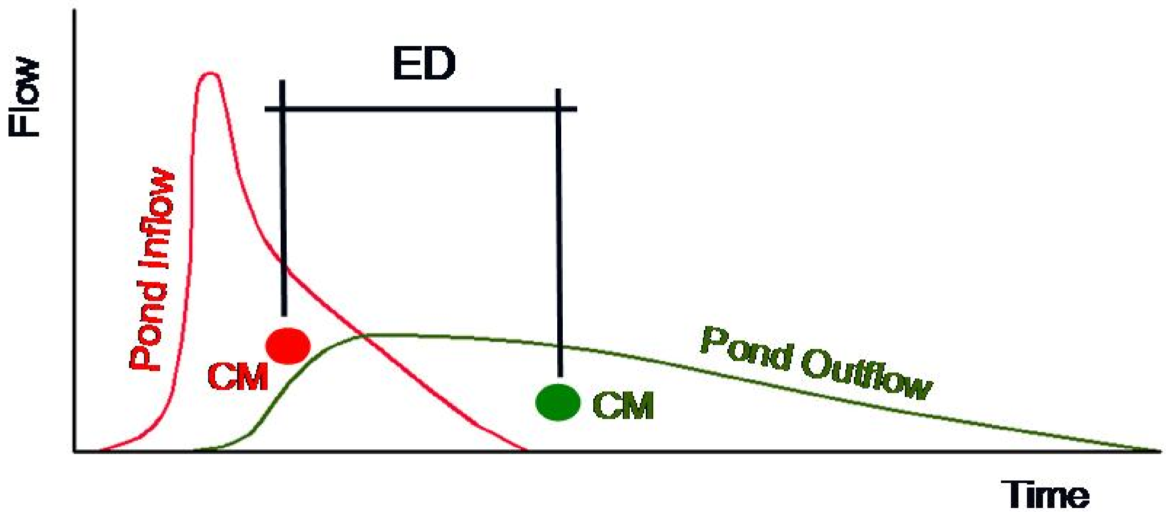
Figure 3-30. Perforated Stand-pipe Orifice Protection



3.3.4 The Design of Channel Protection Outlets

The purpose of channel protection outlets is to prevent the erosive channel-forming flows that occur during the 1 to 2 year storm. This purpose is accomplished by extending the detention of the 1-year 24-hour design storm to 24 hours. The detention time is measured from the centroid of the inflow hydrograph to the centroid of the outflow hydrograph as shown below.

Figure 3-31. Channel Protection Hydrograph



3.3.4.1 Outlet Sizing

Channel protection outlets, then, must be sized using hydrograph routing techniques. The channel protection volume estimated in section 3.2.5 will have a channel protection outlet placed at the bottom of it. The size of the outlet can only be estimated initially. Routing the 1-year 24-hour inflow hydrograph through the pond will provide an outflow hydrograph. If the detention time is less than 24 hours, the channel protection orifice must be made smaller. The water quality orifice may preclude reaching the 24 hour detention time, in which case, the water quality orifice must be made smaller. The water quality and channel protection orifices can be combined so long as both water quality and channel protection criteria are met.

3.3.5 Multi-Stage Outlet Design

A combination outlet such as a multiple orifice plate system or multi-stage riser is often used to provide adequate hydraulic outlet controls for the different design requirements (e.g., WQv, CPv, Qp_2, Qp_{10}, Qp_{25} and Qp_{100}) for stormwater ponds, stormwater wetlands and detention-only facilities. Separate openings or devices at different elevations are used to control the rate of discharge from a facility during multiple design storms. Figure 3-26 (shown previously) is an example of multi-stage combination outlet systems.

A design engineer must be creative to provide the most economical and hydraulically efficient outlet design possible in designing a multi-stage outlet. Many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. The stage-discharge table or rating curve is a composite (previously shown in Figure 3-25) of the different outlets that are used for different elevations within the multi-stage riser.

3.3.5.1 Multi-Stage Outlet Design Procedure

Below are the steps for designing a multi-stage outlet. Note that if a structural control facility will not control one or more of the required storage volumes (WQv, CPv, Qp_2, Qp_{10}, Qp_{25}, and Qp_{100}), then that step in the procedure is skipped.

1. Determine Stormwater Control Volumes. Using the procedures from Sections 3.1 and 3.2, estimate the required storage volumes for water quality treatment (WQv), channel protection (CPv), and overbank flood control (Qp_2, Qp_{10}, Qp_{25}) and extreme flood control (Qp_{100}).

2. Develop Stage-Storage Curve. Using the site geometry and topography, develop the stage-storage curve for the facility in order to provide sufficient storage for the control volumes involved in the design.

3. Design the Water Quality Volume Outlet. Design the water quality orifice. If a permanent pool is incorporated into the design of the facility, a portion of the storage volume for water quality may be above the elevation of the permanent pool. The outlet can be protected using either a reverse slope pipe, a hooded protection device, or another acceptable method. Design the water quality volume outlet using either Method 1 or 2 from subsection 3.3.3.

4. Design the Channel Protection Volume Outlet. For this design, the storage needed for channel protection may be “stacked” on top of the water quality volume storage elevation determined in Step 3. The total stage-discharge rating curve at this point will include the water quality control orifice and the outlet used for stream channel protection. Use hydrograph routing software to ensure that the 1-year 24-hour storm has been detained for a minimum of 24 hours, centroid to centroid. The water quality outlet may be too large for the channel protection criteria to be met, in which case the water quality outlet must be made smaller and the routing re-run. The outlet should be protected in a manner similar to that for the water quality orifice.

5. Design Overbank Flood Protection Outlet. The overbank protection volume is added above the water quality and channel protection storage. Establish the Qp_2, Qp_{10}, Qp_{25} maximum water surface elevations using the stage-storage curve to find the 2-year, 10-year and 25-year maximum head. Select an outlet type and calculate the initial size and geometry based upon maintaining the pre-development 2-year, 10-year and 25-year peak discharge rates. Develop a stage-discharge curve for the combined set of outlets (WQv, CPv and Qp_2, Qp_{10}, Qp_{25}). This procedure is repeated for control (peak flow attenuation) of the 100-year storm (Qp_{100}).

6. Check Performance of the Outlet Structure. Perform a hydraulic analysis of the multi-stage outlet structure using reservoir routing to ensure that all outlets will function as designed. Ensure that post-developed peak flows are less than pre-developed peak flows for all design storms at the pond outlet, and downstream per the 10% rule. Also, check that the CPv criteria are still being met. Several iterations may be required to calibrate and optimize the pond shape and outlets that are used. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This usually requires that the structure have a larger cross-sectional area than the outlet conduit. The hydraulic analysis of the design must take into account the hydraulic changes that will occur as depth of storage changes for the different design storms. As shown in Figure 3-32, as the water passes over the rim of a riser, the riser acts as a weir. However, when the water surface reaches a certain height over the rim of a riser, the riser will begin to act as a submerged orifice. The designer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place for an outlet where this change in hydraulic conditions will occur. Also note in Figure 3-32 that as the elevation of the water increases further, the control can change from barrel inlet flow control to barrel pipe flow control. Figure 3-33 shows another condition where weir flow can change to orifice flow, which must be taken into account in the hydraulics of the rating curve as different design conditions result in changing water surface elevations.

Figure 3-32. Riser Flow Diagrams
(Source: Virginia Department of Conservation and Recreation, 1999)

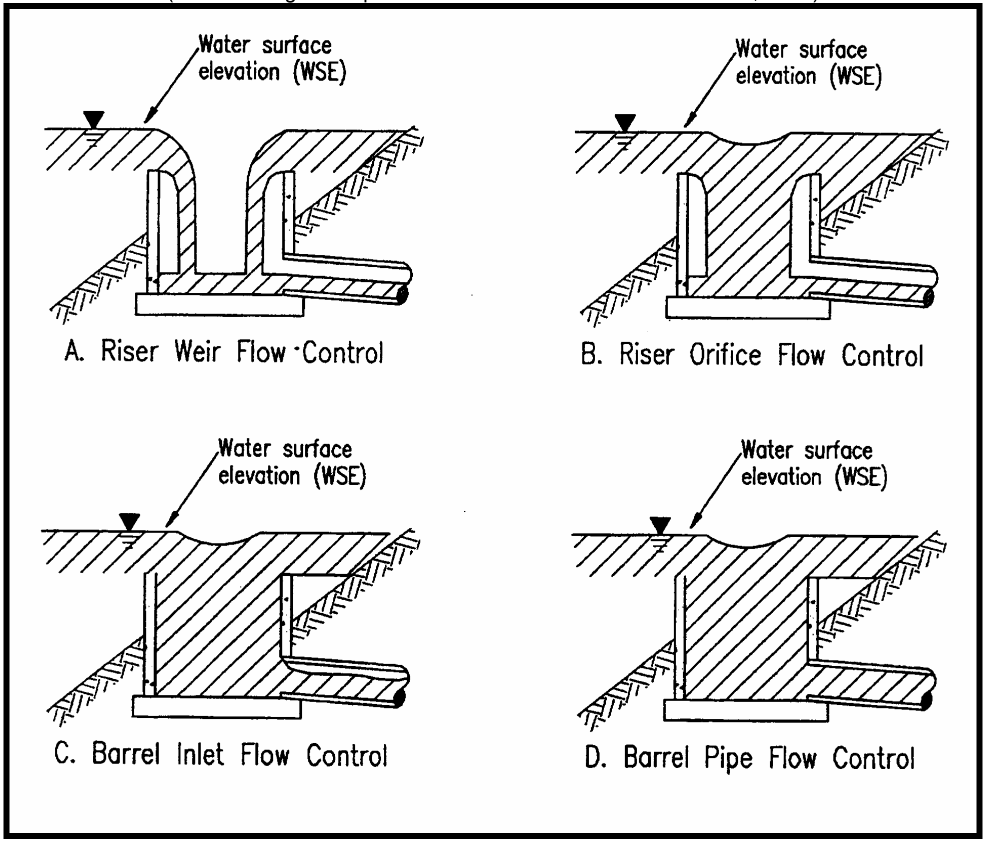
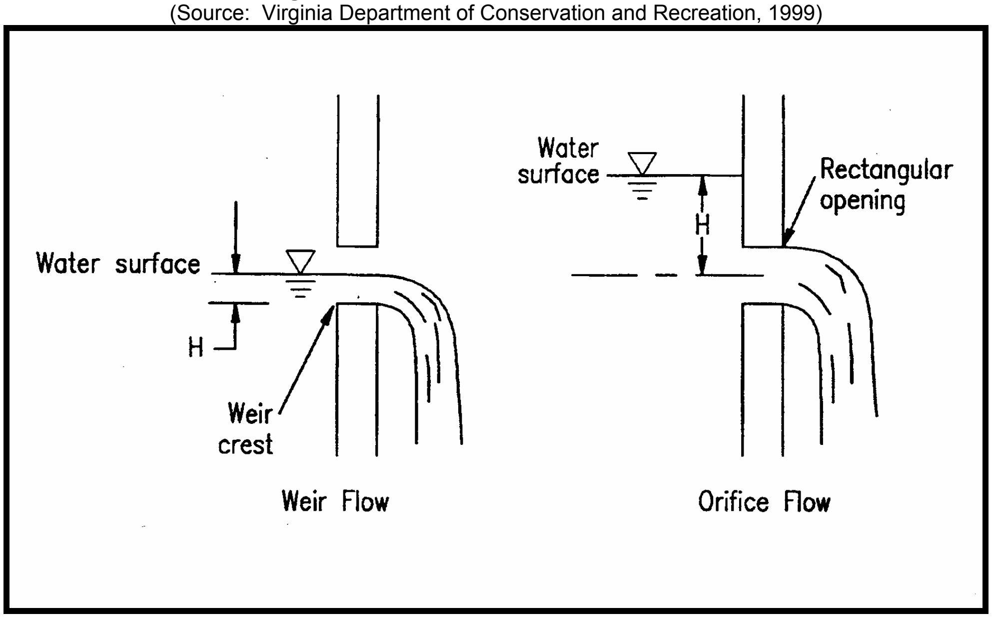


Figure 3-33. Weir and Orifice Flow
(Source: Virginia Department of Conservation and Recreation, 1999)



7. Size the Emergency Spillway. It is recommended that all stormwater impoundment structures have an emergency spillway. An emergency spillway provides a degree of safety to prevent overtopping of an embankment if the primary outlet or principal spillway should become clogged, or otherwise inoperative. The 100-year storm should be routed through the outlet devices and emergency spillway to ensure the hydraulics of the system will operate as designed.
8. Design Outlet Protection. Design necessary outlet protection and energy dissipation facilities to avoid erosion problems downstream from outlet devices and emergency spillway(s). Refer to Chapter 7 for more information.
9. Perform Buoyancy Calculations. Perform buoyancy calculations for the outlet structure and footing. Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water.
10. Provide Seepage Control. Seepage control should be provided for the outflow pipe or culvert through an embankment. The two most common devices for controlling seepage are (1) filter and drainage diaphragms and (2) anti-seep collars.

3.3.6 Trash Racks and Safety Grates

The susceptibility of larger inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances, trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions. When properly designed, installed, and maintained, trash racks:

- keep debris away from the entrance to the outlet works where the debris will not clog the critical portions of the structure;
- capture debris in such a way that relatively easy removal is possible;
- ensure that people and large animals are kept out of confined conveyance and outlet areas; and,
- provide a safety system that prevents humans and animals from being drawn into the outlet and allows them to climb to safety.

Trash racks can serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985; Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors, including head losses through the rack, structural convenience, safety and size of outlet.

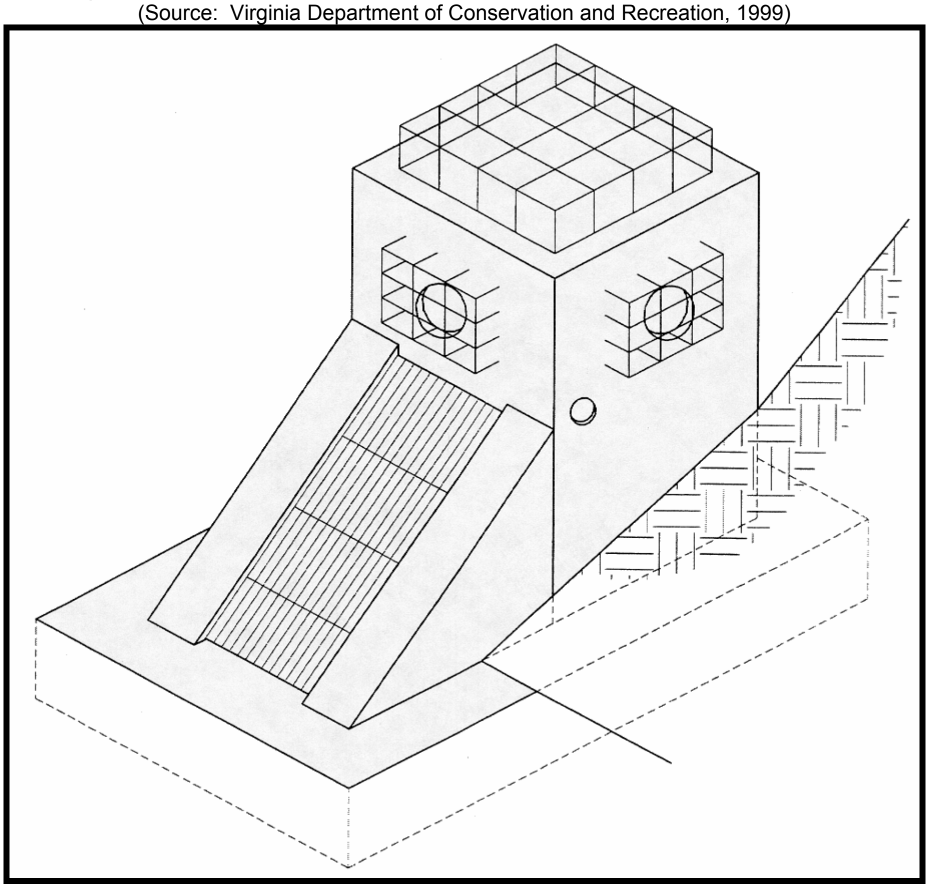
3.3.6.1 Trash Rack Design

An example of trash racks used on a riser outlet structure is shown in Figure 3-34. The inclined bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

Trash racks must be large enough such that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice.

The surface area of all trash racks should be maximized and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

Figure 3-34. Trash Racks Used on a Riser Outlet Structure
(Source: Virginia Department of Conservation and Recreation, 1999)

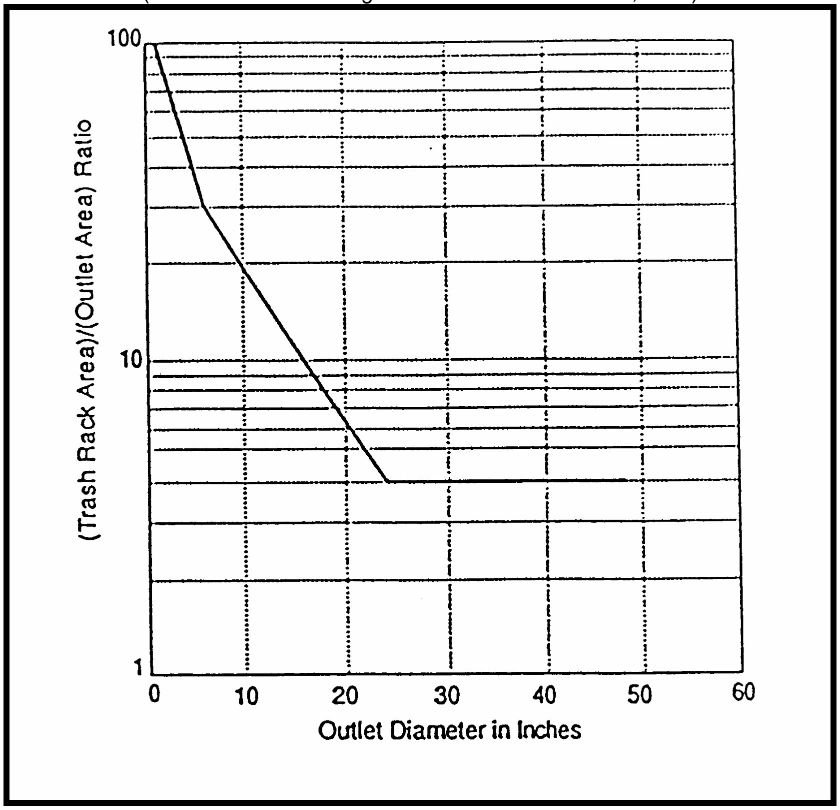


To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level—the slower the approach of flow, the flatter the trash rack angle. Rack opening rules-of-thumb are found in literature. Figure 3-35 gives opening estimates based on outlet diameter (Urban Drainage and Flood Control District, 1999). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

Figure 3-35. Minimum Rack Size vs. Outlet Diameter
(Source: Urban Drainage and Flood Control District, 1999)



The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack. Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (United States Bureau of Reclamation, 1978; Urban Drainage and Flood Control District, 1999). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore, either head losses through the grate should be calculated or Figure 3-35 used to ensure adequate hydraulic capacity through the trash rack. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Equation 3-48 can be used to determine the head loss through a trash rack (Metcalf and Eddy, 1972). In the use of Equation 3-48, grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50% blockage is chosen as a working assumption.

Equation 3-48

$$H_g = K_{g1}\left(\frac{w}{x}\right)^{4/3}\left(\frac{V_u^{\,2}}{2g}\right)\sin\theta_g$$

where:

H_g = head loss through grate (ft)

K_{g1} = bar shape factor:

2.42 – sharp-edged rectangular

1.83 - rectangular bars with semicircular upstream faces

1.79 - circular bars
1.67 - rectangular bars with semicircular up- and downstream faces

w = maximum cross-sectional bar width facing the flow (in)
x = minimum clear spacing between bars (in)
V_u = approach velocity (ft/s)
θ_g = angle of the grate with respect to the horizontal (degrees)
g = acceleration of gravity (32.2 ft/s^2)

The United States Army Corps of Engineers has developed a similar head loss equation for trash racks, shown in Equation 3-49 (USACE, 1988). This equation is for vertical racks, but presumably can be adjusted through multiplication by the sine of the angle of the grate with respect to the horizontal, in a manner similar to the previous equation.

Equation 3-49

$$H_g = \frac{K_{g2} V_u^{\;2}}{2g}$$

where:

H_g = head loss through grate (ft)
K_{g2} = defined as described below
V_u = approach velocity (ft/s)
g = acceleration of gravity (32.2 ft/s^2)

K_{g2} is defined from a series of fit curves, as shown in Table 3-23.

Table 3-23. Fit Curves to Determine K_{g2}

Grate Type	Length/thickness	Curve
Sharp-edged rectangular	10	K_{g2} = 0.00158 - 0.03217 A_r + 7.1786 A_r^2
Sharp-edged rectangular	5	K_{g2} = -0.00731 + 0.69453 A_r + 7.0856 A_r^2
Round edged rectangular	10.9	K_{g2} = -0.00101 + 0.02520 A_r + 6.0000 A_r^2
Circular cross-section	not applicable	K_{g2} = 0.00866 + 0.13589 A_r + 6.0357 A_r^2

3.3.7 Secondary Outlets

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure 3-36 shows an example of an emergency spillway.

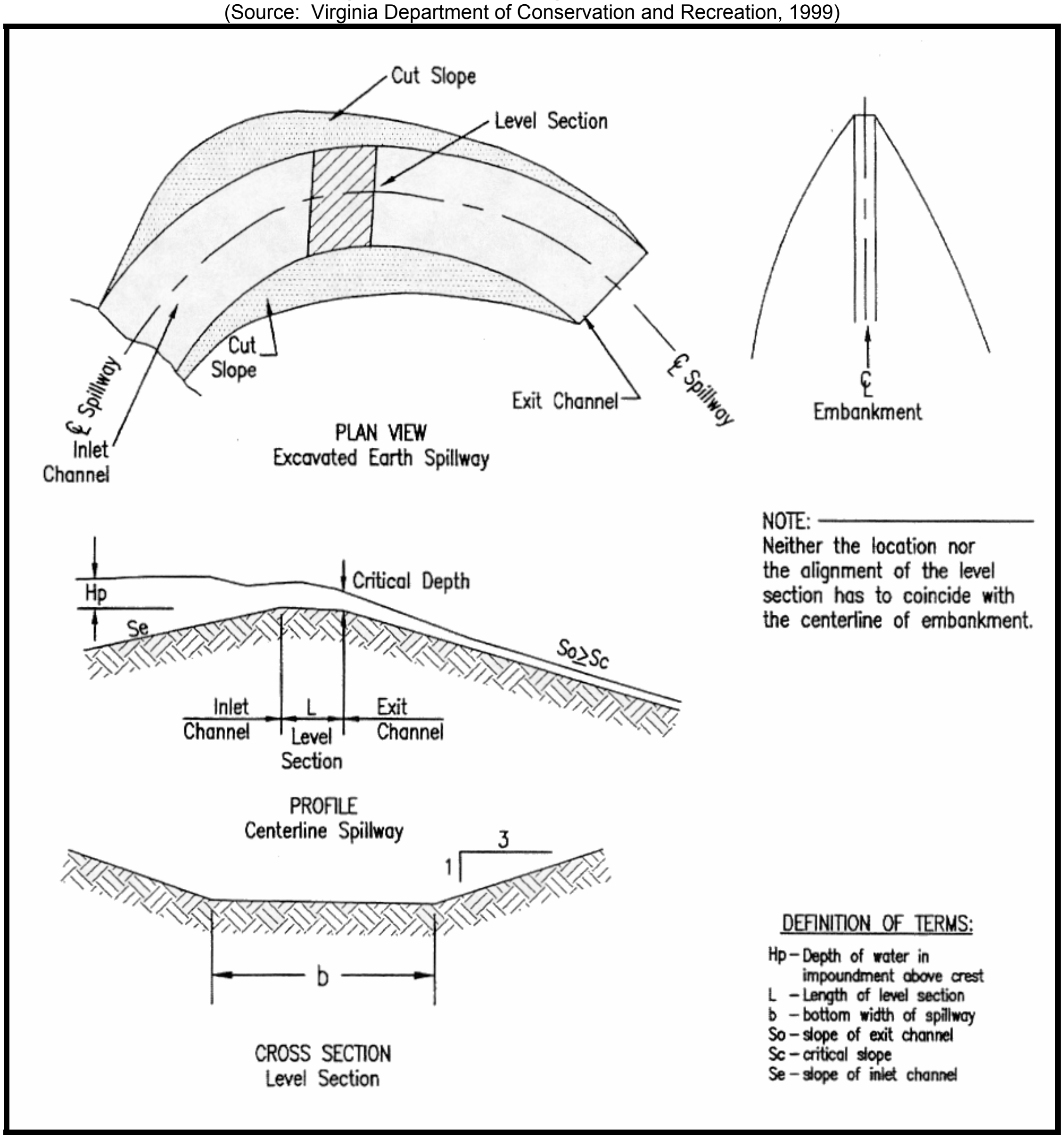
In many cases, on-site stormwater storage facilities do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor.

3.3.7.1 Emergency Spillway Design

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see Figure 3-36). The emergency spillway is proportioned to pass flows in excess of the 100-year flood without allowing excessive velocities and without overtopping of the embankment. Flow in the emergency spillway is open channel flow. Normally, it is assumed that critical depth occurs at the control section. Volume 2, Chapter 7 provides more information on open channel hydraulics.

SCS manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both the inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper the 3:1 (horizontal to vertical). The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated below.

Figure 3-36. Emergency Spillway
(Source: Virginia Department of Conservation and Recreation, 1999)



References

Allred and Coonrod. *Safety Grates in Open Channels.* MS Prof. Paper, University of New Mexico, Albuquerque, 1991.

American Society of Civil Engineers (ASCE), *Stormwater Detention Outlet Control Structures*, Task Committee on the Design of Outlet Control Structures, ASCE, New York, NY, 1985.

Brater, E. F. and H. W. King. *Handbook of Hydraulics. 6th edition.* New York: McGraw Hill Book Company, 1976.

Chow, C. N. *Open Channel Hydraulics.* New York: McGraw Hill Book Company, 1959.

City of Knoxville. *Knoxville BMP Manual.* Stormwater Treatment Paper ST-13, 2003.

Engman, E. T. *Roughness Coefficients for Routing Surface Runoff*. Journal of Irrigation and Drainage, American Society of Civil Engineers, Vol. 112, No. 4, pp 39-53, 1986.

Federal Emergency Management Agency (FEMA). *Flood Insurance Study Knox County, Tennessee and Incorporated Areas.* Flood Insurance Study Number 47093CV002A, not yet dated

Ferguson, B. and Debo, T. *On-Site Stormwater Management.* 1990.

Hershfield, D. M. *Rainfall Frequency Atlas of the United States.* Technical Paper 40, 1961.

Hoggan, Daniel H. *Computer Assisted Floodplain Hydrology and Hydraulics, 2nd edition.* McGraw Hill Book Co., New York, 1997.

Linsley, R.K., Jr., Kohler, M.A., and Paulhus, J.A.H., *Applied Hydrology*. McGraw-Hill Book Co., New York, 1949.

Maryland Department of the Environment. *Maryland Stormwater Design Manual, Volumes I and II.* Prepared by Center for Watershed Protection (CWP), 2000.

McEnroe, B.M., J.M. Steichen and R. M. Schweiger. *Hydraulics of Perforated Riser Inlets for Underground Outlet Terraces.* Trans ASAE, Vol. 31, No. 4, 1988.

Metcalf and Eddy. *Wastewater Engineering.* McGraw-Hill, New York, 1972.

Natural Resources Conservation Service (NRCS). *Engineering Field Manual for Conservation Practices.* Soil Conservation Service, Engineering Division, Washington, D.C., 1984.

NOAA. *Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States.* NOAA Technical Memo, NWS HYDRO-35, 1977.

NOAA. *Evaporation Atlas for the Contiguous 48 United States.* NOAA Technical Report NWS 33, 1982.

Overton, D.E. and Meadows, M.E., *Storm Water Modeling.* Academic Press, New York, pp. 58-88, 1976.

Pitt, Robert. *Small Storm Hydrology.* Unpublished report. Department of Civil Engineering, University of Alabama at Birmingham, 1994.

Sandvik, A. *Proportional Weirs for Stormwater Pond Outlets.* Civil Engineering, ASCE pp. 54-56, March 1985.

Soil Conservation Service (SCS). *Urban Hydrology for Small Watersheds.* Technical Release 55, (TR-55), 1986.

United States Army Corps of Engineers (USACE). *Hydraulic Design Criteria.* USAE Waterways Experiment Station, Vicksburg, Mississippi, 1988.

United States Bureau of Reclamation (USBR). *Water Measurement Manual.* 1978. http://www.usbr.gov/wrrl/fmt/wmm/

United States Geological Survey. *Flood Characteristics of Urban Watersheds in the United States*. USGS Water Supply Paper 2207, 1983.

United States Geological Survey. *Synthesized Flood-frequency for Small Urban Streams in Tennessee.* Water-Resources Investigation Report 84-4182, 1984.

United States Geological Survey. *Techniques for Simulating Flood Hydrographs and Estimate Flood Volumes for Ungaged Basins in East and West Tennessee.* Water-Resources Investigation Report 86-4076, 1986.

Urban Drainage and Flood Control District (UDFCD). *Urban Drainage Criteria Manual-Volume 3.* Denver, Colorado, 1999.

Virginia Department of Conservation and Recreation. *Virginia Stormwater Management Handbook.* 1999.

Wright-McLaughlin Engineers. *Urban Storm Drainage Criteria Manual, Volumes I and II.* Prepared for the Denver Regional Council of Governments, Denver, Colorado, 1969.

Wycoff, R. L. and U. P. Singh. *Preliminary Hydrologic Design of Small Flood Detention Reservoirs.* Water Resources Bulletin, Vol. 12, No. 2, pp 337-49, 1976.

Suggested Reading

Debo, Thomas N., and Andrew J. Reese. *Municipal Storm Water Management.* Lewis Publishers: CRC Press, Inc., Boca Raton, Florida, 1995.

Ferguson, B. *Estimation of Direct Runoff in the Thornthwaite Water Balance.* Prof. Geographer, pp. 263-271, October 1996.

Haan, C. T., B. J. Barfield and J. C. Hayes. *Design Hydrology and Sedimentology for Small Catchments.* 1994.

Metropolitan Government of Nashville and Davidson County. *Stormwater Management Manual - Volume 2 Procedures.* Prepared by AMEC, Inc. (formerly The Edge Group) and CH2M Hill, 1988.

Schueler, Thomas R. *Controlling Urban Runoff.* Washington Metropolitan Water Resources Planning Board, 1987.

Soil Conservation Service. *SCS National Engineering Handbook.* 1985.

United States Department of Transportation (USDOT). *Hydrology.* Federal Highway Administration, Hydraulic Engineering Circular No. 19, 1984.

This page left intentionally blank.

Table of Contents

CHAPTER 4 — DESIGN AND MAINTENANCE OF STRUCTURAL BMPS

Chapter 4 - List of Figures

Chapter 4 - List of Tables

This page left intentionally blank

DESIGN AND MAINTENANCE OF STRUCTURAL BMPS

Structural stormwater best management practices (BMPs) are engineered facilities that are intended to treat stormwater runoff and/or mitigate the effects of increased stormwater runoff peak discharge, volume, and velocity due to urbanization. This chapter provides detailed descriptions and design specifications for the structural stormwater BMPs that can be used to address Knox County's minimum stormwater management standards outlined in Chapter 1 and the design criteria cited in Chapter 3.

In terms of the Integrated Site Design approach, a structural stormwater BMP, or a series of structural BMPs, must:

- Treat the Water Quality Volume (WQv);
- Control the Channel Protection Volume (CPv);
- Control for Overbank Flood Protection (up to Qp_{25}), and
- Provide for Extreme Flood Protection (Qp_{100}).

4.1 Design Standards Policy

The State of Tennessee's NPDES Phase II regulation requires that Knox County implement a post-construction stormwater treatment program for all new developments and redevelopments. To comply with this regulation, the Stormwater Management Ordinance requires that stormwater runoff be treated for pollutants prior to discharge from the site. Further, Knox County policy established in Chapters 1 and 3 (Volume 2) of this manual sets the minimum design standard for stormwater treatment as removal of 80% of the average annual post-development total suspended solids (TSS) load. The structural BMPs presented in this chapter of the Knox County stormwater management manual, used alone or in series, can be used to meet this minimum design standard. For purposes of compliance with local and State regulations, it is presumed that developments and redevelopments are meeting the 80% TSS removal standard so long as stormwater management systems are designed, constructed, and maintained in accordance with the design criteria and specifications discussed in this manual.

Therefore, Knox County requires that all of the structural BMPs presented in this section be designed, constructed and maintained in accordance with the criteria, standards, and specifications presented in the stormwater management ordinance and in this manual.

Proprietary, new, and other BMPs not included in this manual may be approved by the Director of Engineering and Public Works (Director) for treatment of stormwater quality on a case-by-case basis provided that the conditions outlined in Volume 2, Section 2.2.2.1 of this manual are met.

4.2 BMP Description and Selection Information

The structural stormwater BMPs recommended in this manual have been placed into two categories, general application and limited application, based upon generalized acceptance criteria set by Knox County Engineering. These categories are described below.

4.2.1 General Application BMPs

A listing of general application BMPs can be found in Table 4-1 below. Knox County will accept these BMPs for use with a wide variety of land uses and development types. General application BMPs have a demonstrated ability to treat the WQv and many are presumed to be able to achieve the 80% TSS removal standard when designed, constructed and maintained in accordance with recommended specifications. Several of the general application BMPs can also be designed to comply with other stormwater criteria, for downstream channel protection, and overbank and extreme flood protection. Knox County recommends that general application BMPs be utilized for the stormwater management facilities for a site wherever feasible and practical. A detailed discussion of each of the general application BMPs, as well as design criteria and procedures can be found later in this chapter.

Table 4-1. Descriptions of General Application BMPs

Structural BMP	Description
Stormwater Ponds • Wet Pond • Wet ED Pond • Micropool ED Pond • Multiple Pond Systems	Stormwater ponds are constructed stormwater retention basins that have a permanent pool (or micropool) of water. Runoff from each rain event is detained and treated in the pool. ED = Extended Detention. ED is the detention of stored runoff for a minimum of 24 hours.
Detention Basins • Dry Detention Basin • Dry ED Basin	Dry detention basins and dry extended detention (ED) basins are surface facilities intended to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts and will have to be combined with another BMP to achieve the 80% TSS removal goal.
Stormwater Wetlands • Shallow Wetland • ED Shallow Wetland • Pond/Wetland Systems • Pocket Wetland	Stormwater wetlands are constructed wetland systems used for stormwater management. Stormwater wetlands consist of a combination of shallow marsh areas, open water and semi-wet areas above the permanent water surface.
Bioretention Areas	Bioretention areas are shallow stormwater basins or landscaped areas which utilize engineered soils and vegetation to capture, infiltrate and treat stormwater runoff. Runoff may be returned to the conveyance system, or allowed to partially infiltrate into the soil.
Sand Filters • Surface Sand Filter • Perimeter Sand Filter	Sand filters are multi-chamber structures designed to treat stormwater runoff through filtration, using a sand bed as its primary filter media. Filtered runoff may be returned to the conveyance system, or allowed to partially infiltrate into the soil.
Infiltration Trench	An infiltration trench is an excavated trench filled with stone aggregate used to capture and allow infiltration of stormwater runoff into the surrounding soils from the bottom and sides of the trench.
Enhanced Swales • WQ Dry Swale • Wet Swale	Enhanced swales are vegetated open channels that are explicitly designed and constructed to capture and treat stormwater runoff within dry or wet cells formed by check dams or other means.

Structural BMP	Description
Biofilters • Filter Strip • Grass Channel	Both filter strips and grass channels provide "biofiltering" of stormwater runoff as it flows across the grass surface. However, by themselves these controls cannot meet the 80% TSS removal performance goal. Consequently, both filter strips and grass channels should only be used as pretreatment measures or as part of a treatment train approach. Grass channels are open channel practices that are not designed specifically for water quality.
Modular Porous Paver Systems and Porous Pavement/Concrete	Porous surfaces are permeable pavement surfaces with an underlying stone reservoir to temporarily store surface runoff before it infiltrates into the subsoil. These practices are considered source control BMPs rather than treatment BMPs. Areas where porous surfaces have been applied are included in the WQv calculations as pervious surfaces, rather than impervious surfaces. Porous concrete is the term for a mixture of course aggregate, portland cement and water that allows for rapid infiltration of water. Modular porous paver systems consist of open void paver units laid on a gravel subgrade. Both porous concrete and porous paver systems provide water quality and quantities benefits, but may have high workmanship and maintenance requirements.

4.2.2 Limited Application BMPs

Limited application BMPs will be allowed only when the use of general application BMPs is not feasible because special site or design conditions prohibit their use. Limited application BMPs will be approved for use in Knox County on a site-by-site basis. In general, limited application BMPs are intended to address hotspot or specific land use constraints or conditions requiring pretreatment, and may have high installation costs or special maintenance requirements that may preclude their use for most general applications. Limited application BMPs are typically used for water quality treatment only and do not provide additional control for channel or flood protection. Limited application BMPs should be considered primarily for commercial, industrial or institutional developments.

Table 4-2 lists the limited application BMPs, along with the rationale for limited use. These structural BMPs are recommended for use with particular land uses and densities, to meet certain water quality requirements, for limited usage on larger projects, or as part of a stormwater treatment train. A detailed discussion of each of the limited application BMPs, as well as design criteria and procedures can be found later in this chapter.

4.2.3 Pollutant Removal Capabilities

Research has shown that the use of the structural BMPs discussed in this chapter will have benefits for the removal of TSS and other pollutants (i.e., phosphorous, nitrogen, fecal coliform and heavy metals). The ability for both general and limited application BMPs to remove pollutants varies by structural BMP type and by pollutant type. Pollutant removal capabilities for a given BMP are based on a number of factors including the physical, chemical and/or biological processes that take place in the BMP and the design and sizing of the facility. In addition, pollutant removal efficiencies for the same BMP type and facility design can vary widely depending on the tributary land use and area, incoming pollutant concentration, rainfall pattern, time of year, maintenance frequency and numerous other factors.

Table 4-2. Descriptions of Limited Application BMPs

Structural BMP	Description and Rationale for Limited Use
Filtering Practices • Organic Filter	Organic filters are surface sand filters where organic materials such as a leaf compost or peat/sand mixture are used as the filter media. These media may be able to provide enhanced removal of some contaminants, such as heavy metals and nutrients. Given their potentially high maintenance requirements, they should only be used in environments that warrant their use.
• Underground Sand Filter	Underground sand filters are sand filter systems located in an underground vault. These systems should only be considered for extremely high density or space-limited sites.
Wetland Systems • Submerged Gravel Wetlands	Submerged gravel wetlands systems use wetland plants in a submerged gravel or crushed rock media to remove stormwater pollutants. These systems should only be used in mid- to high-density environments where the use of other structural controls may be precluded. The long-term maintenance burden of these systems is uncertain.
Chemical Treatment • Alum Treatment	Alum treatment provides for the removal of suspended solids from stormwater runoff entering a wet pond by injecting liquid alum into the stormwater system. Alum treatment should only be considered for large-scale projects where high water quality is desired and where other BMPs do not provide the level of pollutant removal required for the receiving water.
Proprietary Systems • Commercial Stormwater BMPs	Proprietary BMPs are manufactured structural control systems available from commercial vendors designed to improve stormwater runoff quality and/or provide water quantity control. Proprietary systems often can be used on small sites and in space-limited areas, as well as in pretreatment applications. However, proprietary systems are often more costly than other alternatives, may have high maintenance requirements, and often lack adequate independent performance data, particularly for use in Knox County conditions. Approval of the Knox County Engineering and Public Works Director is required prior to their implementation.
Gravity (oil-grit) Separator	Gravity separators, (also called hydrodynamic BMPs) use the movement of stormwater runoff through a specially designed structure to remove target pollutants. They are typically used on smaller impervious commercial sites and urban hotspots. These BMPs typically do not meet the 80% TSS removal performance goal, and therefore, should only be used as a pretreatment measure and as part of a treatment train approach.

Table 4-3 provides design removal efficiencies assigned to each of the general and limited application BMPs. It should be noted that these values are median pollutant reduction percentages for design purposes that have been derived from existing sampling data, modeling and research. A structural BMP design may be capable of exceeding these performances; however, the values in the table are considered median values that can be assumed to be achieved when the structural BMP is sized, designed, constructed and maintained in accordance with recommended specifications in this manual.

Where the pollutant removal capabilities of an individual structural stormwater BMP are not sufficient for a given site application, additional controls may be used in series in a “treatment train” approach. More detail on the use of stormwater BMPs in series is provided later in this chapter.

Table 4-3. Design Pollutant Removal Efficiencies (in %) for Structural BMPs

Structural BMP	TSS	Total P[1]	Total N[2]	Fecal Coliform	Metals
General Application Structural BMPs					
Stormwater Ponds (Wet ED Pond, Micropool ED Pond, and Multiple Pond Systems)	80	55	30	70*	50
Conventional Dry Detention Pond	10	---	---	---	---
Dry Extended Detention Pond	60	35	25	---	25
Stormwater Wetlands (Shallow Wetlands, ED Wetlands, Pond/Wetland System, Pocket Wetland)	75	45	30	70*	50
Bioretention Areas	85	60	50	---	80
Sand Filters	80	50	30	40	50
Infiltration Trench	90	60	60	90	90
Water Quality (WQ) Dry Swale	90	50	50	---	40
Wet Swale	75	25	40	---	20
Filter Strip	50	20	20	---	40
Grass Channel[3]	30	25	20	---	30
Modular Porous Paver Systems and Porous Pavement/Concrete	**	**	**	**	**
Limited Application Structural BMPs					
Organic Filter	80	60	40	50	75
Underground Sand Filter	80	50	25	40	50
Submerged Gravel Wetland	80	50	20	70	50
Alum Treatment	90	80	60	90	75
Proprietary Systems	***	***	***	***	***
Gravity (oil-grit) Separator	30	5	5	---	---

1 Total phosphorus
2 Total nitrogen
3 Refers to open channel practices not designed specifically for water quality.
* If no resident waterfowl population is present.
** These practices are source controls and are not designed as pollutant removal devices.
*** The performance of specific proprietary commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data.
--- Insufficient data to provide design removal efficiency. See Volume 2, Section 2.2.2.1 for details.

4.2.4 Screening Process for General Application BMPs

Outlined below is a process used in the selection of general application BMPs. This process is intended to assist the site developer and design engineer in determining the most appropriate structural BMP for a development site, and to provide guidance on factors to consider in their location. Knox County's goal of 80% TSS removal is the primary factor in the selection process of BMPs or BMP treatment trains. Information on selection factors related to pollutants other than TSS is provided for informational purposes, and may be useful in the future depending upon local, state and federal water quality regulations at that time.

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

- stormwater treatment suitability;
- water quality performance;
- site applicability;
- implementation considerations.

In addition, for a given site, the following factors should be considered and any specific design criteria or restrictions need to be evaluated:

- physiographic factors;
- soils;
- special watershed or stream considerations.

Finally, environmental regulations that may influence the location of a structural BMP on site, or may require a permit, need to be considered.

Guidance on a selection process for comparing and evaluating various general application structural stormwater BMPs using two screening matrices and a list of location and permitting factors is presented below. These tools are provided to assist the design engineer in selecting the subset of structural BMPs that will meet the stormwater management and design objectives for a development site or project.

Step 1: Evaluate Overall Applicability

Through the use of Table 4-4, the site designer evaluates and screens the overall applicability of the full set of general application structural BMPs as well as the constraints of the site in question. The discussion following the table presents an explanation of the various screening categories and individual characteristics used to evaluate the structural BMPs.

<u>Stormwater Management Suitability</u>

The first columns of Table 4-4 examine the capability of each structural BMP option to provide water quality treatment, downstream channel protection, overbank flood protection, and extreme flood protection. A blank entry means that the structural BMP cannot or is not typically used to meet the aforementioned criteria. This does not necessarily mean that it should be eliminated from consideration, but rather is a reminder that more than one structural BMP may be needed at a site (e.g., a bioretention area used in conjunction with dry detention storage).

> *Ability to treat the Water Quality Volume (WQv).* This indicates whether a structural BMP provides treatment of the WQv and provides the TSS reduction amount assigned to each BMP type.

Table 4-4. General Application BMP Screening Matrix – Overall BMP Applicability

STRUCTURAL BMP CATEGORY	STRUCTURAL BMP	STORMWATER TREATMENT SUITABILITY				WATER QUALITY PERFORMANCE*		SITE APPLICABILITY					IMPLEMENTATION CONSIDERATIONS			
		Water Quality	Channel Protection	Overbank Flood Protection	Extreme Flood Protection	TSS / Sediment Removal Rate	Hotspot Application	Drainage Area (acres)	Space Req'd (% of tributary imp. Area)	Site Slope	Minimum Head Required	Depth to Water Table	Residential Subdivision Use	High Density / Ultra-Urban	Capital Cost	Maintenance Burden
Stormwater Ponds	Wet Pond	✓	✓	✓	✓	80%		25 min**	2-3%	15% max	6 to 8 ft	2 feet, if hotspot or aquifer	✓		Low	Low
	Wet ED Pond	✓	✓	✓	✓								✓		Low	Low
	Micropool ED Pond	✓	✓	✓	✓			10 min**					✓		Low	Moderate
	Multiple Ponds	✓	✓	✓	✓			25 min**					✓		Low	Low
Detention Basins	Extended Detention	✓	✓	✓	✓	60%		any	4-5%	15% max		2 feet min	✓		Low	Low
	Conventional Detention	✓	✓	✓	✓	10%		any	4-5%	15% max		2 feet min	✓		Low	Low
Stormwater Wetlands	Shallow Wetland	✓	✓	✓	✓	75%		25 min	3-5%	8% max	3 to 5 ft	2 feet, if hotspot or aquifer	✓		Moderate	Moderate
	Shallow ED Wetland	✓	✓	✓	✓								✓		Moderate	Moderate
	Pond/Wetland	✓	✓	✓	✓						6 to 8 ft		✓		Moderate	Moderate
	Pocket Wetland	✓	✓					5 min			2 to 3 ft	below WT	✓	✓	Moderate	High
Bioretention	Bioretention Areas	✓	♦			85%		5 max***	5%	6% max	5 ft	2 feet	✓	✓	Moderate	Moderate
Sand Filters	Surface Sand Filter	✓	♦			80%	✓	10 max***	2-3%	6% max	5 ft	2 feet		✓	High	High
	Perimeter Sand Filter	✓	♦				✓	2 max***			2 to 3 ft			✓	High	High
Infiltration	Infiltration Trench	✓	♦			90%		5 max	2-3%	6% max	1 ft	4 feet	✓	✓	High	High
Enhanced Swales	Dry Swale	✓	♦			90%		5 max	10-20%	4% max	3 to 5 ft	2 feet	✓		Moderate	Low
	Wet Swale	✓	♦			75%		5 max			1 ft	below WT	✓		High	Low
Biofilters	Filter Strip	✓				50%		2 max	20-25%	2-6% max		2-4 feet	✓		Low	Moderate
	Grass Channel	✓				30%		5 max	10-20%	4% max		2 feet	✓		Low	Low
Modular Porous Paver Systems and Porous Pavement/Concrete	Porous Pavers, Pavement and Concrete		✓			**		5 max	varies	5%	2 ft	4 feet	✓	✓	Moderate	High

✓ This BMP meets suitability criteria.

♦ This BMP can be incorporated into the structural control in certain situations.

* TSS pollutant removal rates must be used for design purposes. See Volume 1 Chapter 3 for guidance on calculating the % TSS removal for a development site.

** Smaller drainage areas may be approved by the Director with adequate water balance and anti-clogging device.

*** The use of this BMP for larger drainage areas may be approved by the Director if design calculations show that the BMP will achieve its design intentions given a larger drainage area.

Ability to provide Channel Protection (CPv). This indicates whether the structural BMP can be used to provide the extended detention of the CPv. The presence of a check mark indicates that the structural control can be used to meet CPv requirements. A diamond indicates that the structural control may be sized to provide channel protection in certain situations, for instance on small sites.

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

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

Relative Water Quality Performance

The second group of columns in Table 4-4 provides an overview of the pollutant removal performance of each structural control option, when designed, constructed and maintained according to the criteria and specifications in this manual.

Ability to provide TSS Removal. This column indicates the capability of a structural BMP to remove TSS from runoff.

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

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

Ability to accept Hotspot Runoff. This last column indicates the capability of a structural BMP to treat runoff from designated hotspots. Hotspots are land uses or activities with higher potential pollutant loadings. Examples of hotspots might include: gas stations, convenience stores, marinas, public works storage areas, vehicle service and maintenance areas, commercial nurseries, and auto recycling facilities. A check mark indicates that the structural BMP may be used on hotspot site, however it may have specific design restrictions. Please see Section 4.3 for the specific design criteria of the structural BMP for more details.

Site Applicability

The third group of columns in Table 4-4 provides an overview of the specific site conditions or criteria that must be met for a particular structural BMP to be suitable. In some cases, these values are recommended values or limits that can be exceeded or reduced with proper design or depending on specific circumstances. Refer to the specific criteria section of the structural BMP in Section 4.2 for more details.

Drainage Area. This column indicates the approximate minimum or maximum drainage area that is considered suitable for the structural BMP. The Director may approve exceptions to the drainage area maximum or minimum depending on the site conditions and the structural BMP(s) being proposed. The drainage areas indicated for ponds and wetlands should not be considered inflexible limits, and may be increased or decreased depending on water availability (baseflow or groundwater), the mechanisms employed to prevent outlet clogging, or design variations used to maintain a permanent pool (e.g., liners).

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

Slope. This column evaluates the effect of slope on the structural BMP. Specifically, the slope restrictions refer to how flat the area where the facility is installed must be and/or how steep the contributing drainage area or flow length can be.

Minimum Head. This column provides an estimate of the minimum elevation difference needed at a site (from the inflow to the outflow) to allow for gravity operation within the structural BMP.

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

<u>Implementation Considerations</u>
The last group of columns of Table 4-4 provides additional considerations for the applicability of each structural BMP option.

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

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

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

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

Step 2: Specific Criteria

Table 4-5 provides an overview of various design criteria and specifications, or exclusions for a structural BMP that may be present due to a site's general physiographic character, soils, or location in a watershed with special water resources considerations.

<u>Physiographic Factors</u>

Three key factors to consider are low-relief, high-relief, and karst terrain. In Knox County, low relief (very flat) areas and high relief (steep and hilly) areas are found throughout the county. Karst and major carbonaceous rock areas are found throughout portions of Knox County. Special geotechnical testing requirements may be needed in karst areas. Knox County Engineering should be consulted to determine if a project is subject to terrain constraints.

- Low relief areas need special consideration because many structural BMPs require a hydraulic head to move stormwater runoff through the facility.
- High relief areas may limit use of some structural BMPs that need flat or gently sloping areas to settle out sediment or to reduce velocities. In other cases high relief may impact embankment heights to the point that a structural BMP becomes infeasible.
- Karst areas can limit the use of some structural BMPs as the infiltration of polluted waters directly into underground streams found in karst areas may be prohibited. In addition, ponding areas may not reliably hold water in karst areas.

Table 4-5. General Application BMP Screening Matrix – Specific Criteria

STRUCTURAL BMP	PHYSIOGRAPHIC FACTORS			SOILS
	Low Relief	High Relief	Karst	
Stormwater Ponds	Limit maximum normal pool depth to about 4 feet (dugout) Providing pond drain can be problematic	Embankment height restrictions	Require poly or clay liner Max ponding depth Geotechnical tests	"A" soils may require pond liner "B" soils may require infiltration testing
Detention Basins	✷	Embankment height restrictions	Require poly or clay liner Max ponding depth Geotechnical tests	"A" soils may require pond liner "B" soils may require infiltration testing
Stormwater Wetlands	✷	Embankment height restrictions	Require poly-liner Geotechnical tests	"A" soils may require pond liner
Bioretention & Sand Filters	Several design variations will likely be limited by low head	✷	Use poly-liner or impermeable membrane to seal bottom	Clay or silty soils may require pretreatment
Infiltration	Minimum distance to water table of 2 feet	Maximum slope of 6% Trenches must have flat bottom	Generally not allowed	Infiltration rate > 0.5 inch/hr
Enhanced Swales	Generally feasible however slope <1% may lead to standing water in dry swales	Often infeasible if slopes are 4% or greater	✷	✷
Biofilters (Filter Strips & Grass Channels)	✷	✷	✷	✷
Modular Pavers/Porous Pavement	✷	Maximum slope of 5%	✷	Underdrain system required for C and D soils

✷ - These BMPs typically have no limiting factors or constraints for physiographic factors or soils.

<u>Soils</u>

The key evaluation factors are based on an initial investigation of the Natural Resources Conservation Service (NRCS) hydrologic soils groups at the site. Note that more detailed geotechnical tests are usually required for infiltration feasibility and during design to confirm permeability and other factors.

Additionally, the design of structural stormwater controls is fundamentally influenced by the nature of the downstream water body that will be receiving the stormwater discharge. In some cases, higher pollutant removal or environmental performance is needed to fully protect aquatic resources and/or human health and safety within a particular watershed or receiving water. Special design criteria for a particular structural control or the exclusion of one or more controls may need to be considered within these watersheds or areas. An important watershed factor to consider is the protection of drinking water sources, wellheads and surface reservoirs. Wellhead protection areas that recharge existing public water supply wells present a unique management challenge. The key design constraint is to prevent possible groundwater contamination by preventing infiltration of hotspot runoff. At the same time, recharge of unpolluted stormwater is encouraged to maintain flow in streams and wells during dry weather. Watersheds that deliver surface runoff to a public water supply reservoir or impoundment are a special concern also. Depending on the treatment available at the water intake, it may be necessary to achieve a greater level of pollutant removal for the pollutants of concern, such as bacteria pathogens, nutrients, sediment or metals. One particular management concern for reservoirs is ensuring that stormwater hotspots are adequately treated so that they do not contaminate drinking water.

Step 3: Location and Permitting Considerations

In the last step, a site designer assesses the physical and environmental features at the site to determine the optimal location for the selected structural BMP or group of BMPs. Table 4-6 provides a condensed summary of current restrictions as they relate to common site features that may be regulated under local, state or federal law. These restrictions fall into one of three general categories:

- Locating a structural BMP within an area that is expressly prohibited by law.
- Locating a structural BMP within an area that is strongly discouraged, and is only allowed on a case-by-case basis. Local, state and/or federal permits may be needed, and the applicant will need to supply additional documentation to justify locating the BMP within the regulated area.
- Locating a BMP based upon setbacks from a site feature or features.

This checklist is only intended as a general guide to location and permitting requirements as they relate to siting of stormwater structural BMPs. Knox County advises that the appropriate permitting agency be consulted if any of the site features listed in Table 4-6 are encountered on the development or redevelopment site.

4.2.5 Limited Application BMP Screening Process

Outlined below is a screening process for limited application BMPs designed to assist the site designer and design engineer in the evaluation of the performance and applicability of the various limited application BMPs. Through the use of Table 4-7, the site designer can evaluate and screen the list of limited application structural BMPs to determine if a particular BMP or set of BMP(s) is appropriate.

As with the general application BMPs, the site designer assesses the physical and environmental features at the site to determine the optimal location for the selected BMP(s) or group of BMPs using Table 4-7 (Location and Permitting Checklist).

Evaluation Criteria

The following are the details of the various screening categories and individual characteristics used to evaluate the structural BMPs listed in Table 4-7.

Water Quality Treatment

> *% TSS Reduction*. This column indicates the pollutant removal value assigned to each BMP type. If the BMP has a value of less than 80% TSS, then the BMP must be used in a treatment train with other BMPs to meet the overall weighted TSS reduction goal.

Table 4-6. BMP Location and Permitting Checklist

Site Feature and Regulatory Agency	General Location and Permitting Guidance
Jurisdictional Wetlands (Waters of the U.S) U.S. Army Corps of Engineers Section 404 Permit Knox County Engineering	• Jurisdictional wetlands should be delineated prior to siting structural control. • Use of natural wetlands for stormwater quality treatment is contrary to the goals of the Clean Water Act and should be avoided. • Stormwater should be treated prior to discharge into a natural wetland. • Structural BMPs may also be restricted in buffer zones, although they may be utilized as a non-structural filter strip (i.e., accept sheet flow). • Justification must be provided that no practical upland treatment alternatives exist for wetland impacts by structural BMPs. • Where practical, excess stormwater flows should be conveyed away from jurisdictional wetlands or as sheet flow towards the wetland.
Stream Channels (Waters of the U.S) U.S. Army Corps of Engineers Section 404 Permit TDEC Knox County Engineering	• All Waters of the U.S. (streams, ponds, lakes, etc.) should be delineated prior to design. • Waters of the U.S. should not be used for stormwater quality treatment. In-stream ponds for stormwater quality treatment are highly discouraged. • Stormwater should be treated prior to discharging into Waters of the U.S. • Justification must be provided that no practical upland treatment alternatives exist for stream impacts by structural BMPs. • Temporary runoff storage preferred over permanent pools. • Implement measures that reduce downstream warming.
Sinkholes TDEC Knox County Engineering	• The Director may require additional BMPs to prevent flooding or additional information to verify structural integrity.
Wellhead Protection Zones TDEC	• Infiltration BMPs may be prohibited due to proximity to wellhead protection zones for public water supplies. TDEC required setbacks for public water systems categories 1-4 will be enforced.
100 Year Floodplains Knox County Engineering	• Grading and fill for structural control construction is generally discouraged within the ultimate 100 year floodplain, as delineated by FEMA flood insurance rate maps, FEMA flood boundary and floodway maps, or as determined by the Director. • Fill that alters the conveyance and storage capacity of the natural floodplain is prohibited in the flood fringe one-half the linear distance between the floodway line and the 100-year floodplain line.
Water Quality Buffers Knox County Engineering	• Structural BMPs are prohibited in the inner zone of buffers on streams and wetland buffers.
Utilities Local utility district	• Call appropriate utility district to locate existing utilities prior to design. • Note the location of proposed utilities to serve development. • Structural BMPs are discouraged within utility easements or rights-of-way for public or private utilities.
Roads Knox County Engineering TDOT	• Approval must also be obtained for any stormwater discharges to a Knox County or state-owned conveyance channel.
Structures and Property Lines Knox County	• Consult the Volume 2, Chapter 4 of the Knox County Stormwater Management Manual for structural BMP setbacks from structures. • Recommended setbacks for each structural BMP group are provided in the performance criteria in this manual.
Septic Drainfields Knox County Health Department	• Consult Knox County Health Department. • Recommended setback is a minimum of 50 feet from drain field edge.

Site Feature and Regulatory Agency	General Location and Permitting Guidance
Water Wells Knox County Health Department	• 100-foot setback for stormwater infiltration. • 50-foot setback for all other structural controls.

Site Applicability

The next two columns in Table 4-7 provide an overview of the specific site conditions or criteria that must be met for a particular limited application structural BMP to be suitable. Please see the specific criteria for each BMP provided in Section 4.3 for more details.

Drainage Area. This column indicates the approximate minimum or maximum drainage area that is considered suitable for the structural BMP.

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

Implementation Considerations

The last group of columns in Table 4-7 provides additional considerations for the applicability of each structural BMP option.

Residential Subdivision. A check mark in this column identifies whether or not a structural control is suitable for typical residential subdivision development (not including high-density or ultra-urban areas).

High Density / Ultra-Urban. A check mark in this column identifies those structural controls that are appropriate for use in very high-density (ultra-urban) areas, or areas where space is a premium.

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

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

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

4.2.6 Off-Line Versus On-Line Structural BMPs

Structural stormwater controls are designed either as “off-line” or “on-line” stormwater quality treatment controls. Examples of off-line and on-line BMPs are presented in Figure 4-1.

Off-line structural BMPs provide stormwater treatment (or other control) away from the flowpath of the runoff, and therefore, are typically designed only to receive a specified discharge rate (the water quality peak discharge) or volume. After the design runoff flow has been treated and/or controlled it is returned to the conveyance system. In contrast, on-line facilities, such as a stormwater treatment channel, typically provide stormwater control within the flowpath of the runoff. Because of this, on-line facilities often must be able to handle the entire range of design storm discharges, up to the Qp_{25} or Qp_{100} event. Techniques and calculation methods for proper sizing of off-line BMPs are presented in Volume 2, Chapter 3 of this manual

A flow regulator (e.g., diversion structure, flow splitter, etc.) is used to direct stormwater to off-line structural BMPs. Examples of flow regulators are shown in Figures 4-2 through 4-4 below.

Table 4-7. Limited Application BMP Screening Matrix

STRUCTURAL BMP CATEGORY	STRUCTURAL BMP	WATER QUALITY	SITE APPLICABILITY		IMPLEMENTATION CONSIDERATIONS				
		% TSS Reduction	Drainage Area (acres)	Space Req'd (% of tributary imp. Area)	Residential Subdivision Use	High Density / Ultra-Urban	Capital Cost	Maintenance Burden	Commercially Manufactured Systems Available?
Filtering Practices	Organic Filter	80	10 max**	2-3%		✓	High	High	
	Underground Sand Filter	80	5 max	None		✓	High	High	Yes
Wetland Systems	Submerged Gravel Wetland	80	5 max**	2-3%		✓	High	High	
Porous Surfaces[1]	Porous Concrete	*	5 max	Varies		✓	Medium	High	
	Modular Porous Paver Systems	*	5 max	Varies	✓	✓	High	High	Yes
Chemical Treatment	Alum Treatment System	90	25 min	None	✓	✓	High	High	
Proprietary Systems	Commercial Stormwater Controls*	***	***	***	***	***	***	***	Yes
Separator Units	Oil/Grit Oil/Water Gravity	30	1 max	***		✓	Medium	High	Yes

✓ Meets suitability criteria
* These practices are source controls and are not designed as pollutant removal devices.
** Drainage area can be larger in some instances
*** The application, performance and maintenance requirements of specific commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data
[1] Porous surfaces provide water quantity benefits by reducing the effective impervious area

Figure 4-1. Example of Off-Line versus On-Line Structural Controls

(Source: Center for Watershed Protection, 1996)

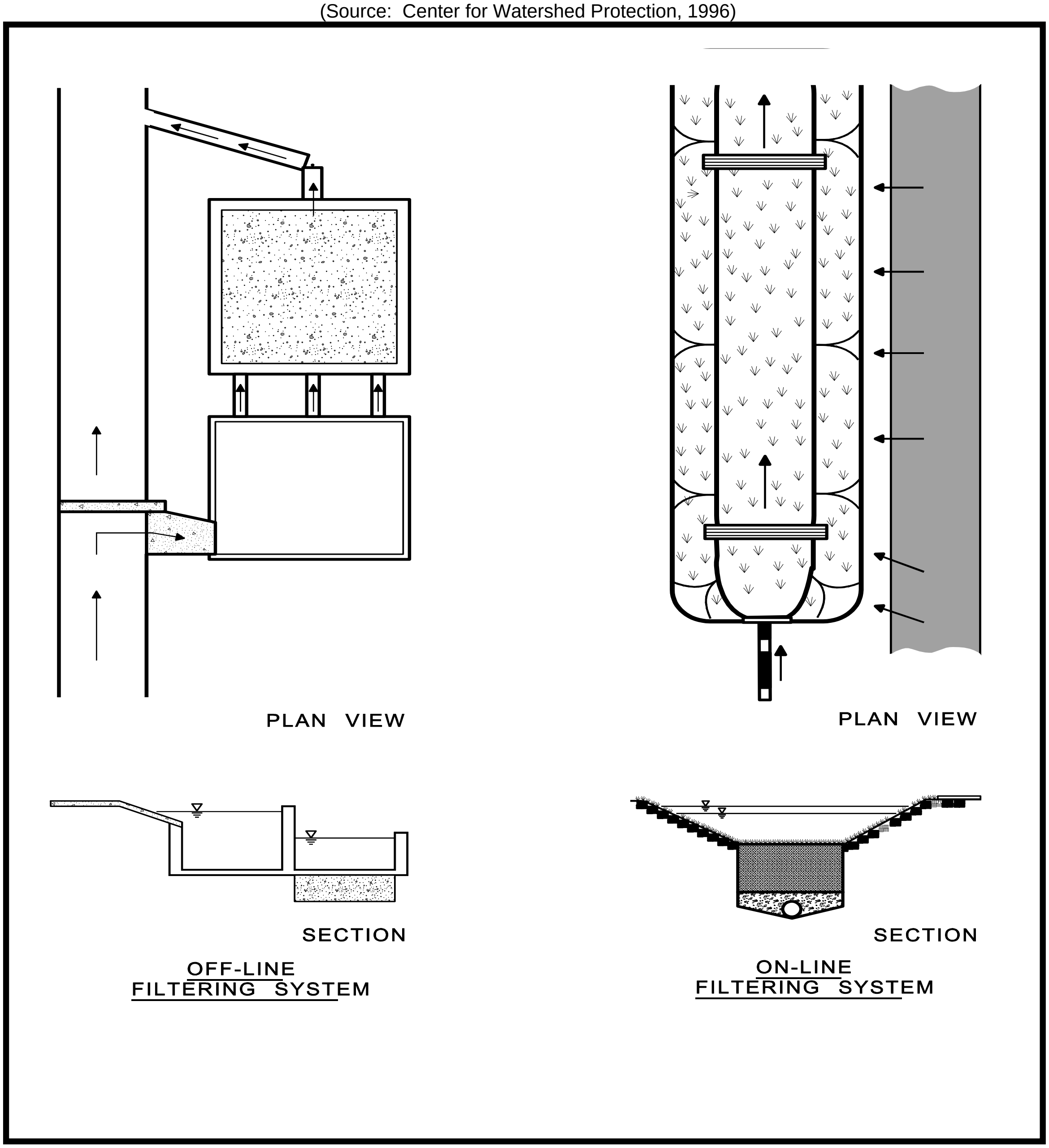


Figure 4-2. Pipe Interceptor Diversion Structure
(Source: City of Sacramento, 2000)

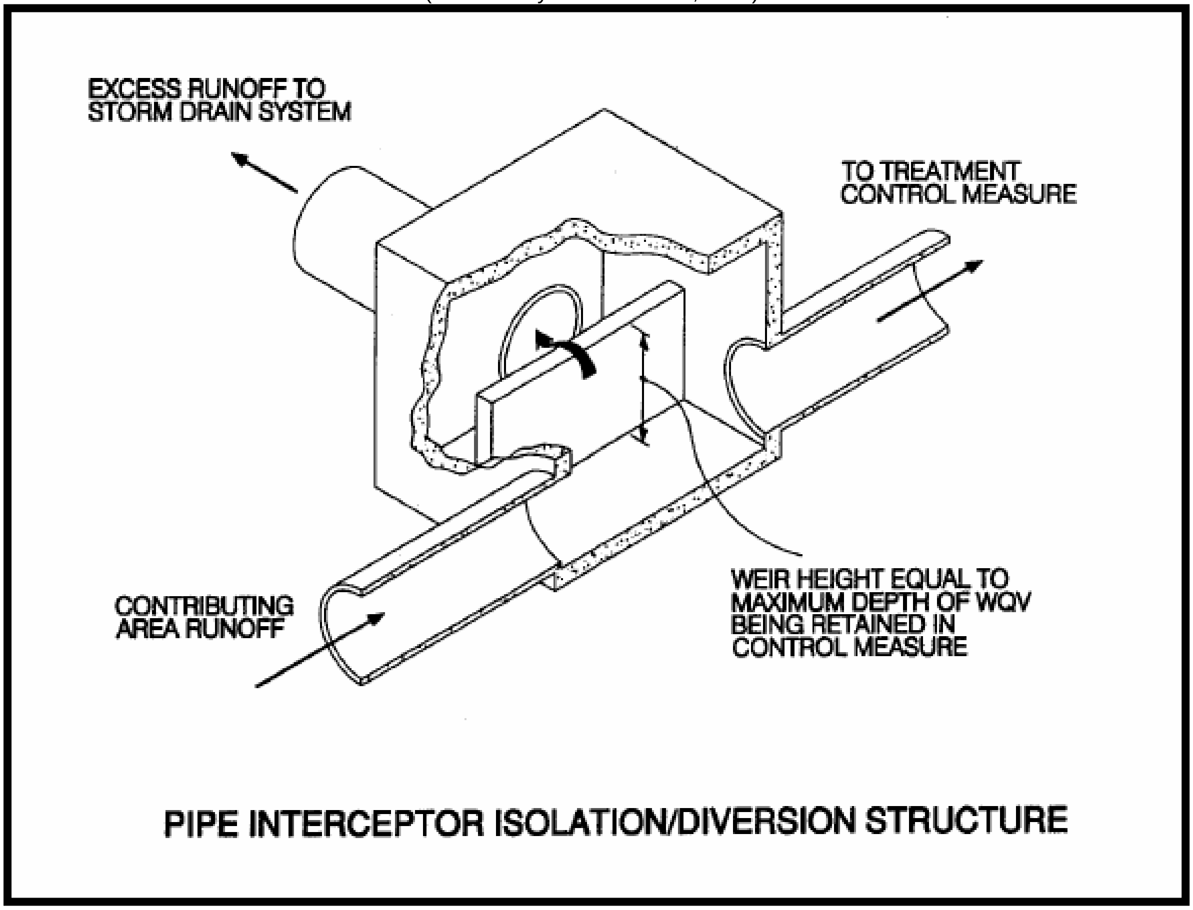


Figure 4-3. Regulator
(Source: City of Sacramento, 2000)

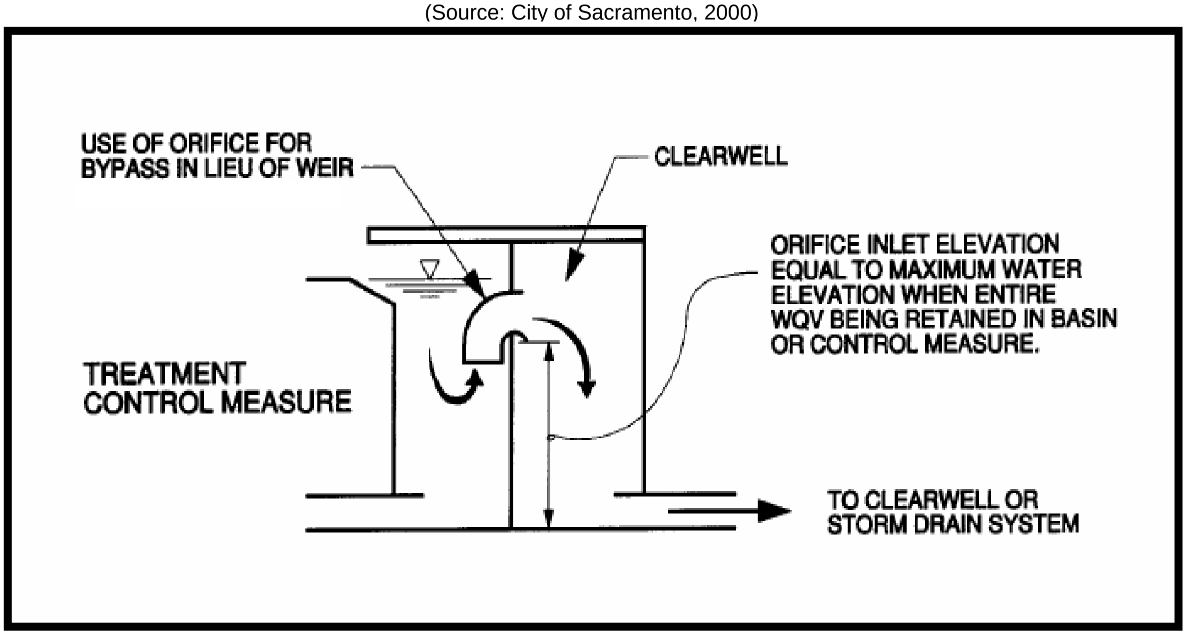
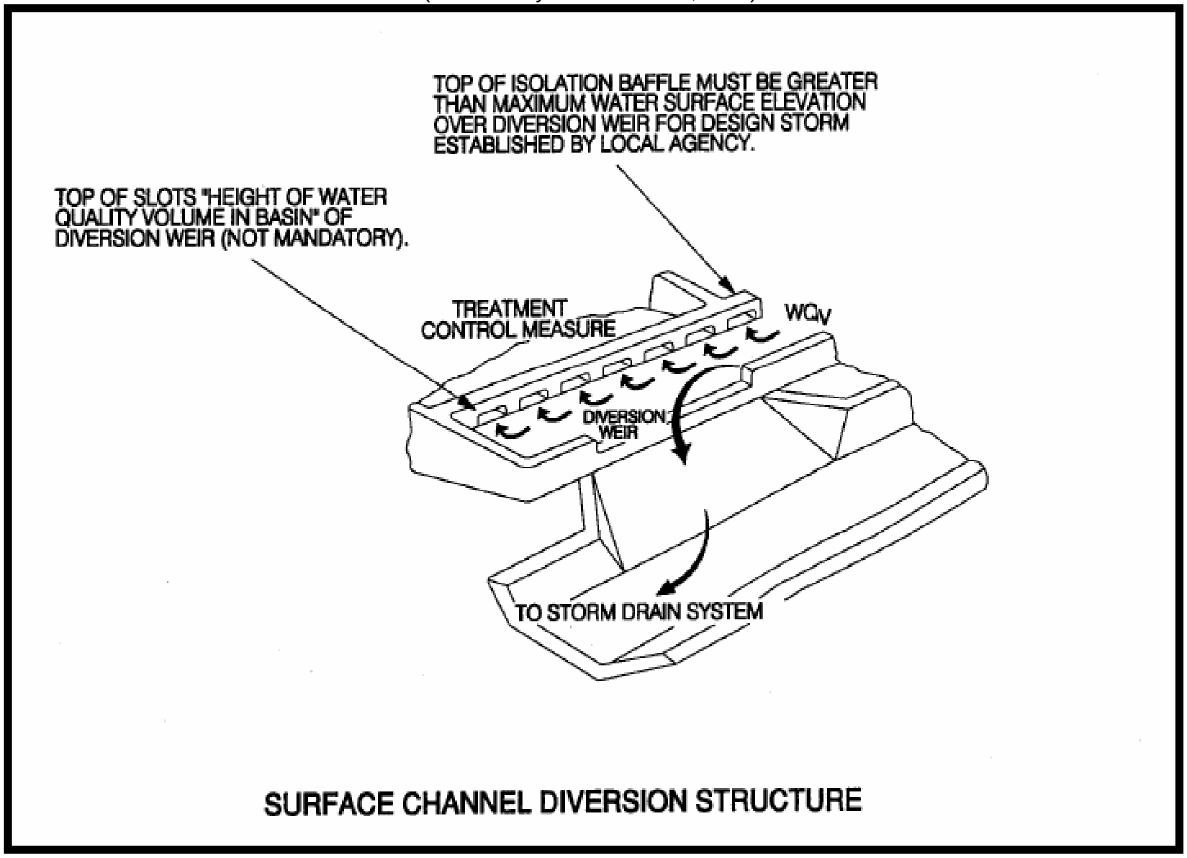


Figure 4-4. Surface Channel Diversion Structure
(Source: City of Sacramento, 2000)



4.2.7 Using Structural Stormwater BMPs in Series

The minimum stormwater management standards are an integrated planning and design approach whose components work together to limit the adverse impacts of urban development on downstream waters and riparian areas. This approach is sometimes called a stormwater “treatment train”, where two or more structural (and sometimes non-structural) BMPs work in series to treat and control stormwater runoff. The calculation of % TSS removal for BMPs in series is discussed in detail in Volume 2, Chapter 2.

When considered comprehensively, a treatment train consists of all the design concepts and nonstructural and structural BMPs that work to attain water quality and quantity goals. This is illustrated in Figure 4-5, and is described below.

Figure 4-5. Generalized Stormwater Treatment Train

<u>Runoff and Load Generation</u> – The initial part of the “train” is located at the source of runoff and pollutant load generation, and consists of better site design and pollution prevention practices that reduce runoff and stormwater pollutants.

Pretreatment – The next step in the treatment train consists of pretreatment measures. These measures typically do not provide sufficient pollutant removal to meet the 80% TSS reduction goal, but do provide calculable water quality benefits that may be applied towards meeting the WQv treatment requirement. These measures include:

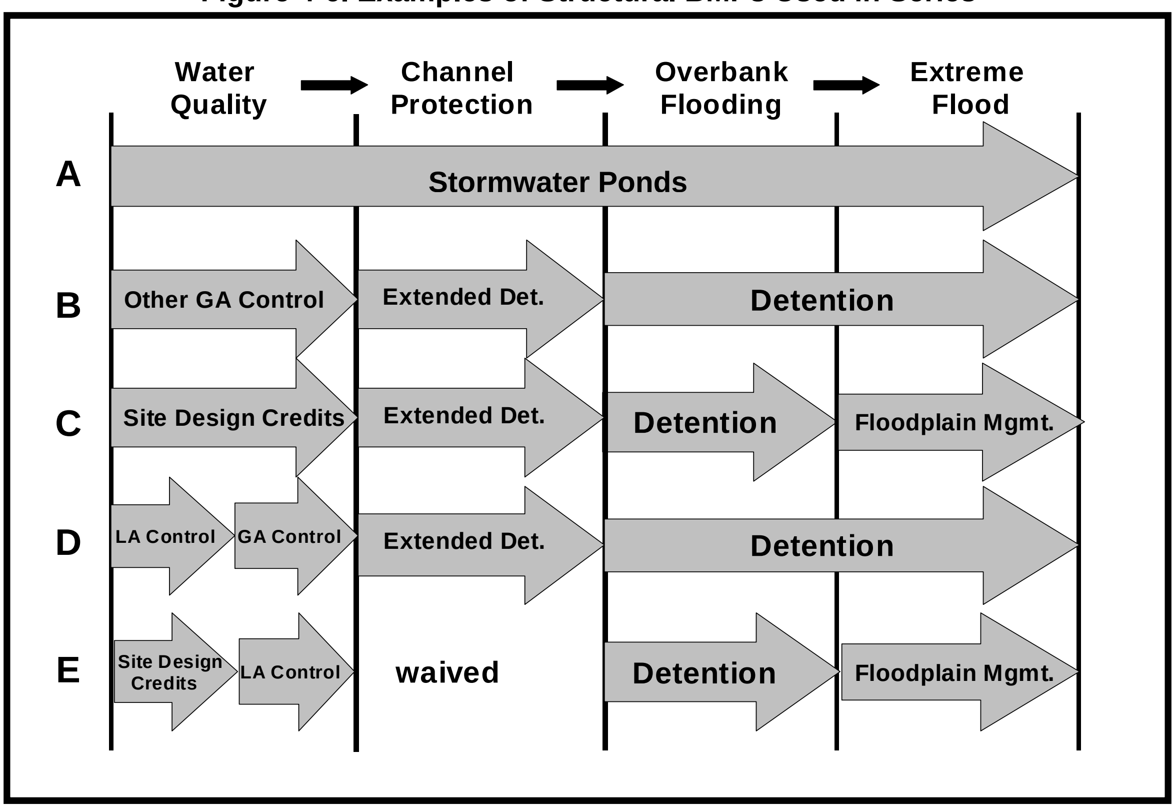
- The use of stormwater better site design practices and site design credits to reduce the generation of stormwater runoff and thereby reducing the WQv
- Limited application BMPs that provide pretreatment
- Pretreatment facilities such as sediment forebays on General Application BMPs

Primary Treatment and/or Quantity Control – The last step is primary water quality treatment and/or quantity (channel protection, overbank flood protection, and/or extreme flood protection) control. This is achieved through the use of general or limited application BMPs, or detention facilities. It should be noted that controls installed to reduce the runoff load and to provide pretreatment can affect the size of the primary treatment control.

4.2.7.1 Use of Multiple Structural BMPs in Series

Many combinations of structural BMPs in series may exist for a site. Figure 4-6 provides a number of hypothetical examples of structural BMPs, that when used in a treatment train, can satisfy Knox County's stormwater design criteria for water quality treatment, channel protection, overbank flooding and extreme flooding. In Figure 4-6, GA indicates General Application BMPs, LA indicates Limited Application BMPs.

Figure 4-6. Examples of Structural BMPs Used in Series



Referring to Figure 4-6 by line letter:

A. One general application BMP, stormwater ponds, can be used as a stand-alone to meet all the design criteria.

B. Other general application (GA) BMPs (bioretention, sand filters, infiltration trench and enhanced swale) are typically used in combination with detention controls to meet the WQv, CPv, Qp_{25} and Qp_{100} criteria. The detention facilities are located downstream from the water quality controls either on-site or combined into a regional or neighborhood facility.

C. Line C indicates the condition where an environmentally sensitive large lot neighborhood (discussed in Volume 2, Chapter 5) has been developed that can be designed so as to waive the water quality treatment requirement altogether. However, detention controls may still be required for downstream channel protection, overbank flood protection and extreme flood protection.

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

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

The combinations of structural stormwater BMPs are limited only by the need to employ measures of proven effectiveness and meet local regulatory and physical site requirements. Figures 4-7, 4-8 and 4-9, illustrate the application of the treatment train concept for: a moderate density residential neighborhood, a small commercial site, and a large shopping mall site, respectively.

In Figure 4-7 rooftop runoff drains over grassed yards to backyard grass channels. Runoff from front yards and driveways reaches roadside grass channels. Finally, all stormwater flows to a micropool ED stormwater pond.

A gas station and convenience store is depicted in Figure 4-8. In this case, the decision was made to intercept hydrocarbons and oils using a commercial gravity (oil-grit) separator located on the site prior to draining to a perimeter sand filter for removal of finer particles and TSS. Overbank and extreme flood protection is provided by a regional stormwater control downstream.

Figure 4-9 shows an example treatment train for a commercial shopping center. In this case, runoff from rooftops and parking lots drains to a depressed parking lot, perimeter grass channels, and bioretention areas. Slotted curbs are used at the entrances to these swales to better distribute the flow and to settle out the very coarse particles at the parking lot edge for sweepers to remove. Runoff is then conveyed to a wet ED pond for additional pollutant removal and channel protection. Overbank and extreme flood protection is provided through parking lot detention.

Figure 4-7. Example Treatment Train – Residential Subdivision
(Adapted from: Atlanta Regional Council, 2001)

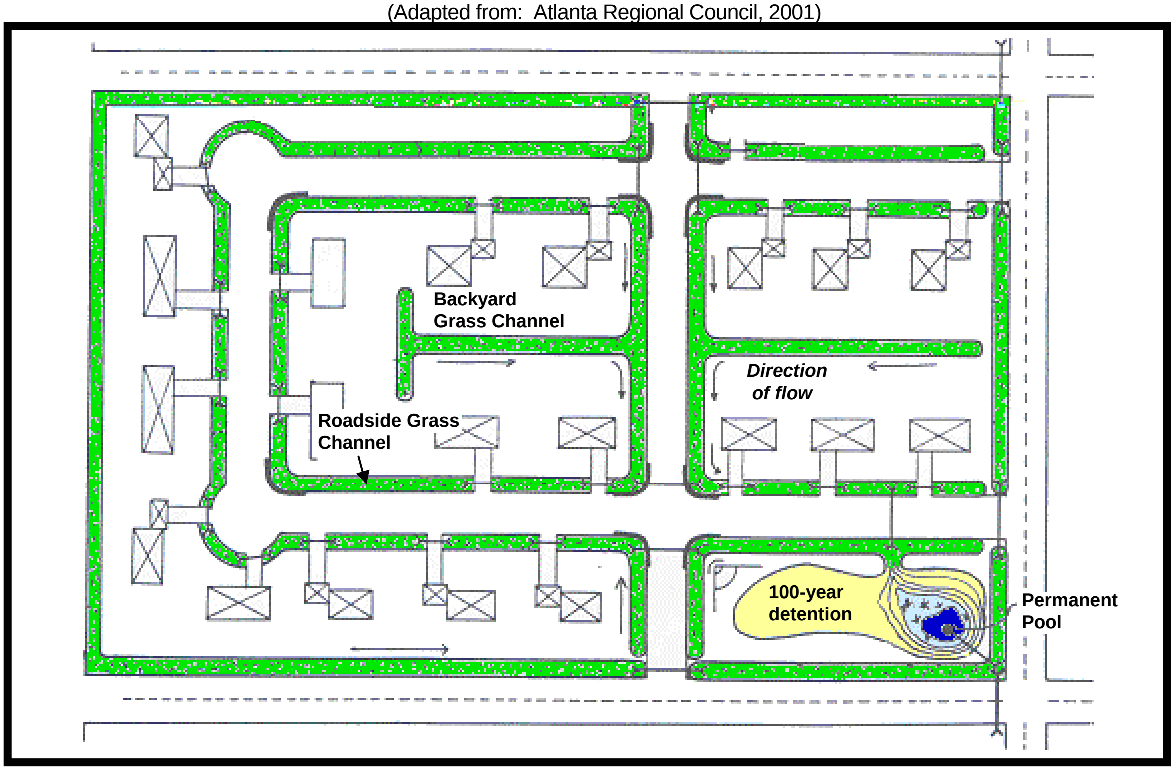


Figure 4-8. Example Treatment Train – Commercial Development

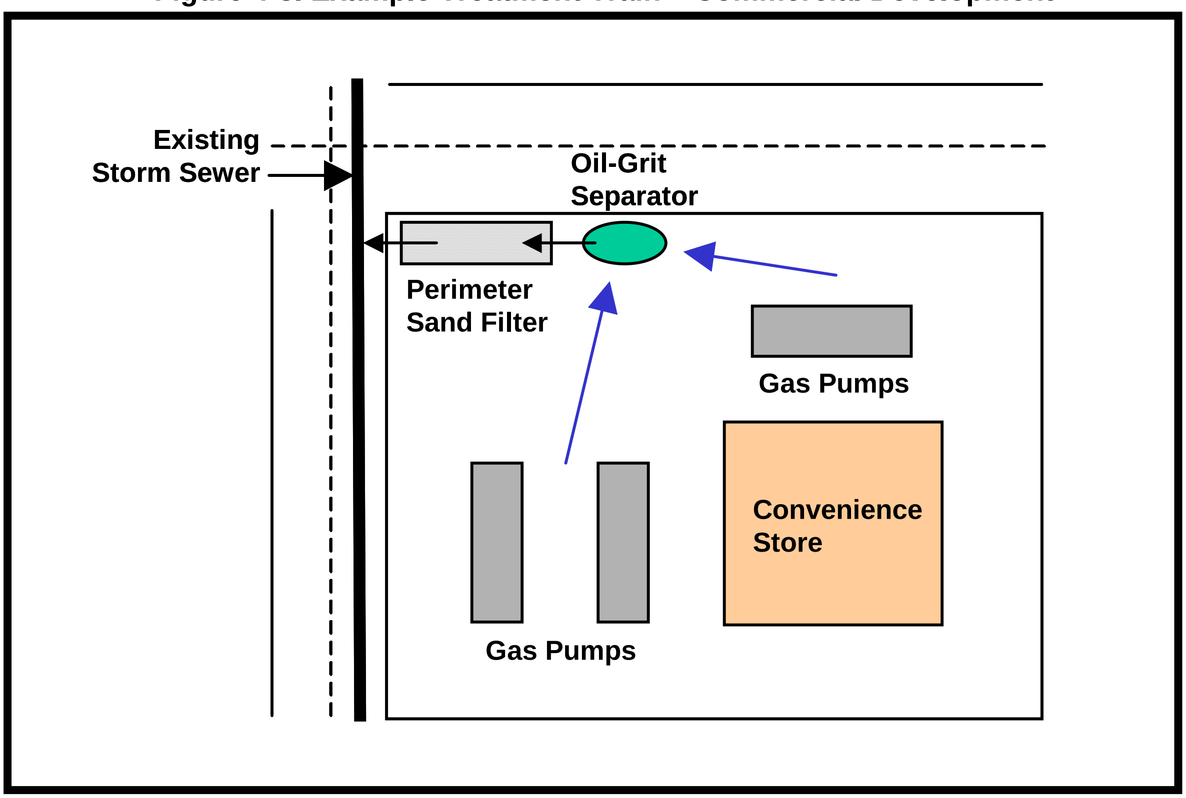
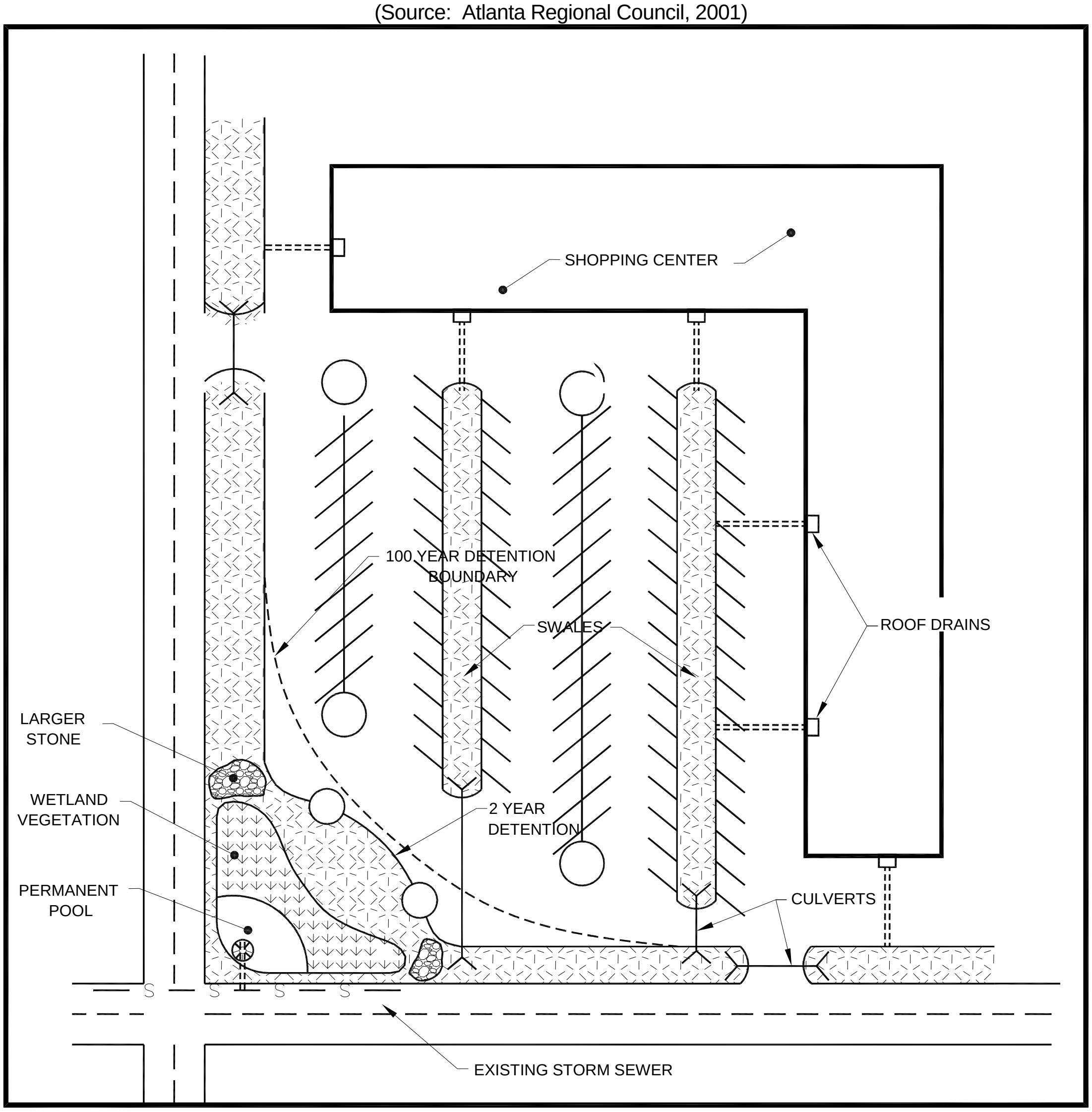


Figure 4-9. Example Treatment Train – Commercial Development
(Source: Atlanta Regional Council, 2001)



References

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

Center for Watershed Protection. *Design of Stormwater Filtering Systems*. Prepared for the Chesapeake Research Consortium 1996.

City of Sacramento Department of Utilities. *Guidance Manual for On-Site Stormwater Quality Control Measures*. 2000.

4.3.1 Stormwater Ponds

General Application Stormwater BMP

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

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Minimum contributing drainage area of 25 acres; 10 acres for micropool ED pond.
- A sediment forebay or equivalent upstream pre-treatment must be provided.
- Minimum length to width ratio for the pond is 1.5:1.
- Maximum depth of the permanent pool shall not exceed 8 feet.
- Side slopes to the pond shall not exceed 3:1 (h:v) on one side of the pond to facilitate access. Slopes as steep as 2:1 will be allowed for other areas, with proper stabilization.

ADVANTAGES / BENEFITS:

- Moderate to high removal rate of urban pollutants.
- High community acceptance if aesthetics are maintained.
- Opportunity for wildlife habitat.

DISADVANTAGES / LIMITATIONS:

- Potential for thermal impacts/downstream warming.
- Dam height restrictions for high relief areas.
- Pond drainage can be difficult for low relief terrain.

MAINTENANCE REQUIREMENTS:

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

STORMWATER MANAGEMENT SUITABILITY

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

Accepts runoff from SPAP land uses: *Yes*

(2' distance required to water table)

FEASIBILITY CONSIDERATIONS

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

Residential/Subdivision Use: *Yes*

Drainage Area: *10-25 acres min.*

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

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

POLLUTANT REMOVAL

H	**Total Suspended Solids**
M	**Nutrients**: Total Phosphorus / Total Nitrogen
M	**Metals**: Cadmium, Copper, Lead, and Zinc
M	**Pathogens**: Coliform, Streptococci, E.Coli

OTHER CONSIDERATIONS

- Outlet clogging
- Safety bench
- Landscaping

4.3.1.1 General Description

Stormwater ponds (also referred to as retention ponds, wet ponds, or wet extended detention ponds) are constructed stormwater retention basins that have a permanent (dead storage) pool of water throughout the year. They can be created by excavating an already existing natural depression or through the construction of embankments.

In a stormwater pond, runoff from each rain event is detained and the water quality volume (WQv) is treated in the pool through gravitational settling and biological uptake until it is displaced by runoff from the next storm. The permanent pool also serves to protect deposited sediments from resuspension. Above the permanent pool level, additional temporary storage (live storage) is provided for runoff quantity control. The upper stages of a stormwater pond are designed to provide extended detention of the downstream channel protection volume (CPv), as well as conventional detention for overbank flood protection (Qp_2, Qp_{10}, and Qp_{25}) and extreme flood protection (Qp_{100}).

Stormwater ponds are among the most cost-effective and widely used stormwater practices. A well-designed and landscaped pond can be an aesthetic feature on a development site when planned and located properly.

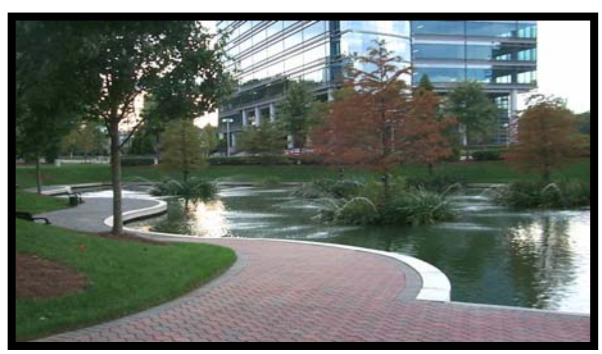
There are several variations of stormwater pond design, the most common of which include the wet pond, the wet extended detention pond, and the micropool extended detention pond. In addition, multiple stormwater ponds can be placed in series or parallel to increase total suspended solids (TSS) removal efficiency or meet site design constraints. Figure 4-10 shows a number of examples of stormwater ponds. Below are descriptions of each design:

Figure 4-10. Stormwater Pond Examples

Wet Pond

Wet Extended Detention Pond

Micropool Extended Detention Pond



Multiple Pond System

- **Wet Pond** – Wet ponds are stormwater basins constructed with a permanent (dead storage) pool of water equal to the WQv. Stormwater runoff is added to the water already present in the pool.

Temporary storage (live storage) can be provided above the permanent pool elevation for larger flows.

- **Wet Extended Detention (ED) Pond** – A wet extended detention pond is a wet pond where the WQv is split evenly between the permanent pool and extended detention (ED) storage provided above the permanent pool. During storm events, water is detained above the permanent pool and released over 24 hours. This design has similar pollutant removal to a traditional wet pond, but consumes less space.
- **Micropool Extended Detention (ED) Pond** – The micropool extended detention pond is a variation of the wet ED pond where only a small "micropool" is maintained at the outlet to the pond. The outlet structure is sized to detain the WQv for 24 hours. The micropool prevents resuspension of previously settled sediments and also prevents clogging of the low flow orifice.
- **Multiple Pond System** – A multiple pond system consists of constructed facilities that provide water quality and quantity volume storage in two or more cells. The additional cells can create longer pollutant removal pathways and improved downstream protection.

4.3.1.2 Stormwater Management Suitability

Stormwater ponds are designed to control both stormwater quantity and quality, and therefore, provide a comprehensive stormwater management BMP. Thus, a stormwater pond can be used to address the minimum design standards for water quality, channel protection and flood protection for a given drainage area.

<u>Water Quality Volume (WQv)</u>
Ponds treat incoming stormwater runoff through physical, biological, and chemical processes. The primary removal mechanism is gravitational settling of particulates, organic matter, metals, bacteria and organics as stormwater runoff resides in the pond. Another mechanism for pollutant removal is uptake by algae and wetland plants in the permanent pool, particularly of nutrients. Volatilization and chemical activity also work to break down and eliminate a number of other stormwater contaminants such as hydrocarbons.

<u>Channel Protection Volume (CPv)</u>
A portion of the storage volume above the permanent pool in a stormwater pond can be used to provide control of the CPv. This is accomplished by capture of the stormwater runoff from the 1-year, 24-hour storm event, and release of that runoff over a minimum 24-hour and a maximum of a 72-hour period, measured from centroid to centroid (extended detention).

<u>Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})</u>
A stormwater pond can also provide storage above the permanent pool/water surface level to reduce the post-development peak flow of the 2-year, 10-year, 25-year and 100-year storms to pre-development levels (detention).

4.3.1.3 Pollutant Removal Capabilities

All of the stormwater pond design variations are presumed capable of removing at least 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the specifications provided in this manual. The TSS removal performance can be reduced by poor design, construction or maintenance.
Additionally, research has shown that use of stormwater ponds will have benefits beyond the removal of TSS, such as the removal of other pollutants (i.e. phosphorous, nitrogen, fecal coliform and heavy metals) as well, which is useful information should the pollutant removal criteria change in the future. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- Total Suspended Solids – 80%
- Total Phosphorus – 55%

- Total Nitrogen – 30%
- Pathogens – 70% (if no resident waterfowl population is present)
- Heavy Metals – 50%

For additional information and data on pollutant removal capabilities for stormwater ponds, see the National Pollutant Removal Performance Database (2nd Edition) available at www.stormwatercenter.net and the International Stormwater Best Management Practices Database at www.bmpdatabase.org.

4.3.1.4 Application and Site Feasibility Criteria

Stormwater ponds are generally applicable to most types of new development and redevelopment, and can be used in both residential and nonresidential areas. Ponds can also be used in retrofit situations. The following criteria should be evaluated to ensure the suitability of a stormwater pond for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for use in residential subdivisions and in non-residential areas.
- Suitable for high density/ultra-urban areas, however, land requirements may preclude their use.
- Suitable for use as a regional stormwater control measure (i.e., controlling runoff from more than one site).

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area: A minimum of 25 acres is needed for wet ponds and wet ED ponds to maintain a permanent pool; 10 acres minimum for micropool ED ponds. The Director may approve a smaller drainage area with an adequate water balance and anti-clogging device.
- Space Required: Approximately 2 to 3% of the contributing drainage area is typically required for most stormwater ponds.
- Site Slope: In general, stormwater ponds can be used on sites that have an upstream slope of up to 15%.
- Minimum Head: Six to eight feet of elevation difference is needed from the inflow of the pond to the outflow.
- Minimum Depth to Water Table: If used on a site with an underlying water supply aquifer (but not within a designated wellhead protection zone) or when treating a land use that requires a Special Pollution Abatement Permit, a separation distance of 2 feet is required between the bottom of the pond and the elevation of the seasonally high water table, as determined through geotechnical data collection or historical data.
- Soils: Underlying soils of hydrologic group “C” or “D” are typically adequate to maintain a permanent pool. Stormwater ponds constructed in group “A” soils and some group “B” soils may require a pond liner. Evaluation of underlying soils should be based upon an actual subsurface analysis and permeability tests.

4.3.1.5 Planning and Design Standards

The following standards shall be considered **minimum** design standards for the design of a stormwater pond facility. Stormwater ponds that are not designed to these standards will not be approved. The Director shall have the authority to require additional design conditions if deemed necessary.

A. LOCATION AND SITING

- Stormwater ponds must have a minimum contributing drainage area of 25 acres or more for a wet pond or wet ED pond to maintain a permanent pool. For a micropool ED pond, the minimum drainage area is 10 acres. The Director may consider allowing the use of a stormwater pond for a smaller drainage area when water availability can be confirmed (such as from a groundwater source or areas that typically have a high water table). In such situations, the Director may require calculation of a water balance for the pond (see Chapter 3 for details). It is important that ponds that serve smaller drainage areas have an adequate anti-clogging device provided for the pond outlet.
- Although not required, Knox County recommends that stormwater ponds be located where the topography allows for maximum runoff storage at minimum excavation or embankment construction costs. When locating a stormwater pond, the site designers should also consider the location and use of other site features, such as buffers and undisturbed natural areas, and should attempt to aesthetically blend the facility into the adjacent landscape.
- Stormwater ponds shall not be located on unstable slopes or slopes greater than 15%.
- Stormwater ponds shall not be located in a stream or any other navigable waters of the United States, including natural (i.e., not constructed) wetlands. Where an appeal or variance of this policy is desired, the property owner must obtain coverage under a Section 404 permit under the Clean Water Act and/or an Aquatic Resource Alteration Permit (ARAP) and provide proof of such coverage with the Stormwater Management Plan.
- Each stormwater pond shall be placed in a water quality easement. The water quality easement shall be defined at the outer edge of the safety bench, or a minimum of 15 feet from the normal water pool elevation (measured perpendicular from the pool elevation boundary) if a safety bench is not included in the pond design. The easement limit should be located no closer than as follows unless otherwise specified by the Director:
  - From a public water system well – TDEC specified distance per designated category
  - From a private well – 50 feet; if the well is downgradient from a land use that must obtain a Special Pollution Abatement Permit, then the minimum is 250 feet
  - From a septic system tank/leach field – 50 feet
- The minimum setback for habitable structures from the water quality easement shall be 15 feet. The first floor elevation (FFE) for any structure adjacent to the pond shall have an elevation no lower than 1 foot above the top of the berm.
- All utilities shall be located outside of the water quality easement.

B. GENERAL DESIGN

- A stormwater pond shall consist of the following elements, designed in accordance with the specifications provided in this section.

  (1) Permanent pool of water;

  (2) A sediment forebay at each pond inlet (unless the inlet provides less than 10% of the total inflow to the pond);

  (3) Overlying zone in which runoff control volumes are stored;

  (4) Shallow littoral zone (aquatic bench) along the edge of the permanent pool that acts as a biological filter;

  (5) An emergency spillway;

(6) Maintenance access;

(7) Safety bench (if pond side slopes are 4:1 or greater); and,

(8) Appropriate native landscaping.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

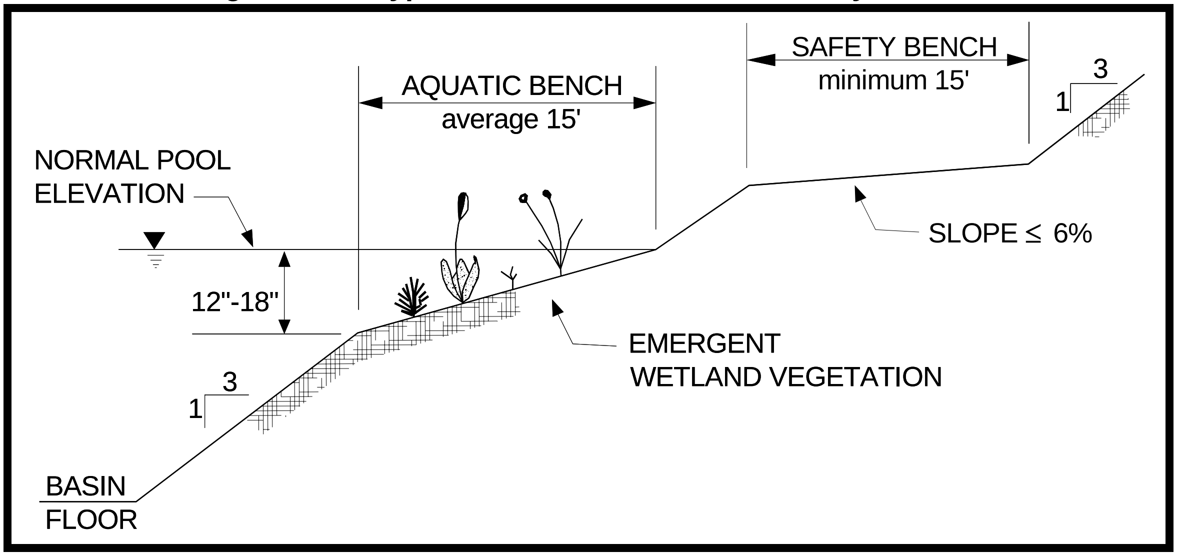
In general, pond designs are unique for each site and application. However, there are a number of geometric ratios and limiting depths for pond design that must be observed for adequate pollutant removal, ease of maintenance, and improved safety.

- Permanent pool volume shall be sized as follows:
  - Standard wet ponds: 100% of the water quality treatment volume (1.0 X WQv);
  - Wet ED ponds: 50% of the water quality treatment volume (0.5 X WQv);
  - Micropool ED ponds: Approximately 0.1 foot per impervious acre (4356 ft^3).
- The pretreatment storage volume is part of the total WQv design requirement and may be subtracted from the WQv for permanent pool sizing. See Part D below for more information.
- Proper geometric design is essential to prevent hydraulic short-circuiting (unequal distribution of inflow), which results in the failure of the pond to achieve adequate levels of pollutant removal. The minimum length-to-width ratio permitted for the permanent pool shape is 1.5:1, and should ideally be greater than 3:1 to avoid short-circuiting. In addition, ponds should be wedge-shaped when possible so that flow enters the pond and gradually spreads out, improving the sedimentation process. Baffles, pond shaping or islands can be added within the permanent pool to increase the flow path.
- Maximum depth of the permanent pool shall not exceed 8 feet to avoid stratification and anoxic conditions. The Director may approve a greater depth in the event that measures are taken that will eliminate the possibility of such conditions and safety precautions are adequately considered. Minimum depth for the permanent pool should be 3 to 4 feet. Deeper depths near the outlet will result in cooler bottom water discharges from the pond, which may mitigate downstream thermal effects caused by discharges of warm stormwater runoff.
- Side slopes shall not exceed 3:1 (horizontal to vertical) on one side of the pond to facilitate access for maintenance and repair. The remainder of the pond shall have side slopes no steeper than 2:1 although 3:1 is preferred. Benching of the slope (see safety bench in Figure 4-11) is required for embankments greater than 10 feet in height and having greater than a 3:1 side slope. Riprap-protected embankments shall be no steeper than 2:1.
- The perimeter of all deep pool areas (4 feet or greater in depth) shall be surrounded by two benches: safety and aquatic. For large ponds, the safety bench shall extend no less than 15 feet outward from the normal water edge to the toe of the pond side slope. The slope of the safety bench shall not exceed 6%. The requirements for a safety bench may be waived if pond side slopes are 4:1 or gentler. The aquatic bench shall have an average width of 15 feet, and shall extend inward from the normal pool edge and shall have a maximum depth of 18 inches below the normal pool water surface elevation (see Figure 4-11).
- The contours and shape of the permanent pool should be irregular to provide a more natural landscaping effect.

D. PRETREATMENT / INLETS

- Each pond shall have a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the stormwater flow prior to dispersal in a larger permanent pool. The forebay shall consist of a separate cell, formed by an acceptable barrier. A forebay must be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. In some design configurations, the pretreatment volume may be located within the permanent pool.

Figure 4-11. Typical Stormwater Pond Geometry Criteria



- The forebay shall be sized to contain 0.1 inch per impervious acre (363 ft^3) of contributing drainage and shall be no more than 4 to 6 feet deep. The pretreatment storage volume is part of the total WQv design requirement and may be subtracted from the WQv for permanent pool sizing.
- A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.
- Inflow channels shall be stabilized with flared riprap aprons, or the equivalent. Inlet pipes to the pond can be partially submerged. Exit velocities of discharges from the forebay to the pond must be non-erosive.

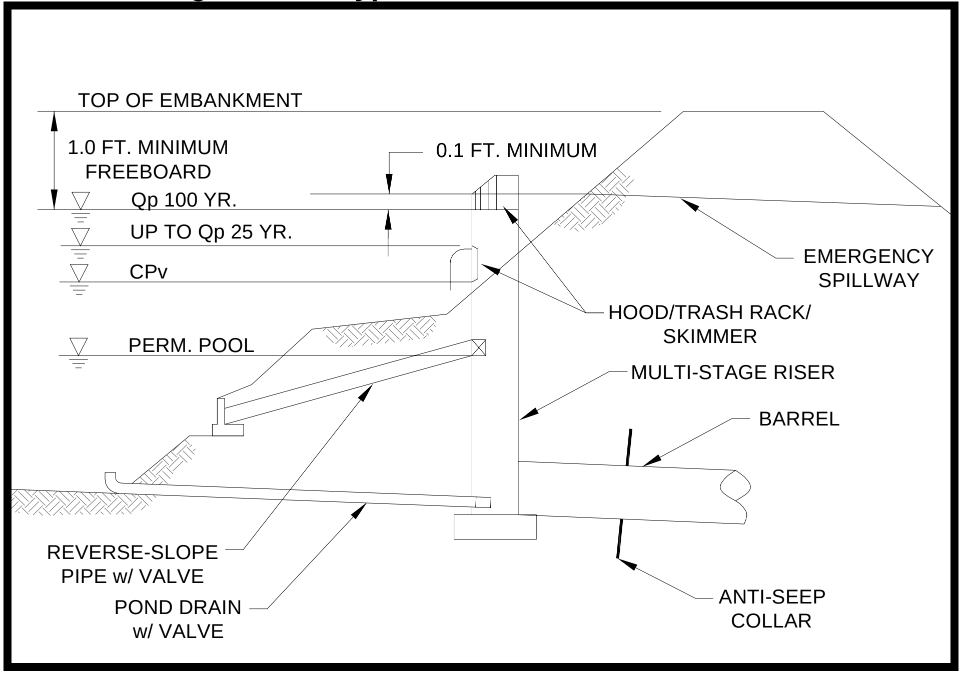
E. OUTLET STRUCTURES

- Flow control from a stormwater pond is typically accomplished with the use of a riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the base of the pond with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (see Figure 4-12). The riser may be located within the pond embankment for maintenance access, safety and aesthetics, unless flow distribution or the potential for erosion around the rise exists. The outlet barrel shall be of reinforced concrete.
- A number of outlets at varying depths in the riser provide internal flow control for routing of the WQv, CPv, Qp_2, Qp_{10}, Qp_{25} and Qp_{100}. The number of orifices can vary and is usually a function of the pond design.

  For example, a wet pond riser configuration is typically comprised of a channel protection (CPv) outlet (usually an orifice), an overbank flood protection (Qp_2, Qp_{10}, Qp_{25}) outlet (often a slot or weir), and the extreme flood protection (Qp_{100}) outlet. The channel protection orifice is sized to release the channel protection storage volume over a 24-hour period. Since the water quality volume is fully contained in the permanent pool, no orifice sizing is necessary for this volume. As runoff from a water quality event enters the wet pond, it simply displaces that same volume through the channel protection orifice. Thus an off-line wet pond providing *only* water quality treatment can use a simple overflow weir as the outlet structure.

  In the case of a wet ED pond or micropool ED pond, there is generally a need for an additional outlet (usually an orifice) that is sized to pass the extended detention water quality volume that is surcharged on top of the permanent pool. Flow will first pass through this orifice, which is sized to release the water quality ED volume in 24 hours. The preferred design is a reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool to

Figure 4-12. Typical Pond Outlet Structure



prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond. The next outlet is sized for the release of the channel protection storage volume. The outlet (often an orifice) invert is located at the maximum elevation associated with the extended detention water quality volume and is sized to release the channel protection storage volume over a 24-hour period. The final orifice invert is located at the extreme flood elevation.

Alternative hydraulic control methods to an orifice can be used and include the use of a broad-crested, rectangular, V-notch, or proportional weir, or an outlet pipe protected by a hood that extends at least 12 inches below the normal pool.

- The water quality outlet (if the design is for a wet ED or micropool ED pond) and channel protection outlet shall be fitted with adjustable slide gates or another mechanism that can be used to adjust detention time.
- Higher flows (Qp_2, Qp_{10}, Qp_{25}, Qp_{100}) pass through openings or slots protected by trash racks further up on the riser.
- After entering the riser, flow is conveyed through the barrel and is discharged downstream. Anti-seep collars shall be installed on the outlet barrel to reduce the potential for pipe or embankment failure.
- Riprap, plunge pads or pools, or other energy dissipators shall be placed at the outlet of the barrel to prevent scouring and erosion. If a pond outlet discharges immediately to a channel that carries dry weather flow, care shall be taken to minimize disturbance along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. See Chapter 7 (Stormwater Drainage Design) for more guidance on outlet designs and Chapter 6 (Water Quality Buffers) for rules and regulations pertaining to encroachments in a water quality buffer.
- Each pond shall have a bottom drain pipe with an adjustable slide gate that can completely or partially drain the pond within 24 hours.
- Ponds shall not be drained until at least 24 hours after the completion of a rain event, so that water quality and channel protection objectives can be met.

See the design procedures in 4.3.1.6 as well as in Volume 2, Chapter 3 for additional information and specifications on pond routing and outlet operations.

F. EMERGENCY SPILLWAY

- An emergency spillway shall be included in the stormwater pond design, sized to safely pass the Qp_{100}. The spillway prevents pond water levels from overtopping the embankment and causing structural damage to the embankment. The emergency spillway shall be located so that downstream structures will not be impacted by spillway discharges.
- A minimum of 1 foot of freeboard shall be provided, measured from the top of the water surface elevation for the extreme flood to the lowest point of the dam embankment, not counting the emergency spillway.

G. MAINTENANCE ACCESS

- A minimum 20' wide maintenance right-of-way or easement shall be provided to the pond from a driveway, public road or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles.
- The maintenance access shall extend to the forebay, safety bench, riser, and outlet, and, to the extent feasible, be designed to allow vehicles to turn around.
- Access to the riser shall be provided by lockable manhole covers, and manhole steps within easy reach of valves and other controls.

H. SAFETY FEATURES

- A safety bench shall be provided for embankments greater than 10 feet in height and having greater than a 3:1 side slope. For large ponds, the safety bench shall extend no less than 15 feet outward from the normal water edge to the toe of the pond side slope. The slope of the safety bench shall not exceed 6%.
- All embankments and spillways shall be designed to TDEC rules and regulations as applied to the Safe Dams Act of 1973, where applicable.
- The property owner may consider fencing the pond for the purpose of safety management.
- All outlet structures shall be designed so as not to permit access by children. Knox County encourages the posting of warning signs near the pond to prohibit swimming and fishing in the facility.

I. LANDSCAPING

- Aquatic vegetation can play an important role in pollutant removal in a stormwater pond. In addition, vegetation can enhance the appearance of the pond, stabilize side slopes, serve as wildlife habitat, and can temporarily conceal unsightly trash and debris. Therefore, wetland plants should be encouraged in a pond design, along the aquatic bench (fringe wetlands), the safety bench and side slopes (ED ponds), and within shallow areas of the pool itself. The best elevations for establishing wetland plants, either through transplantation or volunteer colonization, are within 6 inches (plus or minus) of the normal pool elevation. More information on wetland plants can be found at the following websites:
    - http://wetlands.fws.gov/
    - http://www.npwrc.usgs.gov/resource/plants/floraso/species.htm
- Woody vegetation shall not be planted on the embankment or allowed to grow within 15 feet of the toe of the embankment and 25 feet from the principal spillway structure.
- Native species of fish can be stocked in a pond to aid in mosquito prevention. Knox County strongly discourages the use non-native fish species in a stormwater facility due to the possibility that the fish will enter downstream receiving waters.
- A fountain or aerator may be used for oxygenation of water in the permanent pool and to aid in mosquito breeding prevention.

- Water quality buffers, as defined and described in Volume 2, Chapter 6 of this manual, are not required for stormwater ponds that are constructed for the purpose of stormwater quality or quantity control. However, it should be noted that vegetated buffers can be utilized for water quality treatment and can result in a volume credit that reduces the WQv. The criteria for the vegetated buffer credit are presented in Volume 2, Chapter 5 of this manual.

J. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

There are a number of additional site specific design criteria and issues (listed below) that must be considered in the design of a stormwater pond.

Physiographic Factors - Local terrain design constraints:

- Low Relief – Maximum normal pool depth is limited; providing pond drain can be problematic;
- Karst – Requires poly or clay liner to sustain a permanent pool of water and protect aquifers; limits on ponding depth; geotechnical tests may be required.

Soils

- Hydrologic group “A” soils generally require a pond liner; group “B” soils may require infiltration testing.

Wellhead Protection

- Reduce potential groundwater contamination in wellhead protection areas by preventing infiltration of runoff from areas that require a Special Pollution Abatement Permit, or provide pretreatment of this runoff for the target pollutants that may discharge from the land use.
- Wellhead protection may require liner for type “A” and “B” soils.
- A minimum of two (2) to four (4) feet separation distance of the pond from water table shall be provided.

4.3.1.6 Design Procedures

In general, site designers should perform the following design procedures when designing a stormwater pond.

Step 1. Compute runoff control volumes

Calculate WQv, CPv, Qp_2, Qp_{10}, Qp_{25}, and Qp_{100}, in accordance with the guidance presented in Volume 2, Chapter 3.

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

Consider the Application and Site Feasibility Criteria in sections 4.3.1.4 and 4.3.1.5.

Step 3. Confirm additional design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 4.3.1.5. Check with Knox County Engineering, TDEC, or other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply to the site.

Step 4. Determine pretreatment volume

A sediment forebay is provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. The forebay should be sized to contain 0.1 inch per impervious acre (363 ft^3) of contributing drainage and should be 4 to 6 feet deep. The forebay storage volume counts toward the total WQv requirement and may be subtracted from the WQv for subsequent calculations.

Step 5. Determine permanent pool volume (and water quality ED volume)

Wet Pond: Size permanent pool volume to 1.0 WQv less any forebay storage volume.

Wet ED Pond: Size permanent pool volume to 0.5 WQv less any forebay storage volume. Size extended detention volume to 0.5 WQv less any forebay storage volume.

Micropool ED Pond: Size permanent pool volume at 0.1 foot per impervious acre (4356 ft^3) less any forebay storage volume. Size extended detention volume to remainder of WQv.

Step 6. Determine pond location and preliminary geometry. Conduct pond grading design and determine storage available for permanent pool (and water quality extended detention if wet ED pond or micropool ED pond)

This step involves initially designing the grading of the pond (establishing contours) and determining the elevation-storage relationship for the pond. See subsection 4.3.1.5 for more details.

- Include safety and aquatic benches, if required.
- Set WQv permanent pool elevation (and WQv-ED elevation for wet ED and micropool ED pond) based on volumes calculated earlier.

Step 7. Compute extended detention orifice release rate(s) and size(s), and establish CPv elevation

Wet Pond: The CPv elevation is determined from the stage-storage relationship and the orifice is then sized to detain the channel protection storage volume for a 24-hour period, measured from centroid to centroid. The channel protection orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool, is a recommended design. Orifice diameters less than three inches must employ internal orifice protection (i.e., an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable slide gates can also be used to achieve this equivalent diameter.

Wet ED Pond and Micropool ED Pond: Based on the elevations established in Step 6 for the extended detention portion of the water quality volume, the water quality orifice is sized to release this extended detention volume in 24 hours. The water quality orifice should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool, is a recommended design. Orifice diameters less than three inches must employ internal orifice protection (i.e., an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable slide gates can also be used to achieve this equivalent diameter. The CPv elevation is then determined from the stage-storage relationship. The invert of the channel protection orifice is located at the water quality extended detention elevation, and the orifice is sized to detain the channel protection storage volume for a 24-hour period, measured from centroid to centroid.

Step 8. Calculate Qp_2, Qp_{10}, Qp_{25} and Qp_{100} release rates and water surface elevations

Set up a stage-storage-discharge relationship for the control structure for the extended detention (CPv) requirement, the 2, 10, and 25-year storms, and the 100-year storm orifices.

Step 9. Design embankment(s) and spillway(s)

Using the 100-year water surface elevation, set the top of the embankment elevation, and size the emergency spillway, ensuring safe passage of the Qp_{100}. Set the invert elevation of the emergency spillway 0.1 foot above the 100-year water surface elevation.

Step 10. Investigate potential pond hazard classification

The design and construction of stormwater management ponds are required to follow the latest version of the TDEC Rules and Regulations Application to the Safe Dams Act of 1973.

Step 11. Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features

See subsection 4.3.1.5-D through H for more details.

Step 12. Design vegetation

A vegetation scheme for a stormwater pond and its buffer should be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation.

See subsection 4.3.1.5-I for more details.

4.3.1.7 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.1.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of stormwater ponds as designed. It is the responsibility of the property owner to maintain all stormwater facilities in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for stormwater ponds, along with a suggested frequency for each activity. Individual stormwater ponds may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e.., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the pond in proper operating condition at all times.

Inspection Activities	Suggested Schedule
• After several storm events or an extreme storm event, inspect for: bank stability; signs of erosion; and damage to, or clogging of, the inlet/outlet structures and pilot channels.	As needed
• Inspect for: trash and debris; clogging of the inlet/outlet structures and any pilot channels; excessive erosion; sediment accumulation in the basin, forebay and inlet/outlet structures; tree growth on dam or embankment; the presence of burrowing animals; standing water where there should be none; vigor and density of the grass turf on the basin side slopes and floor; differential settlement; cracking; leakage; and slope stability.	Semi-annually
• Inspect that the inlet/outlet structures, pipes, sediment forebays, and upstream, downstream, and pilot channels are free of debris and are operational. • Check for signs of unhealthy or overpopulation of plants and/or fish (if utilized). • Note signs of algal growth or pollution, such as oil sheens, discolored water, or unpleasant odors. • Check sediment marker(s) for sediment accumulation in the facility and forebay. • Check for proper operation of control gates, valves or other mechanical devices. • Note changes to the wet pond or contributing drainage area as such changes may affect pond performance.	Annually
Maintenance Activities	**Suggested Schedule**
• Clean and remove debris from inlet and outlet structures. • Mow side slopes (embankment) and maintenance access. Periodic mowing is only required along maintenance rights-of-way and the embankment. The remaining pond buffer can be managed as a meadow (mowing every other year) or forest.	Monthly
• If wetland vegetation is included, remove invasive vegetation.	Semi-annually
• Repair damage to pond, outlet structures, embankments, control gates, valves, or other mechanical devices; repair undercut or eroded areas. • Remove pollutants or algal overgrowth as appropriate.	As Needed
• Perform wetland plant management and harvesting.	Annually (if needed)
• Remove sediment from the forebay. Sediments excavated from stormwater ponds that do not receive runoff from land uses that require a Special Pollution Abatement Permit (SPAP) are not considered toxic or hazardous material and can be safely disposed of by either land application or landfilling. Sediment testing is required prior to sediment disposal when the pond receives discharge from a land use that requires a SPAP. Dispose of sediments per Section 10.3.	5 to 7 years or after 50% of the total forebay capacity has been lost
• Monitor sediment accumulations, and remove sediment when the pond volume has become reduced significantly or the pond is not providing a healthy habitat for vegetation and fish (if used). Discharges of pond water may be considered an illegal discharge, as per the Knox County Stormwater Management Ordinance. Care should be exercised during pond drawdowns to prevent downstream discharge of sediments, anoxic water, or high flows with erosive velocities. Knox County should be notified before draining a stormwater pond.	10 to 20 years or after 25% of the permanent pool volume has been lost

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of stormwater ponds. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the stormwater pond. Questions regarding stormwater facility inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
STORMWATER POND INSPECTION CHECKLIST

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ____

Date: ___________ Time: ______________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Embankment and Emergency Spillway		
Healthy vegetation?		
Erosion on embankment?		
Animal burrows in embankment?		
Cracking, sliding, bulging of dam?		
Blocked or malfunctioning drains?		
Leaks or seeps on embankment?		
Obstructions of spillway(s)?		
Erosion in/around emergency spillway?		
Other (describe)?		
Inlet/Outlet Structures and Channels		
Clear of debris and functional?		
Trash rack clear of debris and functional?		
Sediment accumulation?		
Condition of concrete/masonry?		
Pipes in good condition?		
Slide gate operation?		
Pond drain valve operation?		
Outfall channels function, not eroding?		
Other (describe)?		
Sediment Forebays		
Evidence of sediment accumulation?		
Permanent Pool Areas (if applicable)		
Undesirable vegetation growth?		
Visible pollution?		
Shoreline erosion?		
Erosion at outfalls into pond?		
Headwalls and endwalls in good condition?		
Encroachment by other activities?		
Evidence of sediment accumulation?		
Dry Pond Areas (if applicable)		
Vegetation adequate?		
Undesirable vegetation growth?		
Excessive sedimentation?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature:________________________________ Inspector Name (printed)____________________________

4.3.1.8 Example Schematics

The example schematics for stormwater wet ponds presented in Figures 4-13 through 4-16 can be used to assist in the design of such BMPs.

Figure 4-13. Schematic of a Standard Wet Pond

(Source: adapted from CWP, 2005)

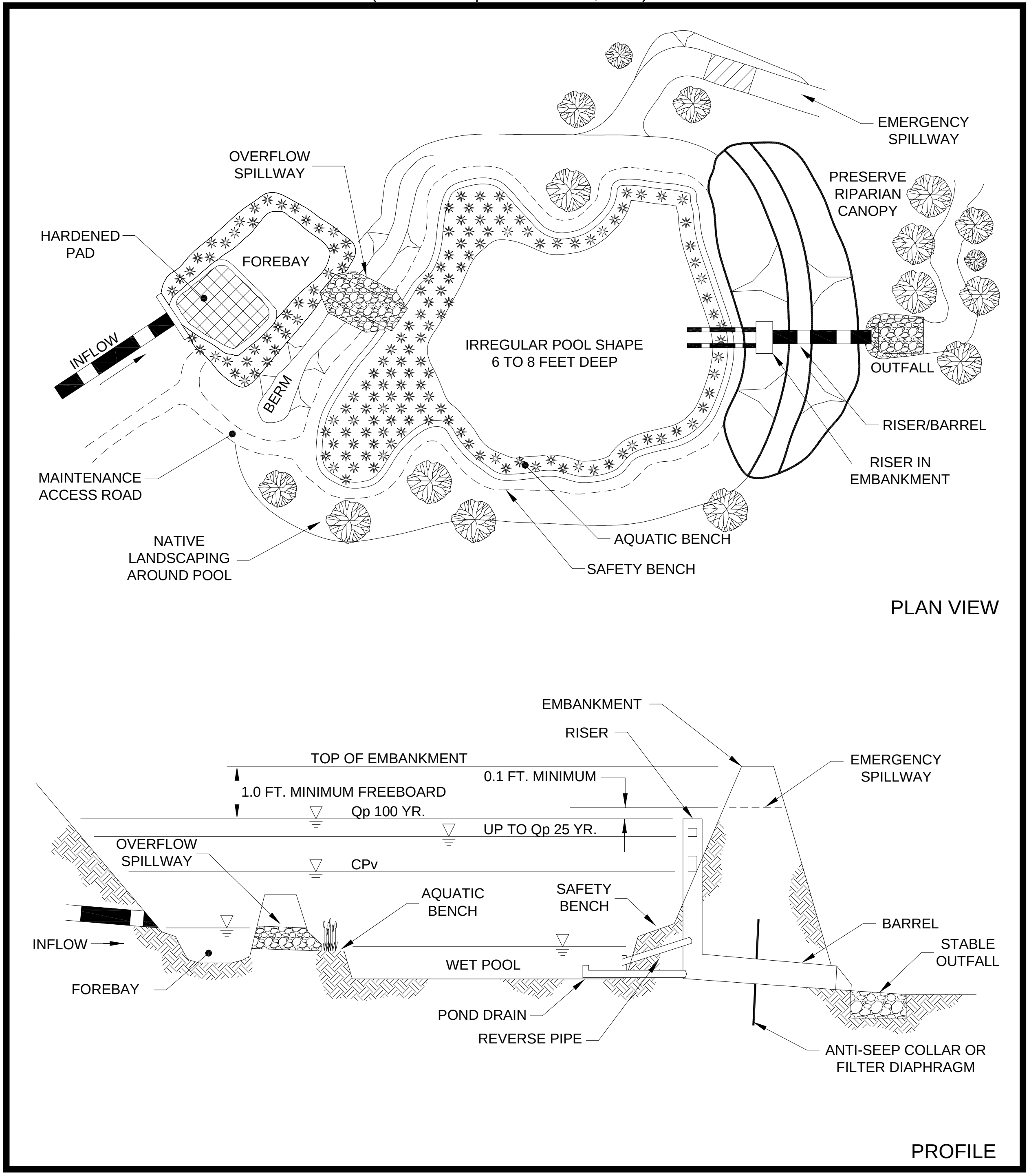


Figure 4-14. Schematic of a Wet Extended Detention Pond
(Source: adapted from CWP, 2005)

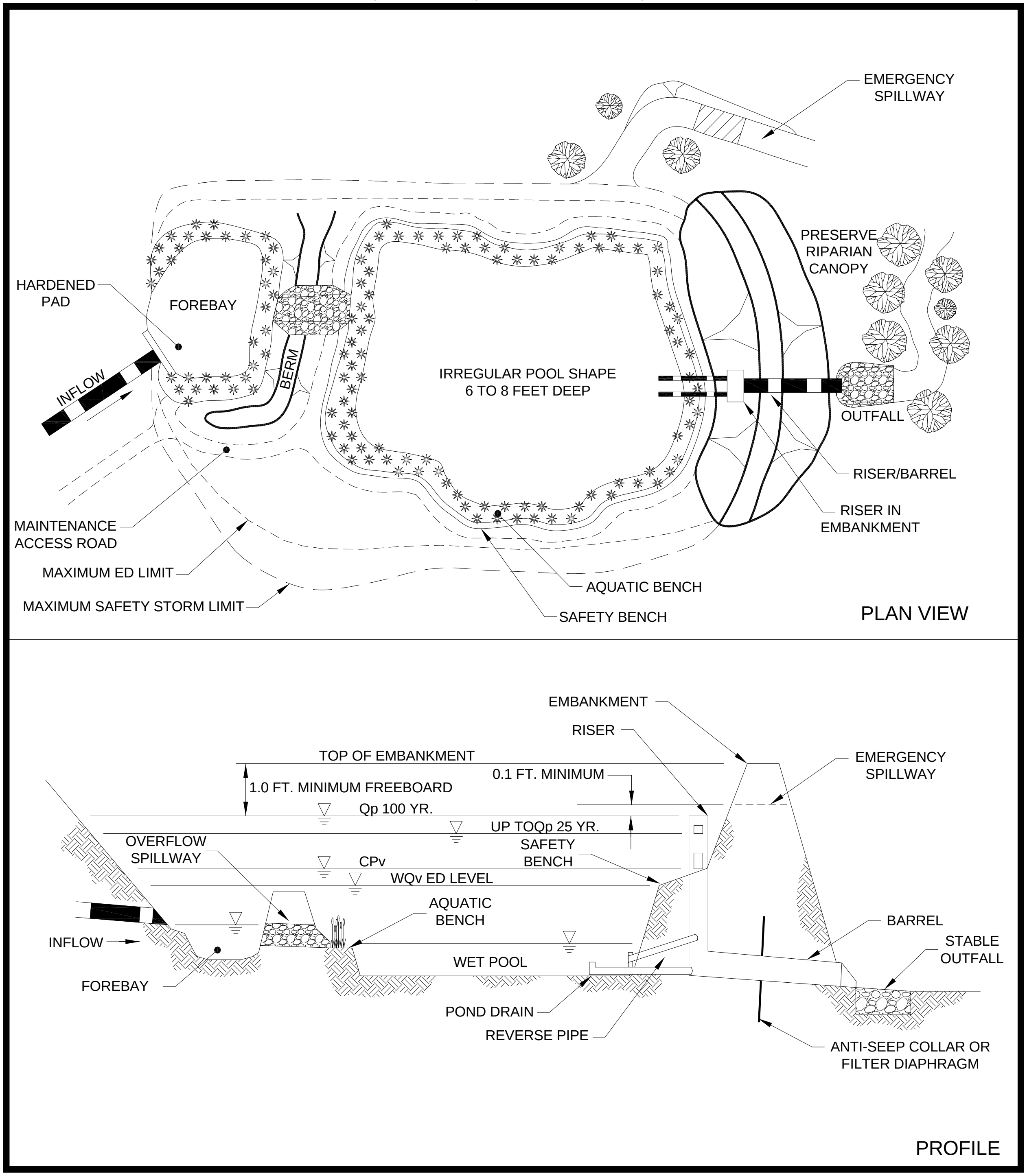


Figure 4-15. Schematic of a Micropool Extended Detention Pond
(Source: adapted from CWP, 2005)

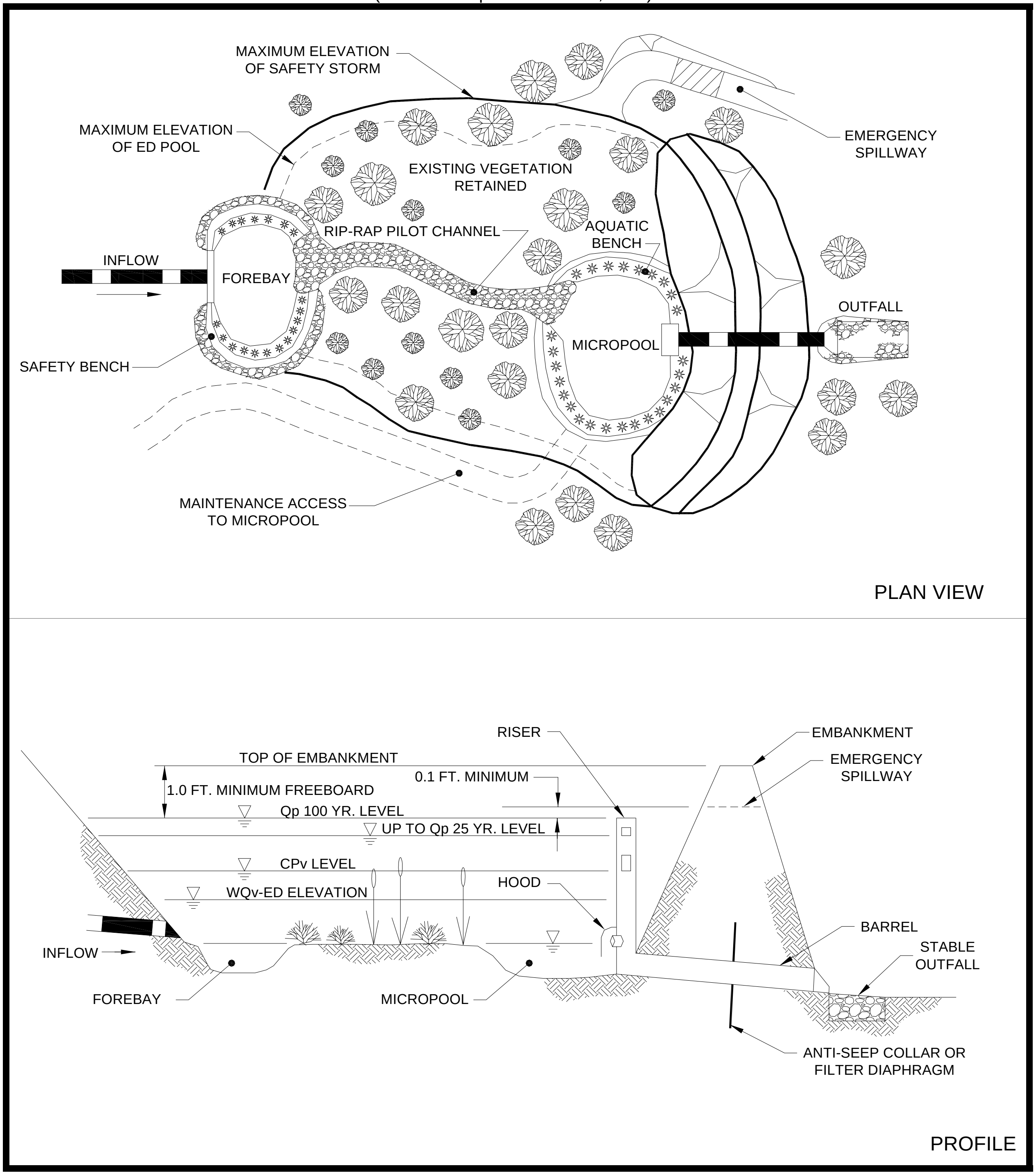
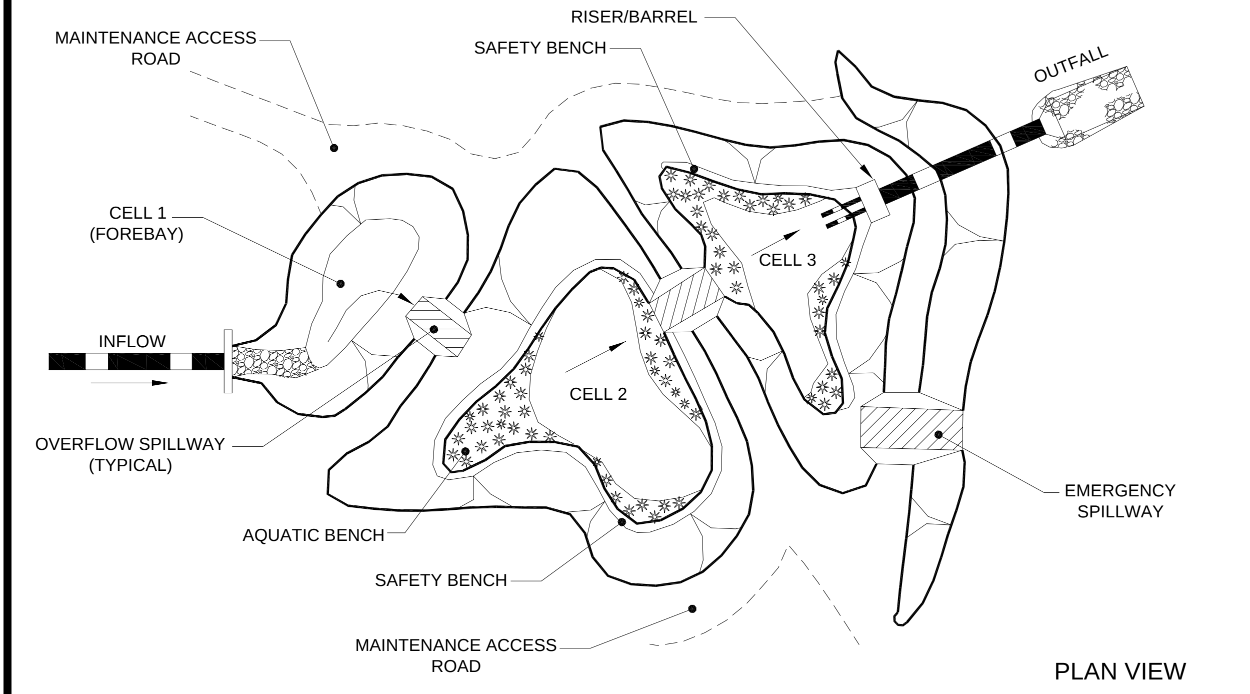
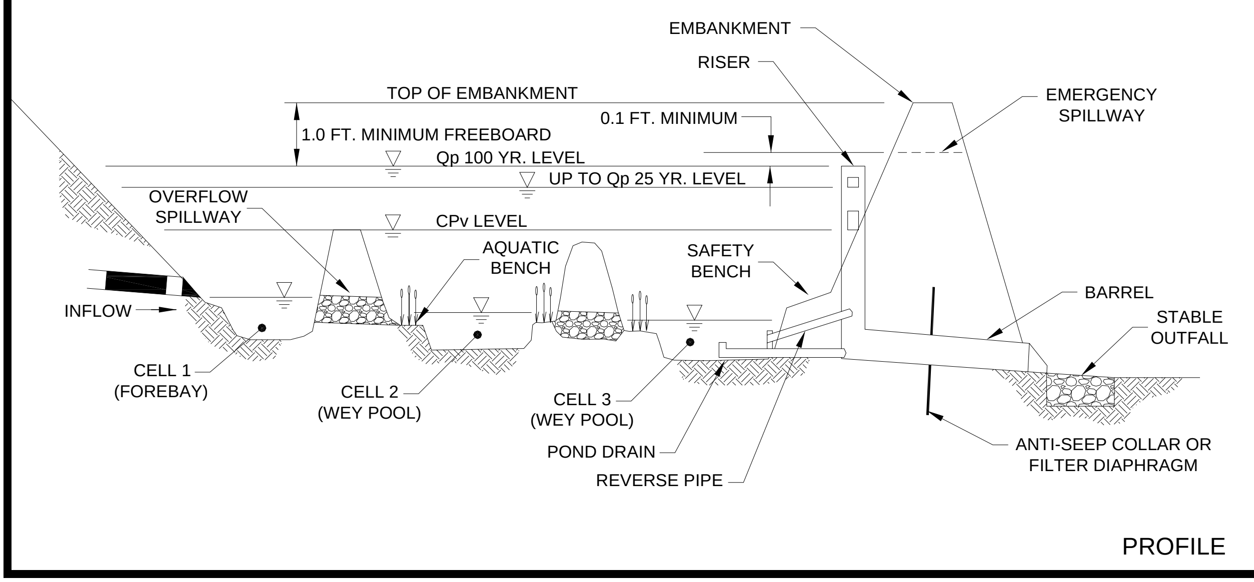


Figure 4-16. Schematic of a Multiple Pond System
(Source: adapted from CWP, 2005)



PLAN VIEW



4.3.1.9 Design Forms

Knox County recommends the use of the following design procedure forms when designing a stormwater wet pond. Proper use and completion of the form may allow a faster review of the Stormwater Management Plan by Knox County Engineering.

Design Procedure Form: Stormwater Ponds

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQv volume requirements
Compute Runoff Coefficient, Rv — Rv = ______
Compute WQv — WQv = ______ acre-ft

1b. Estimate CPv — CPv = ______ acre-ft

1c. Estimate storage volumes
Estimate storage volume required for 2-year storm — 2-year storage = ______ acre-ft
Estimate storage volume required for 10-year storm — 10-year storage = ______ acre-ft
Estimate storage volume required for 25-year storm — 25-year storage = ______ acre-ft
Estimate storage volume required for 100-year storm — 100-year storage = ______ acre-ft

STORMWATER POND DESIGN

2. Is the use of a stormwater pond appropriate? — ***See subsections 4.3.1.4 and 4.3.1.5 - A***

3. Confirm additional design criteria and applicability. — ***See subsection 4.3.1.5 - J***

4. Pretreatment Volume (Forebay)
V_{pre} =(I)(.1")(1'/12") — V_{pre} = ______ acre-ft

5. Allocation of Permanent Pool Volume and ED Volume

Wet Pond — V_{pool}=1.0(WQv)-Vol_{pre} — V_{pool}= ______ acre-ft

Wet ED Pond — V_{pool}=0.5(WQv)-Vol_{pre} — V_{pool}= ______ acre-ft
V_{ED}=0.5(WQv) — V_{ED}= ______ acre-ft

Micropool ED Volume — V_{pool}=(I)(.1")(1'/12") — V_{pool}= ______ acre-ft

6. Conduct grading design and determine storage available for permanent pool (and WQv-ED volume if applicable) — Prepare an elevation-storage table and curve using the average area method for computing volumes.

Elevation MSL	Area ft^2	Ave. Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3	Volume above Permanent Pool acre-ft

Design Procedure Form: Stormwater Ponds (continued)

7. WQv and CPv Orifice Computations

Average ED release rate (if applicable)	release rate= ________ cfs
Average head, h= (ED elev.- Permanent Pool elev.) /2	head= ________ ft
Area of orifice from orifice equation	Area= ________ ft^2
$Q=CA(2gh)^{0.5}$	diameter ________ inches
(C varies with orifice condition. Refer to Chapter 3, Section 3.3.2.3 for guidance)	
Establish CPv top elevation using stage-storage curve	CPv WSEL= ________ ft-NGVD
Estimate orifice size	CPv orifice diameter = ________ inches
Perform hydrograph routing to check detention time	centroid-centroid det. = ________ hours
Iterate to final orifice size	Final CPv orifice diameter = ________ inches

8. Calculate Qp_2, Qp_{10}, Qp_{25} and Qp_{100} release rates and WSEL

Set up a stage-storage-discharge relationship

Elevation	Storage	Low Flow	Riser			Barrel		Emergency	Total
		WQv-ED	CPv-ED	High Storage		Inlet	Pipe	Spillway	Outflow
				Orif.	Weir				
MSL	acre-ft	H(ft) Q(cfs)	H(ft) Q(cfs)	H Q	H Q	H(ft) Q(cfs)	H(ft) Q(cfs)	H(ft) Q(cfs)	Q(cfs)

Qp_2=pre-dev. Peak discharge - (WQv-ED release + CPv-ED release)	Qp_2= ________ cfs
Qp_{10}=pre-dev. Peak discharge - (WQv-ED release + CPv-ED release)	Qp_{10}= ________ cfs
Qp_{25}=pre-dev. Peak discharge - (WQv-ED release + CPv-ED release)	Qp_{25}= ________ cfs
Qp_{100}=pre-dev. Peak discharge - (WQv-ED release + CPv-ED release)	Qp_{100}= ________ cfs
Maximum head =	H= ________ ft
Use weir equation for slot length ($Q=CLH^{3/2}$)	L= ________ ft
Check inlet condition Check outlet conditions	**Use culvert design guidance in Chapter 7**

9. Size emergency spillway using the Qp_{100} and set top of embankment elevation and emergency spillway elevation based on $WSEL_{100}$

$Q_{ES}=Qp_{100}$ ________ cfs
$WSEL_{100}$= ________ ft
$El_{embank}=WSEL_{100}+1.0$ ________ ft
$El_{ES}=WSEL_{100}+.01$ feet ________ ft

10. Investigate potential pond hazard classification — **See TN Safe Dams Act of 1973**

11. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features — **See Section 4.3.1.5 - D through H**

12. Design vegetation according to guidance in Chapter 6

13. Verify peak flow control, water quality drawdown time and channel protection detention time

4.3.1.10 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001*.*

Center for Watershed Protection. *Manual Builder.* Stormwater Manager's Resource Center, Accessed July 2005. *www.stormwatercenter.net*

4.3.1.11 Suggested Reading

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

City of Austin, TX. *Water Quality Management.* Environmental Criteria Manual, Environmental and Conservation Services, 1988.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

Claytor, R.A., and T.R. Schueler. *Design of Stormwater Filtering Systems.* The Center for Watershed Protection, Silver Spring, MD, 1996.

Maryland Department of the Environment. *Maryland Stormwater Design Manual, Volumes I and II.* Prepared by Center for Watershed Protection (CWP), 2000.

Metropolitan Washington Council of Governments (MWCOG). *A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone.* March, 1992.

United States Environmental Protection Agency. *Methodology for Analysis of Detention Basins for Control of Urban Runoff Quality,* 1986.

Urban Drainage and Flood Control District. *Urban Storm Drainage Criteria Manual – Volume 3 – Best Management Practices – Stormwater Quality.* Denver, Colorado, September 1992.

Walker, W. *Phosphorus Removal by Urban Runoff Detention Basins.* Lake and Reservoir Management, North American Society for Lake Management, 314, 1987.

Wanielista. M. *Final Report on Efficiency Optimization of Wet Detention Ponds for Urban Stormwater Management.* University of Central Florida, 1989.

This page left intentionally blank

4.3.2 Conventional Dry Detention

General Application Stormwater BMP

Description: A surface storage basin or facility designed to provide water quantity control through detention of stormwater runoff.

KEY CONSIDERATIONS

- Conventional detention ponds provide control for overbank and extreme flood protection only. These ponds are not intended to provide water quality treatment.
- Typically less costly than stormwater (wet) ponds for equivalent flood storage, as less excavation is required.
- Must be used in conjunction with other BMPs that can adequately meet Knox County's minimum standard of 80% removal of TSS.
- Dry detention basins can be used to provide recreational and other open space opportunities between storm runoff events when the pond bottom is dry.

MAINTENANCE REQUIREMENTS:

- Remove debris from inlet and outlet structures.
- Maintain side slopes and outlet structure.
- Monitor sediment accumulation and remove periodically.

STORMWATER MANAGEMENT SUITABILITY

- ☐ **Water Quality**
- ☐ **Channel/Flood Protection**
- ☑ **Overbank Flood Protection**
- ☑ **Extreme Flood Protection**

Accepts runoff from SPAP land uses: *Yes*

FEASIBILITY CONSIDERATIONS

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

Residential/Subdivision Use: *Yes*

High Density/Ultra-Urban: *No*

Drainage Area: *unlimited.*

4.3.2.1 General Description

Conventional dry detention ponds are surface facilities intended to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts. These facilities temporarily detain stormwater runoff, releasing the flow over a period of time. They are designed to completely drain following a storm event and are normally dry between rain events.

Dry detention ponds can be utilized to provide overbank flood protection (Qp_2, Qp_{10}, and Qp_{25}) and extreme flood protection for the 100-year storm event (Qp_{100}). Such ponds provide limited pollutant removal benefits and are not intended for water quality treatment. Because conventional detention-only facilities can not provide a significant degree of water quality treatment, they must be used in conjunction with other structural controls that provide treatment of the water quality volume (WQv). Volume 2, Chapter 2 provides more information on treatment trains.

4.3.2.2 Planning and Design Standards

The following standards shall be considered **minimum** design standards for the design of a dry detention pond. Detention ponds that are not designed to these standards will not be approved. The Director of Engineering and Public Works (the Director) shall have the authority to require additional design conditions if deemed necessary.

<u>A. LOCATION AND SITING</u>

- It is strongly recommended that dry detention ponds be located where the topography allows for maximum runoff storage at minimum excavation or embankment construction costs. When locating a detention basin, the site designers should also consider the location and use of other land use features, such as planned open spaces and recreational areas, and should attempt to achieve a multi-use objective with the pond where this can be safely achieved.
- Detention ponds shall not be located on unstable slopes or slopes greater than 15%.
- Flood protection controls for peak discharge control (Qp_2, Qp_{10}, Qp_{25} and Qp_{100}) should be designed as final controls for on-site stormwater. Therefore, dry detention ponds will typically be located downstream of structural stormwater BMPs that are designed to provide treatment of the water quality volume (WQv) and channel protection volume (CPv).
- Detention ponds shall not be located in a stream or any other navigable waters of the United States, including natural (i.e., not constructed) wetlands. Where an appeal or variance of this policy is desired, the property owner must obtain coverage under a Section 404 permit under the Clean Water Act and/or an Aquatic Resource Alteration Permit (ARAP) and provide proof of such coverage with the Stormwater Management Plan.
- Each detention pond shall be placed in a water quality easement. The water quality easement shall be defined at the outer edge of the safety bench, or a minimum of 15 feet from the normal water pool elevation (measured perpendicular from the pool elevation boundary) if a safety bench is not included in the pond design. The easement limit should be located no closer than as follows unless otherwise specified by the Director:
  - From a public water system well – TDEC specified distance per designated category
  - From a private well – 50 feet; if the well is downgradient from a land use that must obtain a Special Pollution Abatement Permit, then the minimum is 250 feet
  - From a septic system tank/leach field – 50 feet
- The minimum setback for habitable structures from the drainage easement shall be 15 feet. The first floor elevation (FFE) for any structure adjacent to the pond shall have an elevation no lower than 1 foot above the top of the berm.
- All utilities shall be located outside of the pond/basin site.

B. GENERAL DESIGN

- A dry detention pond shall consist of the following elements, designed in accordance with the specifications provided in this section.

  (1) An outlet structure;

  (2) An emergency spillway;

  (3) Maintenance access; and,

  (4) Appropriate landscaping.

- Dry detention basins shall be sized to attenuate peak discharges for the overbank flood (Qp_2, Qp_{10}, Qp_{25}) protection and/or extreme flood protection (Qp_{100}) design criteria (see Chapter 2). Routing calculations must be used to demonstrate that the storage volume is adequate. See Volume 2, Chapter 7 for procedures on the design of detention storage.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

- Vegetated embankments shall be less than 20 feet in height. Side slopes shall not exceed 3:1 (horizontal to vertical) on one side of the pond to facilitate access for maintenance and repair. The remainder of the pond shall have side slopes no steeper than 2:1 although 3:1 is preferred. Benching of the slope is required for embankments greater than 10 feet in height and having greater than a 3:1 side slope. Riprap-protected embankments shall be no steeper than 2:1. Geotechnical slope stability analysis is recommended for embankments greater than 10 feet in height and is mandatory for embankment slopes steeper than those given above. All embankments must be designed to State of Tennessee guidelines for dam safety.

- The maximum depth of the basin shall not exceed 10 feet.

- Areas above the normal high water elevations of the detention pond shall be sloped toward the basin to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The pond bottom shall be graded toward the outlet to prevent standing water. A low flow or pilot channel across the facility bottom from the inlet to the outlet (often constructed with riprap) is recommended to convey low flows and prevent standing water conditions.

Inlet and Outlet Structures

- Inflow channels shall be stabilized with flared riprap aprons, or the equivalent. A sediment forebay shall be provided for dry detention ponds that are located in a treatment train with off-line water quality treatment structural controls. The sediment forebay shall be sized to contain 0.1 inch per impervious acre (363 ft^3) of contributing drainage and shall be no more than 4 to 6 feet deep.

- The outlet structure shall be sized for Qp_2, Qp_{10}, Qp_{25} and Qp_{100} control (based upon hydrologic routing calculations) and can consist of a weir, orifice, outlet pipe, combination outlet, or other acceptable control structure. Small outlets that will be subject to clogging or are difficult to maintain shall not be permitted. See Volume 2, Chapter 7 for more information on the design of outlet structures. The outlet barrel shall be of reinforced concrete.

- Seepage control or anti-seep collars shall be provided for all outlet pipes.

- Water shall not be discharged from a detention pond in an erosive manner. Riprap, plunge pads or pools, or other energy dissipators shall be placed at the outlet of the barrel to prevent scouring and erosion. If a pond outlet discharges immediately to a channel that carries dry weather flow, care should be taken to minimize disturbance along the downstream channel and streambanks, and to reestablish a forested riparian zone in the shortest possible distance (if the downstream area is located in a water quality buffer).

D. EMERGENCY SPILLWAY

- An emergency spillway shall be included in the stormwater pond design to safely pass Qp_{100}. The spillway prevents pond water levels from overtopping the embankment and causing structural damage. The emergency spillway shall be located so that downstream structures will not be impacted by spillway discharges.
- The emergency spillway shall be located a minimum 0.1 foot above the 100-year water surface elevation.
- A minimum of 1 foot of freeboard shall be provided, measured from the top of the water surface elevation for the extreme flood to the lowest point of the dam embankment, not counting the emergency spillway.

E. MAINTENANCE ACCESS

- A maintenance right-of-way or easement having a minimum width of 20 feet shall be provided to the pond from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles.
- The maintenance access shall extend to the forebay (if included) and outlet works, and, to the extent feasible, be designed to allow vehicles to turn around.

F. SAFETY FEATURES

- A safety bench shall be provided for embankments greater than 10 feet in height and having greater than a 3:1 side slope. For large ponds, the safety bench shall extend no less than 15 feet outward from the normal water edge to the toe of the pond side slope. The slope of the safety bench shall not exceed 6%.
- All embankments and spillways shall be designed to TDEC rules and regulations as applied to the Safe Dams Act of 1973, where applicable.
- The property owner may consider fencing the pond for the purpose of safety management.
- All outlet structures shall be designed so as not to permit access by children. Knox County encourages the posting of warning signs near the pond to prohibit swimming and fishing in the facility.

G. LANDSCAPING

- All areas of the pond shall be stabilized with vegetation to prevent the occurrence of erosion.
- Woody vegetation shall not be planted on the embankment or allowed to grow within 15 feet of the toe of the embankment and 25 feet from the principal spillway structure.
- Water quality buffers, as defined and described in Volume 2, Chapter 6 of this manual, are not required for dry detention ponds. However, it should be noted that vegetated buffers can be utilized for water quality treatment and can result in a volume credit that reduces the WQv. The criteria for the vegetated buffer credit are presented in Volume 2, Chapter 5 of this manual.

4.3.2.3 Design Procedures

In general, site designers should perform the following design procedures when designing a stormwater pond.

Step 1. Compute runoff control volumes.

Calculate Qp_2, Qp_{10}, Qp_{25} and Qp_{100}, in accordance with the guidance presented in Volume 2, Chapter 2.

Step 2. Confirm design criteria and applicability.

Consider any special site-specific design conditions/criteria from subsection 4.3.2.2. Check with

Knox County Engineering, TDEC, or other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply to the site.

Step 3. Determine pond location and preliminary geometry.

This step involves initially designing the grading of the pond (establishing contours) and determining the elevation-storage relationship for the pond. Include consideration of a safety bench, if used or required by the Director.

Step 4. Calculate Qp_2, Qp_{10}, Qp_{25} and Qp_{100} release rates and water surface elevations.

Set up stage-storage-discharge relationships for the control structure for the 2, 10, 25 and 100-year storms.

Step 5. Design embankment(s) and spillway(s).

Size emergency spillway, calculate the 100-year water surface elevation, set the top of the embankment elevation a minimum of 1 foot above the water surface elevation of the 100-year event, and analyze safe passage of the Qp_{100}. Set the emergency spillway elevation a minimum 0.1 foot above the 100-year water surface elevation. At final design, provide safe passage for the 100-year event.

Step 6. Investigate potential pond hazard classification.

The design and construction of stormwater management ponds are required to follow the latest version of the TDEC Rules and Regulations Application to the Safe Dams Act of 1973.

Step 7. Design inlets, outlet structures, maintenance access, and safety features.

See subsection 4.3.2.2 for more details.

Step 8. Design vegetation.

A vegetation scheme for the detention pond should be prepared to indicate how the pond bottom, side slopes and embankments will be stabilized and established with vegetation.

This page left intentionally blank

4.3.2.4 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.2.4 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of the detention pond as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for detention ponds, along with a suggested frequency for each activity. Individual ponds may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the pond in proper operating condition at all times.

Inspection Activities	Suggested Schedule
• After several storm events or an extreme storm event, inspect for: bank stability; signs of erosion; and damage to, or clogging of, the outlet structures and pilot channels.	As Needed
• Inspect for: trash and debris; clogging of the outlet structures and any pilot channels; excessive erosion; sediment accumulation in the basin and inlet/outlet structures; tree growth on dam or embankment; the presence of burrowing animals; standing water where there should be none; vigor and density of the grass turf on the basin side slopes and floor; differential settlement; cracking; leakage; and slope stability.	Semi-annually
• Inspect that the outlet structures, pipes, and downstream and pilot channels are free of debris and are operational. • Note signs of pollution, such as oil sheens, discolored water, or unpleasant odors. • Check for sediment accumulation in the facility. • Check for proper operation of control gates, valves or other mechanical devices.	Annually
Maintenance Activities	**Suggested Schedule**
• Clean and remove debris from inlet and outlet structures. • Mow side slopes (embankment) and maintenance access. Periodic mowing is only required along maintenance rights-of-way and the embankment.	Monthly or as needed
• Repair and revegetate eroded areas. • Remove vegetation that may hinder the operation of the pond. • Repair damage to pond, outlet structures, embankments, control gates, valves, or other mechanical devices; repair undercut or eroded areas.	As Needed
• Monitor sediment accumulations, and remove sediment when the pond volume has become reduced significantly.	As Needed (typically every 20 to 50 years)

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of dry detention ponds. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the dry detention pond. Questions regarding stormwater facility inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
CONVENTIONAL DRY DETENTION POND INSPECTION CHECKLIST

Location: ______________________________ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ______________________________

Date: ___________ Time: ______________ Site conditions:______________________________

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Embankment and Emergency Spillway		
Vegetation coverage adequate?		
Erosion on embankment?		
Animal burrows in embankment?		
Cracking, sliding, bulging of dam?		
Blocked or malfunctioning drains?		
Leaks or seeps on embankment?		
Obstructions of spillway(s)?		
Erosion in/around emergency spillway?		
Other (describe)?		
Inlet/Outlet Structures and Channels		
Clear of debris and functional?		
Trash rack clear of debris and functional?		
Sediment accumulation?		
Condition of concrete/masonry?		
Pipes in good condition?		
Slide gate operational?		
Pond drain valve operational?		
Outfall channels function, not eroding?		
Other (describe)?		
Pond Bottom		
Vegetation adequate?		
Undesirable vegetation growth?		
Excessive sedimentation?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

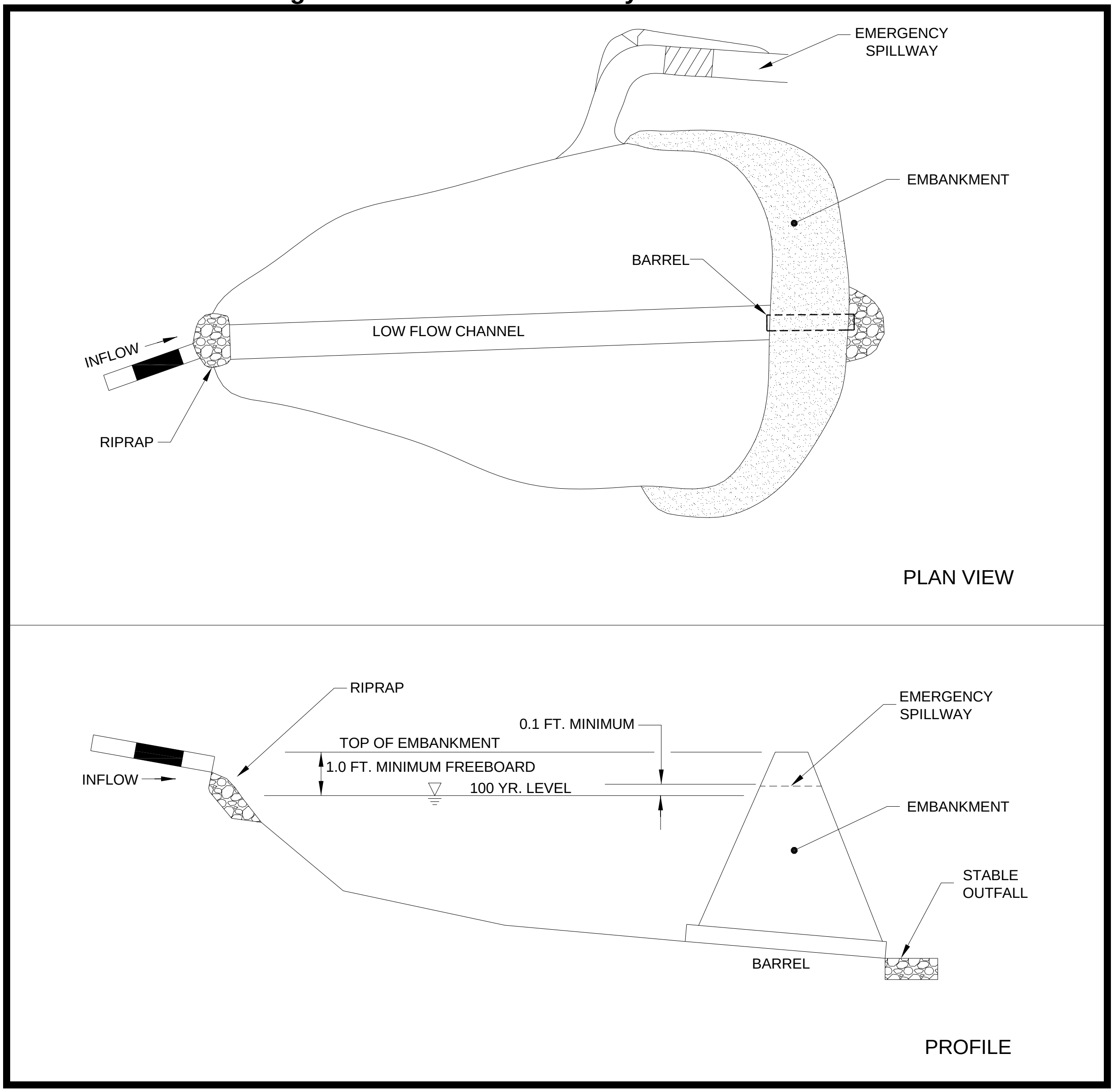
Corrective Action Needed	Due Date

Inspector Signature:______________________________ Inspector Name (printed)______________________________

4.3.2.5 Example Schematic

The example schematics for dry detention ponds presented in Figure 4-17 can be used to assist in the design of such a BMP.

Figure 4-17. Schematic of Dry Detention Basin



4.3.2.6 Design Form

Knox County recommends the use of the following design procedure forms when designing a conventional dry detention pond. Proper use and completion of the form may allow a faster review of the Stormwater Management Plan by Knox County Engineering.

Design Procedure Form: Dry Detention Ponds

PRELIMINARY HYDROLOGIC CALCULATIONS

1 Estimate storage volume required for 2-year storm — 2-year storage = ________ acre-ft
Estimate storage volume required for 10-year storm — 10-year storage = ________ acre-ft
Estimate storage volume required for 25-year storm — 25-year storage = ________ acre-ft
Estimate storage volume required for 100-year storm — 100-year storage = ________ acre-ft

DRY DETENTION POND DESIGN

2. Confirm design criteria and applicability. — ***See subsection 4.3.2.2***

3. Conduct grading design and determine storage available — Prepare an elevation-storage table and curve using the average area method for computing volumes.

Elevation MSL	Area ft^2	Ave. Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3

4. Calculate Qp_2, Qp_{10}, Qp_{25} and Qp_{100} release rates and WSEL — Set up a stage-storage-discharge relationship

Elevation	Storage	Low Flow	Riser			Barrel		Emergency	Total
		WQv-ED	CPv.ED	High Storage		Inlet	Pipe	Spillway	Storage
				Orif.	Weir				
MSL	acre-ft	H(ft) Q(cfs)	H(ft) Q(cfs)	H Q	H Q	H(ft) Q(cfs)	H(ft) Q(cfs)	H(ft) Q(cfs)	acre-ft

Calculate Qp_2 — Qp_2= ________ cfs
Calculate Qp_{10} — Qp_{10}= ________ cfs
Calculate Qp_{25} — Qp_{25}= ________ cfs
Calculate Qp_{100} — Qp_{100}= ________ cfs

Maximum head = — H= ________ ft
Use weir equation for slot length ($Q=CLH^{3/2}$) — L= ________ ft

Check inlet condition — **Use culvert design guidance in Chapter 7**
Check outlet conditions

5. Size emergency spillway using the Qp_{100} and set top of embankment elevation and emergency spillway elevation based on $WSEL_{100}$
   $Q_{ES}=Qp_{100}$ ________ cfs
   $WSEL_{100}$= ________ ft
   $El_{embank}=WSEL_{100+}1.0$ ________ ft
   $El_{ES}=WSEL_{100}+.01$ feet ________ ft

6. Investigate potential pond hazard classification — **See TN Safe Dams Act of 1973**

7. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features — **See Section 4.3.2.2**

8. Design vegetation according to guidance in Chapter 6

9. Verify peak flow control

4.3.2.7 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

4.3.2.8 Suggested Reading

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

City of Austin, TX. *Water Quality Management.* Environmental Criteria Manual, Environmental and Conservation Services. 1988.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities. 2000.

Merritt, F.S., Loftin, M.K., Ricketts, J.T. *Standard Handbook for Civil Engineers*. Fourth Edition McGraw-Hill. 1996.

Metropolitan Washington Council of Governments (MWCOG). *A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone.* March 1992.

This page left intentionally blank

4.3.3 Dry Extended Detention Ponds

General Application Stormwater BMP

Description: A surface storage basin or facility designed to provide water quantity control through detention of stormwater runoff. A dry extended detention pond can be used for water quality treatment purposes as well as for flood control.

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- A sediment forebay or equivalent upstream pretreatment must be provided.
- Minimum flow length to width ratio for the pond is 1.5:1. The pond shall be sized to detain the volume of runoff to be treated for a minimum of 24 hours.
- Side slopes to the pond shall not exceed 3:1 (h:v) on one side of the pond to facilitate access. Slopes as steep as 2:1 will be allowed for other areas, with proper stabilization.

ADVANTAGES / BENEFITS:

- Moderate removal rate of urban pollutants.
- High community acceptance.
- Useful for water quality treatment and flood control.

DISADVANTAGES / LIMITATIONS:

- Potential for thermal impacts/downstream warming.
- Dam height restrictions for high relief areas.
- Pond drainage can be problematic for low relief terrain.

MAINTENANCE REQUIREMENTS:

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

OTHER CONSIDERATIONS:

- Outlet clogging
- Safety bench
- Landscaping

STORMWATER MANAGEMENT SUITABILITY

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

Accepts runoff from SPAP land uses: *Yes*

FEASIBILITY CONSIDERATIONS

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

Residential/Subdivision Use: *Yes*

High Density/Ultra-Urban: *No*

Drainage Area: *unlimited.*

POLLUTANT REMOVAL

M	**Total Suspended Solids**
L	**Nutrients**: Total Phosphorus / Total Nitrogen
L	**Metals**: Cadmium, Copper, Lead, and Zinc
No Data	**Pathogens**: Coliform, Streptococci, E.Coli

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

4.3.3.1 General Description

Dry extended detention (ED) ponds are surface facilities that provide for the temporary storage of stormwater runoff for some minimum time (e.g., 24 to 72 hours) to allow suspended sediments and other associated pollutants to settle to the pond bottom, and therefore, not discharge to downstream channels. Dry ED ponds provide moderate treatment of the water quality volume (WQv), are useful for control of the channel protection volume (CPv), and can provide overbank flood protection (Qp_2, Qp_{10}, and Qp_{25}) and extreme flood protection (Qp_{100}) as well.

4.3.3.2 Pollutant Removal Capabilities

Dry ED ponds are presumed capable of removing at least 60% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the specifications provided in this manual. The TSS removal performance can be reduced by poor design, construction or maintenance.

Additionally, research has shown that use of dry ED ponds will have moderate benefits beyond the removal of TSS, such as the removal of other pollutants (i.e. phosphorous, nitrogen, fecal coliform and heavy metals), as well, which is useful information should the pollutant removal criteria change in the future. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or "treatment train" approach.

- Total Suspended Solids – 60%
- Total Phosphorus – 35%
- Total Nitrogen – 25%
- Pathogens – Insufficient data to provide a pollutant removal value
- Heavy Metals – 25%

For additional information and data on dry ED ponds, see the National Pollutant Removal Performance Database (2nd Edition) available at www.stormwatercenter.net and the International Stormwater Best Management Practices Database at www.bmpdatabase.org.

Because dry ED ponds cannot alone provide adequate treatment of the water quality volume, they must be utilized in a treatment train approach with other structural controls to achieve the goal of 80% removal of total suspended solids (TSS). Volume 2, Chapter 2 provides more information on treatment trains.

4.3.3.3 Planning and Design Standards

The following criteria shall be considered **minimum** design standards for the design of a dry ED pond. Dry ED ponds that are not designed to these standards will not be approved. The Director of Engineering and Public Works (the Director) shall have the authority to require additional design conditions if deemed necessary.

A. LOCATION AND SITING

- It is strongly recommended that dry ED ponds be located where the topography allows for maximum runoff storage at minimum excavation or embankment construction costs. When locating a detention basin, the site designers should also consider the location and use of other land use features, such as planned open spaces and recreational areas, and should attempt to achieve a multi-use objective with the pond where this can be safely achieved.
- Detention ponds shall not be located on unstable slopes or slopes greater than 15%.
- Flood protection controls for control of the Qp_2, Qp_{10}, Qp_{25} and Qp_{100} peak discharges should be designed as final controls for on-site stormwater. Because most dry ED ponds will be used for flood protection and are not capable of achieving Knox County's required 80% TSS removal standard, they

will typically be located downstream of structural stormwater BMPs that are used in conjunction with the dry ED pond to provide 80% treatment of the WQv.

- Dry ED ponds shall not be located in a stream or any other navigable waters of the United States, including natural (i.e., not constructed) wetlands. Where an appeal or variance of this policy is desired, the property owner must obtain coverage under a Section 404 permit under the Clean Water Act and/or an Aquatic Resource Alteration Permit (ARAP) and provide proof of such coverage with the Stormwater Management Plan.
- Each dry ED pond shall be placed in a water quality easement. The water quality easement shall be defined at the outer edge of the safety bench, or a minimum of 15 feet from the normal water pool elevation (measured perpendicular from the pool elevation boundary) if a safety bench is not included in the pond design. The easement limit should be located no closer than as follows unless otherwise specified by the Director:
  - From a public water system well – TDEC specified distance per designated category
  - From a private well – 50 feet; if the well is downgradient from a land use that must obtain a Special Pollution Abatement Permit, then the minimum is 250 feet
  - From a septic system tank/leach field – 50 feet
- The minimum setback for habitable structures from the water quality easement shall be 15 feet. The first floor elevation (FFE) for any structure adjacent to the pond shall have an elevation no lower than 1 foot above the top of the berm.
- All utilities shall be located outside of the dry ED pond.

B. GENERAL DESIGN

- A dry ED pond shall consist of the following elements, designed in accordance with the specifications provided in this section:

  (1) An outlet structure;

  (2) An emergency spillway;

  (3) A sediment forebay;

  (4) Maintenance access;

  (5) Appropriate landscaping.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

- Vegetated embankments shall be less than 20 feet in height. Side slopes shall not exceed 3:1 (horizontal to vertical) on one side of the pond to facilitate access for maintenance and repair. The remainder of the pond shall have side slopes no steeper than 2:1 although 3:1 is preferred. Benching of the slope is required for embankments greater than 10 feet in height and having greater than a 3:1 side slope. Riprap-protected embankments shall be no steeper than 2:1. Geotechnical slope stability analysis is recommended for embankments greater than 10 feet in height and is mandatory for embankment slopes steeper than those given above. All embankments must be designed to State of Tennessee guidelines for dam safety.
- The maximum depth of the basin shall not exceed 10 feet.
- Areas above the normal high water elevations of the detention pond shall be sloped toward the basin to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The pond bottom shall be graded toward the outlet to prevent standing water. A low flow or pilot channel across the facility bottom from the inlet to the outlet (often constructed with riprap) is recommended to convey low flows and prevent standing water conditions.

Inlet and Outlet Structures

- Inflow channels shall be stabilized with flared riprap aprons, or the equivalent.

- The outlet structure shall be sized for Qp_2, Qp_{10}, Qp_{25} and Qp_{100} control (based upon hydrologic routing calculations) and can consist of a weir, orifice, outlet pipe, combination outlet, or other acceptable control structure. Small outlets that will be subject to clogging or are difficult to maintain shall not be permitted. Routing calculations must be used to demonstrate that the storage volume is adequate. See Volume 2, Chapter 7 for more information on the design of outlet works.
- Seepage control or anti-seep collars shall be provided for all outlet pipes.
- Water shall not be discharged from a dry ED pond in an erosive manner. Riprap, plunge pads or pools, or other energy dissipators shall be placed at the outlet of the barrel to prevent scouring and erosion. If a pond outlet discharges immediately to a channel that carries dry weather flow, care should be taken to minimize disturbance along the downstream channel and streambanks, and to reestablish a forested riparian zone in the shortest possible distance (if the downstream area is located in a water quality buffer).
- The outlet barrel shall be of reinforced concrete.

D. PRETREATMENT / INLETS

- A sediment forebay shall be provided for dry ED ponds that are located in a treatment train with other water quality treatment structural controls. The sediment forebay is utilized to remove incoming sediment from the stormwater flow prior to dispersal into the larger pond area. The forebay shall consist of a separate cell, formed by an acceptable barrier. A forebay must be provided at each inlet to the dry ED pond, unless the inlet provides less than 10% of the total design storm inflow to the pond.
- The sediment forebay shall be sized to contain 0.1 inch per impervious acre (363 ft^3) of contributing drainage and shall be no more than 4 to 6 feet deep.
- A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.
- Inflow channels to the forebay shall be stabilized with flared riprap aprons, or the equivalent. Exit velocities of discharges from the forebay to the pond must be non-erosive.

E. OUTLET STRUCTURES

- Flow control from a dry ED pond that is used for control of the WQv, CPv and Qp_2, Qp_{10}, Qp_{25} and Qp_{100} is typically accomplished with the use of a riser and barrel. The riser is a vertical pipe or inlet structure that is located at the base of the pond. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment. The riser shall be located within the pond embankment for maintenance access, safety and aesthetics.
- A number of outlets at varying depths in the riser provide internal flow control for routing of the WQv, CPv, Qp_2, Qp_{10}, Qp_{25} and the Qp_{100}. The number of orifices can vary and is usually a function of the pond design. A dry ED pond riser configuration is typically comprised of an outlet that provides water quality (WQv) and channel protection (CPv) outlet (usually an orifice), an overbank flood protection (Qp_2, Qp_{10}, Qp_{25}) outlet (often a slot or weir), and the extreme flood protection (Qp_{100}) outlet. The channel protection orifice is sized to release the channel protection storage volume for a minimum 24-hour period.
- Extended detention ponds can have a bottom drain pipe with an adjustable slide gate that can completely or partially drain the pond within 24 hours.
- Ponds shall not be drained until at least 24 hours after the completion of a rain event, so that water quality and channel protection objectives can be met.
- Higher flows (Qp_2, Qp_{10}, Qp_{25}, Qp_{100}) pass through openings or slots protected by trash racks further up on the riser.
- After entering the riser, flow is conveyed through the barrel and is discharged downstream. Anti-seep collars shall be installed on the outlet barrel to reduce the potential for pipe or embankment failure.

- Water shall not be discharged from a detention pond in an erosive manner. Riprap, plunge pads or pools, or other energy dissipators shall be placed at the outlet of the barrel to prevent scouring and erosion. If a pond outlet discharges immediately to a channel that carries dry weather flow, care should be taken to minimize disturbance along the downstream channel and streambanks, and to reestablish a forested riparian zone in the shortest possible distance (if the downstream area is located in a water quality buffer).

F. EMERGENCY SPILLWAY

- An emergency spillway shall be included in the stormwater pond design to safely pass Qp_{100}. The spillway prevents pond water levels from overtopping the embankment and causing structural damage. The emergency spillway shall be located so that downstream structures will not be impacted by spillway discharges.
- The emergency spillway shall be located a minimum 0.1 foot above the 100-year water surface elevation.
- A minimum of 1 foot of freeboard shall be provided, measured from the top of the water surface elevation for the extreme flood to the lowest point of the dam embankment, not counting the emergency spillway.

G. MAINTENANCE ACCESS

- A maintenance right-of-way or easement having a minimum width of 20 feet shall be provided to the pond from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles.
- The maintenance access shall extend to the forebay (if included) and outlet structure, and, to the extent feasible, be designed to allow vehicles to turn around.

H. SAFETY FEATURES

- A safety bench shall be provided for embankments greater than 10 feet in height and having greater than a 3:1 side slope. For large ponds, the safety bench shall extend no less than 15 feet outward from the normal water edge to the toe of the pond side slope. The slope of the safety bench shall not exceed 6%.
- All embankments and spillways shall be designed to TDEC rules and regulations as applied to the Safe Dams Act of 1973, where applicable.
- The property owner may consider fencing the pond for the purpose of safety management.
- All outlet structures shall be designed so as not to permit access by children. Knox County encourages the posting of warning signs near the pond to prohibit swimming and fishing in the facility.

I. LANDSCAPING

- All areas of the pond shall be stabilized with appropriate vegetation to prevent the occurrence of erosion.
- Woody vegetation shall not be planted on the embankment or allowed to grow within 15 feet of the toe of the embankment and 25 feet from the principal spillway structure.
- Water quality buffers, as defined and described in Volume 2, Chapter 6 of this manual, are not required for dry ED ponds. However, it should be noted that vegetated buffers can be utilized for water quality treatment and can result in a volume credit that reduces the WQv. The criteria for the vegetated buffer credit are presented in Volume 2, Chapter 5 of this manual.

4.3.3.4 Design Procedures

In general, site designers should perform the following design procedures when designing a dry ED pond.

Step 1. Compute runoff control volumes

Calculate WQv, CPv, Qp_2, Qp_{10}, Qp_{25}, and Qp_{100}, in accordance with the guidance presented in Volume 2.

Step 2. Confirm design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 4.3.3.3. Check with Knox County Engineering, TDEC or other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply to the site.

Step 3. Determine pretreatment volume

A sediment forebay is provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. The forebay should be sized to contain 0.1 inch per impervious acre (363 ft^3) of contributing drainage and should be 4 to 6 feet deep.

Step 4. Determine pond location and preliminary geometry

This step involves initially designing the grading of the pond (establishing contours) and determining the elevation-storage relationship for the pond. Include safety bench, if required or used. See subsection 4.3.3.3 for more details.

Step 5. Compute extended detention orifice release rate(s) and size(s), and establish CPv elevation

The water quality orifice is sized to release the calculated WQv over a minimum of 24 hours. Orifice diameters less than three inches must employ internal orifice protection (i.e., an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). The CPv elevation is then determined from the stage-storage relationship. The invert of the channel protection orifice is located at the water quality extended detention elevation, and the orifice is sized to detain the channel protection storage volume for a 24-hour period, measured from centroid to centroid.

Step 6. Calculate Qp_2, Qp_{10}, Qp_{25} and Qp_{100} release rates and water surface elevations

Set up a stage-storage-discharge relationship for the control structure for the extended detention, the 2, 10, 25 and 100-year storm orifices.

Step 7. Design embankment(s) and spillway(s)

Size emergency spillway, calculate the 100-year water surface elevation, set the top of the embankment elevation, and analyze safe passage of the Qp_{100}. Set the invert elevation of the emergency spillway 0.1 foot above the 100-year water surface elevation.

Step 8. Investigate potential pond hazard classification

The design and construction of stormwater management ponds are required to follow the latest version of the "TDEC Rules and Regulations Applied to the Safe Dams Act of 1973" (Chapter 1200-5-7).

Step 9. Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features

See subsection 4.3.3.3 for more details.

Step 10. Design vegetation

A vegetation scheme for the dry ED pond shall be prepared to indicate how the pond bottom, side slopes and embankment will be stabilized and established with vegetation.

4.3.3.5 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.3.5 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of the dry extended detention (ED) pond as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for dry ED ponds, along with a suggested frequency for each activity. Individual ponds may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e.., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the pond in proper operating condition at all times.

Inspection Activities	Suggested Schedule
• After several storm events or an extreme storm event, inspect for: bank stability; signs of erosion; and damage to, or clogging of, the outlet structures and pilot channels.	As needed
• Inspect for: trash and debris; clogging of the outlet structures and any pilot channels; excessive erosion; sediment accumulation in the basin, forbay and inlet/outlet structures; tree growth on dam or embankment; the presence of burrowing animals; standing water where there should be none; vigor and density of the grass turf on the basin side slopes and floor; differential settlement; cracking; leakage; and slope stability.	Semi-annually
• Inspect that the outlet structures, pipes, and downstream and pilot channels are free of debris and are operational. • Note signs of pollution, such as oil sheens, discolored water, or unpleasant odors. • Check for sediment accumulation in the facility. • Check for proper operation of control gates, valves or other mechanical devices.	Annually
Maintenance Activities	**Suggested Schedule**
• Clean and remove debris from inlet and outlet structures. • Mow side slopes (embankment) and maintenance access. Periodic mowing is only required along maintenance rights-of-way and the embankment.	Monthly or as needed
• Repair and revegetate eroded areas. • Remove vegetation that may hinder the operation of the pond. • Repair damage to pond, outlet structures, embankments, control gates, valves, or other mechanical devices; repair undercut or eroded areas.	As Needed
• Monitor sediment accumulations, and remove sediment when the pond volume has become reduced significantly.	As Needed (typically every 15 to 50 years)

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of dry extended detention ponds. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the dry detention pond. Questions regarding stormwater facility inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
DRY EXTENDED DETENTION POND INSPECTION CHECKLIST

Location: ______________________________ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ______________________________

Date: ___________ Time: ______________ Site conditions:______________________________

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Embankment and Emergency Spillway		
Vegetation coverage adequate?		
Erosion on embankment?		
Animal burrows in embankment?		
Cracking, sliding, bulging of dam?		
Blocked or malfunctioning drains?		
Leaks or seeps on embankment?		
Obstructions of spillway(s)?		
Erosion in/around emergency spillway?		
Other (describe)?		
Inlet/Outlet Structures and Channels		
Clear of debris and functional?		
Trash rack clear of debris and functional?		
Sediment accumulation?		
Condition of concrete/masonry?		
Pipes in good condition?		
Slide gate operational?		
Pond drain valve operational?		
Outfall channels function, not eroding?		
Other (describe)?		
Pond Bottom		
Vegetation adequate?		
Undesirable vegetation growth?		
Excessive sedimentation?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

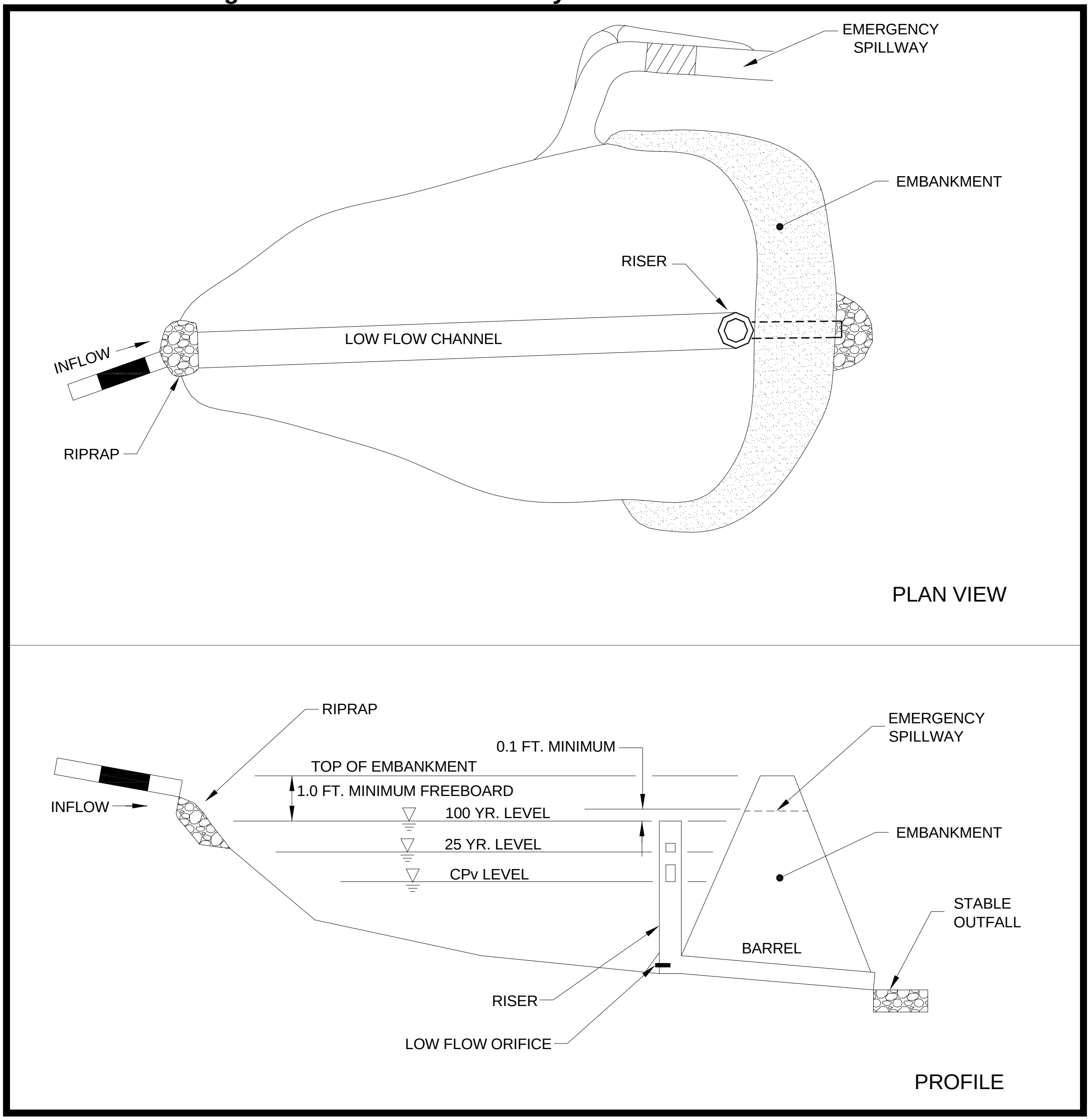
Corrective Action Needed	Due Date

Inspector Signature:______________________________ Inspector Name (printed)______________________________

4.3.3.6 Example Schematics

The example schematic for a dry extended detention basin presented in Figure 4-18 can be used to assist in the design of such BMPs.

Figure 4-18. Schematic of Dry Extended Detention Basin



4.3.3.7 Design Procedures Form

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQv volume requirements
Compute Runoff Coefficient, Rv — Rv = ________
Compute WQv — WQv = ________ acre-ft

1b. Estimate CPv — CPv = ________ acre-ft

1c. Estimate storage volumes
Estimate storage volume required for 2-year storm — 2-year storage = ________ acre-ft
Estimate storage volume required for 10-year storm — 10-year storage = ________ acre-ft
Estimate storage volume required for 25-year storm — 25-year storage = ________ acre-ft
Estimate storage volume required for 100-year storm — 100-year storage = ________ acre-ft

DRY EXTENDED DETENTION PONDS DESIGN

2. Is the use of a dry extended detention pond appropriate? — ***See subsections 4.3.3.1***

3. Confirm design criteria and applicability. — ***See subsection 4.3.3.3***

4. Pretreatment Volume (Forebay)
V_{pre} =(I)(.1")(1'/12") — V_{pre} = ________ acre-ft

5. Conduct grading design and determine storage available — Prepare an elevation-storage table and curve using the average area method for computing volumes.

Elevation MSL	Area ft^2	Ave. Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3

6. WQv and CPv Orifice Computations
   Average ED release rate (if applicable) — release rate= ____________ cfs
   Average head, h= (ED elev.- Permanent Pool elev.) /2 — head= ____________ ft
   Area of orifice from orifice equation — Area= ____________ ft^2
   $Q=CA(2gh)^{0.5}$ — diameter ____________ inches
   (C varies with orifice condition. Refer to Chapter 3, Section 3.3.2.3 for guidance)

   Establish CPv top elevation using stage-storage curve — CPv WSEL= ____________ ft-NGVD
   Estimate orifice size — CPv orifice diameter = ____________ inches
   Perform hydrograph routing to check detention time — centroid-centroid det. = ____________ hours
   Iterate to final orifice size — Final CPv orifice diameter = ____________ inches

7. Calculate Qp_2, Qp_{10}, Qp_{25} and Qp_{100} release rates and WSEL — Set up a stage-storage-discharge relationship

Elevation	Storage	Low Flow	Riser			Barrel		Emergency	Total
		WQv-ED	CPv-ED	High Storage		Inlet	Pipe	Spillway	Storage
				Orif.	Weir				
MSL	acre-ft	H(ft) Q(cfs)	H(ft) Q(cfs)	H Q	H Q	H(ft) Q(cfs)	H(ft) Q(cfs)	H(ft) Q(cfs)	acre-ft

   Qp_2=pre-dev. Peak discharge - (WQv-ED release + CPv-ED release) — Qp_2= ____________ cfs
   Qp_{10}=pre-dev. Peak discharge - (WQv-ED release + CPv-ED release) — Qp_{10}= ____________ cfs
   Qp_{25}=pre-dev. Peak discharge - (WQv-ED release + CPv-ED release) — Qp_{25}= ____________ cfs
   Qp_{100}=pre-dev. Peak discharge - (WQv-ED release + CPv-ED release) — Qp_{100}= ____________ cfs

   Maximum head = — H= ____________ ft
   Use weir equation for slot length ($Q=CLH^{3/2}$) — L= ____________ ft

   Check inlet condition
   Check outlet conditions — **Use culvert design guidance in Chapter 7**

8. Size emergency spillway using the Qp_{100} and set top of embankment elevation and emergency spillway elevation based on $WSEL_{100}$
   $Q_{ES}=Qp_{100}$ ____________ cfs
   $WSEL_{100}$= ____________ ft
   $El_{embank}=WSEL_{100}+1.0$ ____________ ft
   $El_{ES}=WSEL_{100}+.01$ feet ____________ ft

9. Investigate potential pond hazard classification — **See TN Safe Dams Act of 1973**

10. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features — **See Section 4.3.3.3**

11. Design vegetation according to guidance in Chapter 6

12. Verify peak flow control, water quality drawdown time and channel protection detention time

4.3.3.8 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001*.*

Center for Watershed Protection. *Manual Builder.* Stormwater Manager's Resource Center, Accessed July 2005. www.stormwatercenter.net

4.3.3.9 Suggested Reading

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

City of Austin, TX. *Water Quality Management.* Environmental Criteria Manual, Environmental and Conservation Services, 1988.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

Maryland Department of the Environment. *Maryland Stormwater Design Manual, Volumes I and II.* Prepared by Center for Watershed Protection (CWP), 2000.

Metropolitan Washington Council of Governments (MWCOG). *A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone.* March, 1992.

United States Environmental Protection Agency. *Methodology for Analysis of Detention Basins for Control of Urban Runoff Quality.* 1986.

Urban Drainage and Flood Control District. *Urban Storm Drainage Criteria Manual – Volume 3 – Best Management Practices – Stormwater Quality.* Denver, Colorado, September 1992.

Walker, W. *Phosphorus Removal by Urban Runoff Detention Basins.* Lake and Reservoir Management, North American Society for Lake Management, 314, 1987.

4.3.4 Stormwater Wetlands

General Application Stormwater BMP

Description: A constructed wetland system used for stormwater management. Runoff volume is both stored and treated in the wetland facility.

KEY DESIGN CONSIDERATIONS

DESIGN GUIDELINES:

- Minimum contributing drainage area of 25 acres; 5 acres for a pocket wetland.
- Minimum dry weather flow path of 2:1 (length:width) should be provided from inflow to outflow.
- Minimum of 35% of total surface area should have a depth of 6 inches or less; 10 to 20% of surface area should be deep pool (1.5- to 6-foot depth).

ADVANTAGES / BENEFITS:

- Good nutrient removal.
- Provides natural wildlife habitat.
- Relatively low maintenance costs.

DISADVANTAGES / LIMITATIONS:

- Requires large land area.
- Needs continuous baseflow for viable wetland.
- Regular sediment removal is critical.

MAINTENANCE REQUIREMENTS:

- Replace wetland vegetation to maintain at least 50% surface area coverage.
- Remove invasive vegetation.
- Monitor and remove sediment accumulation.

STORMWATER MANAGEMENT SUITABILITY

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

Provides pretreatment for SPAP land uses? *No*

FEASIBILITY CONSIDERATIONS

M-H	**Land Requirement**
M	**Capital Cost**
	Maintenance Burden:
M	Shallow Wetland
M	ED Shallow Wetland
H	Pocket Wetland
M	Pond/Wetland

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

POLLUTANT REMOVAL

H	**Total Suspended Solids**
M	**Nutrients** - Total Phosphorus / Total Nitrogen
M	**Metals** - Cadmium, Copper, Lead, and Zinc
M	**Pathogens** - Coliform, Streptococci, E.Coli

OTHER CONSIDERATIONS

Residential Subdivision Use: *Yes*

High-Density/Ultra-Urban: *Yes*

Drainage Area: *25 acres min. (5 to 10 acres for Pocket Wetlands)*

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

4.3.4.1 General Description

Stormwater wetlands (also referred to as *constructed wetlands*) are constructed shallow marsh systems that are designed to both treat urban stormwater and control runoff volumes. As stormwater runoff flows through the wetland facility, pollutant removal is achieved through settling and uptake by marsh vegetation. Wetlands can be utilized effectively for pollutant removal and also offer aesthetic value and wildlife habitat.

Constructed stormwater wetlands differ from natural wetland systems in that they are engineered facilities designed specifically for the purpose of treating stormwater runoff and typically have less biodiversity than natural wetlands both in terms of plant and animal life. However, as with natural wetlands, stormwater wetlands require a continuous base flow or a high water table to support aquatic vegetation.

There are several design variations of the stormwater wetland, each design differing in the relative amounts of shallow and deep water, and dry storage above the wetland. The variations are shown in Figure 4-19. These include the shallow wetland, the extended detention shallow wetland, pond/wetland system and pocket wetland.

Figure 4-19. Stormwater Wetland Examples

Shallow Wetland

Extended Detention Shallow Wetland

Pocket wetland

Newly Constructed Shallow Wetland

Below are descriptions of each stormwater wetland design variant:

- **Shallow Wetland** – In the shallow wetland design, most of the water quality treatment volume is in the relatively shallow high marsh or low marsh depths. The only deep portions of the shallow wetland design are the forebay at the inlet to the wetland, and the micropool at the outlet. One disadvantage of this design is that, since the pool is very shallow, a relatively large amount of land is typically needed to store the water quality volume.
- **Extended Detention (ED) Shallow Wetland** – The extended detention (ED) shallow wetland design is the same as the shallow wetland; however, part of the water quality treatment volume is provided as extended detention above the surface of the marsh and released over a period of 24 hours. This design can treat a greater volume of stormwater in a smaller space than the shallow wetland design. In the extended detention shallow wetland option, plants that can tolerate both wet and dry periods need to be specified in the ED zone.

- **Pond/Wetland System** – The pond/wetland system has two separate cells: a wet pond and a shallow marsh. The wet pond traps sediments and reduces runoff velocities prior to entry into the wetland, where stormwater flows receive additional treatment. Less land is required for a pond/wetland system than for the shallow wetland or the ED shallow wetland systems.
- **Pocket Wetland** – A pocket wetland is intended for smaller drainage areas of 5 to 10 acres and typically requires excavation down to the water table for a reliable water source to support the wetland system.

Certain types of wetlands, such as *submerged gravel wetland systems* are not recommended for general application use to meet stormwater management goals due to limited performance data. They may be applicable in special or retrofit situations where there are severe limitations on what can be implemented. Please see a further discussion of submerged gravel wetlands in Section 4.4.3.

4.3.4.2 Stormwater Management Suitability

Similar to stormwater ponds, stormwater wetlands are designed to control both stormwater quantity and quality. Thus, a stormwater wetland can be used to address the minimum design standards for water quality, channel protection and flood protection for a given drainage area.

<u>Water Quality Volume (WQv)</u>
Pollutants are removed from stormwater runoff in a wetland through uptake by wetland vegetation and algae, vegetative filtering, and through gravitational settling in the slow moving marsh flow. Other pollutant removal mechanisms are also at work in a stormwater wetland, including chemical and biological decomposition, and volatilization. Section 4.3.4.3 provides median pollutant removal efficiencies that can be used for planning and design purposes.

<u>Channel Protection Volume (CPv)</u>
The storage volume above the permanent pool/water surface level in a stormwater wetland is used to provide control of the channel protection volume (CPv). This is accomplished by releasing the 1-year, 24-hour storm runoff volume over 24 hours (extended detention). It is best to do this with minimum vertical water level fluctuation, as extreme fluctuation may stress vegetation.

<u>Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})</u>
A stormwater wetland can also provide storage above the permanent pool/water surface level to reduce the post-development peak flow of the 2, 10, 25 and 100-year storms to pre-development levels (detention). If a wetland facility is not used for flood protection, it should be designed as an off-line system to pass higher flows around rather than through the wetland system, and/or designed to safely pass discharges from extreme storm events.

4.3.4.3 Pollutant Removal Capabilities

All of the stormwater wetland design variants are presumed to be able to remove 75% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the specifications provided in this manual. Since stormwater wetlands cannot achieve the 80% TSS standard by itself, additional stormwater quality BMPs will be required in a treatment train with stormwater wetlands. Undersized or poorly designed wetland facilities can reduce TSS removal performance.

Additionally, research has shown that use of stormwater ponds or wetlands will have benefits beyond the removal of TSS, such as the removal of other pollutants (i.e. phosphorous, nitrogen, fecal coliform and heavy metals), as well, which is useful information should the pollutant removal criteria change in the future. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional stormwater BMPs may be put in place at the given site in a series or “treatment train” approach.

- Total Suspended Solids – 75%
- Total Phosphorus – 45%
- Total Nitrogen – 30%
- Pathogens – 70% (if no resident waterfowl population present)
- Heavy Metals – 50%

For additional information and data on pollutant removal capabilities for stormwater wetlands, see the National Pollutant Removal Performance Database (2nd Edition) available at www.stormwatercenter.net and the International Stormwater Best Management Practices Database at www.bmpdatabase.org.

4.3.4.4 Application and Site Feasibility Criteria

Stormwater wetlands are generally applicable to most types of new development and redevelopment, and can be utilized in both residential and nonresidential areas. However, due to the large land requirements, wetlands may not be practical in higher density areas. The following criteria should be evaluated to ensure the suitability of a stormwater wetland for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage
- Suitable for High Density/Ultra Urban Areas, however, land requirements may preclude use
- Suitable for Regional Stormwater Control

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – A minimum of 25 acres and a positive water balance is needed to maintain wetland conditions; a minimum of 5 acres for pocket wetland. The Knox County Director of Engineering and Public Works (the Director) may approve a smaller drainage area with an adequate water balance and anti-clogging device.
- Space Required – Approximately 3 to 5% of the tributary drainage area
- Site Slope – Wetlands are feasible on sites where the upstream slope (above the wetland) is no more than 15%.
- Minimum Head – Enough elevation drop is required, from inlet to outlet, to allow hydraulic conveyance by gravity. Generally, the minimum head for a pocket wetland is 2 to 3 feet. For all other wetlands the minimum head is 3 to 5 feet.
- Minimum Depth to Water Table – In general, no minimum separation distance to the water table is required for stormwater wetlands. In fact, water table interception may be helpful to sustain a permanent pool. However, some source water protection requirements may dictate a separation distance if there is a sensitive underlying aquifer. In such situations, an impermeable liner, or a minimum separation between 2 to 4 feet is required for portions of the wetland that will have standing water.
- Soils – Permeable soils are not well suited for a constructed stormwater wetland without a high water table. Underlying soils of hydrologic group “C” or “D” should be adequate to maintain wetland conditions. Most group “A” soils and some group “B” soils will require a liner. Evaluation of soils should be based upon an actual subsurface analysis and permeability tests.

4.3.4.5 Planning and Design Standards

The following standards shall be considered **minimum** design standards for the design of a stormwater wetland facility. Stormwater wetlands that are not designed to these standards will not be approved. The Director shall have the authority to require additional design conditions if deemed necessary.

A. LOCATION AND SITING

- Stormwater wetlands should normally have a minimum contributing drainage area of 25 acres or more. For a pocket wetland, the minimum drainage area is 5 acres. The Director may consider allowing the use of a stormwater wetland for a smaller drainage area when water availability can be confirmed (such as from a groundwater source or areas that typically have a high water table). It is important that wetlands that serve smaller drainage areas have an adequate anti-clogging device provided for the wetland outlet.
- A continuous base flow or high water table is required to support wetland vegetation. A water balance shall be performed to demonstrate that a stormwater wetland can withstand a 30-day drought at summer evaporation rates without completely drawing down (see Chapter 3 for details).
- When determining an appropriate location for a stormwater wetland, the site designer should also take into account the location and use of other site features such as natural depressions, buffers, and undisturbed natural areas. The site designer should attempt to aesthetically “fit” the wetland into the landscape.
- Stormwater wetlands shall not be located in a stream or any other navigable waters of the United States, including natural (i.e., not constructed) wetlands. Where an appeal or variance of this policy is desired, the property owner must obtain coverage under a Section 404 permit under the Clean Water Act and/or an Aquatic Resource Alteration Permit (ARAP) and provide proof of such coverage with the Stormwater Management Plan. The Director may approve the conversion of an existing degraded wetland into a stormwater wetland where appropriate for local watershed restoration efforts, and when prior approval for such a conversion is obtained from all applicable State and Federal agencies.
- If a wetland facility is not used for overbank and extreme flood protection, it shall be designed as an off-line system to bypass the higher flows rather than passing them through the wetland system.
- Each wetland or wetland system shall be placed in a water quality easement that is recorded with the deed. The water quality easement shall be defined at the outer edge of the safety bench, or a minimum of 15 feet from the normal water pool elevation (measured perpendicular from the pool elevation boundary) if a safety bench is not included in the wetland design. Minimum setback requirements for the easement shall be as follows unless otherwise specified by the Director:
  - from a property line – 10 feet;
  - From a public water system well – TDEC specified distance per designated category;
  - From a private well – 100 feet; if well is downgradient from a land use that requires a Special Pollution Abatement Permit, then the minimum setback is 250 feet;
  - From a septic system tank/leach field – 50 feet.
- All utilities should be located outside of the wetland site.

B. GENERAL DESIGN

- A stormwater wetland shall consist of the following elements, designed in accordance with the specifications provided in this section.
  - Shallow marsh areas of varying depths with wetland vegetation;
  - Permanent micropool;
  - Overlying zone in which runoff control volumes are stored if the wetland will be used for storage of the CPv, Qp_2, Qp_{10}, Qp_{25} and Qp_{100}.
  - Emergency spillway;
  - Maintenance access;
  - Safety bench;
  - Sediment forebay at each wetland inlet (unless the inlet provides less than 10% of the total inflow to the wetland);

- Wetland buffer (this is not the same as a regulatory water quality buffer – see section I-Landscaping for more information); and
- Appropriate wetland vegetation and native landscaping.
- Pond/wetland systems that also include stormwater pond facilities must meet all of the design parameters in Section 4.3.1 for pond design.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

In general, wetland designs are unique for each site and application. However, there are number of geometric ratios and limiting depths for the design of a stormwater wetland that shall be observed for adequate pollutant removal, ease of maintenance, and improved safety. Table 4-8 provides the recommended physical specifications and geometry for the various stormwater wetland design variants.

Table 4-8. Recommended Design Criteria for Stormwater Wetlands
(Source: Modified from Massachusetts DEP, 1997; Schueler, 1992)

Design Criteria	Shallow Wetland	ED Shallow Wetland	Pond/ Wetland	Pocket Wetland
Length to Width Ratio (minimum)	2:1	2:1	2:1	2:1
Extended Detention (ED)	No	Yes	Optional	Optional
Allocation of WQv Volume (pool/marsh/ED) in %	25/75/0	25/25/50	70/30/0 (includes pond volume)	25/75/0
Allocation of Surface Area (deepwater/low marsh/high marsh/semi-wet)[1] in %	20/35/40/5	10/35/45/10	45/25/25/5 (includes pond surface area)	10/45/40/5
Forebay	Required	Required	Required	See section D below
Micropool	Required	Required	Required	Required
Outlet Configuration	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Hooded broad-crested weir

1 – Depth Considerations:
Deepwater: 1.5 to 6 feet below normal pool elevation
Low marsh: 6 to 18 inches below normal pool elevation
High marsh: 6 inches or less below normal pool elevation
Semi-wet zone: Above normal pool elevation

- The stormwater wetland shall be designed with the recommended proportion of “depth zones.” Each of the four wetland design variants has depth zone allocations which are given as a percentage of the stormwater wetland surface area. Target allocations are found in Table 4-8. The four basic depth zones are:

Deepwater zone
From 1.5 to 6 feet deep. Includes the outlet micropool and deepwater channels through the wetland facility. This zone supports little emergent wetland vegetation, but may support submerged or floating vegetation.

Low marsh zone
From 6 to 18 inches below the normal permanent pool or water surface elevation. This zone is suitable for the growth of several emergent wetland plant species.

High marsh zone

From 6 inches below the pool to the normal pool elevation. This zone will support a greater density and diversity of wetland species than the low marsh zone. The high marsh zone should have a higher surface area to volume ratio than the low marsh zone.

Semi-wet zone
Those areas above the permanent pool that are inundated during larger storm events. This zone supports a number of species that can survive flooding.

- A dry weather flow path shall be provided from inflow to outlet across the stormwater wetland. The path shall have a minimum length to width ratio of 2:1. Ideally, the path length to width ratio should be greater than 3:1. This path may be achieved by constructing internal dikes or berms, using marsh plantings, and/or by using multiple cells. Finger dikes are commonly used in surface flow systems to create serpentine configurations and prevent short-circuiting. Microtopography (contours along the bottom of a wetland or marsh that provide a variety of conditions for different species needs and increases the surface area to volume ratio) is encouraged to enhance wetland diversity.
- A micropool having a depth no greater than 4 to 6 feet shall be included in the design at the outlet to prevent the outlet from clogging and resuspension of sediments, and to mitigate thermal effects.
- Maximum depth of any permanent pool areas shall not exceed 6 feet.
- The volume that is handled through extended detention shall not comprise more than 50% of the total WQv, and its maximum water surface elevation shall not extend more than 3 feet above the normal pool. Storage of CPv, Qp_2, Qp_{10}, Qp_{25} and Qp_{100} can be provided above the maximum WQv elevation within the wetland.
- The perimeter of all deep pool areas (4 feet or greater in depth) shall be surrounded by safety and aquatic benches similar to those for stormwater ponds (see subsection 4.3.1).
- The contours of the wetland shall be irregular to provide a more natural landscaping effect.

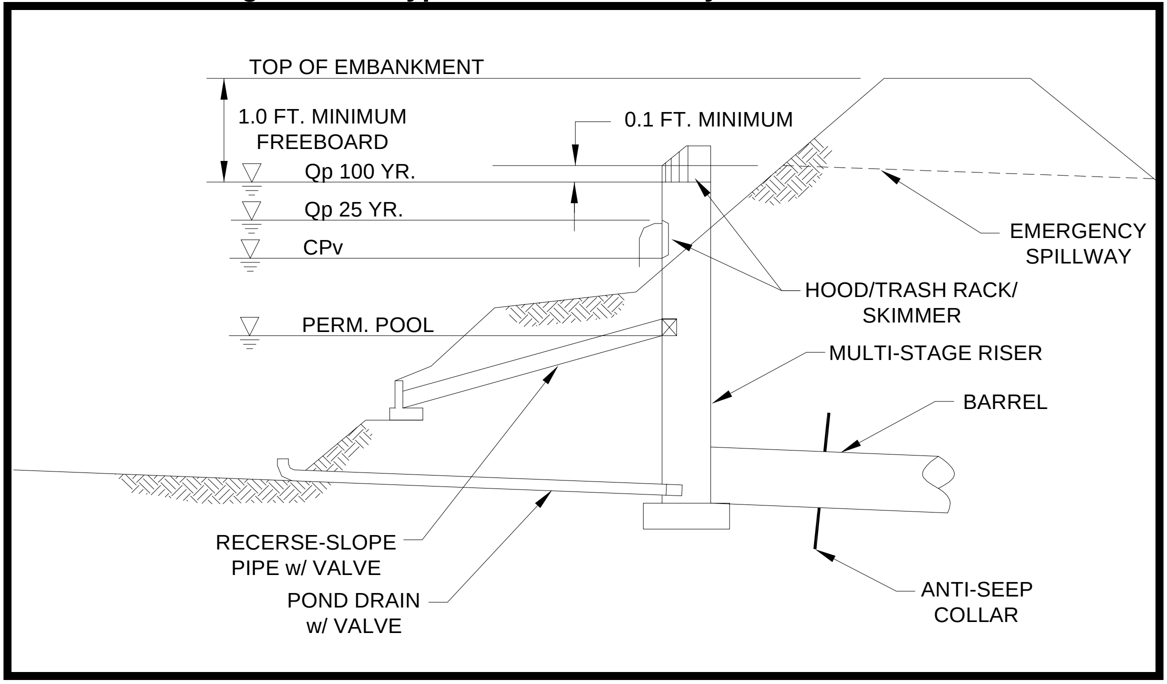
D. PRETREATMENT / INLETS

- Sediment regulation and removal is critical to sustain stormwater wetlands. A wetland facility shall have a sediment forebay or equivalent upstream pretreatment. In some cases, a pocket wetland design may not allow construction of a sediment forebay because of space limitations on small sites. In this case, a smaller "cattail" forebay is recommended to capture trash, debris and oil.
- A sediment forebay is designed to remove incoming sediment from the stormwater flow prior to dispersal into the wetland. The forebay shall consist of a separate cell, formed by an acceptable barrier. A forebay shall be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the wetland facility.
- The forebay shall be sized to contain 0.1 inches per impervious acre (363 ft^3) of contributing drainage and shall be no more than 4 to 6 feet deep. The pretreatment storage volume is part of the total WQv design requirement and may be subtracted from the WQv for wetland storage sizing.
- A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.
- Inflow channels shall be stabilized with flared riprap aprons, or the equivalent. Inlet pipes to the pond can be partially submerged. Exit velocities from the forebay to the wetland shall be nonerosive.

E. OUTLET STRUCTURES

Flow control from a stormwater wetland is typically accomplished with the use of a concrete or corrugated metal riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the base of the micropool with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (see Figure 4-20).

Figure 4-20. Typical Wetland Facility Outlet Structure



- The riser shall be located within the embankment for maintenance access, safety and aesthetics. The outlet barrel shall be of reinforced concrete.
- A number of outlets at varying depths in the riser provide internal flow control for routing of WQv, CPv, Qp_2, Qp_{10}, Qp_{25} and Qp_{100}. The number of orifices can vary and is usually a function of the wetland design.

  For shallow and pocket wetlands, the riser configuration is typically comprised of a channel protection outlet (usually an orifice) and overbank flood protection outlet (often a slot or weir). The channel protection orifice is sized to release the channel protection storage volume over a 24-hour period. Since the water quality volume is fully contained in the permanent pool, no orifice sizing is necessary for this volume. As runoff from a water quality event enters the wet pond, it simply displaces that same volume through the channel protection orifice. Thus an off-line shallow or pocket wetland providing only water quality treatment can use a simple overflow weir as the outlet structure.

  In the case of an extended detention (ED) shallow wetland; there is generally a need for an additional outlet (usually an orifice) that is sized to pass the extended detention water quality volume that is surcharged on top of the permanent pool. Flow will first pass through this orifice, which is sized to release the water quality ED volume in 24 hours. The preferred design is a reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond. The next outlet is sized for the release of the channel protection storage volume. The outlet (often an orifice) invert is located at the maximum elevation associated with the extended detention water quality volume and is sized to release the channel protection storage volume over a 24-hour period.

  Alternative hydraulic control methods to an orifice can be used and include the use of a broad-crested rectangular, V-notch, proportional weir, or an outlet pipe protected by a hood that extends at least 12 inches below the normal pool.

- Wetlands shall have a bottom drain pipe with an adjustable slide gate within the micropool that can completely or partially drain the wetland within 24 hours. The wetland drain shall be sized one pipe size greater than the calculated design diameter.

- Wetlands shall not be drained unless necessary for rescue or maintenance.
- Higher flows (Qp_2, Qp_{10}, Qp_{25} and Qp_{100}) pass through openings or slots protected by trash racks that are located further up on the riser.
- After entering the riser, flow is conveyed through the barrel and is discharged downstream. Anti-seep collars shall be installed on the outlet barrel to reduce the potential for pipe failure.
- Riprap, plunge pools or pads, or other energy dissipators shall be placed at the outlet of the barrel to prevent scouring and erosion. If a wetland facility discharges to a stream that has dry weather flow at any time during the year, care should be taken to minimize land disturbance along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. See Chapter 7 (Stormwater Drainage Design) and Chapter 6 (Water Quality Buffers) for more guidance on outlet designs and rules and regulations for disturbances in a water quality buffer.

See the design procedures in subsection 4.3.4.6 as well as Chapter 3 for additional information and specifications on pond routing and outlet operations.

F. EMERGENCY SPILLWAY

- An emergency spillway shall be included in the stormwater wetland design to safely pass the extreme flood (Qp_{100}) flow. The spillway prevents the wetland's water levels from overtopping the embankment and causing structural damage. The emergency spillway shall be located so that downstream structures will not be impacted by spillway discharges.
- A minimum of 1 foot of freeboard must be provided, measured from the top of the water surface elevation for the extreme flood (Qp_{100}) to the lowest point of the dam embankment, not counting the emergency spillway.

G. MAINTENANCE ACCESS

- A minimum 20' wide maintenance right of way or easement shall be provided from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles.
- The maintenance access shall extend to the forebay, safety bench, riser, and outlet, and, to the extent feasible, be designed to allow vehicles to turn around.
- Access to the riser shall be provided by lockable manhole covers, and manhole steps within easy reach of slide gates and other controls.

H. SAFETY FEATURES

- All embankments and spillways shall be designed to the requirements set by TDEC's Safe Dams Act of 1973.
- Fencing of wetlands is not generally desirable, but may be required by the Director. A preferred method is to manage the contours of deep pool areas through the inclusion of a safety bench (see above) to eliminate dropoffs and reduce the potential for accidental drowning. In addition, the safety bench may be landscaped to deter access to the pool.
- All outlet structures shall be designed so as not to permit access by children.

I. LANDSCAPING

- A landscaping plan shall be developed that indicates the methods used to establish and maintain wetland coverage. Minimum considerations of the plan include: delineation of landscaping zones, selection of corresponding plant species, planting plan, sequence for preparing wetland bed (including soil amendments, if needed) and sources of plant material. More information on wetland plants can be found at the following websites:

- http://wetlands.fws.gov/
- http://www.npwrc.usgs.gov/resource/plants/floraso/species.htm
- http://www.tva.gov/river/landandshore/stabilization/plantsearch.htm

- Landscaping zones include low marsh, high marsh, and semi-wet zones. The low marsh zone ranges from 6 to 18 inches below the normal pool. This zone is suitable for the growth of several emergent plant species. The high marsh zone ranges from 6 inches below the pool up to the normal pool. This zone will support greater density and diversity of emergent wetland plant species. The high marsh zone should have a higher surface area to volume ratio than the low marsh zone. The semi-wet zone refers to those areas above the permanent pool that are inundated on an infrequent basis and can be expected to support wetland plants.
- The landscaping plan should provide elements that promote greater wildlife and waterfowl use within the wetland and buffers.
- Woody vegetation shall not be planted on the embankment or allowed to grow within 15 feet of the toe of the embankment and 25 feet from the principal spillway structure.
- Water quality buffers, as defined and described in Volume 2, Chapter 6 of this manual, are not required for wetlands that are constructed for the purpose of stormwater quality or quantity control. However, it should be noted that vegetated buffers can be utilized for water quality treatment and can result in a volume credit that reduces the WQv. The criteria for the vegetated buffer credit are presented in Volume 2, Chapter 5 of this manual.
- Existing trees should be preserved in the wetland area during construction, or should be replanted in another location if they are intolerant of having a saturated root zone. It is desirable to locate forest conservation areas adjacent to wetlands. To discourage resident waterfowl populations, the wetland buffer can be planted with trees, shrubs and native ground covers.
- The soils in planting areas within and surrounding a wetland are often severely compacted during the construction process to ensure stability. The density of these compacted soils is so great that it effectively prevents root penetration and therefore may lead to premature mortality or loss of vigor. Consequently, it is advisable to excavate large and deep holes around the proposed planting sites and backfill these with uncompacted topsoil.
- Native species of fish can be stocked in the permanent pool to aid in mosquito prevention. Knox County strongly discourages the use non-native fish species in a stormwater facility due to the possibility that the fish will enter downstream receiving waters.
- A fountain or aerator may be used for oxygenation of water in the permanent pool and to aid in mosquito breeding prevention.

J. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

There are a number of additional site specific design criteria and issues (listed below) that must be considered in the design of wetlands.

Physiographic Factors - Local terrain design constraints

- Low Relief – Providing wetland drain can be problematic.
- Karst – Requires poly or clay liner to sustain a permanent pool of water and protect aquifers; limits on ponding depth; geotechnical tests may be required. Stormwater wetlands are the preferred BMP over ponds in the karst areas.

Soils

- Hydrologic group “A” soils and some group “B” soils may require liner (not relevant for pocket wetland).

Special Watershed Considerations

- Wellhead Protection – The potential for groundwater contamination (in required wellhead protection areas) shall be reduced through pretreatment of runoff, and installation of a liner for type “A” and “B” soils; pretreat hotspots; 2 to 4 foot separation distance from water table.

4.3.4.6 Design Procedures

Step 1. Compute runoff control volumes

Calculate WQv, CPv, Qp_2, Qp_{10}, Qp_{25} and Qp_{100}, in accordance with the guidance presented in Volume 2, Chapter 2.

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

Consider the Application and Site Feasibility Criteria in subsections 4.3.4.4 and 4.3.4.5-A (Location and Siting).

Step 3. Confirm design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 4.3.4.5-J (Additional Site-Specific Design Criteria and Issues).

Check with Knox County, TDEC, TDOT or other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4. Determine pretreatment volume

A sediment forebay is provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond. The forebay shall be sized to contain 0.1 inches per impervious acre (363 ft^3) of contributing drainage and shall be 4 to 6 feet deep. The forebay storage volume counts toward the total WQv requirement and may be subtracted from the WQv for subsequent calculations.

Step 5. Allocate the WQv volume among marsh, micropool, and ED volumes

Use recommended criteria from Table 4-8.

Step 6. Determine wetland location and preliminary geometry, including distribution of wetland depth zones

This step involves initially laying out the wetland design and determining the distribution of wetland surface area among the various depth zones (high marsh, low marsh, and deepwater). Set WQv permanent pool elevation (and WQv-ED elevation for ED shallow wetland) based on volumes calculated earlier.

Step 7. Compute extended detention orifice release rate(s) and size(s), and establish CPv elevation

Shallow Wetland and Pocket Wetland: The CPv elevation is determined from the stage-storage relationship and the orifice is then sized to detain the channel protection storage volume for a 24-hour period, measured from centroid to centroid. The channel protection orifice should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 foot below the elevation of the permanent pool, is a recommended design. The orifice diameters less than three inches must have internal orifice protection (i.e., an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable slide gates can also be used to achieve this equivalent diameter.

ED Shallow Wetland: Based on the elevations established in Step 6 for the extended detention portion of the water quality volume, the water quality orifice is sized to release this extended detention volume in 24 hours. The water quality orifice be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged one foot below the elevation of the permanent pool, is a recommended design. Orifice diameters less than three inches must employ internal orifice protection (i.e., an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wirecloth and a stone filtering jacket). Adjustable slide gates can also be used to achieve this equivalent diameter. The CPv elevation is then determined from the stage-storage relationship. The invert of the channel protection orifice is located at the water quality extended detention elevation, and the orifice is sized to detain the channel protection storage volume for a 24-hour period, measured from centroid to centroid.

Step 8. Calculate Qp2, Qp10, Qp25 and Qp100 release rates and water surface elevations

Set up a stage-storage-discharge relationship for the control structure for the extended detention orifice(s) and the 25-year storm.

Step 9. Design embankment(s) and spillway(s)

Calculate the 100-year water surface elevation, set the top of the embankment elevation, and size the emergency spillway, ensuring safe passage of the Qp_{100}. Set the invert elevation of the emergency spillway 0.1 foot above the 100-year water surface elevation.

Step 10. Investigate potential pond/wetland hazard classification

The design and construction of stormwater management facilities are required to follow the latest version of the TDEC Rules and Regulations Application to the Safe Dams Act of 1973.

Step 11. Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features

See subsection 4.3.4.5-D through H for more details.

Step 12. Design landscape plan

A landscape plan for a stormwater wetland and its buffer shall be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation. See subsection 4.3.4.5-I (Landscaping) for more details.

4.3.4.7 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.4.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of stormwater wetlands as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary. This page provides guidance on maintenance activities that are typically required for stormwater wetlands, along with a suggested frequency for each activity. Individual wetlands may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the pond in proper operating condition at all times.

Inspection Activities	Suggested Schedule
• After several storm events or an extreme storm event, inspect for: bank stability; signs of erosion; vegetation growth; drainage system function; and structural damage.	As needed
• Inspect for: invasive vegetation; trash and debris; clogging of the inlet/outlet structures and any pilot or low flow channels; excessive erosion; sediment accumulation in the basin, forebay and inlet/outlet structures; tree growth on dam or embankment; the presence of burrowing animals; standing water where there should be none; vigor and density of the grass turf on the basin side slopes and floor; differential settlement; cracking; leakage; and slope stability.	Semi-annually
• Inspect the inlet/outlet structures, pipes, sediment forebays, and upstream, downstream, and pilot channels to ensure they are free of debris and are operational. • Check for signs of unhealthy or overpopulation of plants and/or fish (if utilized). • Note signs of pollution, such as oil sheens, discolored water, or unpleasant odors. • Check sediment marker(s) for sediment accumulation in the facility and forebay. • Check for proper operation of control gates, valves or other mechanical devices. • Note changes to the wetland or contributing drainage area as such changes may affect wetland performance.	Annually
Maintenance Activities	**Suggested Schedule**
• Replace wetland vegetation to maintain at least 50% surface area coverage in wetland plants after the second growing season.	One-time
• Clean and remove debris from inlet and outlet structures. Mow side slopes (embankment) and maintenance access. Periodic mowing is only required along maintenance rights-of-way and the embankment. The wetland buffer surrounding the wetland can be managed as a meadow (mowing every other year) or forest.	Frequently (3 to 4 times per year)
• Supplement wetland plants if a significant portion have not established (at least 50% of the surface area). Remove unhealthy, invasive or nuisance plant species and replant with appropriate species if necessary. Harvest plant species if vegetation becomes too thick causing flow backup and flooding, or an overabundance of undesirable wildlife.	Annually (if needed)
• Repair damage to pond, outlet structures, embankments, control gates, valves, or other mechanical devices; repair undercut or eroded areas. Remove litter, debris, pollutants as appropriate.	As needed
• Remove sediment from the forebay. Sediments excavated from stormwater wetlands that receive treated runoff from land uses that require a Special Pollution Abatement Permit (SPAP) are not considered toxic or hazardous material and can be safely disposed of by either land application or landfilling.	As needed (typically every 5 to 7 years)
• Monitor sediment accumulations, and remove sediment when the volume in the wetland, forebay, or micropool has become reduced significantly or the wetland area is not providing a healthy habitat for vegetation and fish (if used). Discharges of turbid or untreated stormwater from the wetland may be considered an illegal discharge, as per the Knox County Stormwater Management Ordinance. Care should be exercised during wetland drawdowns to prevent downstream discharge of sediments, anoxic water, or high flows with erosive velocities. Knox County should be notified before draining a stormwater wetland.	As needed (typically every 20 to 50 years)

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of stormwater wetlands. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the wetland. Questions regarding stormwater facility inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
STORMWATER WETLAND INSPECTION CHECKLIST

Location: ______________________________ Owner Change since last inspection? Y N
Owner Name, Address, Phone: ______________________________
Date: ___________ Time: ______________ Site conditions: ______________________________

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Embankment and Emergency Spillway		
Healthy vegetation?		
Erosion on embankment?		
Animal burrows in embankment?		
Cracking, sliding, bulging of dam?		
Blocked or malfunctioning drains?		
Leaks or seeps on embankment?		
Obstructions of spillway(s)?		
Erosion in/around emergency spillway?		
Other (describe)?		
Inlet/Outlet Structures and Channels		
Clear of debris and functional?		
Trash rack clear of debris and functional?		
Sediment accumulation?		
Condition of concrete/masonry?		
Pipes in good condition?		
Slide gate operation?		
Drain valve operation?		
Outfall channels function, not eroding?		
Other (describe)?		
Sediment Forebays		
Evidence of sediment accumulation?		
Permanent Pool Areas (if applicable)		
Undesirable vegetation growth?		
Visible pollution?		
Shoreline erosion?		
Erosion at outfalls into wetland?		
Headwalls and endwalls in good condition?		
Encroachment by other activities?		
Evidence of sediment accumulation?		
Wetland Vegetation Areas		
Vegetation adequate?		
Undesirable vegetation growth?		
Excessive sedimentation?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature: ______________________________ Inspector Name (printed) ______________________________

4.3.4.8 Example Schematics

Figure 4-21. Schematic of Shallow Wetland

(Source: Adapted from Atlanta Regional Council, 2000)

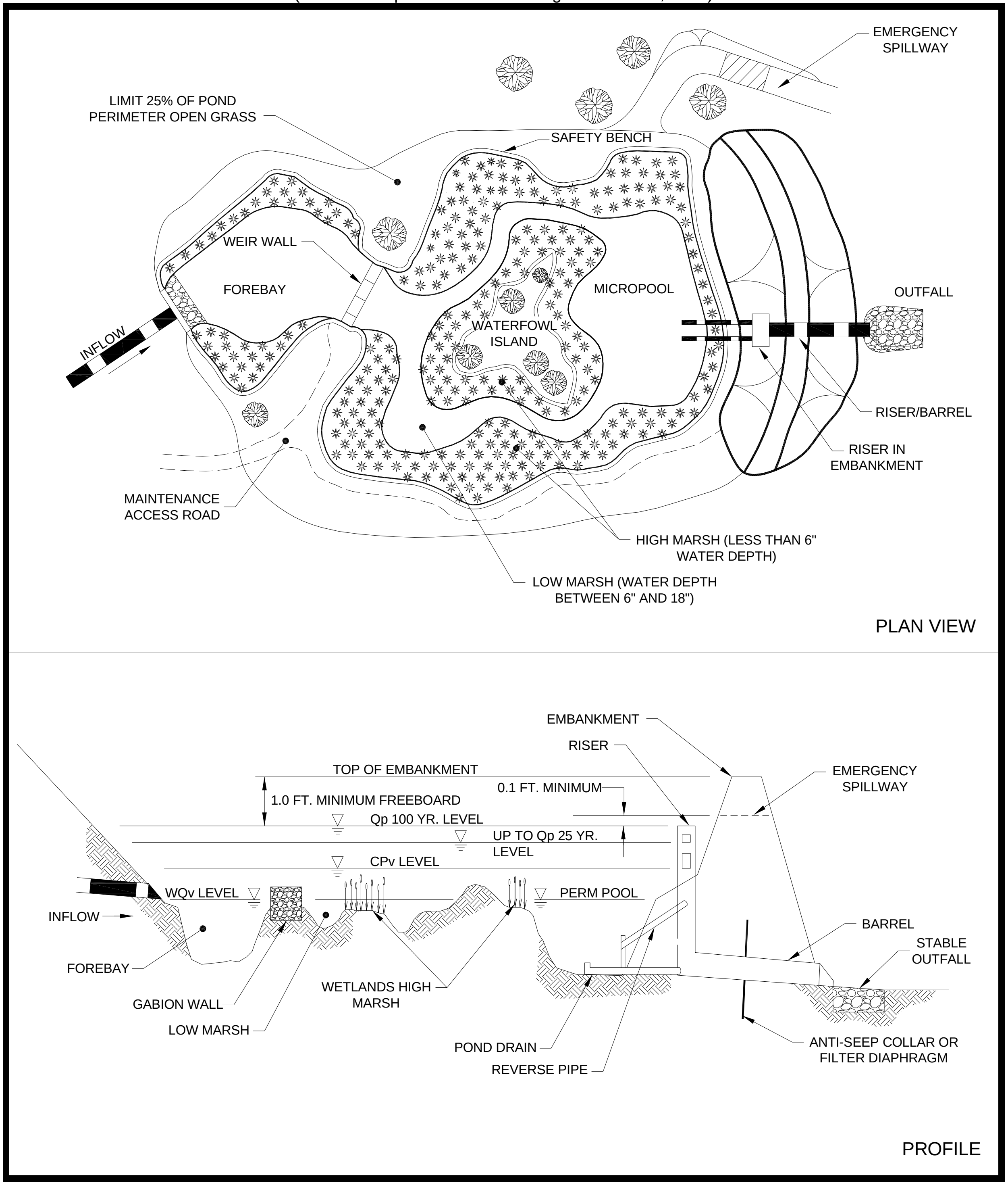


Figure 4-22. Schematic of Extended Detention Shallow Wetland

(Source: Adapted from Atlanta Regional Council, 2000)

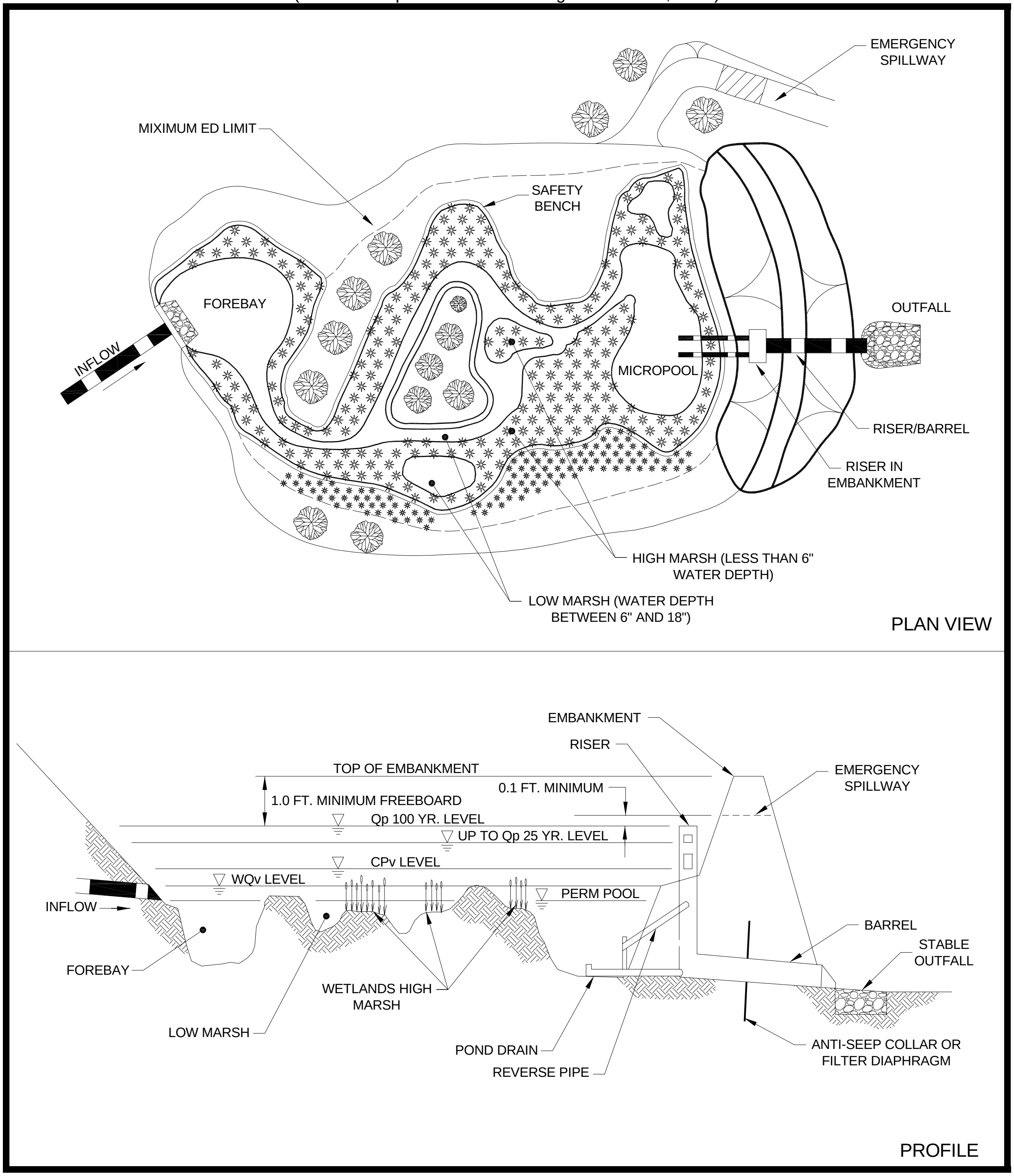


Figure 4-23. Schematic of Pond/Wetland System

(Source: Adapted from Atlanta Regional Council, 2000)

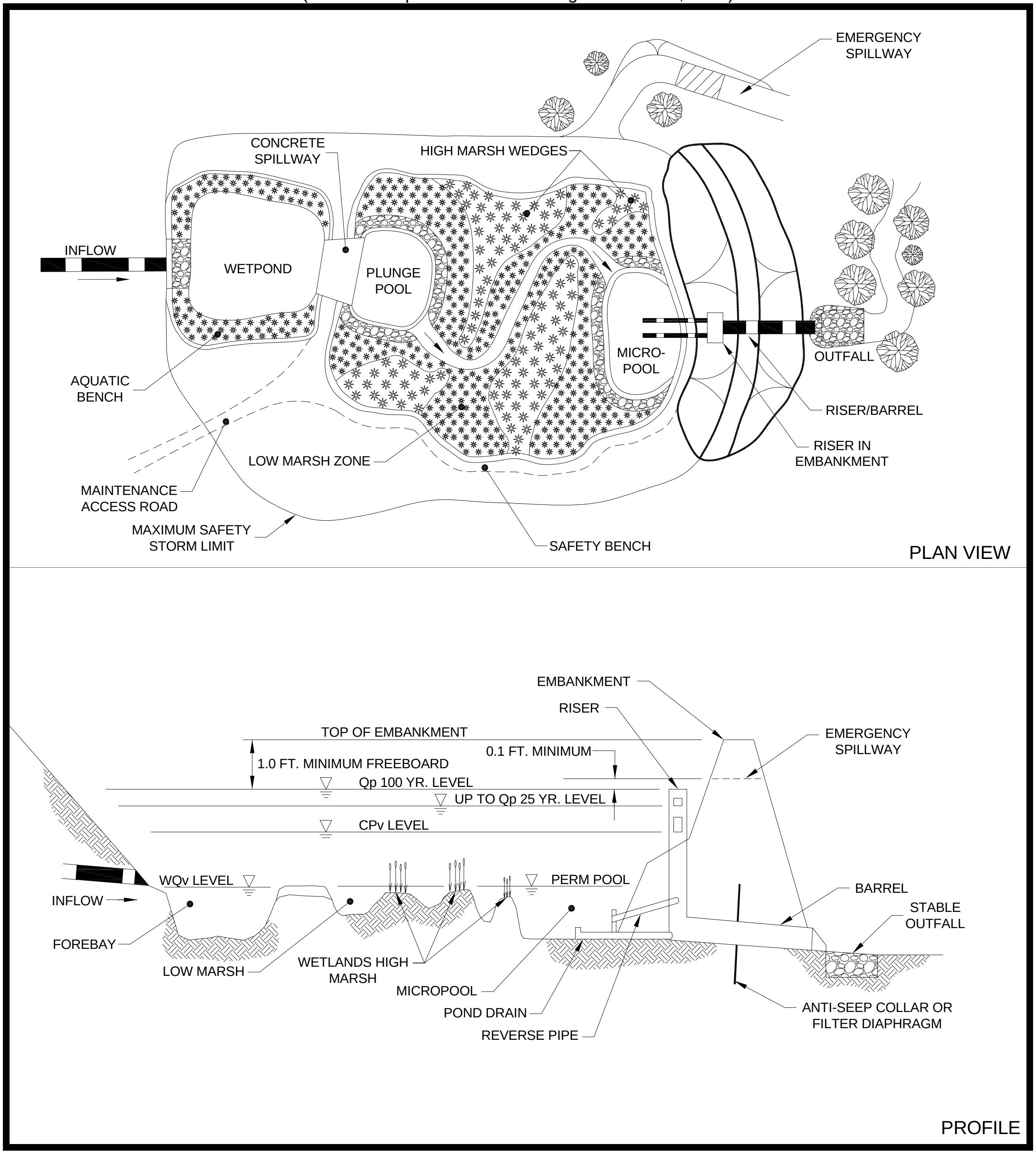
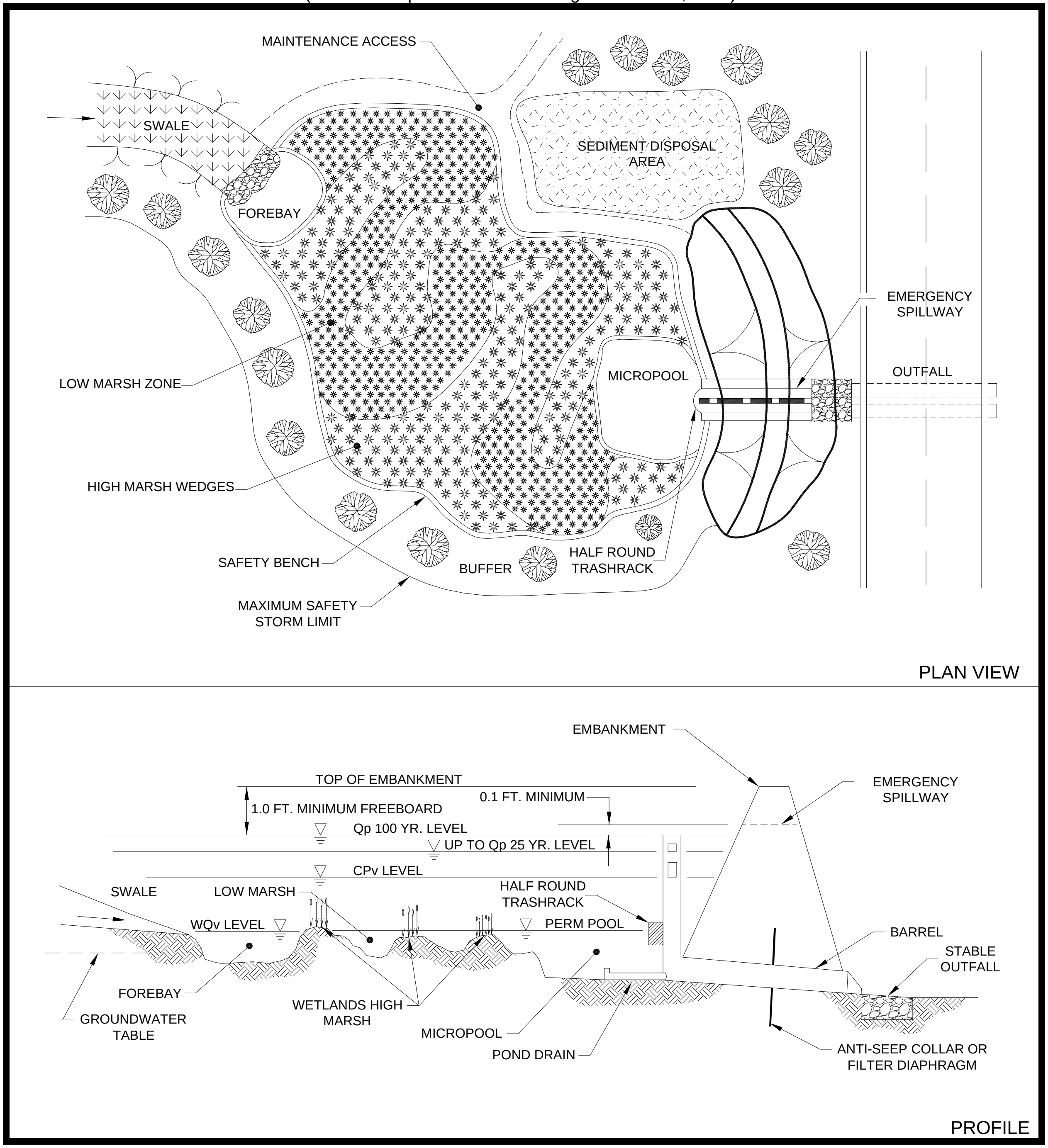


Figure 4-24. Schematic of Pocket Wetland

(Source: Adapted from Atlanta Regional Council, 2000)



4.3.4.9 Design Forms

Knox County recommends the use of the following design procedure forms when designing a stormwater wetland. Proper use and completion of the form may allow a faster review of the Stormwater Management Plan by Knox County Engineering.

Design Procedure Form: Stormwater Wetlands

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute Runoff Coefficient, Rv — Rv = ________
Compute WQv — WQv = ________ acre-ft

1b. Estimate CPv — CPv = ________ acre-ft

1c. Estimate storage volumes
Compute storage volume required for 2-year storm — 2-year storage = ________ acre-ft
Compute storage volume required for 10-year storm — 10-year storage = ________ acre-ft
Compute storage volume required for 25-year storm — 25-year storage = ________ acre-ft
Compute storage volume requried for 100-year storm — 100-year storage = ________ acre-ft

STORMWATER WETLAND DESIGN

2. Is the use of a stormwater wetland appropriate? — ***See subsections 4.3.4.4 and 4.3.4.5 - A***

3. Confirm design criteria and applicability. — ***See subsection 4.3.4.5 - J***

4. Pretreatment Volume (Forebay)
V_{pre} =(I)(.1")(1'/12") — Vol_{pre} = ________ acre-ft

5. Allocation of pool, Marsh, and ED Volumes

Shallow Wetland
Vol_{pool}= 0.2(WQv-Vol_{pre}) — Vol_{pool}= ________ acre-ft
Vol_{marsh}= 0.7(WQv-Vol_{pre}) — Vol_{marsh}= ________ acre-ft

Shallow ED Wetland
Vol_{pool}= 0.1(WQv-Vol_{pre}) — Vol_{pool}= ________ acre-ft
Vol_{marsh}= 0.3(WQv-Vol_{pre}) — Vol_{marsh}= ________ acre-ft
Vol_{ED}= 0.5(WQv-Vol_{pre}) — Vol_{ED}= ________ acre-ft

Pocket Wetland
Vol_{pool}= 0.1(WQv-Vol_{pre}) — Vol_{pool}= ________ acre-ft
Vol_{marsh}= 0.8(WQv-Vol_{pre}) — Vol_{marsh}= ________ acre-ft

6. Allocation of Surface Area

Pool/Deepwater Wetland Zone (1.5-6 feet deep) — $Area_{water}$= ________ acres, %= ________
Low Marsh Wetland Zone (6-18 inches deep) — $Area_{low}$= ________ acres, %= ________
High Marsh Wetland Zone (0-6 inches deep) — $Area_{high}$= ________ acres, %= ________
Semi-Wet Wetland Zone (above pool depth) — $Area_{semi}$= ________ acres, %= ________
100.00%

Conduct grading and determine storage available for marsh zones (and ED if applicable), and compute orifice size.

Prepare an elevation-storage table and curve using the average area method for computing volumes.

Elevation MSL	Area ft^2	Ave. Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3	Cumulative Volume acre-ft	Volume above Permanent Pool acre-ft

Design Procedure Form: Stormwater Wetlands (continued)

7. WQv Orifice Computations
   Average ED release rate (if applicable) — release rate= ______ cfs
   Average head, h = (ED elev. - Permanent pool elev.) / 2 — head= ______ ft
   Area of orifice from orifice equation — Area= ______ ft^2
   $Q=CA(2gh)^{0.5}$ C varies with orifice condition — diameter ______ inches

   Establish CPv top elevation using stage-storage curve — CPv WSEL= ______ ft-NGVD
   Estimate orifice size — CPv orifice diameter = ______ inches
   Perform hydrograph routing to check detention time — centroid-centroid det. = ______ hours
   Iterate to final orifice size — Final CPv orifice diameter = ______ inches

8. Calculate Qp_2, Qp_{10}, Qp_{25} and Qp_{100} release rates and WSELs — Set up a stage-storage-discharge relationship

Elevation	Storage	Low Flow	Riser			Barrel		Emergency	Total
		WQv-ED	CPv-ED	High Storage		Inlet	Pipe	Spillway	Outflow
				Orif.	Weir				
MSL	acre-ft	H(ft) Q(cfs)	H(ft) Q(cfs)	H Q	H Q	H(ft) Q(cfs)	H(ft) Q(cfs)	H(ft) Q(cfs)	Q (cfs)

Qp_2=pre-dev. Peak discharge - (WQv-ED release + CPv-ED release) — Qp_2= ______ cfs
Qp_{10}=pre-dev. Peak discharge - (WQv-ED release + CPv-ED release) — Qp_{10}= ______ cfs
Qp_{25} = pre-dev. Peak discharge - (WQ_v-ED release + CPv-ED release) — Qp_{25}= ______ cfs

Maximum head = — H= ______ ft
Use weir equation for slot length ($Q=CLH^{3/2}$) — L= ______ ft

Check inlet condition
Check outlet condition — **Use culvert design guidance in Chapter 7**

9. Size emergency spillway using the Qp_{100} and set top of embankment elevation and emergency spillway elevation based on $WSEL_{100}$
   $Q_{ES}=Qp_{100}$ ______ cfs
   $WSEL_{100}$= ______ ft
   El_{embank} ______ ft
   El_{ES}= ______ ft

10. Investigate potential pond hazard classification — **See TN Safe Dams Act of 1973**

11. Design inlets, sediment forebays, outlet structures, maintenance access, and safety features — **See subsection 4.3.4.5 - D through H**

12. Design vegetation according to guidance in Chapter 6

13. Verify peak flow control, water quality drawdown time and channel protection detention time

4.3.4.10 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2000.

Center for Watershed Protection. *Manual Builder.* Stormwater Manager's Resource Center, Accessed July 2005. *www.stormwatercenter.net*

Massachusetts Department of Environmental Protection and the Massachusetts Office of Coastal Zone Management. *Stormwater Management -- Volume One: Stormwater Policy Handbook, and Volume Two: Stormwater Technical Handbook*, 1997.

Schueler, T.J. *Design of Stormwater Wetland Systems*, Metrolpolitan Washington Council of Governments, Washington, D.C., April, 1992.

4.3.4.11 Suggested Reading

Adams, L., L.E. Dove, D.L. Leedy, and T. Franklin. *Urban Wetlands for Stormwater Control and Wildlife Enhancement – Analysis and Evaluation.* Urban Wildlife Research Center, Columbia, Maryland, 1983.

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

City of Austin, TX. *Water Quality Management.* Environmental Criteria Manual, Environmental and Conservation Services, 1988.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

Claytor, R.A., and T.R. Schueler. *Design of Stormwater Filtering Systems.* The Center for Watershed Protection, Silver Spring, MD, 1996.

US EPA. *Storm Water Technology Fact Sheet: Storm Water Wetlands.* EPA 832-F-99-025, Office of Water, 1999.

Faulkner, S. and C. Richardson. *Physical and Chemical Characteristics of Freshwater Wetland Soils.* Constructed Wetlands for Wastewater Treatment, ed. D. Hammer, Lewis Publishers, 831 pp, 1991.

Guntenspergen, G.R., F. Stearns, and J. A. Kadlec. *Wetland Vegetation.* Constructed Wetlands for Wastewater Treatment, ed. D. A. Hammer, Lewis Publishers, 1991.

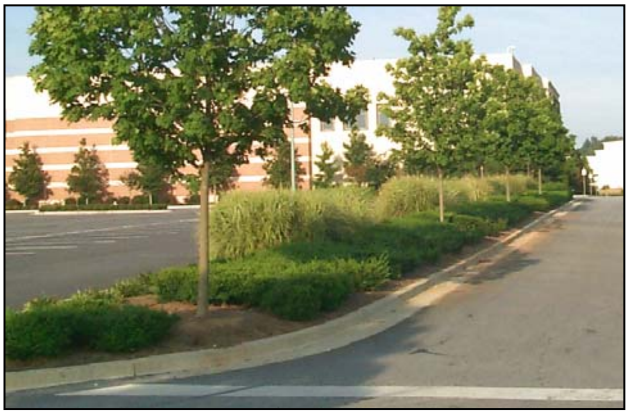
Maryland Department of the Environment. *Maryland Stormwater Design Manual, Volumes I and II.* Prepared by Center for Watershed Protection (CWP), 2000.

Metropolitan Washington Council of Governments (MWCOG). *A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone.* March, 1992.

This page left intentionally blank

4.3.5 Bioretention Areas

General Application Stormwater BMP



Description: Shallow stormwater basin or landscaped area that utilizes engineered soils and vegetation to capture and treat runoff.

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Maximum contributing drainage area of 5 acres.
- Often located in "landscaping islands."
- Treatment area consists of grass filter, sand bed, ponding area, organic/mulch layer, planting soil, and vegetation.
- Typically requires 5 feet of elevation difference from inflow to outflow.

ADVANTAGES / BENEFITS:

- Applicable to small drainage areas.
- Good for highly impervious areas, particularly parking lots.
- Good retrofit capability.
- Relatively low maintenance requirements.
- Can be planned as an aesthetic feature.

DISADVANTAGES / LIMITATIONS:

- Requires extensive landscaping.
- Not recommended for areas with steep slopes.

MAINTENANCE REQUIREMENTS:

- Inspect and repair/replace treatment area components.

STORMWATER MANAGEMENT SUITABILITY

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

Provides pretreatment for SPAP land uses? *Yes*
✷ in certain situations

FEASIBILITY CONSIDERATIONS

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

Residential/Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *5 acres max.*

Soils: *Planting soils must meet specified criteria; no restriction on surrounding soils.*

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

POLLUTANT REMOVAL

H	**Total Suspended Solids**
M	**Nutrients** - Total Phosphorus / Total Nitrogen
M	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E.Coli

OTHER CONSIDERATIONS:

- Use of native plants is recommended

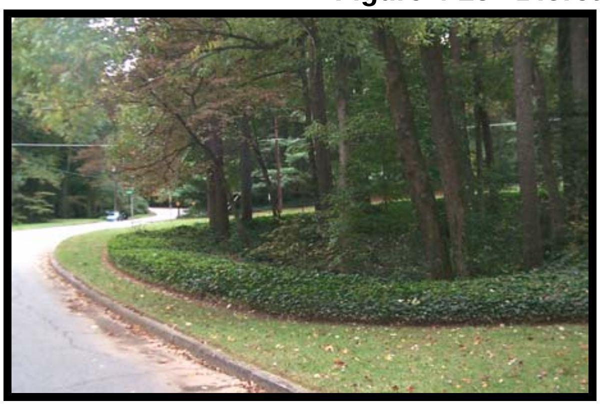
4.3.5.1 General Description

Bioretention areas (also referred to as *bioretention filters* or *rain gardens*) are structural stormwater controls that capture and temporarily store the water quality volume (WQv) using soils and vegetation in shallow basins or landscaped areas to remove pollutants from stormwater runoff.

Bioretention areas are engineered facilities in which runoff is conveyed as sheet flow to the "treatment area," which consists of a grass buffer strip, ponding area, organic or mulch layer, planting soil, and vegetation. An optional sand bed can also be included in the design to provide aeration and drainage of the planting soil. The filtered runoff is typically collected and returned to the conveyance system, though it can also permeate into the surrounding soil in areas with porous soils.

There are numerous design applications, both on- and off-line, for bioretention areas. These include use on single-family residential lots (*rain gardens*), as off-line facilities adjacent to parking lots, along road drainage swales, within larger landscaped pervious areas, and as landscaped islands in impervious or high-density environments. Figures 4-25 and 4-26 illustrate a number of examples of bioretention facilities in both photographs and drawings.

Figure 4-25. Bioretention Area Examples



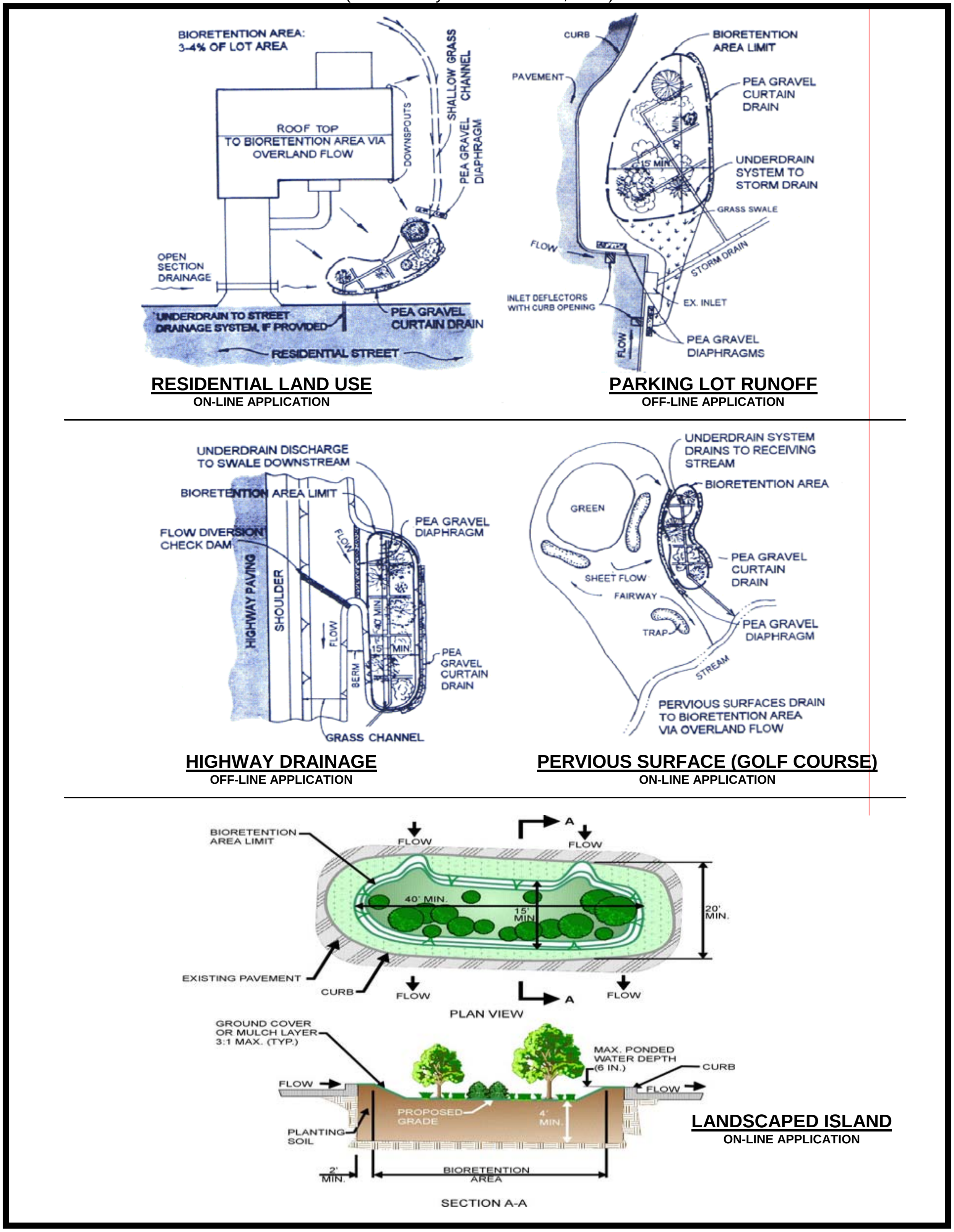
Single-Family Residential "Rain Garden"

Landscaped Island

Newly Constructed Bioretention Area

Newly Planted Bioretention Area After Storm Event

Figure 4-26. Bioretention Area Applications
(Source: Claytor and Schueler, 1996)



4.3.5.2 Stormwater Management Suitability

Bioretention areas are designed primarily for stormwater quality and can provide limited runoff quantity control, primarily for smaller storm events. These facilities may sometimes be used to partially or completely meet channel protection volume (CPv) requirements on smaller sites. However, bioretention areas will typically need to be used in conjunction with other structural BMPs to provide channel protection as well as overbank flood volume (Qp_2, Qp_{10}, and Qp_{25}) protection. It is important to ensure that a bioretention area safely bypasses higher flows.

Water Quality (WQv)
Bioretention is an excellent stormwater treatment practice due to the variety of pollutant removal mechanisms. Each of the components of the bioretention area is designed to perform a specific function (see Figure 4-27). The *grass filter strip (or grass channel)* reduces incoming runoff velocity and filters particulates from the runoff. The *ponding area* provides for temporary storage of stormwater runoff prior to its evaporation, infiltration, or uptake and provides additional settling capacity. The *organic or mulch layer* provides filtration as well as an environment conducive to the growth of microorganisms that degrade hydrocarbons and organic material. The *planting soil* in the bioretention facility acts as a filtration system, and clay in the soil provides adsorption sites for heavy metals, nutrients and other pollutants. Both *woody and herbaceous plants* in the ponding area provide vegetative uptake of runoff and pollutants and also serve to stabilize the surrounding soils. Finally, an *underdrain system* provides for positive drainage and aerobic conditions in the planting soil.

Section 4.3.5.3 provides median pollutant removal efficiencies that can be used for planning and design purposes.

Channel Protection (CPv)
For smaller sites, a bioretention area may be designed to capture the entire channel protection volume (CPv) in either an off or on-line configuration. Given that a bioretention facility must be designed to completely drain over 48 hours, the requirement of extended detention of the 1-year, 24-hour storm runoff volume will be met. For larger sites or where only the WQv is diverted to the bioretention facility, another structural BMP must be used to provide CPv extended detention.

Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})
Another structural BMP must be used in conjunction with a bioretention area to reduce the post-development peak flow of the 2, 10, 25, and 100-year storm events to pre-development levels (detention). Bioretention areas must provide flow diversion and/or be designed to safely pass larger storm flows and protect the ponding area, mulch layer and vegetation. *The volume of runoff removed and treated in the bioretention area may be considered in the calculations for overbank and extreme flood protection.*

4.3.5.3 Pollutant Removal Capabilities

Bioretention areas are presumed to be able to remove 85% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed bioretention areas can reduce TSS removal performance.

Additionally, research has shown that use of bioretention areas will have benefits beyond the removal of TSS, such as the removal of other pollutants (i.e. phosphorous, nitrogen, fecal coliform and heavy metals), as well, which is useful information should the pollutant removal criteria change in the future. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional BMPs may be put in place at the given site in a series or “treatment train” approach.

- Total Suspended Solids – 85%
- Total Phosphorus – 60%
- Total Nitrogen – 50%

- Pathogens – Insufficient data to provide a pollutant removal value
- Heavy Metals – 80%

For additional information and data on pollutant removal capabilities for bioretention areas, see the National Pollutant Removal Performance Database (2nd Edition) available at www.stormwatercenter.net and the International Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

4.3.5.4 Application and Site Feasibility Criteria

Bioretention areas are suitable for many types of development, from single-family residential to high-density commercial projects. Bioretention is also well suited for small lots, including those of 1 acre or less. Because of its ability to be incorporated in landscaped areas, the use of bioretention is extremely flexible. Bioretention areas are an ideal structural stormwater BMP for use as roadway median strips and parking lot islands and are also good candidates for the treatment of runoff from pervious areas, such as a lawn area. Bioretention can also be used to retrofit existing development with stormwater quality treatment capacity.

The following criteria should be evaluated to ensure the suitability of a bioretention area for meeting stormwater management objectives on a site or development.

<u>General Feasibility</u>

- Suitable for Residential Subdivision Usage – YES
- Suitable for Regional Stormwater Control – NO

<u>Physical Feasibility - Physical Constraints at Project Site</u>

- <u>Drainage Area</u> – 5 acres maximum; 0.5 to 2 acres are preferred.
- <u>Space Required</u> – Approximately 5% of the tributary impervious area is required; minimum 200 ft^2 area for small sites (10 feet x 20 feet).
- <u>Site Slope</u> – No more than 6% slope in the contributing drainage area.
- <u>Minimum Head</u> – Elevation difference needed at a site from the inflow to the outflow underdrain or peak gravel under-layer: 5 feet.
- <u>Minimum Depth to Water Table</u> – A separation distance of 2 feet is recommended between the bottom of the bioretention facility and the elevation of the seasonally high water table.
- <u>Soils</u> – No restrictions; engineered media required. Karst areas may require a liner.

<u>Other Constraints / Considerations</u>

- <u>Wellhead Protection</u> – Reduce potential groundwater contamination (in required wellhead protection areas) by preventing infiltration of hotspot runoff. May require liner for type “A” and “B” soils; pretreat hotspots; 2 to 4 foot separation distance from water table. Wellhead protection areas may require guidance from other agencies, such as TDEC.

4.3.5.5 Planning and Design Standards

The following standards are to be considered **minimum** standards for the design of a bioretention facility. Consult with Knox County to determine if there are any variations to these criteria or additional standards that must be followed.

A. LOCATION AND SITING

- Bioretention areas should have a maximum contributing drainage area of 5 acres or less; 0.5 to 2 acres are preferred. Multiple bioretention areas can be used for larger areas.
- Bioretention areas can either be used to capture sheet flow from a drainage area or function as an off-line device. On-line designs should be limited to a maximum drainage area of 0.5 acres.
- When used in an off-line configuration, the WQv is diverted to the bioretention area through the use of a flow splitter or other means. Stormwater flows greater than the WQv are diverted to other controls or downstream (see Chapter 4, Section 4.2 for more discussion of off-line systems and design guidance for diversion structures and flow splitters).
- Bioretention systems are designed for intermittent flow and must be allowed to drain and reaerate between rainfall events. Bioretention systems will not be allowed for sites that have a continuous flow from groundwater, sump pumps, or other sources.
- Aesthetic considerations should be taken into account in the siting and design of bioretention areas. Elevations must be carefully determined to ensure that the desired runoff flow enters the facility with no more than the maximum design depth.
- Ideally, the construction of an bioretention area should take place after the construction site has been stabilized to avoid deposition of sediment in the media. In the event that the bioretention area is not constructed after site stabilization, diversion of site runoff around the bioretention area and installation and maintenance of appropriate erosion prevention and sediment controls prior to site stabilization is required. Erosion prevention and sediment controls shall be maintained around the bioretention area to prevent runoff and sediment from entering the trench during construction.
- Alternatively, the empty pit can be used as a sediment trap while the site is unstabilized. The media, however, cannot be placed into the bioretention area until all sediment has been removed and the site completely stabilized.
- During and after excavation of the bioretention area, all excavated materials shall be placed downstream, away from the bioretention area, to prevent redeposit of the material during runoff events.

B. GENERAL DESIGN

- A bioretention area shall consist of:
  - (1) A grass filter strip (or grass channel) between the contributing drainage area and the ponding area,
  - (2) A ponding area containing vegetation with a planting soil bed,
  - (3) An organic/mulch layer,
  - (4) A gravel and perforated pipe underdrain system to collect runoff that has filtered through the soil layers (bioretention areas can optionally be designed to infiltrate into the soil – see description of infiltration trenches for infiltration criteria).
- A bioretention area design may also include some of the following:
  - (1) An optional sand filter layer with geotextile fabric to spread flow, filter runoff, and aid in aeration and drainage of the planting soil, located between the underdrain and planting soil.
  - (2) A pea gravel diaphragm at the beginning of the grass filter strip to reduce runoff velocities and spread flow into the grass filter.
  - (3) Energy dissipation techniques will be required for contributing drainage areas that have a 6% slope or greater.
  - (4) Inflow diversion or an overflow structure(s) that is designed based on one of five main methods:

- Use of a flow diversion structure;
- Use of curbed pavements as an inlet deflector (see Figure 4-30);
- Use of a slotted curb along with the design of parking lot grades to divert the WQv into the bioretention facility. Additional runoff will be bypassed to a downstream catch basin inlet. The alternative requires temporary ponding in the parking lot (see Figure 4-29);
- Figure 4-29 illustrates the use of a short deflector weir (maximum height 6 inches) designed to divert the maximum water quality peak flow into the bioretention area;
- Use of an in-system overflow consisting of an overflow catch basin inlet and/or a pea gravel curtain drain overflow.

See Figure 4-27 for an overview of the various components of a bioretention area. Figure 4-28 provides a plan view and profile schematic of an on-line bioretention area. An example of an off-line facility is shown in Figure 4-29.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

- The minimum dimensions of a bioretention area shall be 10 feet wide by 20 feet long, or 200 square feet in area for roughly circular designs. All designs, except small residential applications such as bio-retention areas placed in cul-de-sac islands to treat runoff from the surrounding street, shall maintain a length to width ratio of at least 2:1.
- The planting soil filter bed shall be sized using a Darcy's Law equation with a filter bed drain time of 48 hours and a coefficient of permeability (k) of 0.5 ft/day.
- The maximum ponding depth of a bioretention area is 6 inches.
- The planting soil bed shall be at least 4 feet in depth when trees are planted in the bioretention area but can be a minimum of 2 feet deep in facilities that will utilize plants other than trees. Planting soils shall consist of a sandy loam, loamy sand, or loam texture with a clay content ranging from 10 to 25%. The soil must have an infiltration rate of at least 0.5 inches per hour and a pH between 5.5 and 6.5. In addition, the planting soil must have a 1.5 to 3% organic content and a maximum 500 ppm concentration of soluble salts.
- The mulch layer must consist of 2 to 4 inches of commercially available fine shredded hardwood mulch or shredded hardwood chips.
- Pea gravel for the diaphragm and curtain, when used, should be ASTM D 448 size No. 6 ($^1/_8$" to $^1/_4$").
- The underdrain collection system shall include a 4 to 6 inch pipe wrapped in a 6 to 8 inch gravel layer. The pipe shall have 3/8-inch perforations, spaced at 6-inch centers, with a minimum of 4 holes per row around the circumference of the pipe. The pipe spacing shall be at a maximum of 10 feet on center and a minimum grade of 0.5% must be maintained. A permeable filter fabric shall be required between the gravel layer and the planting soil bed. High density polyethylene (HDPE) pipe is the preferred pipe material, however other suitable pipe materials may be approved.

D. PRETREATMENT / INLETS

- Adequate pretreatment and inlet protection for bioretention systems shall be provided, such as: a grass filter strip below a flow spreader, or a grass channel, or a pea gravel diaphragm.
- For on-line configurations, a grass filter strip with a pea gravel diaphragm or other flow spreader shall be utilized (see Figure 4-28) as the pretreatment measure. The required length of the filter strip depends on the drainage area, imperviousness, and the filter strip slope. Design guidance on filter strips for pretreatment can be found in Volume 2, Chapter 4, Section 4.3.9 of this manual.
- For off-line applications, a grass channel with a pea gravel diaphragm or other flow spreader shall be used for pretreatment. The length of the grass channel depends on the drainage area, land use, and channel slope. The minimum grassed channel length shall be 20 feet. Design guidance on grass channels for pretreatment can be found in Volume 2, Chapter 4, Section 4.3.10 of this manual.

E. OUTLET STRUCTURES

- For bioretention areas placed in soils having a hydrologic soil group designation of C or D, an outlet pipe shall be provided from the underdrain system to the facility discharge. Outlet pipes are optional for group B soils. Discharges shall not exit the outlet pipe in an erosive manner. Due to the slow rate of discharge, outlet erosion protection is generally unnecessary. The outlet barrel shall be of reinforced concrete for bioretention areas that are used for flood protection.

F. EMERGENCY SPILLWAY

- An overflow structure and nonerosive overflow channel must be provided to safely pass flows that exceed the storage capacity of the bioretention area to a stabilized downstream area or watercourse. If the system is located off-line, the overflow shall be set above the shallow ponding limit.
- A high flow overflow system within a bioretention structure may consist of a yard drain catchbasin (Figure 4-27), though any number of conventional systems could be used. The throat of the catch basin inlet located in a bioretention facility must be no more than 6 inches above the mulch layer at the elevation of the shallow ponding area.

G. MAINTENANCE ACCESS

- A minimum 20' wide maintenance right-of-way or easement shall be provided from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles.
- The maintenance access shall be designed such that all areas of the bioretention area can be easily accessed, and shall be designed to allow vehicles to turn around.

H. SAFETY FEATURES

- Bioretention areas generally do not require any special safety features. Fencing of bioretention facilities is not generally desirable.

I. LANDSCAPING

- Landscaping is critical to the performance and function of bioretention areas.
- A dense and vigorous vegetative cover that is appropriate for use in a bioretention area shall be established over the contributing pervious drainage areas before runoff can be accepted into the facility. While the contributing drainage area is disturbed or unstabilized, sediment laden runoff shall not be allowed to reach an established bioretention area. A sediment trap can be converted to a bioretention area, but only after removal of all accumulated sediment.
- In general, vegetation utilized in the bioretention area should be native to East Tennessee, resistant to drought and inundation, tolerant of pollutants, have low fertilization requirements, and be easily maintained. Grasses, shrubs, and trees are all permissible vegetation types for bioretention areas, as long as the species used meet the general guidance provided herein.
- Bioretention areas that will contain trees shall be vegetated as follows:
  - The bioretention area shall be vegetated to resemble a terrestrial forest ecosystem, with a mature tree canopy, subcanopy of understory trees, shrub layer, and herbaceous ground cover. Three species each of both trees and shrubs are recommended to be planted.
  - The tree-to-shrub ratio should be 2:1 to 3:1. On average, the trees should be spaced 8 feet apart. Plants should be placed at regular intervals to replicate a natural forest. Woody vegetation should not be specified at inflow locations.
  - After the trees and shrubs are established, the ground cover and mulch should be established.

Additional information and guidance on bioretention area design and vegetation can be found on the EPA website at http://cfpub.epa.gov/npdes/stormwater and on the North Carolina State University Biological and Agricultural Engineering website at http://bae.ncsu.edu/stormwater/ .

J. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

Physiographic Factors - Local terrain design constraints

- Low Relief – Use of bioretention areas may be limited by low head.
- High Relief – Ponding area surface must be relatively level.
- Karst – Use poly-liner or impermeable membrane to seal bottom.

Soils

- No restrictions, however, planting soil must meet the required design infiltration rate.

Special Downstream Watershed Considerations

- Wellhead Protection – Reduce potential groundwater contamination (in required wellhead protection areas) by preventing infiltration of hotspot runoff. May require liner for type "A" and "B" soils; pretreat hotspots; 2 to 4 foot separation distance from water table. Wellhead protection areas may require guidance from other agencies, such as TDEC.

4.3.5.6 Design Procedures

Step 1. Compute runoff control volumes

Calculate WQv, CPv, Qp_2, Qp_{10}, Qp_{25}, and Qp_{100}, in accordance with the guidance presented in Volume 2, Chapter 3.

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

Consider the Application and Site Feasibility Criteria in subsections 4.3.5.4 and 4.3.5.5-A (Location and Siting).

Step 3. Confirm design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 4.3.5.5-J (Additional Site-Specific Design Criteria and Issues).

Check with Knox County and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4. Compute WQv peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion structures (see Volume 2, Chapter 3 for more detail).

(a) Using WQv (or total volume to be captured), compute CN

(b) Compute time of concentration using TR-55 method

(c) Determine appropriate unit peak discharge from time of concentration

(d) Compute Q_{wq} from unit peak discharge, drainage area, and WQv.

Step 5. Size flow diversion structure, if needed

A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQv to the bioretention area.

Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 6. Determine size of bioretention ponding/filter area

The required planting soil filter bed area is computed using the following equation (based on Darcy's Law):

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

where:

A_f = surface area of ponding area (ft^2)
WQv = water quality volume (or total volume to be captured)
d_f = filter bed depth
(4 feet minimum)
k = coefficient of permeability of filter media (ft/day)
(use 0.5 ft/day for silt-loam)
h_f = average height of water above filter bed (ft)
(typically 3 inches, which is half of the 6-inch ponding depth)
t_f = design filter bed drain time (days)
(2.0 days or 48 hours is recommended maximum)

Step 7. Set design elevations and dimensions of facility

See subsection 4.3.5.5-C (Physical Specifications/Geometry).

Step 8. Design conveyances to facility (off-line systems)

See the example figures to determine the type of conveyances needed for the site.

Step 9. Design pretreatment

Pretreat with a grass filter strip (on-line configuration) or grass channel (off-line), and stone diaphragm.

Step 10. Size underdrain system

See subsection 4.3.5.5-C (Physical Specifications/Geometry)

Step 11. Design emergency overflow

An overflow must be provided to bypass and/or convey larger flows to the downstream drainage system or stabilized watercourse. Nonerosive velocities are required at the outlet point.

Step 12. Design vegetation

A landscaping plan for the bioretention area should be prepared to indicate how it will be established with vegetation.

See subsection 4.3.5.5-I (Landscaping) for more details.

4.3.5.7 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.5.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of bioretention areas as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This section provides guidance on maintenance activities that are typically required for bioretention areas, along with a suggested frequency for each activity. Individual bioretention areas may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e.., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the pond in proper operating condition at all times.

Inspection Activities	Suggested Schedule
• After several storm events or an extreme storm event, inspect for signs of erosion, signs of mulch movement out of the treatment area, signs of damage to plants or dead or diseased vegetation.	As needed
• Inspect: inflow points for clogging (off-line systems), strip/grass channel for erosion or gullying, • Inspect trees, shrubs and other vegetation to evaluate their health and replace any dead or diseased vegetation. • Inspect surrounding drainage area for erosion or signs of sediment delivery to the bioretention area.	Semi-annually
• Check for signs of vegetation overgrowth. • Inspect treatment area during a rain event and visually verify that stormwater recedes within 24-48 hours from the treatment area.	Annually
Maintenance Activities	**Suggested Schedule**
• Replace mulch and repair areas of erosion, when identified. • Replace dead or diseased plants.	As needed
• Remove clogs from the stormwater system inflow and overflow components. • Remove sediments from pretreatment areas and restabilize with stone or vegetation as appropriate.	Semi-annually
• Harvest overgrown vegetation and remove from the bioretention area.	As needed
• The planting soils should be tested for pH to establish acidic levels. If the pH is below 5.2, limestone should be applied. If the pH is above 7.0 to 8.0, then iron sulfate plus sulfur can be added to reduce the pH. • Check that planting soils still have specified infiltration rate.	Annually
• Replace mulch over the entire area. • Replace pea gravel diaphragm if warranted. • Note that the surface of the ponding area may become clogged with fine sediment over time. Core aeration or cultivating of un-vegetated areas may be required to ensure adequate filtration.	2 to 3 years

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of bioretention areas. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the bioretention area. Questions regarding stormwater facility inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
BIORETENTION AREA INSPECTION CHECKLIST

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone:____

Date: ___________ Time: ______________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Inflow and Overflow Points		
Clear of debris and functional?		
Sediment accumulation?		
Vegetation in good condition?		
Signs of erosion?		
Other (describe)?		
Sediment Pretreament		
Evidence of sediment accumulation?		
Treatment Area and Vegetation		
Signs of erosion or movement of mulch?		
Vegetation healthy or damaged?		
Signs of sediment?		
Signs of thinning mulch layer?		
Vegetation overgrown and in need of harvesting?		
Standing water for more than 24-48 hours after rain events?		
Other (describe)?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature:________________________________ Inspector Name (printed)___________________________

4.3.5.8 Example Schematics

Figure 4-27. Schematic of a Typical Bioretention Area

(Source: Claytor and Schueler, 1996)

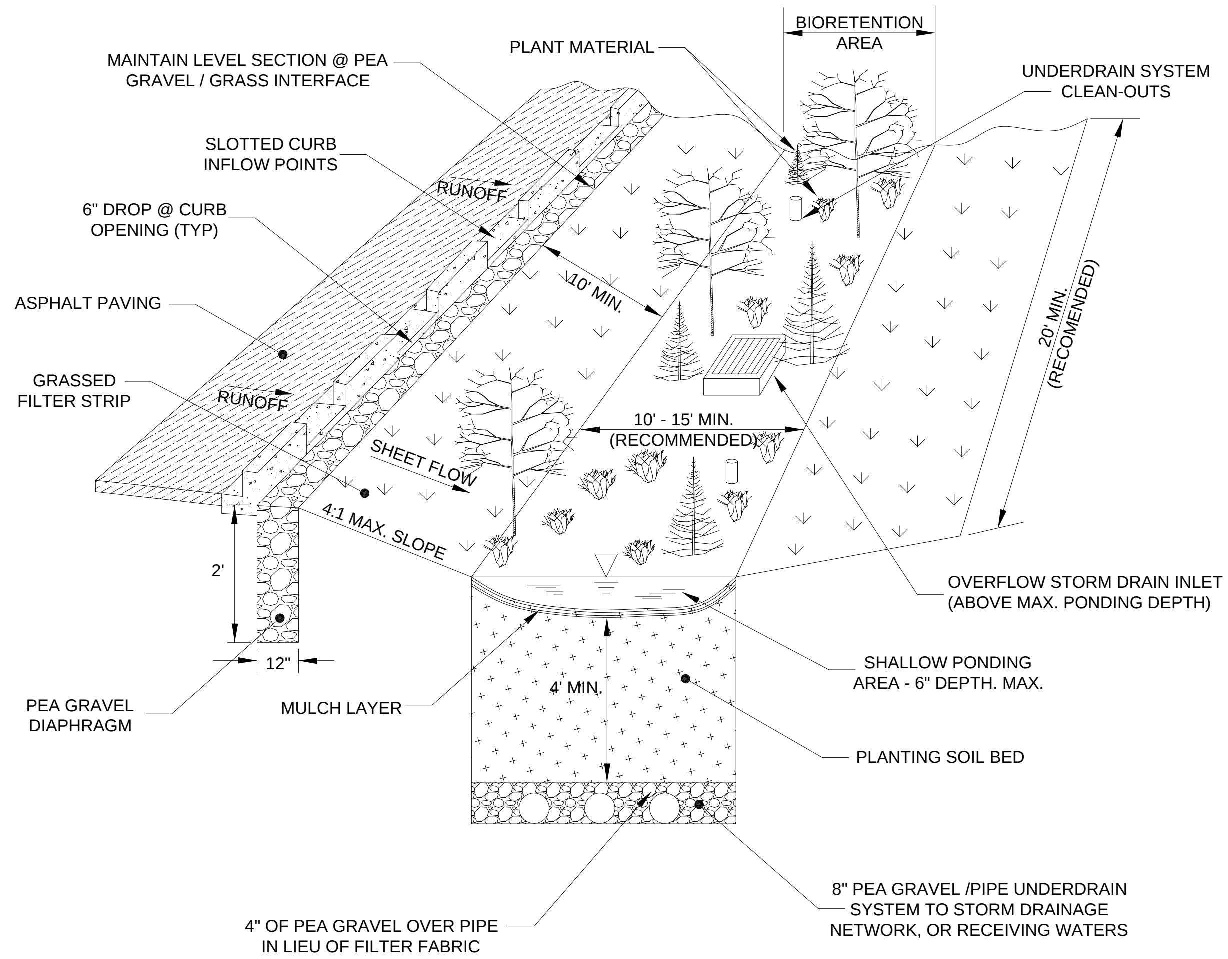


Figure 4-28. Schematic of a Typical On-line Bioretention Area
(Source: Claytor and Schueler, 1996)

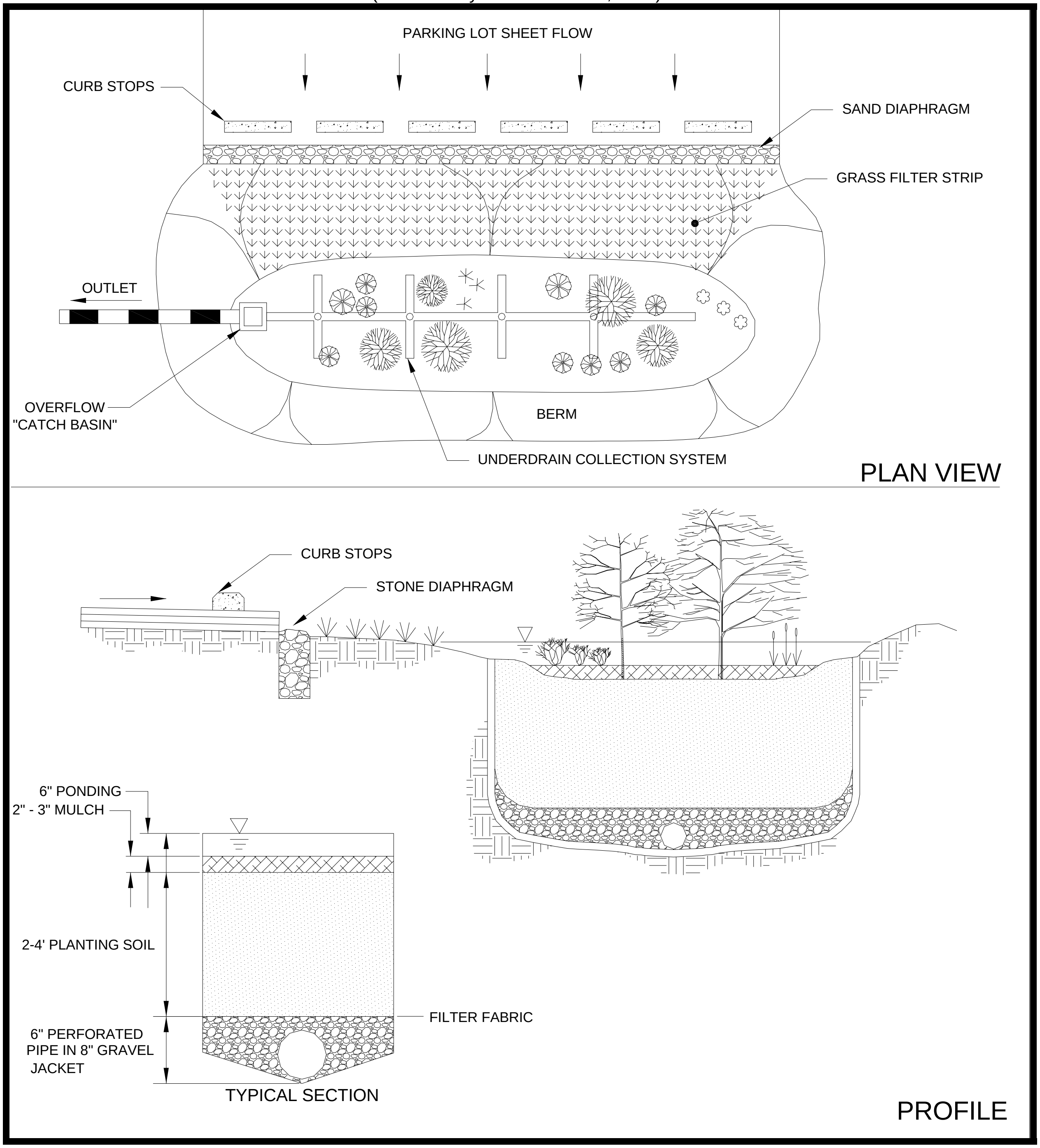


Figure 4-29. Schematic of a Typical Off-line Bioretention Area
(Source: Claytor and Schueler, 1996)

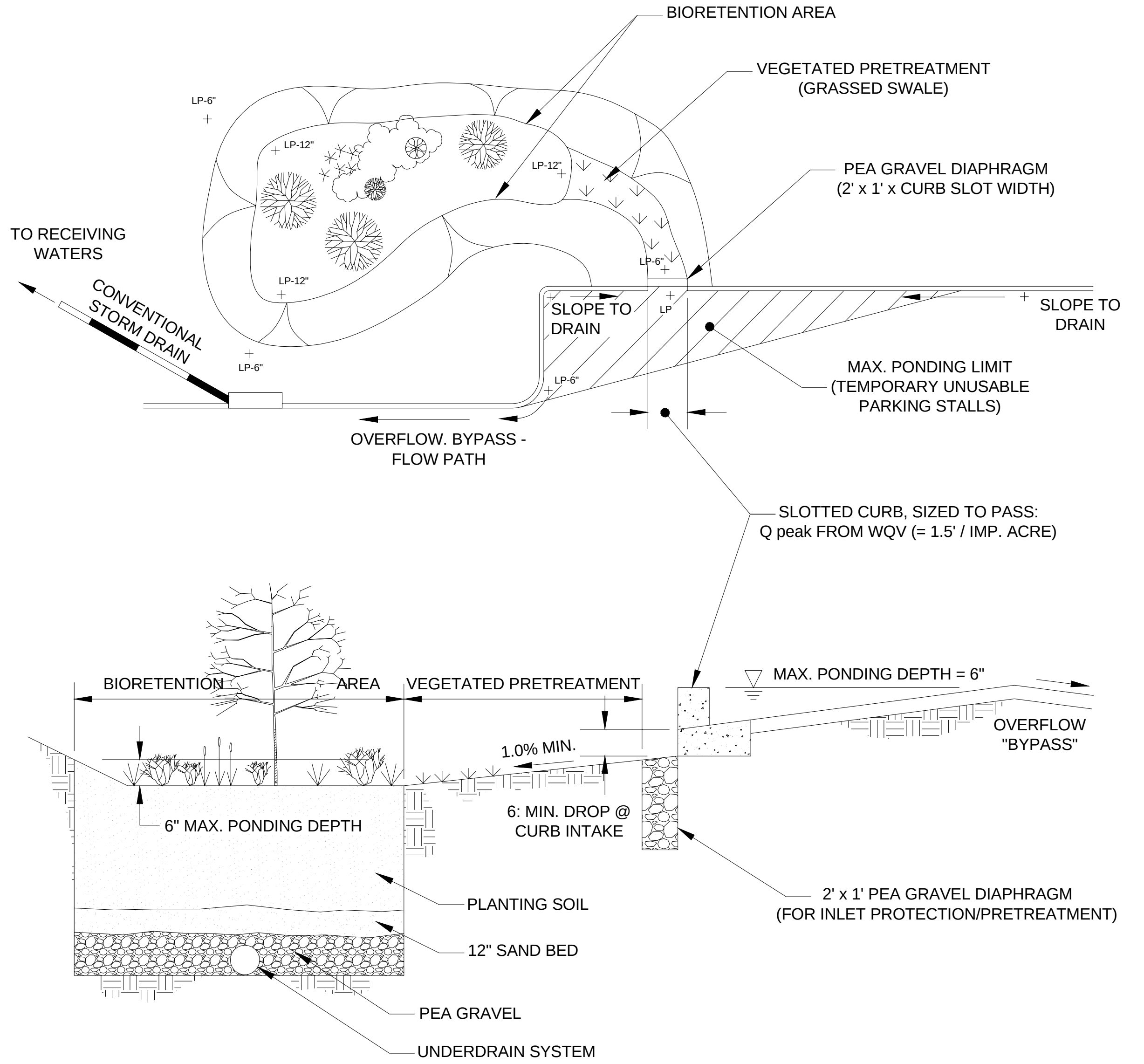
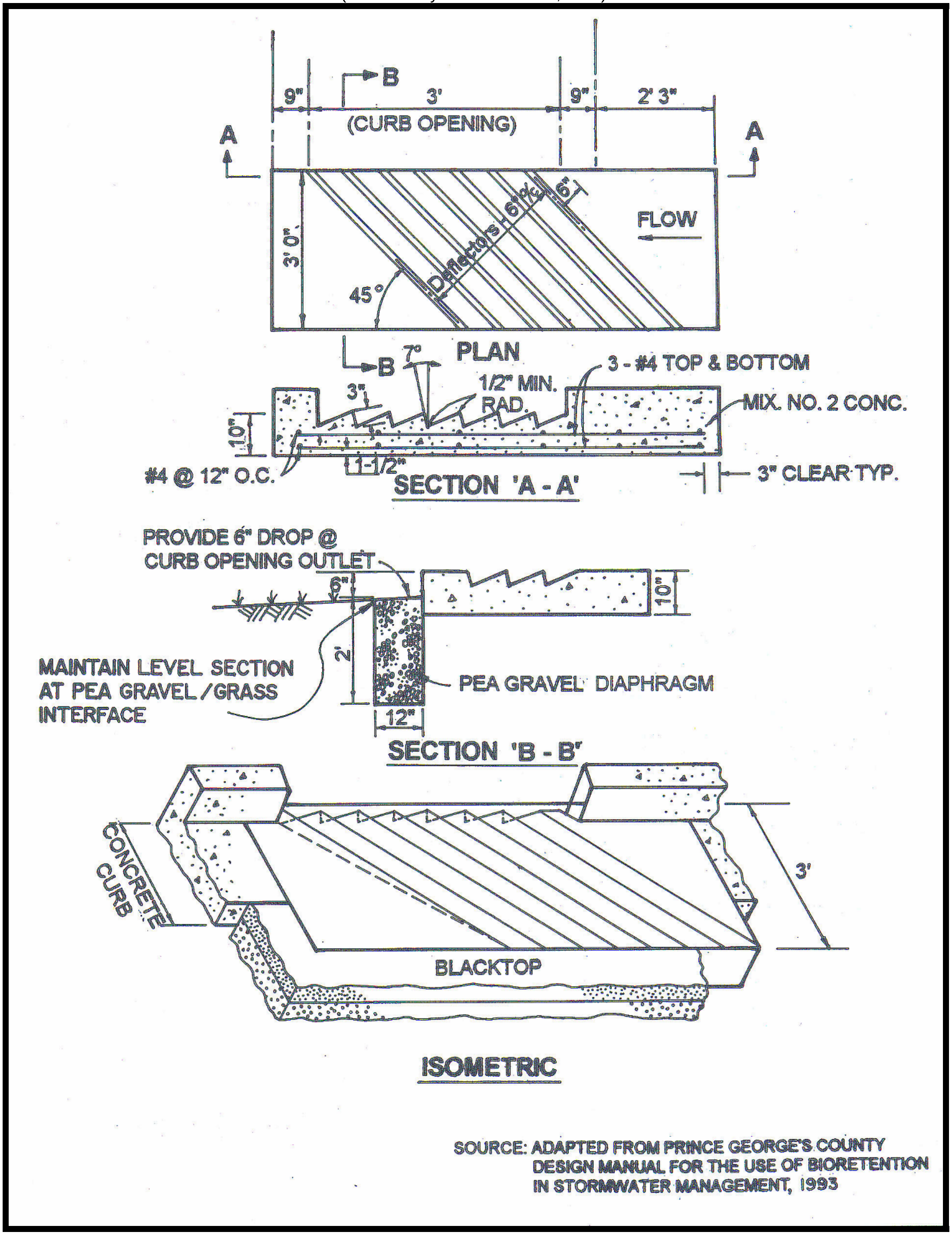


Figure 4-30. Schematic of a Typical Inlet Deflector
(Source: Claytor and Schueler, 1996)



4.3.5.9. Design Form

Knox County recommends the use of the following design procedure forms when designing a bioretention area. Proper use and completion of the form may allow a faster review of the Stormwater Management Plan by Knox County Engineering.

Design Procedure Form: Bioretention Areas

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute Runoff Coefficient, Rv	Rv = ________
Compute WQv	WQv = ________ acre-ft
1b. Estimate CPv	CPv = ________ acre-ft
1c. Estimate storage volumes	
Estimate storage volume required for 2-year storm	2-year storage = ________ acre-ft
Estimate storage volume required for 10-year storm	10-year storage = ________ acre-ft
Estimate storage volume required for 25-year storm	25-year storage = ________ acre-ft
Estimate storage volume required for 100-year storm	100-year storage = ________ acre-ft
BIORETENTION AREA DESIGN	
2. Is the use of a bioretention area appropriate?	***See subsections 4.3.5.4 and 4.3.5.5 - A***
3. Confirm design criteria and applicability.	***See subsection 4.3.5.5 - J***
4. Determine size of bioretention filter area	A_f= ________ ft^2
5. Set design elevations and dimensions	Length = ________ ft Width = ________ ft ________ elevation top of facility ________ other elev: ________ ________ other elev: ________ ________ other elev: ________
6. Conveyance to bioretention facility	________ Online or ________ Offline ?
7. Pretreatment	Type: ________________
8. Size underdrain area Based on guidance: Approx. 10% A_f	Length= ________ ft
9. Overdrain design	Type: ________________ Size: ________
10. Emergency storm weir design Overflow weir - Weir equation	Length= ________ ft
11. Choose plants for planting area	Select native plants based on resistance to drought and inundation, cost, aeshetics, maintenance, etc.
12. Verify peak flow control (if applicable), water quality drawdown time and channel protection detention time	

4.3.5.10 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual, Volume 2: Technical Handbook.* 2001.

Center for Watershed Protection. *Manual Builder.* Stormwater Manager's Resource Center, Accessed July 2005. www.stormwatercenter.net

City of Portland, OR. *Stormwater Management Manual.* 2004.

Claytor, R.A., and T.R. Schueler. *Design of Stormwater Filtering Systems.* The Center for Watershed Protection, Silver Spring, MD, 1996.

Prince George's County. *Design Manual for Use of Bioretention in Stormwater Management.* Department of Environmental Resources, Prince George's County, Landover, MD, 1993.

4.3.5.11 Suggested Reading

Bell, W. *BMP Technologies for Ultra-Urban Settings. In Proceedings of Effective Land Management for Reduced Environmental Impact.* Tidewater's Land Management Conference on Water Quality, August 22, 1996.

City of Austin, TX. *Water Quality Management.* Environmental Criteria Manual, Environmental and Conservation Services, 1988.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

US EPA. *Storm Water Technology Fact Sheet: Bioretention.* EPA 832-F-99-012, Office of Water, 1999.

Maryland Department of the Environment. *Maryland Stormwater Design Manual, Volumes I and II.* Prepared by Center for Watershed Protection (CWP), 2000.

Washington State Department of Transportation (WSDOT). *Highway Runoff Manual.* Washington State Department of Transportation, 1995.

4.3.6 Surface Sand Filters

General Application Stormwater BMP

Description: Surface sand filters are multi-chamber structures located above ground that are designed to treat stormwater runoff through filtration, using a sediment forebay, a sand bed as its primary filter media and, typically, an underdrain collection system.

KEY DESIGN CONSIDERATIONS

DESIGN GUIDELINES:

- Typically requires 2 to 6 feet of head.
- Maximum contributing drainage area of 10 acres for surface sand filter; 2 acres for perimeter sand filter.
- Sand filter media with underdrain system.

ADVANTAGES / BENEFITS:

- Applicable to small drainage areas.
- Good for highly impervious areas.
- Good retrofit capability.

DISADVANTAGES / LIMITATIONS:

- High maintenance burden.
- Not recommended for areas with high stormwater sediment or clay/silt runoff areas.
- Relatively costly.
- Possible odor problems.
- Cannot be installed until site work is complete.

MAINTENANCE REQUIREMENTS:

- Inspect for clogging – rake first inch of sand.
- Remove sediment from forebay/chamber.
- Replace sand filter media as needed.

STORMWATER MANAGEMENT SUITABILITY

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

Accepts Hotspot Runoff: *Yes (requires impermeable liner)*

✷ in certain situations

FEASABILITY CONSIDERATIONS

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

Residential/Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *2-10 acres max.*

Soils: *Not recommended for clay/silt drainage areas that are not stabilized.*

POLLUTANT REMOVAL

H	**Total Suspended Solids**
M	**Nutrients - Total Phosphorus / Total Nitrogen**
M	**Metals - Cadmium, Copper, Lead, and Zinc**
M	**Pathogens - Coliform, Streptococci, E.Coli**

OTHER CONSIDERATIONS:

- Typically needs to be combined with other controls to provide water quantity control

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

4.3.6.1 General Description

Surface sand filters (also referred to as *sand filters* or *filtration basins*) are ground-level, open air structures that capture and temporarily store stormwater runoff and pass it through a filter bed of sand. An example of a surface sand filter is presented in Figure 4-31. Underground sand filters, discussed in Section 4.4.2, treat stormwater in the same manner, but are located below the ground surface. Because of the increased maintenance requirements, underground sand filters are considered Limited Application BMPs.

Figure 4-31. Example of a Surface Sand Filter

Most sand filter systems, surface and underground, consist of two-chamber structures. The first chamber is a sediment forebay or sedimentation chamber, which removes floatables and heavy sediments. The second is the filtration chamber, which removes finer sediments and other pollutants by filtering the runoff through a sand bed. The filtered runoff is typically collected and returned to the conveyance system, though it can also partially or fully permeate into the surrounding soil in areas with porous soils.

This system can treat drainage areas up to 10 acres in size and is typically located off-line. Surface sand filters can be designed as an excavation with earthen embankments or as a concrete or block structure. Because they have few site constraints beside head requirements, sand filters can be used on development sites where the use of other structural BMPs may be precluded. However, sand filter systems can be relatively expensive to construct and install, and require a relatively high level of maintenance and inspection. Because of this, surface sand filters are not recommended for use in residential areas.

4.3.6.2 Stormwater Management Suitability

Surface sand filter systems are designed primarily as off-line systems for treatment of the water quality volume and will typically need to be used in conjunction with another structural BMP that can provide downstream channel protection, overbank flood protection, and extreme flood protection. However, under certain circumstances, filters can provide limited runoff quantity control, particularly for smaller storm events.

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

Channel Protection (CPv)
For smaller sites, a sand filter may be designed to capture the entire channel protection volume (CPv) in either an off- or on-line configuration. Given that a sand filter system is typically designed to completely drain over 40 hours, the channel protection design requirement for extended detention of the 1-year, 24-hour storm runoff volume can be met. For larger sites or where only the WQv is diverted to the sand filter facility, another structural control must be used to provide extended detention of the CPv.

Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})
Sand filters are not useful for flood protection. Another structural control, such as a conventional detention pond must be used in conjunction with a sand filter system to control stormwater peak discharges. Further, sand filter facilities must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the filter bed and facility.

4.3.6.3 Pollutant Removal Capabilities

Surface sand filters are presumed to be able to remove 80% of the total suspended solids (TSS) load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed sand filters can reduce TSS removal performance.

Additionally, research has shown that use of sand filters will have benefits beyond the removal of TSS, such as the removal of other pollutants (i.e. phosphorous, nitrogen, fecal coliform and heavy metals), as well, which is useful information should the pollutant removal criteria change in the future. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data.

- Total Suspended Solids – 80%
- Total Phosphorus – 50%
- Total Nitrogen – 30%
- Pathogens – 40%
- Heavy Metals – 50%

For additional information and data on pollutant removal capabilities for sand filters, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the International Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

4.3.6.4 Application and Site Feasibility Criteria

Surface sand filter systems are well-suited for highly impervious areas where land available for structural BMPs is limited. Sand filters should primarily be considered for new construction or retrofit opportunities for commercial, industrial, and institutional areas where the sediment load is relatively low, such as: parking lots, driveways, loading docks, gas stations, garages, airport runways/taxiways, and storage yards. Sand filters may also be feasible and appropriate in some multi-family residential developments where maintenance is performed by a landscaping (or other suitably capable) company.

To avoid rapid clogging and failure of the filter media, the use of sand filters should be avoided in areas with less than 50% impervious cover, or high sediment yield sites with clay/silt soils.

The following basic criteria should be evaluated to ensure the suitability of a sand filter facility for meeting stormwater management objectives on a site or development.

General Feasibility

- Not suitable for use in a residential subdivision
- Suitable for use in high density/ultra-urban areas
- Not suitable for use as a regional stormwater control. On-site applications are typically most feasible.

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 10 acres maximum for surface sand filter; 2 acres maximum for perimeter sand filter
- Space Required – Function of available head at site
- Minimum Head – The surface slope across the filter location should be no greater than 6%. The elevation difference needed at a site from the inflow to the outflow: 5 feet for surface sand filters; 2 to 3 feet for perimeter sand filters.
- Minimum Depth to Water Table – If used on a site with an underlying water supply aquifer, a separation distance of 2 feet is required between the bottom of the sand filter and the elevation of the seasonally high water table to prevent groundwater contamination.
- Soils – Not recommended for clay/silt drainage areas that are not stabilized. Karst areas may require a liner.

Other Constraints / Considerations

- Aquifer Protection – Do not allow infiltration of filtered hotspot runoff into groundwater

4.3.6.5 Planning and Design Standards

The following standards shall be considered **minimum** design standards for the design of sand filters. Sand filters that are not designed to these standards will not be approved. The Director of Engineering and Public Works (the Director) shall have the authority to require additional design conditions if deemed necessary.

A. CONSTRUCTION SEQUENCING

- Care shall be taken during construction to minimize the risk of premature failure of the sand filter due to deposition of sediments from disturbed, unstabilized areas. This can be minimized or avoided by proper construction sequencing.
- Ideally, the construction of a sand filter shall take place **after** the construction site has been stabilized. In the event that the sand filter is not constructed after site stabilization, diversion of site runoff around the sand filter and installation and maintenance of appropriate erosion prevention and sediment controls prior to site stabilization is required.
- Diversion berms shall be maintained around a sand filter during all phases of construction. No runoff shall enter the sand filter area prior to completion of construction and the complete stabilization of construction areas. Erosion prevention and sediment controls shall be maintained around the sand filter to prevent runoff and sediment from entering the sand filter during construction.
- Sand filters shall not be used as a temporary sediment trap for construction activities.
- During and after excavation of the sand filter, all excavated materials shall be placed downstream, away from the sand filters, to prevent redeposit of the material during runoff events.

B. LOCATION AND SITING

- Surface sand filters shall have a contributing drainage area of 10 acres or less.
- Surface sand filter systems are generally applied to land uses with a high percentage of impervious surfaces. Sand filters shall not be utilized for sites that have less than 50% impervious cover.

Pretreatment must be provided as described in part D below, due to the potential for high clay/silt sediment loads that could result in clogging and failure of the filter bed. Any disturbed or denuded areas located within the area draining to and treated by the sand filter shall be stabilized prior to construction and use of the sand filter. The sand filter shall only be constructed after the construction site is stabilized.

- It is preferred that surface sand filters are to be used in an off-line configuration where the water quality volume (WQv) is diverted to the filter facility through the use of a flow diversion structure and flow splitter. Stormwater flows greater than the WQv shall be diverted to other controls or downstream using a diversion structure or flow splitter. In certain situations, as determined by the Director, a surface sand filter may be used in an on-line configuration.
- Sand filter systems shall be designed for intermittent flow and must be allowed to drain and re-aerate between rainfall events. They shall not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

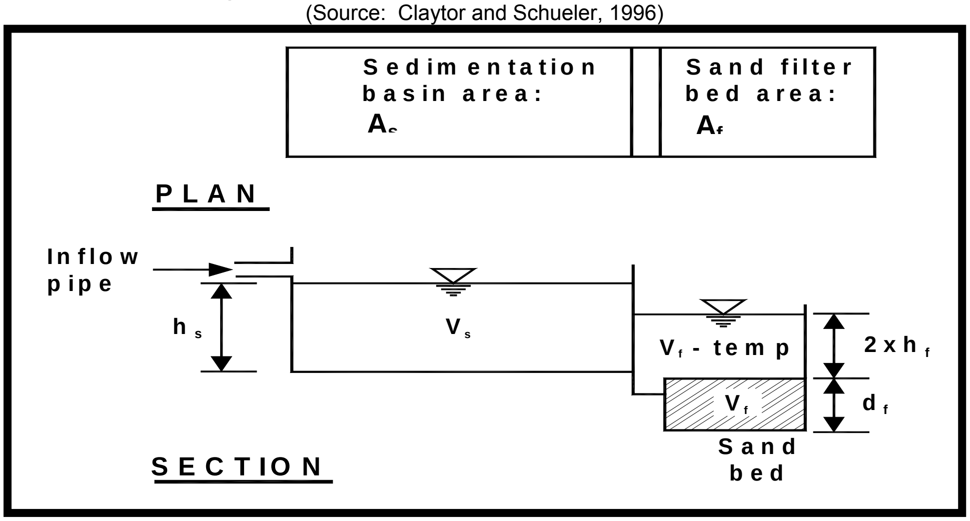
C. GENERAL DESIGN

- A surface sand filter facility shall consist of a two-chamber open-air structure, which is located at ground-level. The first chamber is the sediment forebay (commonly referred to as the sedimentation chamber) while the second chamber houses the sand filter bed. Flow enters the sedimentation chamber where settling of larger sediment particles occurs. Runoff is then discharged from the sedimentation chamber through a perforated standpipe into the filtration chamber. After passing though the filter bed, runoff is collected by a perforated pipe and gravel underdrain system.

D. PHYSICAL SPECIFICATIONS / GEOMETRY

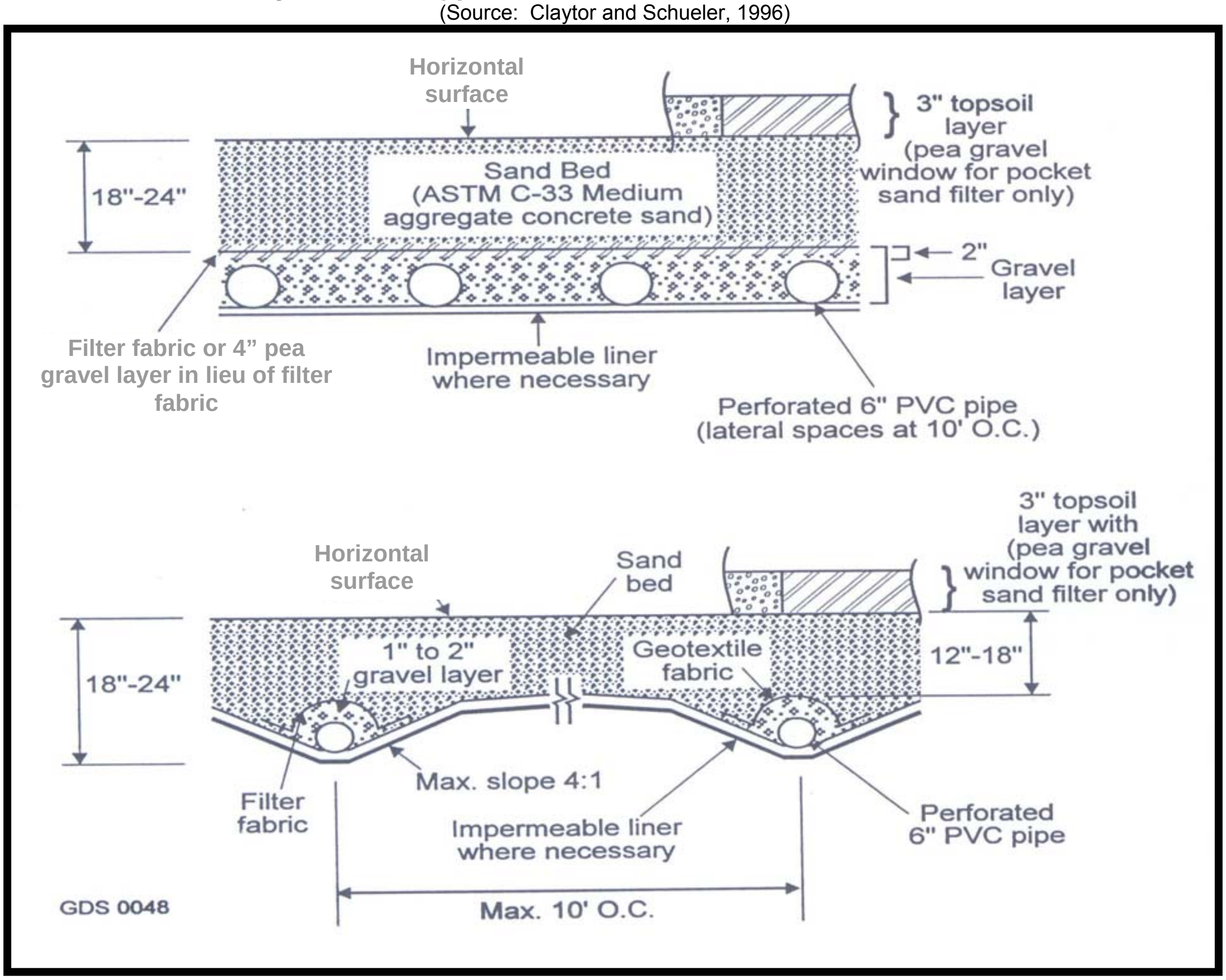
- The entire treatment system (including the sedimentation chamber) shall be designed to temporarily hold at least 75% of the WQv prior to filtration. Figure 4-32 illustrates the distribution of the treatment volume (0.75 WQv) among the various components of the surface sand filter, including:
  - V_s – volume within the sedimentation basin
  - V_f – volume within the voids in the filter bed
  - $V_{f\text{-}temp}$ – temporary volume stored above the filter bed
  - A_s – the surface area of the sedimentation basin
  - A_f – surface area of the filter media
  - h_s – height of water in the sedimentation basin
  - h_f – average height of water above the filter media
  - d_f – depth of filter media

Figure 4-32. Surface Sand Filter Volumes
(Source: Claytor and Schueler, 1996)



- The sedimentation chamber shall be sized to hold at least 25% of the computed WQv and have a length-to-width ratio of at least 2:1. Inlet and outlet structures should be located at opposite ends of the chamber.
- The filter area shall be sized based on the principles of Darcy's Law. A coefficient of permeability (k) of 3.5 ft/day for sand shall be used. The filter bed shall be designed to completely drain in 40 hours or less.
- The filter media shall consist of an 18-inch layer of clean washed medium aggregate concrete sand (ASTM C-33) on top of the underdrain system. Three inches of topsoil shall be placed over the sand bed. Permeable filter fabric shall be placed both above and below the sand bed to prevent clogging of the sand filter and the underdrain system. Figure 4-33 illustrates a typical media cross section.

Figure 4-33. Typical Sand Filter Media Cross Sections
(Source: Claytor and Schueler, 1996)



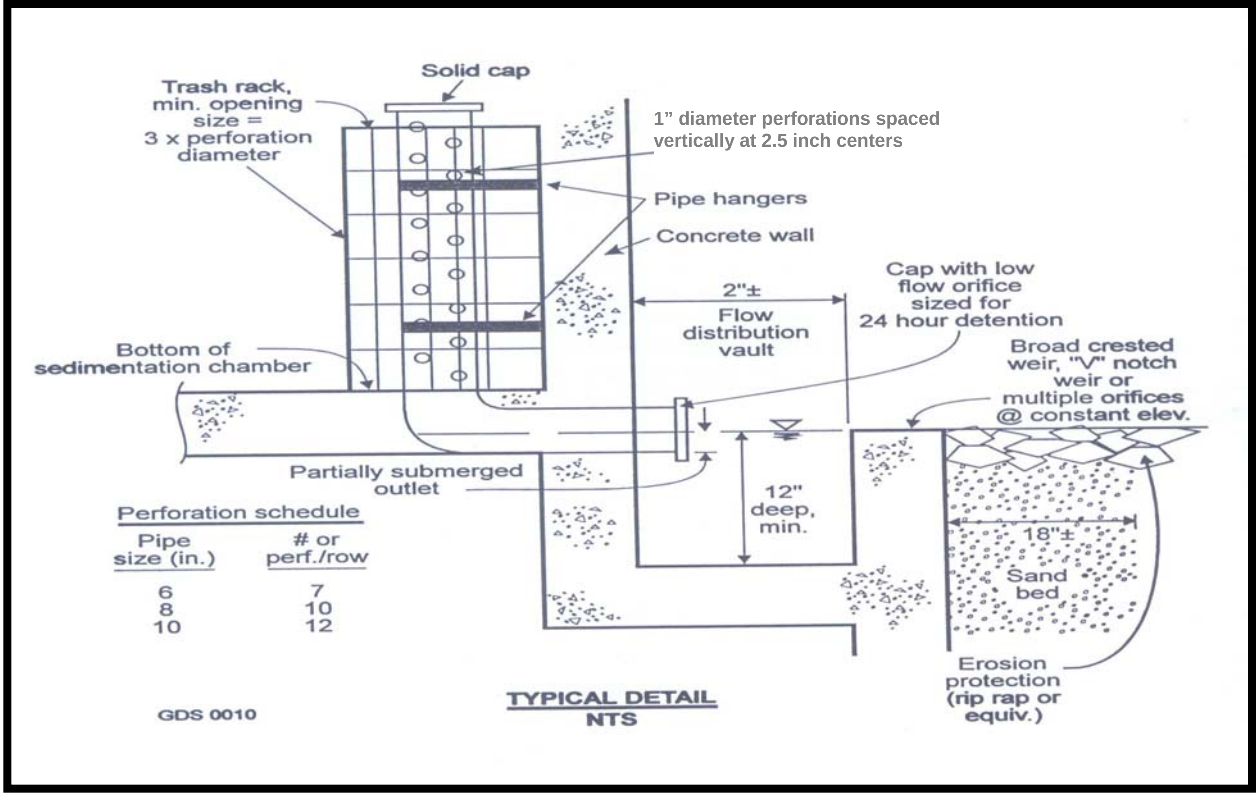
- The filter bed shall be equipped with a 6-inch perforated pipe underdrain (PVC AASHTO M 252, HDPE, or other suitable pipe material) in a gravel layer. The underdrain shall have a minimum grade of 1/8-inch per foot (1% slope). Holes shall be 3/8-inch diameter and spaced approximately 6 inches on center. Gravel shall be clean-washed aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches with a void space of about 40%. Aggregate contaminated with soil shall not be used.
- The structure of the surface sand filter may be constructed of impermeable media such as concrete, or through the use of excavations and earthen embankments. When constructed with earthen

walls/embankments, filter fabric shall be used to line the bottom and side slopes of the structures before installation of the underdrain system and filter media.

E. PRETREATMENT / INLETS

- Pretreatment of runoff in a sand filter system shall be by a sedimentation chamber, designed in accordance with the criteria stated above.
- Energy dissipators shall be used at the inlets to surface sand filters. Figure 4-34 shows a typical inlet pipe from the sedimentation basin to the filter media basin for the surface sand filter.
- The sand filter shall be designed such that runoff exits the sedimentation chamber at a non-erosive velocity.

Figure 4-34. Surface Sand Filter Perforated Stand-Pipe
(Source: Claytor and Schueler, 1996)



F. OUTLET STRUCTURES

- An outlet pipe shall be provided from the underdrain system to the facility discharge. Due to the slow rate of filtration, outlet protection is generally unnecessary (except for emergency overflows and spillways). However, the design shall ensure that the discharges from the underdrain system occur in a non-erosive manner.

G. EMERGENCY SPILLWAY

- An emergency or bypass spillway must be included in the surface sand filter design to safely pass flows that exceed the WQv (and CPv if the filter is utilized for channel protection purposes). The spillway prevents filter water levels from overtopping the embankment and causing structural damage. The emergency spillway shall be located so that embankments, downstream buildings and structures will not be impacted by spillway discharges.

H. MAINTENANCE ACCESS

- A minimum 20' wide maintenance right of way or drainage easement shall be provided for a sand filter from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles. Facility designs must enable maintenance personnel to easily remove and replace upper layers of the filter media.

I. SAFETY FEATURES

- Where necessary, surface sand filter facilities can be fenced to prevent access.

J. LANDSCAPING

- Surface sand filters can be designed with a grass cover to aid in pollutant removal and prevent clogging. The grass should be capable of withstanding frequent periods of inundation and drought.

K. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

Physiographic Factors - Local terrain design constraints

- Low Relief – Use of surface sand filter may be limited by low head
- High Relief – Filter bed surface must be level
- Karst – Use liner or impermeable membrane to seal bottom earthen surface of the sand filter or use watertight structure

Special Downstream Watershed Considerations

- Wellhead Protection – Reduce potential groundwater contamination (in required wellhead protection areas) by preventing infiltration of hotspot runoff. May require liner for type "A" and "B" soils; Pretreat hotspots; provide 2 to 4 foot separation distance from water table.

4.3.6.6 Design Procedures

Step 1. Compute runoff control volumes

> Calculate WQv, CPv, Qp_2, Qp_{10}, Qp_{25}, and Qp_{100}, in accordance with the guidance presented in Volume 2, Chapter 2.

Step 2. Determine if the development site and conditions are appropriate for the use of a surface sand filter.

> Consider the Application and Site Feasibility Criteria, and the Additional Site Specific Design Criteria and Issues noted above. Check with Knox County Engineering and other agencies as appropriate to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 3. Compute WQv peak discharge (Q_{wq})

> The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion structures (see Volume 2, Chapter 2 for more information on this calculation).
>
> (1) Using WQv, compute CN
>
> (2) Compute time of concentration using TR-55 method
>
> (3) Determine appropriate unit peak discharge from time of concentration
>
> (4) Compute Q_{wq} in inches from unit peak discharge, drainage area, and WQv

Step 4. Size flow diversion structure, if needed

> A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQv to the sand filter facility. Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 5. Size filtration basin chamber

The filter area is sized using the following equation (based on Darcy's Law):

$$A_f = (WQv)(d_f) / [(k)(h_f + d_f)(t_f)]$$

where:

A_f = surface area of filter bed (ft^2)
d_f = filter bed depth (1.5 ft) (at least 18 inches, no more than 24 inches)
k = coefficient of permeability of filter media (ft/day) (use 3.5 ft/day for sand)
h_f = average height of water above filter bed (ft)
(1/2 h_{max}, which varies based on site but h_{max} is typically $\leq$ 6 feet)
t_f = design filter bed drain time (days) (1.67 days or 40 hours is maximum time)

Set preliminary dimensions of filtration basin chamber.

Step 6. Size sedimentation chamber

The sedimentation chamber shall be sized to at least 25% of the computed WQv and have a length-to-width ratio of 2:1. The Camp-Hazen equation is used to compute the required surface area:

$$A_s = -(Q_o/w) * \text{Ln}(1-E)$$

where:

A_s = sedimentation basin surface area (ft^2)
Q_o = rate of outflow = the WQv (ft^3) / 86400 seconds
w = particle settling velocity (ft/sec)
E = trap efficiency

Assuming:

- 90% sediment trap efficiency (0.9)
- particle settling velocity (ft/sec) = 0.0033 ft/sec for imperviousness ≥ 75%
- particle settling velocity (ft/sec) = 0.0004 ft/sec for imperviousness < 75%
- average of 24 hour holding period

Then:

A_s = (0.0081) (WQv) ft^2 for $I \geq 75\%$
A_s = (0.066) (WQv) ft^2 for $I < 75\%$

Set preliminary dimensions of sedimentation chamber.

Step 7. Compute V_{min}

$$V_{min} = 0.75 * WQv$$

Step 8. Compute storage volumes within entire facility and sedimentation chamber orifice size

$$V_{min} = 0.75\ WQv = V_s + V_f + V_{f\text{-}temp}$$

(1) Compute V_f = water volume within filter bed/gravel/pipe = $A_f * d_f * n$
Where: n = porosity = 0.4 for most applications

(2) Compute $V_{f\text{-}temp}$ = temporary storage volume above the filter bed = $2 * h_f * A_f$

(3) Compute V_s = volume within sediment chamber = $V_{min} - V_f - V_{f\text{-}temp}$

(4) Compute h_s = height in sedimentation chamber = V_s/A_s

(5) Ensure h_s and h_f fit available head and other dimensions still fit – change as necessary in design iterations until all site dimensions fit.

(6) Size orifice from sediment chamber to filter chamber to release V_s within 24-hours at average release rate with 0.5 h_s as average head.

(7) Design outlet structure with perforations allowing for a safety factor of 10 times the orifice capacity.

(8) Size distribution chamber to spread flow over filtration media – level spreader weir or orifices.

Step 9. Design inlets, pretreatment facilities, underdrain system, and outlet structures

See design criteria above for more details.

Step 10. Compute overflow weir sizes

(1) Size overflow weir at elevation hs in sedimentation chamber (above perforated stand pipe) to handle surcharge of flow through filter system from 25-year storm.

(2) Plan inlet protection for overflow from sedimentation chamber and size overflow weir at elevation hf in filtration chamber (above perforated stand pipe) to handle surcharge of flow through filter system from 25-year storm.

4.3.6.7 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.6.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of a sand filter as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for sand filters, along with a suggested frequency for each activity. Individual sand filters may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e.., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the sand filter in proper operating condition at all times.

Inspection Activities	Suggested Schedule
• A record should be kept of the dewatering time (i.e., the time required to drain the filter bed completely after a storm event) for a sand filter to determine if maintenance is necessary. The filter bed should drain completely in about 40 hours after the end of the rainfall. • Check to ensure that the filter surface does not clog after storm events.	After Rain Events
• Check the contributing drainage area, facility, inlets and outlets for debris. • Check to ensure that the filter surface is not clogging.	Monthly
• Check to see that the filter bed is clean of sediment, and the sediment chamber is not more than 50% full or 6 inches, whichever is less, of sediment. Remove sediment as necessary. • Make sure that there is no evidence of deterioration, spalling, bulging, or cracking of concrete. • Inspect grates (perimeter sand filter). • Inspect inlets, outlets and overflow spillway to ensure good condition and no evidence of erosion. • Check to see if stormwater flow is bypassing the facility. • Ensure that no noticeable odors are detected outside the facility.	Annually
Maintenance Activities	**Suggested Schedule**
• Mow and stabilize (prevent erosion, vegetate denuded areas) the area draining to the sand filter. Collect and remove grass clippings. Remove trash and debris. • Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system. • If permanent water level is present (perimeter sand filter), ensure that the chamber does not leak, and normal pool level is retained.	Monthly
• Check to see that the filter bed is clean of sediment, and the sediment chamber is not more than 50% full or 6 inches, whichever is less, of sediment. Remove sediment as necessary. • Repair or replace any damaged structural parts. • Stabilize any eroded areas.	Annually
• If filter bed is clogged or partially clogged, manual manipulation of the surface layer of sand may be required. Remove the top few inches of sand, roto-till or otherwise cultivate the surface, and replace media with sand meeting the design specifications. • Replace any filter fabric that has become clogged.	As needed

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of sand filters. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the sand filter. Questions regarding stormwater facility inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
SURFACE SAND FILTER INSPECTION CHECKLIST

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ____

Date: __________ Time: _____________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Sand Filter Inspection List		
Complete drainage of the filter in about 40 hours after a rain event?		
Clogging of filter surface?		
Clogging of inlet/outlet structures?		
Clogging of filter fabric?		
Filter clear of debris and functional?		
Leaks or seeps in filter?		
Obstructions of spillway(s)?		
Animal burrows in filter?		
Sediment accumulation in filter bed (less than 50% is acceptable)?		
Cracking, spalling, bulging or deterioration of concrete?		
Erosion in area draining to sand filter?		
Erosion around inlets, filter bed, or outlets?		
Pipes and other structures in good condition?		
Undesirable vegetation growth?		
Other (describe)?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

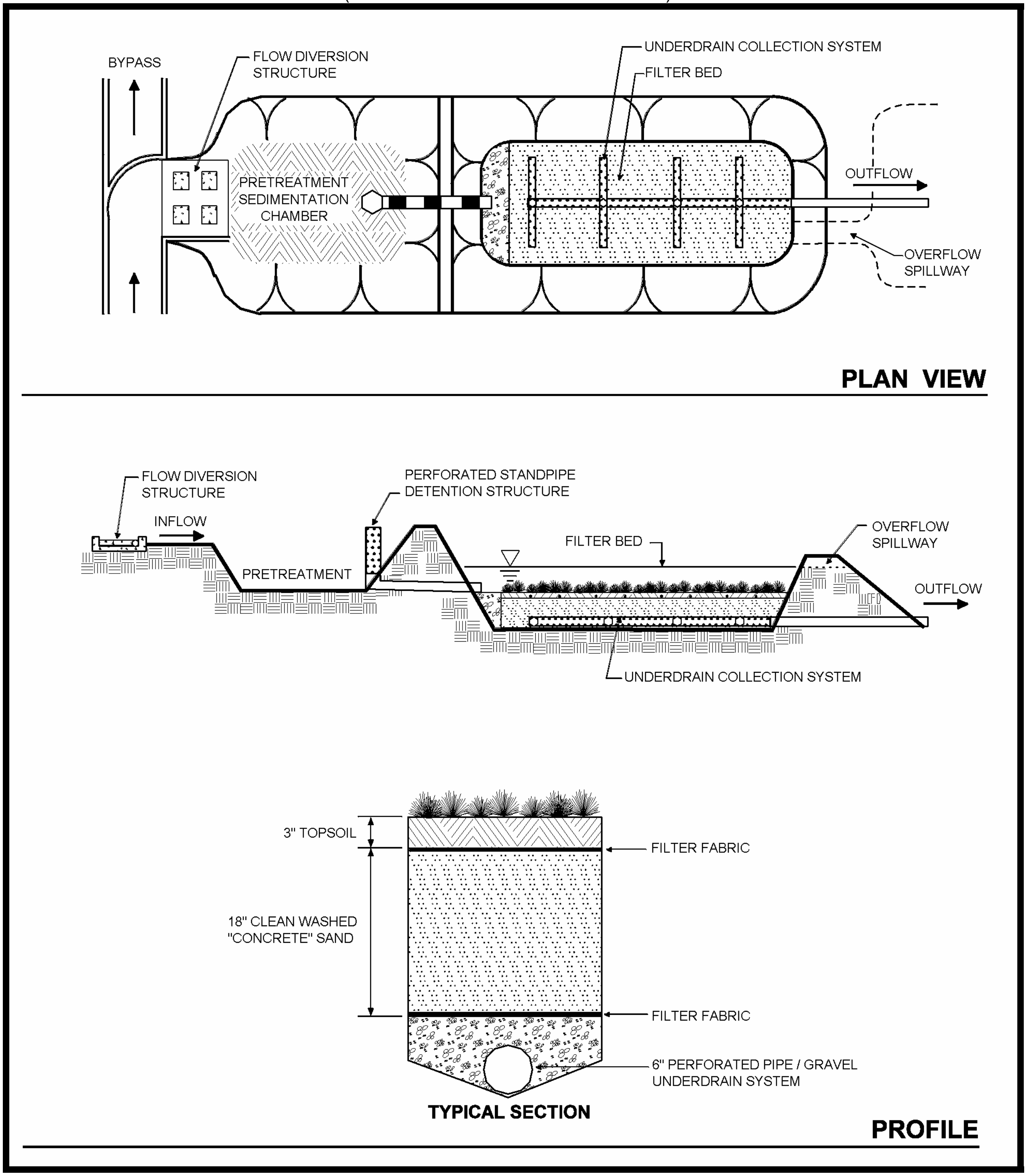
Corrective Action Needed	Due Date

Inspector Signature:______________________________ Inspector Name (printed)__________________________

4.3.6.8 Example Schematics

Figure 4-35. Schematic of Surface Sand Filter

(Source: Center for Watershed Protection)



4.3.6.9 Design Forms

Knox County recommends the use of the following design procedure forms when designing sand filters. Proper use and completion of the form may allow a faster review of the Stormwater Management Plan by Knox County Engineering.

Design Procedure Forms: Sand Filters

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQv volume requirements	
Compute Runoff Coefficient, Rv	Rv = ______
Compute WQv	WQv = ______ acre-ft
1b. Estimate CPv	CPv = ______ acre-ft
1c. Estimate storage volumes	release rate = ______ cfs
Estimate storage volume required for 2-year storm	2-year storage = ______ acre-ft
Estimate storage volume required for 10-year storm	10-year storage = ______ acre-ft
Estimate storage volume required for 25-year storm	25-year storage = ______ acre-ft
Estimate storage volume required for 100-year storm	100-year storage = ______ acre-ft
SAND FILTER DESIGN	
2. Is the use of a sand filter appropriate?	Low point in development area = ______
	Low point at stream invert = ______
	Total available head = ______
	Average depth, h_f = ______
	See subsections 4.3.6.4 and 4.3.6.5 - A
3. Confirm design criteria and applicability.	***See subsection 4.3.6.5 - J***
4. Compute WQv peak discharge (Q_{wq})	
Compute Curve Number	CN = ______
Compute Time of Concentration, t_c	t_c = ______ hour
Compute Q_{wq}	Q_{wq} = ______ cfs
5. Size flow diversion structure	
Low flow orifice - orifice equation	A = ______ ft^2
	diameter = ______ in
Overflow weir - Weir equation	Length = ______ ft
6. Size filtration bed chamber	
Compute area from Darcy's Law	A_f = ______ ft^2
Using length to width (2:1) ratio	L = ______ ft
	W = ______ ft
7. Size sedimentation chamber	
Compute area from Camp-Hazen equation	A_s = ______ ft^2
Given W from step 5, compute Length	L = ______
8. Compute V_{min}	V_{min} = ______ ft^3

Design Procedure Form: Sand Filters (continued)

9. Compute volume within practice

Surface Sand Filter

Volume within filter bed	V_f = ____ ft^3
Temporary storage above filter bed	$V_{f\text{-}temp}$ = ____ ft3
Sedimentation chamber (remaining volume)	V_s = ____ ft3
Height in sedimentation chamber	h_s = ____ ft
Perforated stand pipe - orfice equation	A = ____ ft^2
	diameter = ____ in

Perimeter Sand Filter

Compute volume in filter bed	V_f = ____ ft^3
Compute wet pool storage	V_w = ____ ft^3
Compute temporary storage	$V_{f\text{-}temp}$ = ____ ft^3
	h_{temp} = ____ ft

10. Compute overflow weir sizes

Compute overflow - Orifice equation	Q = ____ cfs
Weir from sedimentation chamber - Weir equation	Length = ____ ft
Weir from filtration chamber - Weir equation	Length = ____ ft

11. Verify peak flow control (if applicable), water quality drawdown time and channel protection detention time

4.3.6.10 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

Center for Watershed Protection. *Manual Builder.* Stormwater Manager's Resource Center, Accessed July 2005. *www.stormwatercenter.net*.

City of Knoxville. *Knoxville Best Management Practices Manual.* City of Knoxville Stormwater Engineering Division, March 2003.

Claytor, R.A., and T.R. Schueler. *Design of Stormwater Filtering Systems.* The Center for Watershed Protection, Silver Spring, MD, 1996.

Connecticut Department of Environmental Protection. *Stormwater Quality Manual.* 2004.

Minnesota Pollution Control Agency. *Minnesota Stormwater Manual.* Accessed January 2006. http://www.pca.state.mn.us/water/stormwater/stormwater-manual.html

New Jersey Department of Environmental Protection. *Stormwater Best Management Practices Manual.* 2004.

StormwaterAuthority.com. *Sand and Organic Filters.* Accessed January 2006. www.stormwaterauthority.com

4.3.6.11 Suggested Reading

Bell, W., L. Stokes, L.J. Gavan, and T. Nguyen. *Assessment of the Pollutant Removal Efficiencies of Delaware Sand Filter BMPs.* City of Alexandria, Department of Transportation and Environmental Services, Alexandria, VA, 1995.

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks,* 1993.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

Horner, R.R., and C.R. Horner. *Design, Construction, and Evaluation of a Sand Filter Stormwater Treatment System. Part II: Performance Monitoring.* Report to Alaska Marine Lines, Seattle, WA, 1995.

Metropolitan Washington Council of Governments (MWCOG). *A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone.* March, 1992.

Northern Virginia Regional Commission (NVRC). *The Northern Virginia BMP Handbook.* Annandale, VA, 1992.

Schueler, T.R. *Developments in Sand Filter Technology to Improve Stormwater Runoff Quality.* Watershed Protection Techniques 1(2):47-54, 1994.

US EPA. *Storm Water Technology Fact Sheet: Sand Filters. EPA 832-F-99-007.* Office of Water. 1999.

Young, G.K., S. Stein, P. Cole, T. Kammer, F. Graziano, and F. Bank. *Evaluation and Management of Highway Runoff Water Quality.* FHWA-PD-96-032. Federal Highway Administration, Office of Environment and Planning, 1996.

4.3.7 Infiltration Trench

General Application Stormwater BMP

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

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Soil infiltration rate of 0.5 in/hr or greater required.
- Excavated trench (3 to 8 foot depth) filled with stone media (1.5- to 2.5-inch diameter); pea gravel and sand filter layers.
- A sediment forebay and grass channel or equivalent pretreatment must be provided.
- Observation well to monitor percolation.
- 5 acre maximum drainage area.

ADVANTAGES / BENEFITS:

- Provides for groundwater recharge.
- Good for small sites with porous soils.

DISADVANTAGES / LIMITATIONS:

- Potential for groundwater contamination.
- High clogging potential; should not be used on sites with clays or silts in the drainage area.
- Significant setback requirements.
- Restrictions in karst areas.
- Two geotechnical test borings per trench required.

MAINTENANCE REQUIREMENTS:

- Inspect for clogging.
- Remove sediment from forebay.
- Replace pea gravel layer as needed.

STORMWATER MANAGEMENT SUITABILITY

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

Accepts Hotspot Runoff: *No*

✷ in certain situations

FEASIBILITY CONSIDERATIONS

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

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *Yes*

Drainage Area: *5 acres max.*

Soils: *Pervious soils required (0.5 in/hr or greater)*

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

POLLUTANT REMOVAL

H	**Total Suspended Solids**
M	**Nutrients** - Total Phosphorus / Total Nitrogen
H	**Metals** - Cadmium, Copper, Lead, and Zinc
H	**Pathogens** - Coliform, Streptococci, E.Coli

OTHER CONSIDERATIONS:

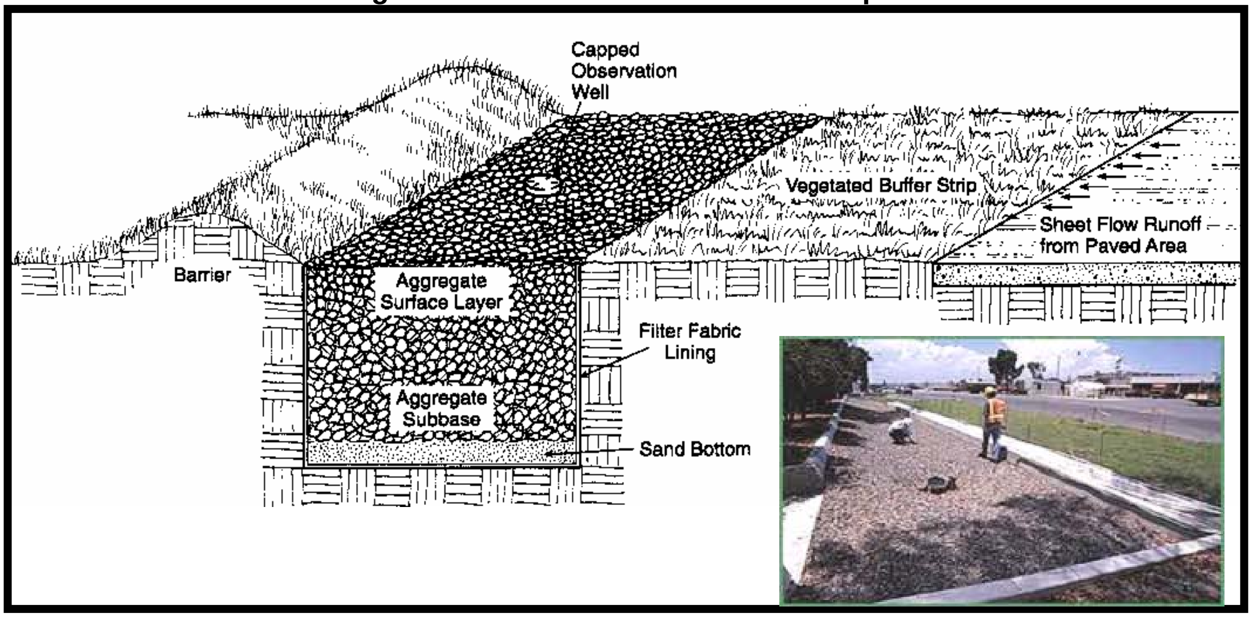
- Must not be placed under pavement or concrete

4.3.7.1 General Description

Infiltration trenches are excavations typically filled with stone to create an underground reservoir for stormwater runoff (see Figure 4-36). This runoff volume gradually infiltrates through the bottom and sides of the trench into the subsoil over a 2-day period and eventually reaches the water table. By diverting runoff into the soil, an infiltration trench not only treats the water quality volume, but also helps to preserve the natural water balance on a site and can recharge groundwater and preserve baseflow. Due to this fact, infiltration systems are limited to areas with highly porous soils where the water table and/or bedrock are located well below the bottom of the trench. In addition, infiltration trenches must be carefully sited to avoid the potential of groundwater contamination.

Infiltration trenches are not intended to trap sediment and must always be designed with a sediment forebay and grass channel or filter strip, or other appropriate pretreatment measures to prevent clogging and failure. Due to their high potential for failure, these facilities must only be considered for sites where upstream sediment control can be ensured.

Figure 4-36. Infiltration Trench Example



4.3.7.2 Stormwater Management Suitability

Infiltration trenches are designed primarily for stormwater quality. However, they can provide limited runoff quantity control, particularly for smaller storm events. For some smaller sites, trenches can be designed to capture and infiltrate the channel protection volume (CPv) in addition to the water quality volume (WQv). An infiltration trench will need to be used in conjunction with another structural BMP to provide overbank and extreme flood protection, if required.

<u>Water Quality (WQv)</u>
Using the natural filtering properties of soil, infiltration trenches can remove a wide variety of pollutants from stormwater through sorption, percolation, filtering, and bacterial and chemical degradation. Sediment load and other suspended solids are removed from runoff by pretreatment measures in the facility that treats flows before they reach the trench surface.

<u>Channel Protection (CPv)</u>
For smaller sites, an infiltration trench may be designed to capture and infiltrate the entire CPv in either an off- or on-line configuration. For larger sites, or where only the WQv is diverted to the trench, another structural BMP must be used to provide CPv extended detention.

<u>Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})</u>

Another structural BMP, such as a conventional detention pond, must be used in conjunction with an infiltration trench system to provide flood protection. All infiltration trench facilities must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the filter bed and facility. The volume of runoff removed and treated by the infiltration trench may be considered in the calculations for overbank and extreme flood protection. Volume 2, Chapter 3 of this manual will provide additional information on the design and sizing calculations.

4.3.7.3 Pollutant Removal Capabilities

An infiltration trench is presumed to be able to remove 90% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. The TSS removal performance is reduced for undersized, poorly designed, or unmaintained infiltration trenches.

Additionally, research has shown that use of infiltration trenches will have benefits beyond the removal of TSS, such as the removal of other pollutants (i.e., phosphorous, nitrogen, fecal coliform and heavy metals), as well, which is useful information should the pollutant removal criteria change in the future. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or “treatment train” approach.

- Total Suspended Solids – 90%
- Total Phosphorus – 60%
- Total Nitrogen – 60%
- Pathogens – 90%
- Heavy Metals – 90%

For additional information and data on pollutant removal capabilities for infiltration trenches, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the International Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

4.3.7.4 Application and Site Feasibility Criteria

Infiltration trenches are generally suited for medium-to-high density residential and commercial developments where the subsoil is sufficiently permeable to provide a reasonable infiltration rate and the water table is low enough to prevent groundwater contamination. They are applicable primarily for impervious areas where there are not high levels of fine particulates (clay/silt soils) in the runoff and should only be considered for sites where the sediment load is relatively low.

Infiltration trenches can either be used to capture sheet flow from a drainage area or function as an off-line device. Due to the relatively narrow shape, infiltration trenches can be adapted to many different types of sites and can be utilized in retrofit situations. Unlike some other structural stormwater BMPs, they can easily fit into the margin, perimeter, or other unused areas of developed sites.
To protect groundwater from potential contamination, infiltration trenches cannot be utilized to treat runoff from land uses that require a Special Pollution Abatement Permit without additional upstream best management practices (BMPs) that are designed to capture and/or treat the SPAP pollutants. Further, infiltration trenches will not be allowed for other pollutant hotspot land uses or activities, as identified by the Director of Engineering and Public Works (the Director). For example, infiltration trenches should not be used for manufacturing and industrial sites, where there is a potential for high concentrations of soluble pollutants and heavy metals, or for areas that may have a high pesticide concentration. Infiltration trenches are also not suitable in areas with karst geology without adequate geotechnical testing by qualified individuals.

The following criteria should be evaluated to ensure the suitability of an infiltration trench for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for use in a residential subdivision
- Suitable for use in high density/ultra-urban areas
- Not suitable for use as a regional (i.e., off-site or treating more than one site) stormwater control

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 5 acres maximum
- Space Required – Will vary depending on the depth of the facility
- Site Slope – No more than 6% slope (for pre-construction facility footprint) across the location of the infiltration trench
- Minimum Head – Elevation difference needed from the inflow of the infiltration trench to the outflow: 1 foot
- Minimum Depth to Water Table – 4 feet recommended between the bottom of the infiltration trench and the elevation of the seasonally high water table
- Soils – Infiltration rate greater than 0.5 inches per hour required (typically hydrologic group "A", some group "B" soils); to be verified with a minimum of two geotechnical borings or one boring per 5,000 SF, whichever is greater.

Other Constraints/Considerations

- Wellhead Protection – Reduce potential groundwater contamination (in required wellhead protection areas) by preventing infiltration of hotspot runoff. May require liner for type "A" and "B" soils; pretreat hotspots; provide 2 to 4 foot separation distance from water table.

4.3.7.5 Planning and Design Standards

The following standards are to be considered **minimum** standards for the design of an infiltration trench facility. Consult with Knox County Engineering to determine if there are any variations to these criteria or additional standards that must be followed.

A. CONSTRUCTION SEQUENCING

- Ideally, the construction of an infiltration trench should take place **after** the construction site has been stabilized to avoid deposition of sediment in the media. In the event that the trench is not constructed after site stabilization, diversion of site runoff around the trench and installation and maintenance of appropriate erosion prevention and sediment controls prior to site stabilization is required. Erosion prevention and sediment controls shall be maintained around the infiltration trench to prevent runoff and sediment from entering the trench during construction.
- Alternatively, the empty infiltration trench can be used as a sediment trap while the site is unstabilized. The media, however, cannot be placed into the trench until all sediment has been removed and the site completely stabilized.
- During and after excavation of the infiltration trench, all excavated materials shall be placed downstream, away from the trench, to prevent redeposit of the material during runoff events.

B. LOCATION AND SITING

- To be suitable for infiltration, underlying soils should have an infiltration rate (f_c) of 0.5 inches per hour or greater, as initially determined from NRCS soil textural classification, and subsequently confirmed by field geotechnical tests. The minimum geotechnical testing is one test hole per 5,000 square feet, with a minimum of two borings per facility (taken within the proposed limits of the facility). Infiltration trenches cannot be used in fill soils.
- Heavy equipment shall not be utilized in the area where the infiltration trench will be located. Soil compaction will adversely affect the performance of the trench. Infiltration trench sites should be roped-off and flagged during construction.

- During excavation and trench construction, only light equipment such as backhoes or wheel and ladder type trenchers should be used to minimize compaction of surrounding soils.
- Infiltration trenches should have a contributing drainage area of 5 acres or less.
- Soils on the drainage area tributary to an infiltration trench should have a clay content of less than 20% and a silt/clay content of less than 40% to prevent clogging and failure.
- There should be at least 4 feet between the bottom of the infiltration trench and the elevation of the seasonally high water table.
- Infiltration rate shall be based on the most restrictive clay lenses, bedrock or other restrictive layers between the trench bottom and water table.

Minimum setback requirements for infiltration trench facilities (when not specified by Knox County):

- ➢ From a building foundation – 25 feet
- ➢ From a private well – 100 feet
- ➢ From a public water supply well – 1,200 feet
- ➢ From a septic system tank/leach field – 100 feet
- ➢ From surface waters – 100 feet
- ➢ From surface drinking water sources – 400 feet (100 feet for a tributary)

- When used in an off-line configuration, the water quality volume (WQv) is diverted to the infiltration trench through the use of a flow splitter. Stormwater flows greater than the WQv are diverted to other controls or downstream using a diversion structure or flow splitter.
- To reduce the potential for costly maintenance and/or system reconstruction, it is strongly recommended that the trench be located in an open or lawn area, with the top of the structure as close to the ground surface as possible. Infiltration trenches shall not be located beneath paved surfaces, such as parking lots.
- Infiltration trenches are designed for intermittent flow and must be allowed to drain and allow reaeration of the surrounding soil between rainfall events. They must not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

C. GENERAL DESIGN

- A well-designed infiltration trench consists of:
  (1) Excavated shallow trench backfilled with sand, coarse stone, and pea gravel, and lined with a filter fabric;
  (2) Appropriate pretreatment measures; and
  (3) One or more observation wells to show how quickly the trench dewaters or to determine if the device is clogged.

Figure 4-37 provides a plan view and profile schematic for the design of an off-line infiltration trench facility. An example of an observation well is shown in Figure 4-38.

D. PHYSICAL SPECIFICATIONS / GEOMETRY

- The required trench storage volume is equal to the water quality volume (WQv). For smaller sites, an infiltration trench can be designed with a larger storage volume to include the channel protection volume (CPv).
- A trench must be designed to fully dewater the entire WQv within 24 to 48 hours after a rainfall event. The slowest infiltration rate obtained from tests performed at the site should be used in the design calculations.

- Trench depths should be between 3 and 8 feet, to provide for easier maintenance. The width of a trench must be less than 25 feet.
- Broad, shallow trenches reduce the risk of clogging by spreading the flow over a larger area for infiltration.
- The surface area required is calculated based on the trench depth, soil infiltration rate, aggregate void space, and fill time (assume a fill time of 2 hours for most designs).
- The bottom slope of a trench should be flat across its length and width to evenly distribute flows, encourage uniform infiltration through the bottom, and reduce the risk of clogging.
- The stone aggregate used in the trench should be washed, bank-run gravel, 1.5 to 2.5 inches in diameter with a void space of about 40%. Aggregate contaminated with soil shall not be used. A porosity value (void space/total volume) of 0.32 should be used in calculations, unless aggregate specific data exist.
- A 6-inch layer of clean, washed sand is placed on the bottom of the trench to encourage drainage and prevent compaction of the native soil while the stone aggregate is added.
- The infiltration trench is lined on the sides and top by an appropriate geotextile filter fabric that prevents soil piping but has greater permeability than the parent soil. The top layer of filter fabric is located 2 to 6 inches from the top of the trench and serves to prevent sediment from passing into the stone aggregate. Since this top layer serves as a sediment barrier, it will need to be replaced more frequently and must be readily separated from the side sections.
- The top surface of the infiltration trench above the filter fabric is typically covered with pea gravel. The pea gravel layer improves sediment filtering and maximizes the pollutant removal in the top of the trench. In addition, it can easily be removed and replaced should the device begin to clog. Alternatively, the trench can be covered with permeable topsoil and planted with grass in a landscaped area.
- An observation well must be installed in every infiltration trench and should consist of a perforated PVC pipe, 4 to 6 inches in diameter, extending to the bottom of the trench. The observation well will show the rate of dewatering after a storm, as well as provide a means of determining sediment levels at the bottom and when the filter fabric at the top is clogged and maintenance is needed. It should be installed along the centerline of the structure, flush with the ground elevation of the trench. A visible floating marker should be provided to indicate the water level. The top of the well should be capped and locked to discourage vandalism and tampering.
- The trench excavation should be limited to the width and depth specified in the design. Excavated material should be placed away from the open trench so as not to jeopardize the stability of the trench sidewalls. The bottom of the excavated trench shall not be loaded in a way that causes soil compaction, and should be scarified prior to placement of sand. The sides of the trench shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling. All infiltration trench facilities should be protected during site construction and should be constructed after upstream areas have been stabilized.
- Smearing of the soil at its interface with the trench bottom or sides must be avoided or corrected. Smearing can be corrected by raking or roto-tilling.

E. PRETREATMENT / INLETS

- Pretreatment facilities **must always** be used in conjunction with an infiltration trench to prevent clogging and failure.
- For a trench receiving sheet flow from an adjacent drainage area, the pretreatment system should consist of a vegetated filter strip with a minimum 25-foot length. A vegetated buffer strip around the entire trench is required if the facility is receiving runoff from both directions. If the infiltration rate for the underlying soils is greater than 2 inches per hour, 50% of the WQv should be pretreated by another method prior to reaching the infiltration trench.

- For an off-line configuration, pretreatment should consist of a sediment forebay, vault, plunge pool, or similar sedimentation chamber (with energy dissipators) sized to 25% of the water quality volume (WQv). Exit velocities from the pretreatment chamber must be non-erosive for the 2-year design storm.

F. OUTLET STRUCTURES

- Outlet structures are not required for infiltration trenches.

G. EMERGENCY SPILLWAY

- Typically for off-line designs, there is no need for an emergency spillway. However, a non-erosive overflow channel should be provided to safely pass flows that exceed the storage capacity of the trench to a stabilized downstream area or watercourse.

H. MAINTENANCE ACCESS

- A minimum 20' wide maintenance right-of-way or drainage easement shall be provided for an infiltration trench, from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles. Adequate access must be provided to the grates to the filter bed for perimeter sand filter design. Facility designs must enable maintenance personnel to easily remove and replace upper layers of the infiltration media.

I. SAFETY FEATURES

- In general, infiltration trenches are not likely to pose a physical threat to the public and do not need to be fenced.

J. LANDSCAPING

- Vegetated filter strips and buffers should fit into and blend with surrounding area. Native grasses are preferable, if compatible. The trench may be covered with permeable topsoil and planted with grass in a landscaped area

K. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

There are a number of additional site specific design criteria and issues (listed below) that must be considered in the design of a infiltration trench.

Physiographic Factors - Local terrain design constraints

- High Relief – Maximum site slope of 6%
- Karst – Not suitable without adequate geotechnical testing

Special Downstream Watershed Considerations

- Wellhead Protection – Reduce potential groundwater contamination (in required wellhead protection areas) by preventing infiltration of hotspot runoff. May require liner for type "A" and "B" soils; Pretreat hotspots; provide a minimum 4 foot separation distance from water table.

4.3.7.6 Design Procedures

Step 1. Compute runoff control volumes

> Calculate WQv, CPv, Qp_2, Qp_{10}, Qp_{25}, and Qp_{100}, in accordance with the guidance presented in Volume 2, Chapter 3.

Step 2. Determine if the development site and conditions are appropriate for the use of an infiltration trench.

> Consider the Application and Site Feasibility Criteria in subsections 4.3.7.4 and 4.3.7.5-B (Location and Siting).

Step 3. Confirm design criteria and applicability

Consider any special site-specific design conditions/criteria from subsection 4.3.7.5-K (Additional Site-Specific Design Criteria and Issues).

Check with Knox County and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4. Compute WQv peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion (see Volume 2, Chapter 3 for more information).

Step 5. Size flow diversion structure, if needed

A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQv to the infiltration trench.

Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 6. Size infiltration trench

The area of the trench can be determined from the following equation:

$$A = \frac{WQv}{(nd + kT/12)}$$

where:

A = Surface Area
WQv = Water Quality Volume (or total volume to be infiltrated)
n = porosity
d = trench depth (feet)
k = percolation (inches/hour)
T = Fill Time (time for the practice to fill with water), in hours

A porosity value n = 0.32 should be used.

All infiltration systems should be designed to fully dewater the entire WQv within 24 to 48 hours after the rainfall event.

A fill time T=2 hours can be used for most designs.
See subsection 4.3.7.5-D (Physical Specifications/Geometry) for more specifications.

Step 7. Determine pretreatment volume and design pretreatment measures

Size pretreatment facility to treat 25% of the water quality volume (WQv) for off-line configurations. See subsection 4.3.7.5-E (Pretreatment / Inlets) for more details.

Step 8. Design spillway(s)

Adequate stormwater outfalls should be provided for the overflow exceeding the capacity of the trench, ensuring non-erosive velocities on the down-slope.

4.3.7.7 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.7.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of an infiltration trench as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for infiltration trenches, along with a suggested frequency for each activity. Individual trenches may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e.., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the infiltration trench in proper operating condition at all times.

Inspection Activities	Suggested Schedule
• A record should be kept of the dewatering time (i.e., the time required to drain the infiltration trench completely after a storm event) of the trench to determine if maintenance is necessary. The trench should drain completely in about 24 hours after the end of the rainfall. Ponded water inside the trench (as visible from the observation well or on the surface) longer than 48 hours or several days after a storm event is an indication that the trench is clogged.	After Rain Events
• Check that the area draining to the trench, the trench and its inlets are clear of debris. • Check the area draining to the trench for evidence of erosion.	Monthly
• Check observation wells following 3 days of dry weather. Failure to percolate within this time period indicates clogging. • Inspect pretreatment devices and diversion structures for sediment build-up and structural damage.	Semi-annual Inspection
Maintenance Activities	**Suggested Schedule**
• Remove sediment and oil/grease from pretreatment devices, as well as overflow structures. • Mow grass filter strips as necessary. Remove grass clippings.	Monthly
• Remove trees that start to grow in the vicinity of the trench.	Semi-annual Inspection
• Replace pea gravel/topsoil and top surface filter fabric (when clogged). Removed sediment and media may usually be disposed of in a landfill. • Stabilize (i.e., vegetate or cover) areas of erosion in the area draining to the trench.	As needed
• Perform total rehabilitation of the trench to maintain design storage capacity. • Excavate trench walls to expose clean soil.	Upon Failure

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of an infiltration trench. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the trench. Questions regarding stormwater facility inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
INFILTRATION TRENCH INSPECTION CHECKLIST

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ____

Date: ___________ Time: ______________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Inspection List		
Complete drainage of the filter in about 24 to 48 hours after a rain event?		
Clogging of trench surface?		
Clogging of inlet/outlet structures?		
Standing water in observation well after three rainless days?		
Trench clear of debris and functional?		
Evidence of leaks or seeps?		
Animal burrows in trench?		
Cracking, spalling, bulging or deterioration of concrete?		
Erosion in area draining to trench?		
Erosion around inlets, trench, or outlets?		
Pipes and other structures in good condition?		
Undesirable vegetation growth?		
Other (describe)?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature:________________________________ Inspector Name (printed)____________________________

4.3.7.8 Example Schematics

Figure 4-37. Schematic of an Infiltration Trench
(Source: Center for Watershed Protection)

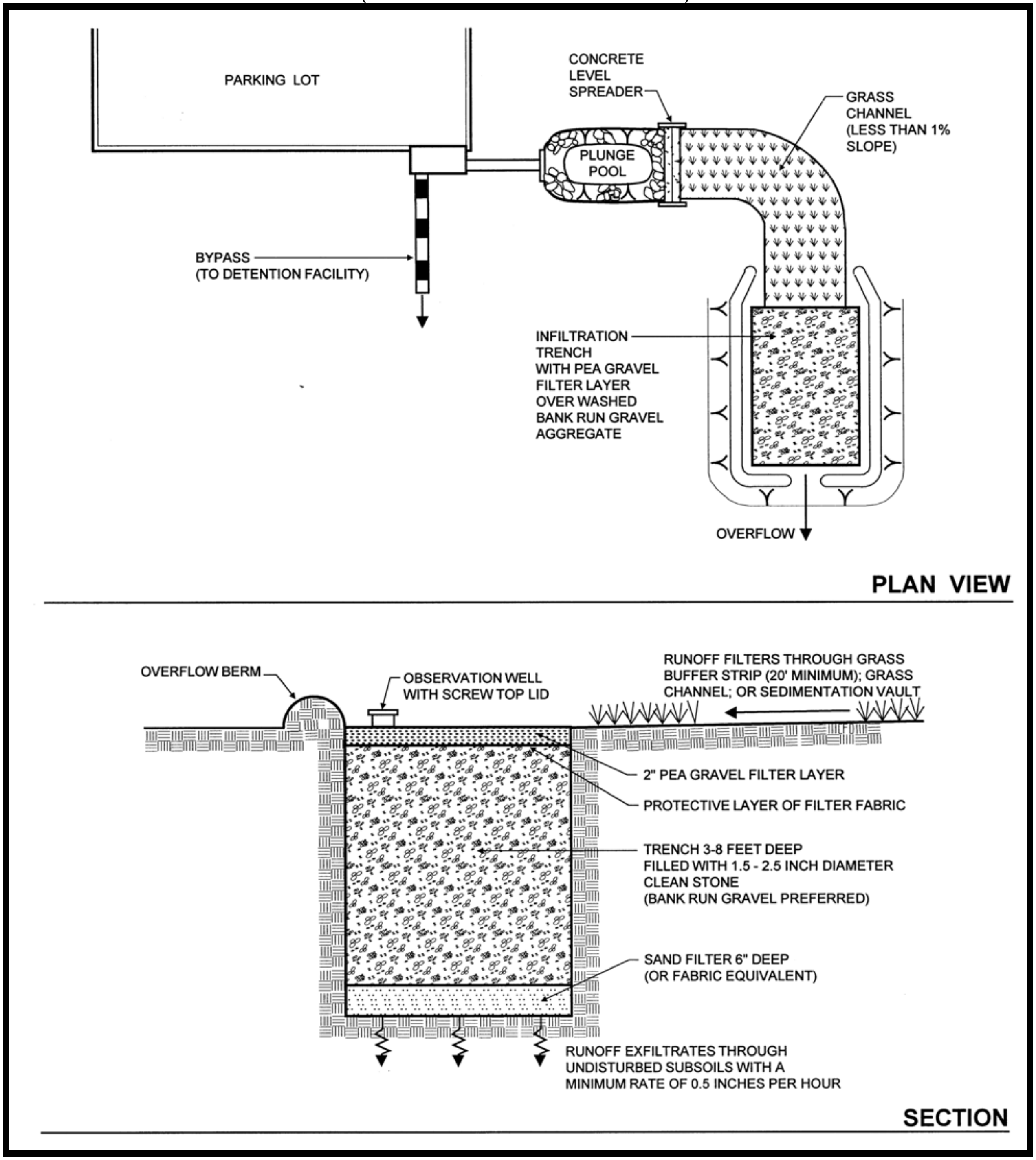
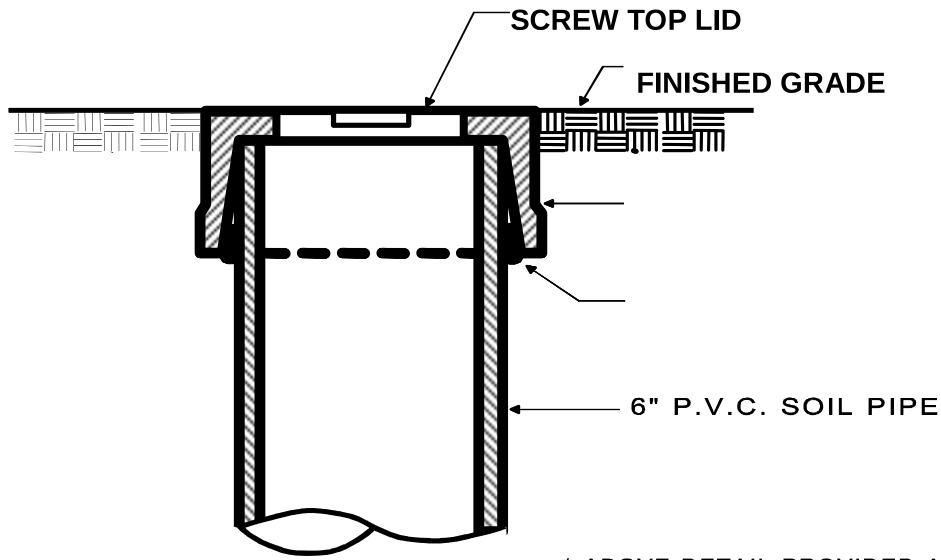


Figure 4-38. Observation Well Detail



* ABOVE DETAIL PROVIDED AS SCHEMATIC
SCREW TOP P.V.C. WELL CAP ONLY

EACH OBSERVATION WELL / CLEANOUT SHALL INCLUDE THE FOLLOWING:

1. FOR AN UNDERGROUND FLUSH MOUNTED OBSERVATION WELL / CLEANOUT, PROVIDE A TUBE MADE OF NON-CORROSIVE MATERIAL, SCHEDULE 40 OR EQUAL, AT LEAST THREE FEET LONG WITH AN INSIDE DIAMETER OF AT LEAST 6 INCHES.

2. THE TUBE SHALL HAVE A FACTORY ATTACHED CAST IRON OR HIGH IMPACT PLASTIC COLLAR WITH RIBS TO PREVENT ROTATION WHEN REMOVING SCREW TOP LID. THE SCREW TOP LID SHALL BE CAST IRON OR HIGH IMPACT PLASTIC THAT WILL WITHSTAND ULTRA-VIOLET RAYS.

4.3.7.9 Design Form

Knox County recommends the use of the following design procedure forms when designing infiltration trenches. Proper use and completion of the form may allow a faster review of the Stormwater Management Plan by Knox County Engineering.

Design Procedure Form: Infiltration Trench

PRELIMINARY HYDROLOGIC CALCULATIONS	
1a. Compute WQv volume requirements	
Compute Runoff Coefficient, R_v	Rv = ________
Compute WQv	WQv = ________ acre-ft
1b. Estimate CPv	CPv = ________ acre-ft
1c. Estimate storage volumes	release rate = ________ cfs
Estimate storage volume required for 2-year storm	2-year storage = ________ acre-ft
Estimate storage volume required for 10-year storm	10-year storage = ________ acre-ft
Estimate storage volume required for 25-year storm	25-year storage = ________ acre-ft
Estimate storage volume required for 100-year storm	100-year storage = ________ acre-ft
INFILTRATION TRENCH DESIGN	
2. Is the use of an infiltration trench appropriate?	***See subsections 4.3.7.4 and 4.3.7.5 - B***
3. Confirm local design criteria and applicability.	***See subsection 4.3.7.5 - K***
4. Compute WQv peak discharge (Q_{wq})	
Compute Curve Number	CN = ________
Compute Time of Concentration t_c	t_c = ________ hour
Compute Q_{wq}	Q_{wq} = ________ cfs
5. Size infiltration trench	Area = ________ ft^2
Width must be less than 25 ft	Width = ________ ft
	Length = ________ ft
6. Size the flow diversion structures	
Low flow orifice from orifice equation	
$Q = CA(2gh)^{0.5}$	A = ________ ft^2
C varies with orifice condition	diam. = ________ inch
Overflow weir from weir equation	
$Q = CLH^{3/2}$	Length = ________ ft
7. Pretreatment volume (for offine designs)	
$Vol_{pre} = 0.25(WQv)$	Vol_{pre} = ________ ft^3
8. Design spillway(s)	
9. Verify peak flow control, water quality drawdown time and channel protection detention time	

4.3.7.10 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

Center for Watershed Protection. *Manual Builder.* Stormwater Manager's Resource Center, Accessed July 2005. *www.stormwatercenter.net*.

Federal Highway Administration (FHWA). *Stormwater Best Management Practices in an Ultra-Urban Setting: Selection and Monitoring.* United States Department of Transportation, Accessed January 2006. http://www.fhwa.dot.gov/environment/ultraurb/index.htm

4.3.7.11 Suggested Reading

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

City of Austin, TX. *Water Quality Management.* Environmental Criteria Manual, Environmental and Conservation Services, 1988.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

Claytor, R.A., and T.R. Schueler. *Design of Stormwater Filtering Systems.* The Center for Watershed Protection, Silver Spring, MD, 1996.

US EPA. *Storm Water Technology Fact Sheet: Storm Water Wetlands.* EPA 832-F-99-025. Office of Water, 1999.

Faulkner, S. and C. Richardson. *Physical and Chemical Characteristics of Freshwater Wetland Soils.* Constructed Wetlands for Wastewater Treatment, ed. D. Hammer, Lewis Publishers, 831 pp, 1991.

Guntenspergen, G.R., F. Stearns, and J. A. Kadlec. *Wetland Vegetation.* Constructed Wetlands for Wastewater Treatment, ed. D. A. Hammer, Lewis Publishers, 1991.

Maryland Department of the Environment. *Maryland Stormwater Design Manual, Volumes I and II.* Prepared by Center for Watershed Protection (CWP), 2000.

Metropolitan Washington Council of Governments (MWCOG). *A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone.* March, 1992.

4.3.8 Enhanced Swales

General Application Stormwater BMP

Description: Enhanced swales are vegetated open channels that are explicitly designed and constructed to capture and treat stormwater runoff within dry or wet cells formed by check dams or other means.

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Maximum contributing drainage area of 5 acres.
- Longitudinal slopes must be less than 4%.
- Bottom width of 2 to 8 feet.
- Side slopes 2:1 or flatter; 4:1 recommended.
- Convey the 25-year storm event with a minimum of 6 inches of freeboard.

ADVANTAGES / BENEFITS:

- Combines stormwater treatment with runoff conveyance system.
- Less expensive than curb and gutter.
- Reduces runoff velocity and the potential for channel/ditch erosion.

DISADVANTAGES / LIMITATIONS:

- Higher maintenance than curb and gutter.
- Cannot be used on steep slopes.
- Possible resuspension of sediment.
- Potential for odor / mosquitoes (wet swale).

MAINTENANCE REQUIREMENTS:

- Maintain grass heights of approximately 4 to 6 inches (dry swale).
- Sediment removal from forebay and channel.

STORMWATER MANAGEMENT SUITABILITY

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

Accepts runoff from SPAP land uses: *Yes (requires impermeable liner)*

✷ in certain situations

FEASIBILITY CONSIDERATIONS

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

Residential Subdivision Use: *Yes*

High Density/Ultra-Urban: *No*

Drainage Area: *5 acres max.*

Soils: *No restrictions*

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

POLLUTANT REMOVAL (DRY SWALE)

H	**Total Suspended Solids**
M	**Nutrients** - Total Phosphorus / Total Nitrogen
M	**Metals** - Cadmium, Copper, Lead, and Zinc
No data	**Pathogens** - Coliform, Streptococci, E.Coli

OTHER CONSIDERATIONS:

- Permeable soil layer (dry swale)
- Wetland plants (wet swale)

4.3.8.1 General Description

Enhanced swales (also referred to as *vegetated open channels* or *water quality swales*) are conveyance channels engineered to capture and treat the water quality volume (WQv) for a drainage area. They differ from a normal drainage channel or conventional swale because they incorporate specific features that enhance stormwater pollutant removal effectiveness.

Enhanced swales are designed with limited longitudinal slopes to force the stormwater flow to be slow and shallow, thus allowing for particulates to settle and limiting the effects of erosion. Berms and/or check dams installed perpendicular to the flow path promote settling and infiltration.

There are two primary enhanced swale designs, the *dry swale* and the *wet swale* (or *wetland channel*). Figure 4-39 depicts both types of enhanced swales.

Figure 4-39. Enhanced Swale Examples

Enhanced Dry Swale

Enhanced Wet Swale

- **Dry Swale** – The dry swale is a vegetated conveyance channel designed to include a filter bed of prepared soil that overlays an underdrain system. Dry swales are sized to allow the entire WQv to be filtered or infiltrated through the bottom of the swale. Because they are dry most of the time, they are often the preferred option in residential settings.
- **Wet Swale (Wetland Channel)** – The wet swale is a vegetated channel designed to retain water or marshy conditions that support wetland vegetation. A high water table or poorly drained soils are necessary to retain water. The wet swale essentially acts as a linear shallow wetland treatment system, where the WQv is retained.

Enhanced swales must not to be confused with a *filter strip* or *grass channel*, because they afford a much higher level of water quality treatment than the latter BMPs. Ordinary *grass channels* are not engineered to provide the same treatment capability as a well-designed dry swale with filter media. *Filter strips* are designed to accommodate overland flow rather than channelized flow and can be used as stormwater credits to help reduce the total water quality treatment volume for a site. Both of these practices may be used for pretreatment or included in a "treatment train" approach where redundant treatment is provided. Please see a further discussion of these structural controls in subsections 4.3.9 and 4.3.10, respectively.

4.3.8.2 Stormwater Management Suitability

Enhanced swale systems are designed primarily for stormwater quality and have only a limited ability to provide channel protection or flood protection.

<u>Water Quality (WQv) and Channel Protection (CPv)</u>
Dry swale systems rely primarily on filtration through an engineered media to provide removal of stormwater contaminants. Wet swales achieve pollutant removal both from sediment accumulation and biological removal. Generally only the WQv is treated by a dry or wet swale, and another structural BMP must be used to provide extended detention of the CPv. However, for some smaller sites, a swale may be designed to capture and detain the full CPv.

Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})
Enhanced swales will not provide overbank or extreme flood protection. Another structural BMP, such as a detention pond, must be used in conjunction with an enhanced swale system to achieve the Qp_2, Qp_{10}, Qp_{25} and Qp_{100} design criteria. However, because enhanced swales will typically be located upstream of detention facilities, they must be designed to provide a flow diversion for the WQv, and/or be designed to safely pass the post-development peak flow of the 25-year and 100-year storms in accordance with Knox County design regulations.

4.3.8.3 Pollutant Removal Capabilities

The dry enhanced swale is presumed to be able to remove 90% of the TSS load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. The TSS removal value for wet swales is 75%. Undersized or poorly designed swales can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or “treatment train” approach.

- Total Suspended Solids – Dry Swale 90% / Wet Swale 75%
- Total Phosphorus – Dry Swale 50% / Wet Swale 25%
- Total Nitrogen – Dry Swale 50% / Wet Swale 40%
- Pathogens – Insufficient data to provide a pollutant removal value
- Heavy Metals – Dry Swale 40% / Wet Swale 20%

Additionally, research has shown that use of enhanced swales will have benefits beyond the removal of TSS, such as the removal of other pollutants (i.e. phosphorous, nitrogen, fecal coliform and heavy metals), as well, which is useful information should the pollutant removal criteria change in the future. For additional information and data on pollutant removal capabilities for enhanced dry and wet swales, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the International Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

4.3.8.4 Application and Feasibility Criteria

Enhanced swales can be used in a variety of development types; however, they are primarily applicable to residential and commercial areas of low to moderate density where the impervious cover in the contributing drainage area is relatively small, and along roads and highways. Dry swales are mainly used in moderate to large lot residential developments, small impervious areas (parking lots and rooftops), and along rural highways. Wet swales tend to be used for highway runoff applications, small parking areas, and in commercial developments as part of a landscaped area.

Because of their relatively large land requirement, enhanced swales are generally not used in higher density areas. In addition, wet swales may not be desirable for some residential applications, due to the presence of standing and stagnant water, which may create nuisance odor or mosquito problems.

The topography and soils of a site will determine the applicability of one of the two enhanced swale designs. Overall, the topography should allow for the design of a swale with sufficient slope and cross-sectional area to maintain non-erosive velocities. The following criteria should be evaluated to ensure the suitability of enhanced swales for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for use in residential subdivisions and in non-residential areas.
- Not generally suitable for high density/ultra-urban areas, as land requirements may preclude their use.
- Not suitable for use as a regional stormwater control.

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 5 acres maximum
- Space Required – Approximately 10 to 20% of the tributary impervious area
- Channel Slope – Channel slope shall not exceed 4%
- Minimum Head – Elevation difference needed between any two berms/checkdams: 3 to 5 feet for dry swale; 1 foot for wet swale
- Minimum Depth to Water Table – 2 feet required between the bottom of a dry swale and the elevation of the seasonally high water table, if an aquifer or treating a stormwater discharging from a land use that requires a special pollution abatement permit (SPAP); wet swale is below water table or placed in poorly drained soils
- Soils – Engineered media for dry swale

Other Constraints / Considerations

- Aquifer Protection – Exfiltration from the enhanced swale should be prevented in enhanced swales that serve SPAP land uses.

4.3.8.5 Planning and Design Standards

The following standards shall be considered **minimum** design standards for the design of an enhanced swale. Enhanced swales that are not designed to these standards will not be approved. The Director shall have the authority to require additional design conditions if deemed necessary.

A. LOCATION AND SITING

- A dry or wet swale shall be located on a property such that the topography allows for the design of a channel with sufficiently mild slope, as discussed in part C below (unless small drop structures are used), and sufficient cross-sectional area to maintain non-erosive velocities. Site designers shall also take into account the location and use of other site features, such as buffers and undisturbed natural areas when determining the location of an enhanced swale, and should attempt to aesthetically “fit” the facility into the landscape.
- Enhanced swale systems shall have a contributing drainage area of 5 acres or less.
- A wet swale shall only be used where the water table is at or near the soil surface, or where there is a sufficient water balance in poorly drained soils to support a wetland plant community.

B. GENERAL DESIGN

- Enhanced swales that are located “on-line” shall also be designed to safely pass larger flows in accordance with Knox County’s design criteria for open channels (Chapter 2). Flow enters the channel through a pretreatment forebay. Runoff can also enter along the sides of the channel as sheet flow through the use of a pea gravel flow spreader trench located along the top of the bank of the swale.

Dry Swale

- A dry swale system shall consist of an open conveyance channel with a filter bed of permeable soils that overlay an underdrain system. Flow passes into and is detained in the main portion of the channel where it is filtered through the soil bed. Runoff is collected and conveyed by a perforated pipe and gravel underdrain system to the outlet. Figure 4-40 presented at the end of this section provides a plan view and profile schematic for the design of a dry swale system.

Wet Swale

- A wet swale or wetland channel shall consist of an open conveyance channel which has been excavated to the water table or to poorly drained soils. Check dams are used to create multiple wetland “cells,” which act as miniature shallow marshes. Figure 4-41 presented at the end of this section provides a plan view and profile schematic for the design of a wet swale system.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

General

- The enhanced swale shall have a minimum slope of 1%, and the slope shall not exceed 4%. A 1% to 2% slope is considered ideal. Where topography necessitates a slope steeper than 2%, 6 to 12-inch drop structures must be designed and constructed to limit the energy slope to within the recommended 1 to 2% range. Energy dissipation is required below the drops. The drops shall be spaced a minimum of 50 feet apart.
- The maximum WQv ponding depth in the enhanced swale shall not exceed 18 inches at the end point of the swale. An average depth of 12-inches shall be maintained.
- Enhanced swales shall have a bottom width ranging from 2 to 8 feet to ensure adequate filtration. Wider channels will be permitted, but must contain berms, walls, or a compound cross-section to prevent channel braiding or uncontrolled sub-channel formation.
- Enhanced swales shall have a trapezoidal or compound cross-section. Side slopes shall not exceed 2:1. The Director may approve side slopes up to 4:1 where side inflows by sheet flow will not be substantial, and where such swales can be easily maintained. Side slopes greater than 2:1 in residential areas are strongly discouraged.
- Enhanced swales shall be designed such that the peak velocity for the 2-year storm must be conveyed in a non-erosive manner, given the soil and vegetative cover provided.
- If the enhanced swale is on-line, the swale shall be sized to convey runoff for the 2, 10, and 25-year storms safely with a minimum of 6 inches of freeboard for the Qp_{25}.

Dry Swale

- Dry swale channels shall be sized to store and infiltrate the entire water quality volume (WQv) with less than 18 inches of ponding and allow for full filtering through the permeable soil layer. Ponding shall occur for no longer than 48 hours, though a 24-hour ponding time is more desirable.
- The bed of a dry swale shall consist of a permeable soil layer of at least 30 inches in depth, above a 4-inch diameter perforated longitudinal underdrain (PVC AASHTO M 252, HDPE or other suitable underdrain pipe material) in a 6-inch gravel layer. The soil media shall have an infiltration rate of at least 1 foot per day (1.5 feet per day maximum) and contain a high level of organic material to facilitate pollutant removal. A permeable filter fabric shall be placed between the gravel layer and the overlying soil.
- Excavation of the dry swale and its associated underdrain shall be limited to the width and depth specified in the design. The bottom of the excavated trench shall not be loaded in a way that causes soil compaction, and shall be scarified prior to placement of gravel and permeable soil. The sides of the channel shall be trimmed of all large roots. The sidewalls shall be uniform with no voids and scarified prior to backfilling.

Wet Swale

- Wet swale channels are sized to retain the entire water quality volume (WQv) with less than 18 inches of ponding at the maximum depth point.
- Check dams can be used to achieve multiple wetland cells. V-notch weirs in the check dams can be utilized to direct low flow volumes.

D. PRETREATMENT / INLETS

- Inlets to enhanced swales shall include energy dissipators, such as riprap.
- Pretreatment of runoff in both a dry and wet swale system shall be provided by a sediment forebay located at the inlet. The pretreatment volume shall be equal to 0.1 inches per impervious acre (363 ft^3). This storage can be obtained by providing check dams at pipe inlets and/or driveway crossings.

- Enhanced swale systems that receive direct concentrated runoff (as opposed to shallow concentrated or overland flow) shall have a 6-inch drop to a pea gravel diaphragm flow spreader at the upstream end of the control.
- A pea gravel diaphragm and gentle side slopes shall be provided along the top of channels to provide pretreatment for lateral sheet inflows.

E. OUTLET STRUCTURES

Dry Swale

- The underdrain system shall discharge to the storm drainage infrastructure or a stable outfall in a non-erosive manner.

Wet Swale

- Outlet protection shall be used at any discharge point from a wet swale to prevent scour and downstream erosion.

F. MAINTENANCE ACCESS

- A minimum 20' wide maintenance right-of-way or drainage easement shall be provided for the length of the enhanced swale from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles. The right-of-way shall be located such that maintenance vehicles and equipment can access the entire enhanced swale.

G. LANDSCAPING

- The stormwater management plan shall specify the landscape design of the enhanced swale, and shall include appropriate grass species and/or wetland plants based on specific site, soils and hydric conditions present along the swale. Vegetation shall be limited to grasses and non-woody wetland plants. Trees and other large woody plant species are not appropriate for use in an enhanced swale and are prohibited.

Dry Swale

- Turf grasses that require minimal maintenance shall be used in dry swales. Native grasses are preferred, but not required. Maintenance of the turf grasses shall be performed as appropriate to maintain a stable and viable coverage of the swale bottom and side slopes.

Wet Swale

- At the time of construction, emergent vegetation shall be planted in the swale, or wetland soils may be spread on the swale bottom for seed stock. More information on wetland plants can be found at the following websites:
    - http://wetlands.fws.gov/
    - http://www.npwrc.usgs.gov/resource/plants/floraso/species.htm
    - http://www.tva.gov/river/landandshore/stabilization/plantsearch.htm
- Where wet swales do not intercept the groundwater table, a water balance calculation shall be performed to ensure an adequate water budget to support the specified wetland species. See Volume 2, Chapter 3 of the Knox County Stormwater Management Manual for guidance on water balance calculations.

H. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

There are a number of additional site specific design criteria and issues (listed below) that must be considered in the design of an enhanced swale.

Physiographic Factors - Local terrain design constraints

- Low Relief – Reduced need for use of check dams
- High Relief – Not feasible if slopes are greater than 4%
- Karst – No exfiltration of runoff from dry swales located in SPAP land uses; an impermeable liner shall be utilized for swales that control stormwater discharges from SPAP land uses.

Special Downstream Watershed Considerations

- Wellhead Protection – Reduce potential groundwater contamination (in required wellhead protection areas) by preventing infiltration of runoff from land uses that have a high pollution potential. May require liner for type "A" and "B" soils; Pretreat runoff from polluted areas and land uses that require a SPAP; 2 to 4 foot separation distance from water table

4.3.8.6 Design Procedures

Step 1. Compute appropriate runoff control volumes and peak discharges

Calculate WQv, CPv, Qp_2, Qp_{10}, Qp_{25}, and Qp_{100}, in accordance with the guidance presented in Volume 2, Chapter 3.

Step 2. Determine if the development site and conditions are appropriate for the use of an enhanced swale system (dry or wet swale).

Consider the Application and Site Feasibility Criteria in subsections 4.3.8.4 and 4.3.8.5-A (Location and Siting). Check with Knox County and other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 3. Determine pretreatment volume

The sediment forebay should be sized to contain 0.1 inches per impervious acre (363 $ft^{3/}$acre) of contributing drainage. The forebay storage volume counts toward the total WQv requirement, and should be subtracted from the WQv for subsequent calculations.

Step 4. Determine swale dimensions and compute number of check dams (or similar structures) required to detain WQv as per the above stated design criteria.

Size bottom width, depth, length, and slope necessary to store WQv with less than 18 inches of ponding at the downstream end.

- Slope cannot exceed 4% (1 to 2% recommended)
- Bottom width should range from 2 to 8 feet
- Ensure that side slopes are no greater than 2:1

Step 5. Calculate draw-down time

Dry swale: Planting soil should pass a maximum rate of 1.5 feet in 24 hours and must completely filter WQv within 48 hours.

Wet swale: Must hold the WQv.

Step 6. Check 2-year and 25-year velocity erosion potential and freeboard

Check for erosive velocities and modify design as appropriate. Provide 6 inches of freeboard for the 25-year event.

Step 7. Design low flow orifice at downstream headwalls and checkdams

Design orifice to pass WQv in six hours.

Step 8. Design inlets, sediment forebay(s), and underdrain system (dry swale)

See design criteria above for further details.

Step 9. Prepare Vegetation and Landscaping Plan

A landscaping plan for a dry or wet swale shall be submitted with the stormwater management plan that indicates the vegetation proposed for the swale, and how the enhanced swale system will be stabilized and established with vegetation.

4.3.8.7 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.8.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of enhanced swales as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for enhanced swales, along with a suggested frequency for each activity. Individual enhanced swales may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e.., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the swale in proper operating condition at all times.

Inspection Activities	Suggested Schedule
• Inspect after seeding and after first major storm for any damage to vegetation, side slopes and bottom.	Post construction
• Inspect for signs of erosion, unhealthy or damaged vegetation, denuded areas, channelization of flow, debris and litter, and areas of sediment accumulation. Perform inspections at the beginning and end of the wet season. Additional inspections after periods of heavy rainfall are desirable.	Semi-annually
• Inspect level spreader for clogging (if applicable), grass along side slopes for erosion and formation of rills or gullies, and sand/soil bed for erosion problems. Inspect pea gravel diaphragm for clogging. • Inspect sediment forebays and/or pretreatment areas for debris and sediment accumulation.	Annually
Maintenance Activities	**Suggested Schedule**
• Mow grass to maintain a height of 3–4 inches, for safety, aesthetic, or other purposes, if needed. Litter should always be removed prior to mowing. Grass clippings, if captured, should not be dumped in the swale. • Irrigate swale during dry season (April through October) or when necessary to maintain the vegetation. • Repair damaged areas (e.g., erosion rills or gullies) and re-establish vegetation where needed. Remove invasive species manually. The use of fertilizers, herbicides and pesticides should occur only when absolutely necessary, and then in minimal amounts.	As needed (frequent, seasonally)
• Remove litter, branches, rocks blockages, and other debris and dispose of properly. • Clear accumulated debris and sediment from the inlet flow spreader (if applicable) and pea gravel diaphragm.	Semi-annually
• Inspect pea gravel diaphragm for clogging and correct the problem. • Plant an alternative grass species if the original grass cover has not been successfully established. Reseed and apply mulch to damaged areas.	Annually (if needed)
• Remove all accumulated sediment that may obstruct flow through the swale. Sediment accumulating near culverts and in channels should be removed when it builds up to 3 in. at any spot, or covers vegetation, or once it has accumulated to 10% of the original design volume. Replace the grass areas damaged in the process. • Remove all accumulated sediment in the sediment forebay and pretreatment areas. • Repair areas of erosion around swale and underdrain outlets. Reestablish soil stabilization measures (e.g., rip-rap stone, turf grasses) as needed. • Rototill or cultivate the surface of the sand/soil bed of dry swales if the swale does not draw down within 48 hours. Re-establish swale vegetation after rototill activities.	As needed (infrequent)

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of enhanced swales. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the enhanced swale.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
ENHANCED SWALE INSPECTION CHECKLIST

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ____

Date: ___________ Time: ______________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Enhanced Swale		
Healthy vegetation?		
Erosion on bottom or side slopes?		
Animal burrows in swale?		
Clear of debris and functional?		
Check dams in place (if applicable)?		
Evidence of sediment accumulation?		
Unintentional obstructions or blockages?		
Clogged pea gravel diaphragm?		
Undesirable vegetation growth?		
Visible pollution?		
Other (describe)?		
Inlet/Outlet Channels		
Clear of debris and functional?		
Sediment accumulation?		
Signs of erosion?		
Other (describe)?		
Sediment Forebays or Pretreatment Areas		
Evidence of sediment accumulation?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature: ________________________________ **Inspector Name (printed)**_________________________

4.3.8.8 Example Schematics

Figure 4-40. Schematic of Dry Swale
(Source: Center for Watershed Protection)

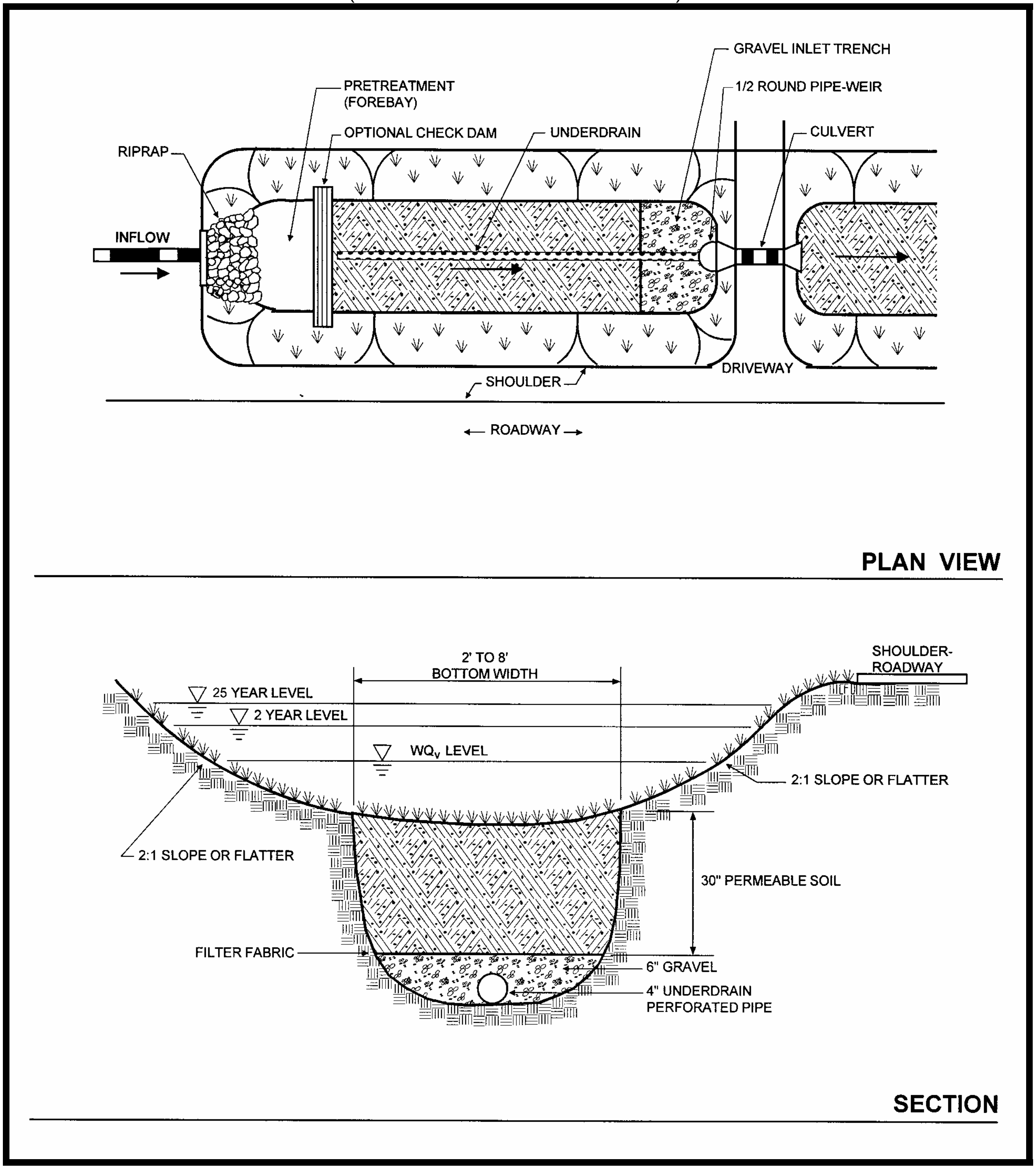
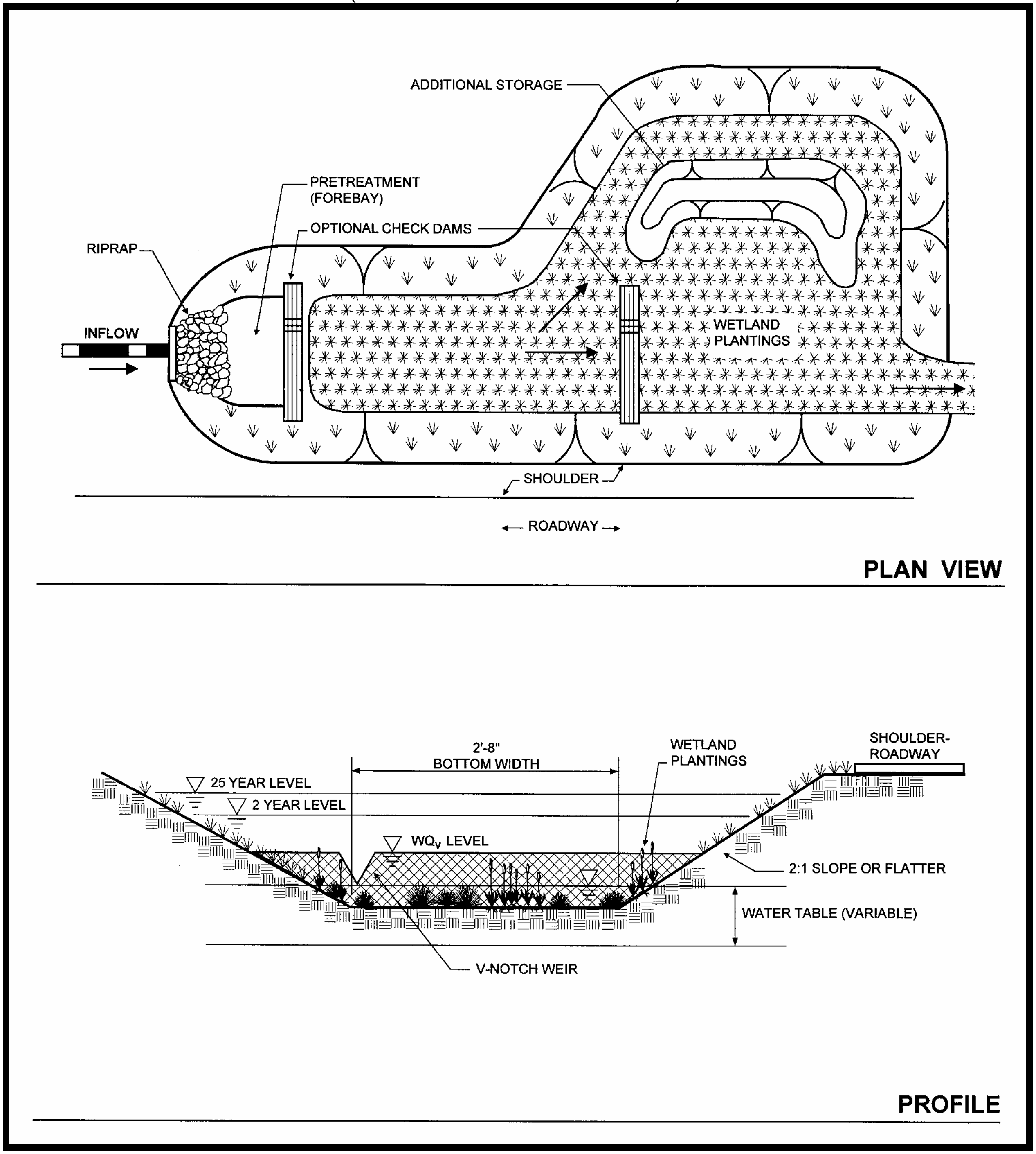


Figure 4-41. Schematic of Wet Swale
(Source: Center for Watershed Protection)



4.3.8.9 Design Form

Knox County recommends the use of the following design procedure forms when designing enhanced swales. Proper use and completion of the form may allow a faster review of the Stormwater Management Plan by Knox County Engineering.

Design Procedure Form: Enhanced Swales

PRELIMINARY HYDROLOGIC CALCULATIONS

1a. Compute WQv volume requirements	
Compute Runoff Coefficient, Rv	Rv = ________
Compute WQv	WQv = ________ acre-ft
1b. Compute CPv	CPv = ________ acre-ft
Compute average release rate	release rate = ________ cfs
Compute storage volume required for 2-year storm	2-year storm = ________ acre-ft
Compute storage volume required for 10-year storm	10-year storm = ________ acre-ft
Compute storage volume required for 25-year storm	25-year storm = ________ acre-ft
Compute storage volume required for 100-year storm	100-year storm = ________ acre-ft

ENHANCED SWALE DESIGN

2. Is the use of an enhanced swale appropriate?	***See subsections 4.3.8.4 and 4.3.8.5 - A***
Confirm design criteria and applicability.	***See subsection 4.3.8.5 - J***
3. Pretreatment Volume (Forebay)	
V_{pre} =(I)(.1")(1'/12")	Vol_{pre} = ________ acre-ft
4. Determine swale dimensions	
Assume trapezoidal channel with max depth of 18 inches	
	Length = ________ ft
	Width = ________ ft
	Side Slopes = ________
	Area = ________ ft^2
Compute number of check dams (or similar structures) required to detain WQv	Slope = ________ ft/ft
	Depth = ________ ft
	Distance = ________ ft
	Number = ________ each
5. Calculate draw-down time	
Require k = 1.5 ft per day for dry swales	t = ________ hr
6. Check 25-year velocity erosion potential and freeboard	V_{min} = ________ fps
Requires separate computer analysis for velocity	
Overflow wier (use weir equation)	Weir Length = ________ ft
Use weir equation for slot length ($Q = CLH^{3/2}$)	
7 Design low flow orifice at headwall	
Area of orifice from orifice equation	Area = ________ ft^2
$Q = CA(2gh)^{0.5}$ C varies with orifice condition	diameter ________ inches
8 Design inlets, sediment forebays, outlet structures, maintenance access, and safety features.	***See subsection 4.3.8.5 - D through H***
9. Design landscaping plan (including wetland vegetation)	

Notes: ______________________________

____

4.3.8.10 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). Georgia *Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

Center for Watershed Protection. *Manual Builder.* Stormwater Manager's Resource Center, Accessed July 2005. *www.stormwatercenter.net*

Connecticut Department of Environmental Protection. *Stormwater Quality Manual.* 2004.

Federal Highway Administration (FHWA), United States Department of Transportation. *Stormwater Best Management Practices in an Ultra-Urban Setting: Selection and Monitoring.* Accessed January 2006. http://www.fhwa.dot.gov/environment/ultraurb/index.htm

Natural Resources Conservation Service (NRCS), United States Department of Agriculture, www.soils.gov

4.3.8.11 Suggested Reading

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

City of Austin, TX. *Water Quality Management.* Environmental Criteria Manual. Environmental and Conservation Services, 1998.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

Claytor, R.A., and T.R. Schueler. *Design of Stormwater Filtering Systems.* The Center for Watershed Protection, Silver Spring, MD, 1996.

Maryland Department of the Environment. *Maryland Stormwater Design Manual, Volumes I and II.* Prepared by Center for Watershed Protection (CWP), 2000.

Metropolitan Washington Council of Governments (MWCOG). *A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone.* March, 1992.

4.3.9 Filter Strip

General Application Stormwater BMP

Description: Filter strips are uniformly graded and densely vegetated sections of land, engineered and designed to treat runoff and remove pollutants through vegetative filtering and infiltration.

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Drainage area size based on flow length and slope.
- Must have slopes between 2% and 6%.
- Must maintain sheet flow across the entire filter strip.
- Minimum 15 ft flow length; the longer the flow length, the higher the pollutant removal.

ADVANTAGES / BENEFITS:

- High community acceptance in any type of setting.
- Easy to maintain once ground cover is established.
- Can be used as pre-treatment for other BMPs, with an effect similar to a sediment forebay.
- Filter strips are easily incorporated into new construction/development designs.

DISADVANTAGES / LIMITATIONS:

- Cannot meet the 80% TSS goal without another BMP in a treatment train. A 50' filter strip is assumed to achieve a 50% TSS removal. A 25 ft strip is assumed to achieve a 10% TSS removal.
- Filter strips and level spreaders have limited drainage areas.
- It can be difficult to construct level spreaders.

MAINTENANCE REQUIREMENTS:

- Maintain a dense, healthy stand of vegetation.
- Repair areas of erosion and re-vegetate as needed.
- Remove sediment build-up.

STORMWATER MANAGEMENT SUITABILITY

- ☑ **Water Quality**
- ☐ **Channel/Flood Protection**
- ☐ **Overbank Flood Protection**
- ☐ **Extreme Flood Protection**

Accepts runoff from SPAP land uses: *Yes, with pre-treatment*

FEASIBILITY CONSIDERATIONS

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

Residential/Subdivision Use: *Yes*

Drainage Area: *Sheet flow requirements result in relatively small drainage areas.*

Soils: *Any soil is suitable – must be fully vegetated with no areas of erosion.*

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

POLLUTANT REMOVAL

L-M	**Total Suspended Solids**
L	**Nutrients**: Total Phosphorus / Total Nitrogen
L	**Metals**: Cadmium, Copper, Lead, and Zinc
No data	**Pathogens**: Coliform, Streptococci, E.Coli

OTHER CONSIDERATIONS

- A level spreader may be needed to achieve sheet flow into the strip.
- Filter strips can be designed with a downstream permeable berm.

4.3.9.1 General Description

Filter strips are uniformly graded and densely vegetated sections of land, engineered and designed to treat runoff and remove pollutants through vegetative filtering and infiltration. Because they cannot accept channelized runoff, filter strips are best suited to treating runoff from roads and highways, roof downspouts, very small parking lots, and pervious surfaces. They are also ideal for use as pre-treatment measures for a stream buffer or structural stormwater controls such as enhanced swales or ponds. Filter strips can serve as a buffer between incompatible land uses, can be landscaped to be aesthetically pleasing, and can provide groundwater recharge in areas with pervious soils.

Pollutant removal from filter strips is highly variable and depends primarily on density of vegetation and contact time for filtration and infiltration. These, in turn, depend on soil and vegetation type, slope, and presence of sheet flow. Pollutant removal efficiencies are based upon a 50-foot long strip. Filter strips with shorter flow lengths are considered to have lower removal efficiencies and should be used as coarse sediment settling areas for other structural controls. Filter strips are often considered to be an integral component of those controls, similar to sediment forebays for stormwater ponds or other structural BMPs. Uniform sheet flow must be maintained through the filter strip to provide pollutant reduction and avoid erosion. To obtain sheet flow when discharging runoff from a developed area, a level spreader may be required.

There are two different filter strip designs: a simple filter strip and a design that includes a permeable berm at the bottom. The presence of the berm increases the contact time between the filter strip and the runoff, thus reducing the overall width of the filter strip required to treat stormwater runoff. An example schematic of a filter strip is presented in Figure 4-42.

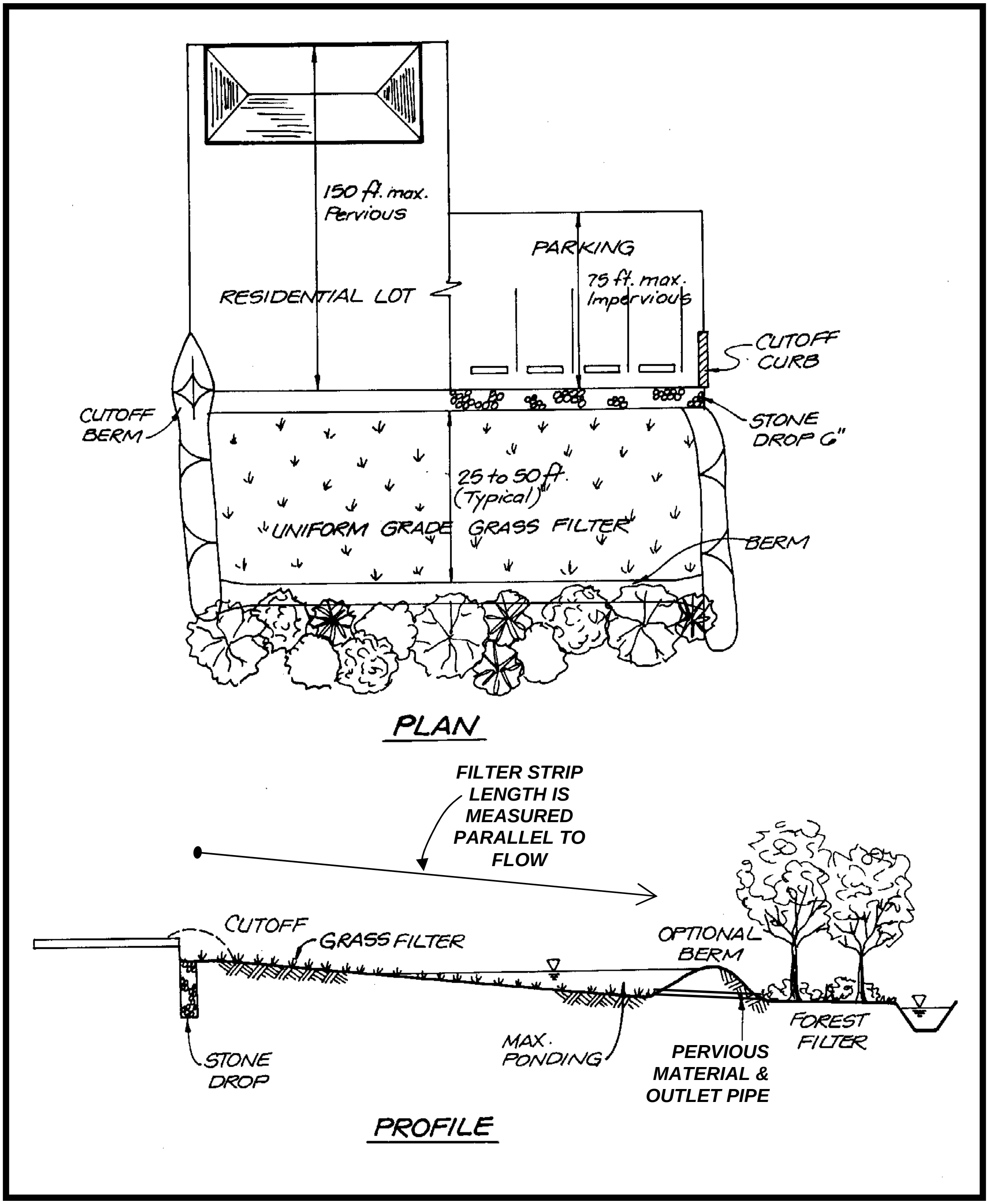
4.3.9.2 Stormwater Management Suitability

Filter strips are designed primarily for stormwater quality and do not have the ability to provide channel protection or flood protection.

<u>Water Quality (WQv)</u>
To treat stormwater runoff, filter strips rely on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from urban stormwater. There can also be a significant reduction in runoff volume for smaller flows that infiltrate through pervious soils within the filter strip. To be effective, however, sheet flow must be maintained across the entire filter strip. Once runoff flow concentrates, it effectively short-circuits the filter strip and reduces any water quality benefits. Therefore, a flow spreader must normally be included in the filter strip design.

<u>Channel Protection (CPv), Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})</u>
Filter strips will not provide for channel protection, overbank or extreme flood protection. Another structural BMP, such as a wet pond that is designed to handle flood control, must be used in conjunction with a filter strip to achieve the CPv, Qp_2, $Qp_{10,}$ Qp_{25} and Qp_{100} design criteria. However, filter strips are typically BMPs that are located “on-line”, so they must be designed to withstand the full range of storm events without eroding.

Figure 4-42. Schematic of Filter Strip (with Berm)



4.3.9.3 Pollutant Removal Capabilities

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. Research indicates that the pollutant removal ability of a filter strip is highly dependant upon the minimum flow path length, as follows.

Filter Strips that have a minimum flow path length of 50 feet or greater:

- Total Suspended Solids – 50%
- Total Phosphorus – 20%
- Total Nitrogen – 20%
- Pathogens – Insufficient data to provide a pollutant removal value
- Heavy Metals – 40%

Filter Strips that have a minimum flow path length between 25 feet and 50 feet (pretreatment control for coarse sediments):

- Total Suspended Solids – 10%
- Total Phosphorus – minimal
- Total Nitrogen – minimal
- Pathogens – Insufficient data to provide a pollutant removal value
- Heavy Metals – minimal

Filter strips that have a flow path length less than 25 feet are assigned a 0% TSS removal value.

4.3.9.4 Application and Feasibility Criteria

Filter strips can be used in a variety of development types. However, because of their relatively large land requirement, filter strips are generally not determined to be useful in higher density areas. The topography and proposed site layout will determine the applicability of filter strips.

General Feasibility

- Suitable for use in residential subdivisions and in non-residential areas.
- Can be used in high density/ultra-urban areas, but land requirements may preclude their use.
- Not suitable for use as a regional stormwater control.

4.3.9.5 Planning and Design Standards

The following standards shall be considered **minimum** design standards for the design of a filter strip. Filter strips that are not designed to these standards will not be approved. The Director shall have the authority to require additional design conditions if deemed necessary.

A. LOCATION AND SITING

- Filter strips are most appropriate for treating the stormwater runoff from small drainage areas. Flow must enter the filter strip as sheet flow spread out over the length (long dimension normal to flow) of the strip. The design depth of flow shall be no greater than 2 inches. As a rule, flow starts to channelize within a maximum of 75 feet for impervious surfaces, and 150 feet for pervious surfaces (CWP, 1996). For longer flow paths, special provision must be made to ensure design flows spread evenly across the filter strip.
- A level spreader may be needed to achieve sheet flow, the design of which should be factored into the location and siting of the filter strip and into the overall site layout. Level spreader design is presented in Volume 2, Chapter 7 of this manual.
- Filter strips should be integrated into site designs.

- Filter strips should be constructed outside the natural stream buffer area whenever possible to maintain a more natural buffer along the streambank.
- Filter strips shall not be in areas or on soils that cannot sustain a dense vegetative cover with high retardance.
- Pedestrian traffic across the filter strip should be limited through channeling onto sidewalks.

B. PHYSICAL SPECIFICATIONS / GEOMETRY

- Filter strips shall be designed having a slope between 2% and 6%. Greater slopes than this will encourage the formation of concentrated flow. Flatter slopes will encourage standing water. Both the top and toe of the slope shall be as flat as possible to encourage sheet flow and prevent erosion.
- The filter strip shall have a minimum length (flow path) of 25 feet long to provide filtration and contact time for water quality treatment. At least fifty (50) feet is necessary to achieve the 50% TSS removal value.
- Flow must enter the filter strip as sheet flow, designed to spread out over the width of the strip with a depth of 1 to 2 inches.
- The design of the filter strip and the area draining to the filter strip shall be such that stormwater flows in excess of the design flow can discharge across or around the strip without causing erosion or other damage. Often a bypass channel or overflow spillway with a protected channel section is designed to handle higher flows.
- An effective flow spreader is to use a pea gravel diaphragm at the top of the slope (ASTM D 448 size no. 6, 1/8" to 3/8"). The pea gravel diaphragm (a small trench running along the top of the filter strip) serves two purposes. First, it acts as a pre-treatment device, settling out sediment particles before they reach the practice. Second, it acts as a level spreader, maintaining sheet flow as runoff flows over the filter strip. Other types of flow spreaders include a concrete sill, curb stops, or curb and gutter with "sawteeth" cut into it. Other level spreader designs can be found in Volume 2, Chapter 7 of this manual.
- Maximum discharge loading per foot of filter strip width (perpendicular to flow path) shall be determined using the Manning equation:

Equation 4.3.9.1

$$q = \frac{0.00236}{n} Y^{\frac{5}{3}} S^{\frac{1}{2}}$$

where: q = discharge per foot of width of filter strip (cfs/ft)
Y = allowable depth of flow (inches) = 2 inches maximum
S = slope of filter strip (percent)
n = Manning's "n" roughness coefficient (use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)

- Using *q* computed above, the minimum width of a filter strip shall be calculated using the following equation:

Equation 4.3.9.2

$$W_{fMIN} = \frac{Q_{wq}}{q}$$

where: W_{fMIN} = minimum filter strip width perpendicular to flow (feet)
Q = peak discharge of stormwater runoff (cfs)
q = discharge per foot of width of filter strip (cfs/ft)

Filter Strips without a permeable berm:

- The length of the filter strip (parallel to flow path across the filter strip) shall be sized to achieve a contact time between the stormwater runoff and filter strip vegetation of no less than five (5) minutes.

- The equation for filter strip length (the flow path) is based on the SCS TR-55 travel time equation (SCS, 1986):

Equation 4.3.9.3

$$L_f = \frac{(T_t)^{1.25}(P_{2-24})^{0.625}(S)^{0.5}}{3.34n}$$

where: L_f = length of filter strip parallel to flow path (ft)
T_t = travel time through filter strip (minutes), minimum 5 minutes
$P_{2\text{-}24}$ = 2-year, 24-hour rainfall depth (inches)
S = slope of filter strip (percent)
n = Manning's "n" roughness coefficient (use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass)

Filter Strips with a permeable berm:

- The filter strip shall be sized to contain the entire WQv within the wedge of water that backs up behind the berm.
- The maximum height of the berm is 12 inches.
- Outlet pipes from the berm shall be sized to ensure that the runoff stored behind the berm drains within 24 hours.
- The outlet pipes shall be designed such that runoff discharges in a non-erosive manner.
- The berm shall be constructed of a mixture of sand, gravel and sandy loam to encourage grass cover. Specifications for sand and gravel are: sand - ASTM C-33 fine aggregate concrete sand 0.02"-0.04"; gravel - AASHTO M-43 ½" to 1".

Filter Strips used for pre-treatment:

- A number of other structural controls, including bioretention areas and infiltration trenches, may utilize a filter strip as a pre-treatment measure. The required length of the filter strip depends on the drainage area, imperviousness, and the filter strip slope. Table 4-9 provides sizing guidance for using filter strips for pre-treatment.

Table 4-9. Sizing of Filter Strips for Pre-treatment
(Source: Adapted from Georgia Stormwater Management Manual)

Parameter	Impervious Areas[1]				Pervious Areas (Lawns, etc)[2]			
Maximum inflow approach length (feet)	35		75		75		100	
Filter strip slope (max = 6%)	≤ 2%	> 2%	≤ 2%	> 2%	≤ 2%	> 2%	≤ 2%	> 2%
Filter strip minimum length (feet)[3]	10	15	20	25	10	12	15	18

1 – 75 feet maximum impervious area flow length to filter strip.
2 – 150 feet maximum pervious area flow length to filter strip.
3 – At least 25 feet is required for minimum pre-treatment credit of 10% TSS removal. Fifty feet is required for 50% removal.

C. SPECIAL CONSIDERATIONS FOR THE AS-BUILT CERTIFICATION

- Like any other water quality BMP, the filter strip must be shown on the as-built certification specifically as a water quality BMP. The following components must be addressed in the as-built certification:
  1. Ensure the design flows are spread evenly across the filter strip.
  2. Ensure the design slope is between 2% and 6%.
  3. The dimensions of the filter strip must be verified.
  4. The type of vegetation used in the filter strip.

D. MAINTENANCE ACCESS

- A minimum 20 foot wide maintenance right-of-way or drainage easement shall be provided for the length and width of the filter strip from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles. The right-of-way shall be located such that maintenance vehicles and equipment can access the entire filter strip.

E. LANDSCAPING

- The vegetation in a filter strip can be grassed, or a combination of grass and woody plants. Filter strips that are vegetated with forest vegetation may be able to qualify as a water quality volume (WQv) credit. See Chapter 5 for more information on the stream and vegetated buffer credit.
- Designers should choose a grass that can withstand relatively high velocity flows at the entrances, and both wet and dry periods.
- For filter strips with a permeable berm, vegetation that can withstand frequent inundation must be utilized in the area where shallow ponding will occur.

4.3.9.6 Design Example

Basic Data

Small commercial lot 150 feet deep x 100 feet wide

- Drainage area (A) = 0.34 acres
- Impervious percentage (I) = 70%
- Slope equals 4%
- Manning's n = 0.25

Step 1: Calculate Maximum Discharge Loading Per Foot of Filter Strip Width (q):

Using Equation 4.3.9.1 above:

$$q = (0.00236/0.25) * (1.0)^{5/3} * (4)^{1/2} = 0.019 \text{ cfs/ft}$$

Step 2: Calculate the Water Quality Flow Rate (Q_{wq}):

(See Chapter 3 for equation information)
Compute the Runoff Peak Volume (Q_{wv}) in inches for 1.1-inch rainfall (P = 1.1):

$$Q_{wv} = PRv = 1.1Rv = 1.1(0.015 + (0.0092)(70)) = 0.72 \text{ inches}$$

Compute modified CN:

$$\begin{aligned} CN &= 1000/[10+5P+10\,Q_{wv} - 10(Q_{wv}{}^2+1.25Q_{wv}P)^{½}] \\ &= 1000/[10+5(1.1)+10(0.72)-10(0.72^2+1.25(0.72)1.1)^{½}] \\ &= 95.98 \text{ (Use CN = 96)} \end{aligned}$$

For CN = 96 and an estimated time of concentration (T_c) of 8 minutes (0.13 hours), compute the Q_{wq} for a 1.1 inch storm.

I_a = 0.083 (from Table 3-14 in Chapter 3), therefore I_a/P = 0.083/1.1 = 0.075.

Using Figure 3-6 in Chapter 3, q_u can be estimated for a Type II storm at approximately 950 csm/in.

q_u = 950 csm/in, and therefore:
$Q_{wq} = q_uA\,Q_{wv}$ = (950 csm/in) (0.34ac/640ac/mi^2) (0.72in) = 0.36 cfs

Step 3: Calculate the Minimum Filter Width

Using Equation 4.3.9.2 above:

$W_{fMIN} = Q_{wq}/q = 0.36/0.019 = 19$ feet

Since the width of the lot is 100 feet, the actual width of the filter strip will depend on site grading and the ability to deliver the drainage to the filter strip in sheet flow through a pea gravel filled trench.

The next step is to calculate the filter length. This calculation is different for a filter designed without a permeable berm (presented in Step 4a), than for a filter designed with a berm (presented in Step 4b).

Step 4a: Calculate the Filter Length (L_f) for a filter without a berm:

Basic Data:

- Depth of 2-year, 24-hour storm = 3.3 inches (see Chapter 3, Table 3-5)
- Use 5 minute travel (contact) time

Using Equation 4.3.9.3 above:

$L_f = (5)^{1.25}(3.3)^{0.625}(4)^{0.5} / (3.34)(0.25) = 37.8$ feet (use 38 feet)

Note: Reducing the filter strip slope to 2% and planting a more dense grass (raising the Manning "n" to 0.35) would reduce the filter strip length to 19 feet.

Step 4b: Calculate the Filter Length (assume filter is designed with a berm):

(See Chapter 3 for equation information)

Basic Data:

- The height of the permeable berm (h) will be 6 inches (0.5 feet).
- Assume the filter width = the maximum lot width (W_f) = 100 feet.

Compute the Water Quality Volume (WQv) in cubic feet:

$WQv = 1.1R_vA/12 = 1.1(0.015 + 0.0092(70))0.34/12 = 0.021$ ac-ft or 895 ft^3

This is the volume of the "wedge" of water that ponds behind the berm.

For a berm height of 6 inches (0.5 feet), the "wedge" of volume captured by the filter strip is:

The area of the "wedge" = $\frac{1}{2}L_fh$, therefore,

The volume of the "wedge" = $W_f\frac{1}{2}L_fh = (100)\ \frac{1}{2}(L_f)(0.5) = 895\ ft^3$

Solving for L_f, the length of the filter = 35.8 feet (use 36 feet).

Note: Increasing the berm height to 1 foot will result in a filter length of 18 feet.

4.3.9.7 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.9.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective use of filter strips as stormwater best management practices. It is the responsibility of the property owner to maintain all stormwater facilities in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for filter strips, along with a suggested frequency for each activity. Individual filter strips may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain filter strips properly at all times.

Inspection Activities	Suggested Schedule
• Inspect pea gravel diaphragm for clogging (i.e., standing water or sediment build-up). • Inspect vegetation for signs of erosion or un-vegetated areas. • Inspect to ensure that grass has established. • Inspect general flow paths to determine if runoff discharges into and across the filter strip in an unchannelized fashion.	Annually (Semi-annually first year)
Maintenance Activities	**Suggested Schedule**
• Maintain a dense, healthy stand of grass and other vegetation by frequent mowing. Grass heights of 3 to 5 inches should be maintained, with a maximum grass height of 8 inches.	Regularly (frequently)
• Repair areas of erosion and re-vegetate. • Re-vegetate as needed to maintain healthy vegetation. • Remove sediment buildup.	As needed

Knox County encourages the use of the inspection checklist presented below for guidance in the inspection and maintenance of the filter strip. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the filter strip. Questions regarding inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST FOR FILTER STRIPS

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ____

Date: ___________ Time: ______________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Healthy vegetation?		
Signs of erosion?		
Clogged pea gravel diaphragm?		
Sediment buildup behind level spreader at top?		
Sediment buildup in filter strip?		
Other (describe)?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature:________________________________ Inspector Name (printed)____________________________

4.3.9.8 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

4.3.9.9 Suggested Reading

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

City of Austin, TX. *Water Quality Management*. Environmental Criteria Manual. Environmental and Conservation Services, 1988.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

Claytor, R.A., and T.R. Schueler. *Design of Stormwater Filtering Systems.* The Center for Watershed Protection, Silver Spring, MD, 1996.

Driscoll, E., and P. Mangarella. *Urban Targeting and BMP Selection.* Prepared by Woodward-Clyde Consultants, Oakland, CA, for U.S. Environmental Protection Agency, Washington, DC, 1990.

Maryland Department of the Environment. *Maryland Stormwater Design Manual, Volumes I and II.* Prepared by Center for Watershed Protection (CWP), 2000.

Metropolitan Washington Council of Governments (MWCOG), *A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone.* March 1992.

Urbonas, B.R., J.T. Doerfer, J. Sorenson, J.T. Wulliman, and T. Fairley. *Urban Storm Drainage Criteria Manual. Vol. 3. Best Management Practices, Stormwater Quality.* Urban Drainage and Flood Control District, Denver, CO, 1992.

Wong, S.L., and R.H. McCuen. *The Design of Vegetative Buffer Strips for Runoff and Sediment Control. Appendix J in Stormwater Management for Coastal Areas.* American Society of Civil Engineers, New York, New York, 1982.

4.3.10 Grass Channel

General Application Stormwater BMP

Description: Grass channels are vegetated open channels that are designed to filter stormwater runoff, as well as slow water for treatment by another structural BMP.

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Broad bottom channel on slopes of 4% or less.
- Gentle side slopes (3:1 (H:V) or less).
- Check dams can be installed to maximize treatment.
- Requires vegetation that can withstand both relatively high velocity flows and wet and dry periods.

ADVANTAGES / BENEFITS:

- Provides pretreatment if used as part of runoff conveyance system.
- Provides infiltration of runoff in some soil conditions.
- Generally less expensive than extruded curb.
- Good for small drainage areas.
- Relatively low maintenance requirements.

DISADVANTAGES / LIMITATIONS:

- Cannot alone achieve 80% removal of TSS.
- Must be carefully designed to achieve low flow rates in the channel for WQv purposes (<1.0 ft/s).
- May re-suspend sediment.
- May not be acceptable for some areas because of standing water in channel.

MAINTENANCE REQUIREMENTS:

- Maintain a dense, healthy stand of grass.
- Repair areas of erosion and re-vegetate as needed.
- Remove sediment buildup.

STORMWATER MANAGEMENT SUITABILITY

- ☑ **Water Quality**
- ☐ **Channel/Flood Protection**
- ☐ **Overbank Flood Protection**
- ☐ **Extreme Flood Protection**

Accepts runoff from SPAP land uses: *Yes, with pre-treatment.*

FEASIBILITY CONSIDERATIONS

L	**Land Requirement**
L	**Capital Cost**
L	**Maintenance Burden**

Residential/Subdivision Use: *Yes*

Drainage Area: *5 acres maximum*

Soils: *Any soil is suitable – must be fully vegetated with no areas of erosion.*

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

POLLUTANT REMOVAL

L	**Total Suspended Solids**
L	**Nutrients**: Total Phosphorus / Total Nitrogen
L	**Metals**: Cadmium, Copper, Lead, and Zinc
No data	**Pathogens**: Coliform, Streptococci, E.Coli

OTHER CONSIDERATIONS

- Grass channels are generally well suited to a large number of applications and land uses.

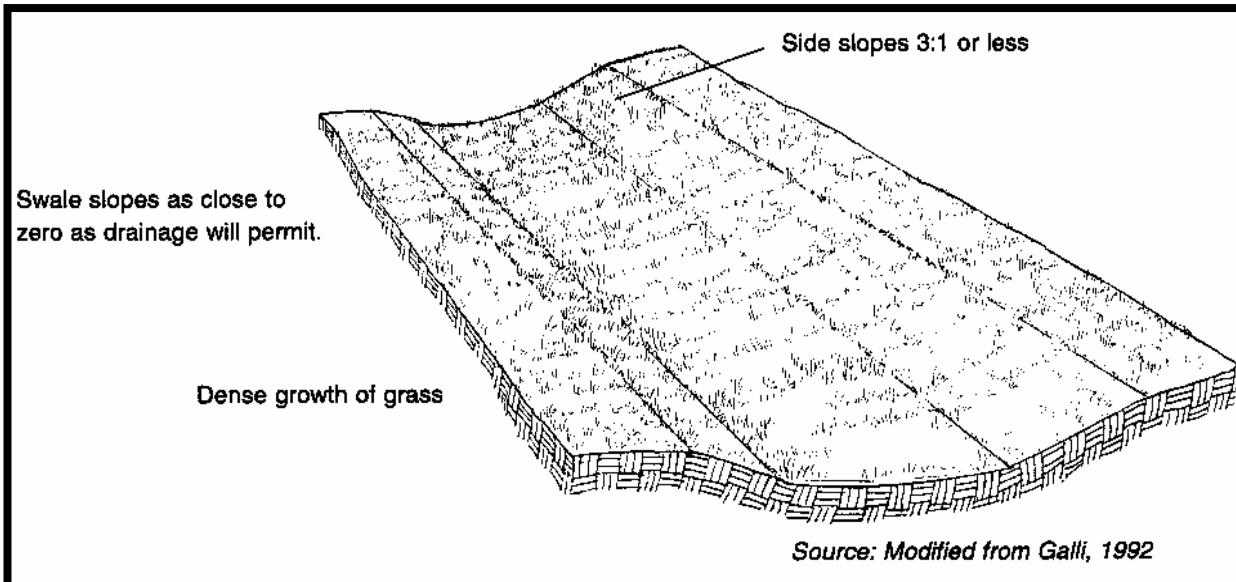
4.3.10.1 General Description

Grass channels, sometimes called biofilters, are conveyance channels that are designed to provide some treatment of runoff, as well as to slow down runoff velocities for treatment in other structural controls. Grass channels are appropriate for a number of applications including treating runoff from paved roads and from pervious areas.

In addition to their ability to provide a minimal level of filtration of pollutants, grass channels can partially infiltrate runoff from small storm events when they are located in areas that have suitable soils (types A, B, and sometimes C). When properly incorporated into a site's layout, grass channels can provide other ancillary benefits, such as reduction of impervious cover, accent of natural features and reduced construction and maintenance costs when compared with traditional extruded curb.

When designing a grass channel, the two primary considerations are channel capacity and minimization of erosion. The channel must be designed with a runoff velocity less than 1.0 foot per second during the peak discharge associated with the water quality design rainfall event, and the total length of a grass channel should provide at least 5 minutes of residence time. To enhance water quality treatment, grass channels must have broader bottoms, lower slopes and denser vegetation than most drainage channels. Additional treatment can be provided by placing check-dams across the channel below pipe inflows, and at various other points along the channel. Example schematics of grass channels are presented in Figures 4-43 and 4-44.

Figure 4-43. Typical Grass Channel



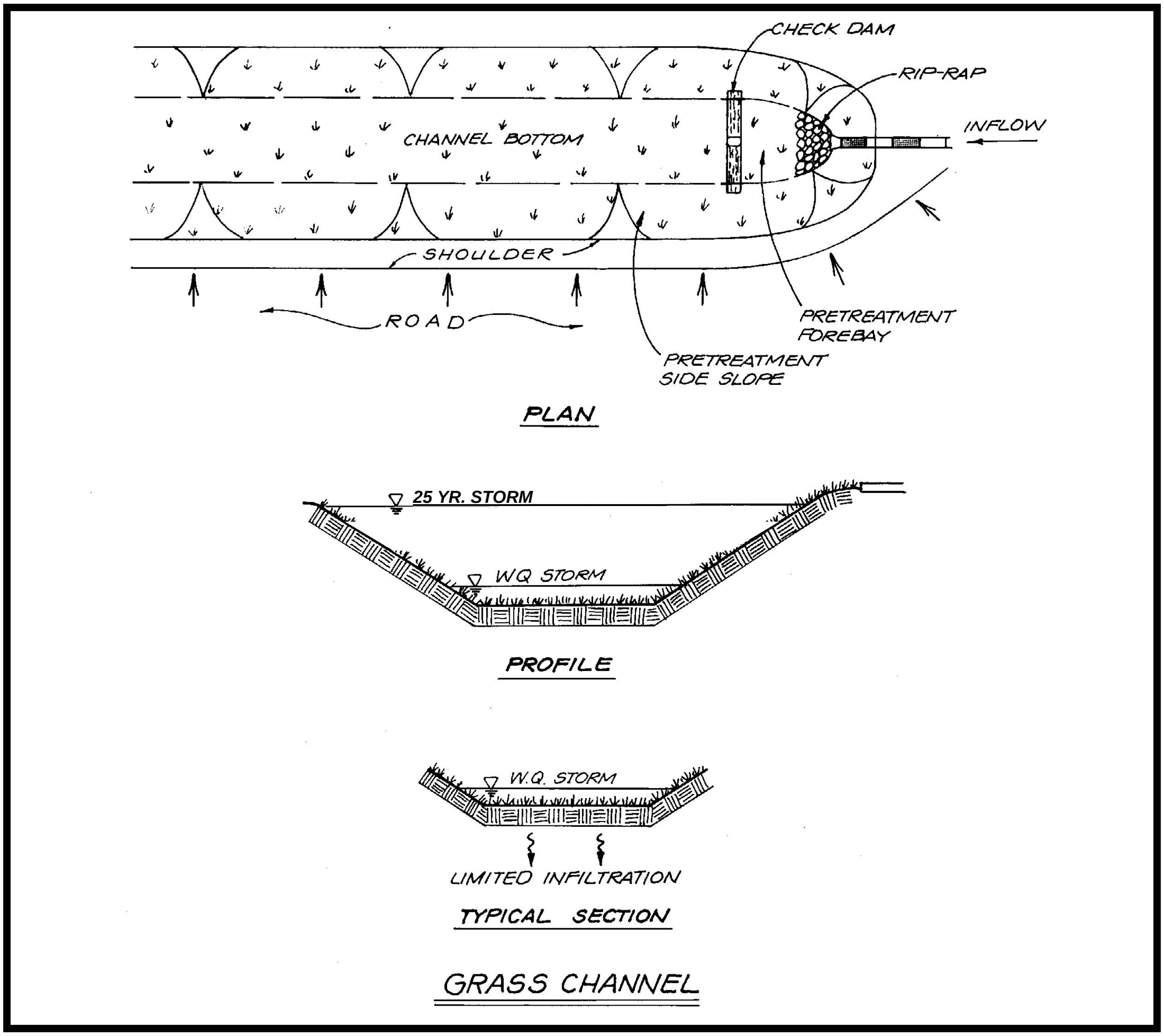
4.3.10.2 Stormwater Management Suitability

Grass channels are designed primarily for stormwater quality treatment and runoff conveyance and do not have the ability to provide channel protection or flood protection.

Water Quality (WQv)

To treat stormwater runoff, grass channels rely on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from urban stormwater. There can also be a reduction in runoff volume for smaller flows that infiltrate through pervious soils within the filter strip.

Figure 4-44. Typical Grass Channel (Plan and Profile Views)



Channel Protection (CPv), Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})
Grass channels will not provide for channel protection, overbank or extreme flood protection. Another structural BMP, such as a wet pond that is designed to handle flood control, must be used in conjunction with the grass channel to achieve the CPv, Qp_2, Qp_{10}, Qp_{25} and Qp_{100} design criteria. However, grass channels are typically BMPs that are located "on-line", so they must be designed to withstand the full range of storm events without eroding.

4.3.10.3 Pollutant Removal Capabilities

Grass channels differ from enhanced swales (discussed in Section 4.3.8 of this manual) in that they do not have an engineered filter media to enhance pollutant removal capabilities. Because of this, grass channels have a lower pollutant removal rate than for a dry or wet (enhanced) swale.

The following design pollutant removal rates are based upon a grass channel that has sufficient length for a runoff residence time (in the channel) of at least 5 minutes. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. Grass channels with residence times less than five (5) minutes may be employed, but no water quality credit will be granted.

- Total Suspended Solids – 30%
- Total Phosphorus – 25%
- Total Nitrogen – 20%
- Pathogens – Insufficient data to provide a pollutant removal value
- Heavy Metals – 30%

4.3.10.4 Application and Feasibility Criteria

Grass channels can be used in a variety of development types. However, because of strict requirements for low slopes, grass channels will generally not be useful in developments that have steep topography.

General Feasibility

- Suitable for use in residential subdivisions and in non-residential areas.
- Can be used in high density/ultra-urban areas, but runoff velocity restrictions may preclude their use.
- Not suitable for use as a regional stormwater control due to small drainage area requirements.

4.3.10.5 Planning and Design Standards

The following standards shall be considered **minimum** design standards for the design of a grass channel. Grass channels that are not designed to these standards will not be approved. The Director shall have the authority to require additional design conditions if deemed necessary.

A. LOCATION AND SITING

- The drainage area (contributing or effective) for a grass channel shall be 5 acres or less. Runoff flows and volumes from larger drainage areas prevent proper filtration and infiltration of stormwater.
- Grass channels can be used on most soils. However, grass channels shall not be used for water quality treatment purposes on soils with infiltration rates less than 0.27 inches per hour.

B. PHYSICAL SPECIFICATIONS / GEOMETRY

The following specifications apply to grass channels that are designed to achieve a % TSS removal rate of 30%. The reader should refer to Volume 2, Chapter 7 for additional specifications and design information on runoff conveyance in open grass channels.

- Grass channels shall be designed on relatively flat slopes of less than 4%; channel slopes between 1% and 2% are recommended.
- A grass channel shall be designed to accommodate the peak flow for the water quality design storm, Q_{wq}, and the 2-year, 24-hour design storm without eroding (see Volume 2, Chapter 3) for more information on Q_{wq}). Larger flows should be accommodated by the channel if dictated by the surrounding conditions. For example, Knox County requires site drainage features to accommodate the 25-year design storm, and in some cases the 100-year storm.
- Grass channels shall have a trapezoidal or parabolic cross-section and shall have side slopes of 3:1 (horizontal:vertical) or flatter.
- For trapezoidal sections, the minimum width of the channel bottom shall be no less than 2 feet. The maximum width of the channel bottom shall be no greater than 6 feet. The minimum width ensures a minimum filtering surface for water quality treatment, and the maximum width prevents braiding, which is the formation of small channels within the swale bottom. The bottom width is a dependent variable in the calculation of velocity based on Manning's equation. If a larger channel is needed, the use of a compound cross section is recommended.
- The channel shall be designed to have a depth of flow no greater than 4-inches, for the WQv design flow. Depth of flow can be greater for the Cp_v, Qp_{25} and Qp_{100} flows.
- Runoff velocities carried in the channel must be non-erosive. The full-channel design velocity will typically govern. Runoff velocity must be less than 1.0 ft/sec for the WQv design flow.

- The channel shall be designed such that the water quality peak flow (Q_{wq}) is contained in the channel for no less than 5-minutes. This residence time may be increased by reducing the slope of the channel, increasing the wetted perimeter, or planting a denser grass (raising the Manning's "n"). Check dams can be utilized in the channel to maximize Q_{wq} retention time. However, the channel must not be designed to hold a permanent pool of standing water. Channel slope shall be sufficient to drain the channel if infiltration does not occur.
- The depth from the bottom of the channel to groundwater shall be at least 2 feet to prevent a moist swale bottom, or contamination of the groundwater.
- Designers should choose a grass that can withstand relatively high velocity flows at the entrances, and both wet and dry periods.

Grass Channels Used for Pretreatment:

- A number of other structural controls, including bioretention areas and infiltration trenches, may utilize a grass channel as a pretreatment measure. The length of the grass channel depends on the drainage area, land use, and channel slope. To be used as a pretreatment measure, the grass channel must have a minimum length of 20 feet. Table 4-10 provides minimum lengths for grass channels based on channel slope and percent imperviousness (of the contributing drainage area).

Table 4-10. Grass Channel Sizing Guidance
(Source: Georgia Stormwater Management Manual)

Parameter	≤ 33% Impervious		Between 34% and 66% Impervious		≥ 67% Impervious	
Slope (max = 4%)	≤ 2%	> 2%	≤ 2%	> 2%	≤ 2%	> 2%
Grass channel min. length (feet) assumes 2-ft bottom width	25	40	30	45	35	50

C. SPECIAL CONSIDERATIONS FOR THE AS-BUILT CERTIFICATION

- Like any other water quality BMP, the grass channel must be shown on the as-built certification specifically as a water quality BMP. The following components must be addressed in the as-built certification:
    1. The channel must be adequately vegetated.
    2. The channel flow velocities must not exceed 1.0 foot per second for the WQv design flow.
    3. A mechanism for overflow of large storm events must be provided.

D. MAINTENANCE ACCESS

- A minimum 20 foot wide maintenance right-of-way or drainage easement shall be provided for the length and width of the grass channel from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles. The right-of-way shall be located such that maintenance vehicles and equipment can access the entire channel.

E. LANDSCAPING

- The vegetation in a grass channel shall be composed entirely of grasses that can withstand relatively high velocity flows at the entrances and periods of inundation and drought.

4.3.10.6 Design Example

Basic Data

Small commercial lot 300 feet deep x 145 feet wide

- Drainage area (A) = 1.0 acres
- Impervious percentage (I) = 70%
- Site slope (S) = 2%

Step 1: Calculate the Water Quality Peak Flow Rate (Q_{wq}):

(See Chapter 3 for equation information)
Compute the Runoff Peak Volume (Q_{wv}) in inches for 1.1-inch rainfall (P = 1.1):

Q_{wv} = PRv = 1.1Rv = 1.1(0.015 + (0.0092)(70)) = 0.72 inches

Compute modified CN:

$$CN = 1000/[10+5P+10\,Q_{wv} - 10(Q_{wv}^{\ 2}+1.25Q_{wv}P)^{1/2}]$$
$$= 1000/[10+5(1.1)+10(0.72)-10(0.72^2+1.25(0.72)1.1)^{1/2}]$$
$$= 95.98 \quad \text{(Use CN = 96)}$$

For CN = 96 and an estimated time of concentration (T_c) of 8 minutes (0.13 hours), compute the Q_{wq} for a 1.1 inch storm.

I_a = 0.083 (from Table 3-14 in Chapter 3), therefore I_a/P = 0.083/1.1 = 0.075.

Using Figure 3-6 in Chapter 3, q_u can be estimated for a Type II storm at approximately 950 csm/in.

q_u = 950 csm/in, and therefore:

$Q_{wq} = q_u A\,Q_{wv}$ = (950 csm/in) (1.0ac/640ac/mi^2) (0.72in) = 1.07 cfs

Step 2: Utilize Q_{wq} to Calculate the Minimum Channel Bottom Width

The maximum flow depth for water quality treatment should be approximately the same height of the grass. A maximum flow depth of 4 inches is allowed for water quality design. A maximum flow velocity of 1.0 foot per second for water quality treatment is required. For Manning's "n" use 0.15 for medium grass, 0.25 for dense grass, and 0.35 for very dense Bermuda-type grass.

Input variables: n = 0.15
S = 0.02 ft/ft
D = 4/12 = 0.33 ft

Then: $Q_{wq} = Q = VA = 1.49/n\ D^{2/3}\ S^{1/2}\ DW$

where: Q = peak flow (cfs)
V = velocity (ft/sec)
A = flow area (ft^2) = WD
W = channel bottom width (ft)
D = flow depth (ft) *(approximates the hydraulic radius for shallow flows)*
S = slope (ft/ft)

The above equation can be solved for the minimum channel bottom width (W), as follows:

$(nQ)/(1.49\ D^{5/3}\ S^{1/2}) = W = (0.15*1.07)/(1.49*0.33^{5/3}*0.02^{1/2})$ = 4.8 feet *(minimum width)*

The velocity of the water quality peak flow rate must be less than 1.0 feet per second (fps). Check this, as follows:

V = Q/(WD) *(where WD approximates the flow area, A, for shallow flows)*
V = 1.07/(4.0 * 4/12) = 0.80 fps *(Design confirmed: the velocity is < 1.0 fps.)*

Step 3: Calculate the Channel Length

The minimum length for a 5-minute (300 seconds) residence time is calculated as follows:

V = L/T

where:
V = velocity (ft/sec)
L = channel length (ft)
T = residence time (seconds)

The above equation can be solved for the minimum channel length (L), as follows:

L = (0.8)(5*60) = 240 feet

Depending on the site geometry, the width or the slope or density of grass (Manning's "n" value) can be adjusted to slow the velocity and shorten the channel within the design specifications discussed above.

Step 4: Complete the Grass Channel design for other design storms

Refer to Volume 2, Chapter 7 to complete the grass channel design for a specified design storm event.

This page left intentionally blank

4.3.10.7 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.10.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective use of grass channels as stormwater best management practices. It is the responsibility of the property owner to maintain all stormwater facilities in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for grass channels, along with a suggested frequency for each activity. Individual grass channels may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain grass channels properly at all times.

Inspection Activities	Suggested Schedule
• Inspect check dams (if used) for clogging (i.e., standing water or sediment build-up). • Inspect vegetation for signs of erosion or un-vegetated areas. • Inspect to ensure that grass is healthy and well-established.	Annually (Semi-annually first year)
Maintenance Activities	**Suggested Schedule**
• Maintain a dense, healthy stand of grass and other vegetation by frequent mowing. Grass heights of 3 to 5 inches should be maintained, with a maximum grass height of 8 inches.	Regularly (frequently)
• Remove trash, debris and sediment accumulated in the channel or behind check dams (if present). • Repair areas of erosion and re-vegetate. • Re-vegetate as need to maintain healthy vegetation.	As-needed

Knox County encourages the use of the inspection checklist presented below for guidance in the inspection and maintenance of the grass channel. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the channel. Questions regarding inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST FOR GRASS CHANNELS

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ____

Date: __________ Time: ____________ Site conditions:____________________________________

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Healthy vegetation?		
Signs of erosion?		
Clogged check dams?		
Sediment build-up on channel bottom?		
Standing water for extended periods?		
Soggy channel bottom for extended periods?		
Other (describe)?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature:______________________________ Inspector Name (printed)__________________________

4.3.10.8 References

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual Volume 4 Best Management Practices.* 2006.

Claytor, R.A., and T.R. Schueler. *Design of Stormwater Filtering Systems.* The Center for Watershed Protection, Silver Spring, MD, 1996.

4.3.10.9 Suggested Reading

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

City of Austin, TX. *Water Quality Management*. Environmental Criteria Manual, Environmental and Conservation Services, 1988.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

Horner, R.R. *Biofiltration Systems for Storm Runoff Water Quality Control.* Washington State Department of Ecology, 1988.

IEP. *Vegetated Buffer Strip Designation Method Guidance Manual.* Narragansett Bay Project, 1991.

Maryland Department of the Environment. *Maryland Stormwater Design Manual, Volumes I and II.* Prepared by Center for Watershed Protection (CWP), 2000.

Metropolitan Washington Council of Governments (MWCOG). *A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone.* March, 1992.

4.3.11 Modular Porous Paver Systems

General Application Stormwater BMP

Description: A pavement surface composed of structural units with void areas that are filled with pervious materials such as sand or grass turf. Porous pavers are installed over a gravel base course that provides storage as runoff infiltrates through the porous paver system into underlying permeable soils.

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Design considerations are similar to any paved area (soil properties, load-bearing design, hydrologic design of pavement and subgrade).
- Soil infiltration rate of 0.5 in/hr or greater is required, if no underdrain is present.
- The infiltration rate of native soil determines appropriateness and need for an underdrain.
- Not appropriate for heavy or high traffic areas.

ADVANTAGES / BENEFITS:

- Reduces runoff volume, attenuates peak runoff rate and outflow.
- Can be used as pretreatment for other BMPs for pollutants other than TSS.
- High level of pollutant removal for pollutants other than TSS.

DISADVANTAGES / LIMITATIONS:

- Sediment-laden runoff can clog modular porous paver systems causing failure.
- Subgrade cannot be overly compacted.
- Construction must be sequenced to avoid compaction and clogging pavement.

MAINTENANCE REQUIREMENTS:

- Vacuum to increase porous paver system life and avoid clogging.
- Ensure that contributing area is clear of debris and areas of erosion.

STORMWATER MANAGEMENT SUITABILITY

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

Accepts runoff from SPAP land uses: *Yes, but does not provide stormwater treatment.*

FEASIBILITY CONSIDERATIONS

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

Residential/Subdivision Use: *Yes*

Soils: *Not appropriate for use with hydrologic soil groups "D" and "C" without underdrain.*

POLLUTANT REMOVAL

L	**Total Suspended Solids**
H	**Nutrients**: Total Phosphorus / Total Nitrogen
H	**Metals**: Cadmium, Copper, Lead, and Zinc
No Data	**Pathogens**: Coliform, Streptococci, E.Coli

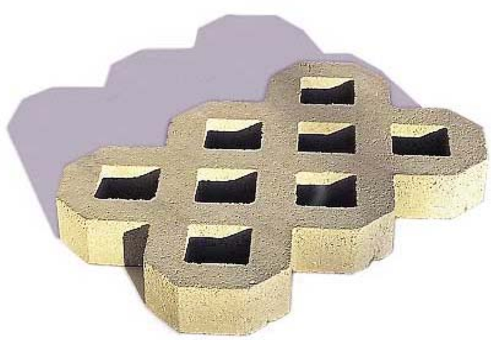
L=Low **M**=Moderate **H**=High

4.3.11.1 General Description

While porous paver systems are not a recommended practice to reduce TSS, they are an excellent application to reduce the effective impervious area on a site, therefore, reducing the Water Quality Volume (WQv) that must be treated. Modular porous pavers are structural units, such as concrete blocks, bricks, or reinforced plastic mats, with regularly dispersed void areas used to create a load-bearing pavement surface. The void areas are filled with pervious materials (gravel, sand, or grass turf) to create a system that allows for the infiltration of stormwater runoff. Porous paver systems provide water quality benefits in addition to groundwater recharge and a reduction in stormwater volume.

There are many different types of modular porous pavers that are available from different manufacturers, including both pre-cast and mold in-place concrete blocks, concrete grids, interlocking bricks, and plastic mats with hollow rings or hexagonal cells (see Figure 4-45). The two main types of modular porous pavement systems are plastic grid and open cell paving grid.

Figure 4-45. Examples of Modular Porous Pavers



Concrete Paver Block

Castellated Block

Lattice Block

Grass / Gravel Paver Mat

Plastic grid systems are often referred to as *geocells* and are defined by manufactured plastic lattices or mattresses that form networks of box-like cells that are filled with earth material. The lattice is typically 3 to 8 inches thick, and the cells range from 2 to 20 inches wide. Porosity and permeability of these systems is entirely dependent on the type of fill and vegetation that exists within each cell. Like any other pavement surface, geocells require a firm gravel base that provides strength and storage capacity as runoff infiltrates. Geocells are lightweight and easy to transport and install. However, they may be jarred easily by moving traffic, resulting in cell failure, cell movement, and possibly the need for replacement.

Open cell paving grids, commonly called *block pavers or grid pavers*, are structural units, such as concrete blocks or bricks with regularly spaced voids that penetrate their entire thickness. Grids are made of concrete or brick and the open cells are filled with porous aggregate or vegetated soil. Block pavers are more rigid and, therefore, can bear larger traffic loads than plastic grid systems.

Modular porous pavement systems are typically placed on a gravel (stone aggregate) base course. Runoff infiltrates through the porous paver surface into the gravel base course, which acts as a storage reservoir as it infiltrates to the underlying soil. The infiltration rate of the soils in the subgrade must be adequate to support drawdown of the entire runoff capture volume within 24 to 48 hours. If the surrounding soil infiltration is insufficient or if the potential for contamination of groundwater exists from pollutants such as chemicals, fertilizers, petroleum products, fats or greases, an underdrain is required to allow discharge of the runoff to additional BMPs for treatment. Additionally, special care must be taken during construction to avoid undue compaction of the underlying soils, which could affect the soils' infiltration capability.

Construction and maintenance costs and requirements should be considered when utilizing porous paver systems. Modular porous paver systems require a high level of construction workmanship to ensure that they function as designed. In addition, the repair or replacement of the surfaces can be costly should they become clogged.

4.3.11.2 Stormwater Management Suitability

Porous paver systems can not be used for stormwater treatment (i.e., 80% TSS removal) or flood control. The major benefit in using these systems lies in the overall reduction of stormwater runoff that can be provided. Areas covered by porous paver systems can be considered as pervious surfaces, thereby reducing water quality treatment and channel protection volumes, and flood protection peak discharges.

<u>Water Quality (WQv)</u>
Porous paver systems do not have the ability to provide stormwater quality treatment for total suspended solids (TSS). However, these systems provide for infiltration of stormwater and can provide for the removal of other pollutants, such as hydrocarbons (e.g., motor oil and gasoline).

<u>Channel Protection (CPv), Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})</u>
Porous paver systems will not provide for channel protection, overbank or extreme flood protection. Another structural BMP, such as an extended dry detention pond, that is designed to handle flood control, must be used in conjunction with the porous paver system to achieve the CPv, Qp_2, $Qp_{10,}$ Qp_{25} and Qp_{100} design criteria.

4.3.11.3 Pollutant Removal Capabilities

As they provide for the infiltration of stormwater runoff, porous paver systems have a high removal of soluble pollutants, where they become trapped, absorbed or broken down in the underlying soil layers. Due to the potential for clogging, porous paver surfaces cannot be used for the removal of sediment or other particulate pollutants.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- Total Suspended Solids – not applicable
- Total Phosphorus – 80%
- Total Nitrogen – 80%
- Pathogens – Insufficient data to provide a pollutant removal value
- Heavy Metals – 90%

4.3.11.4 Application and Feasibility Criteria

Modular porous paver systems are typically used in low-traffic areas, such as:

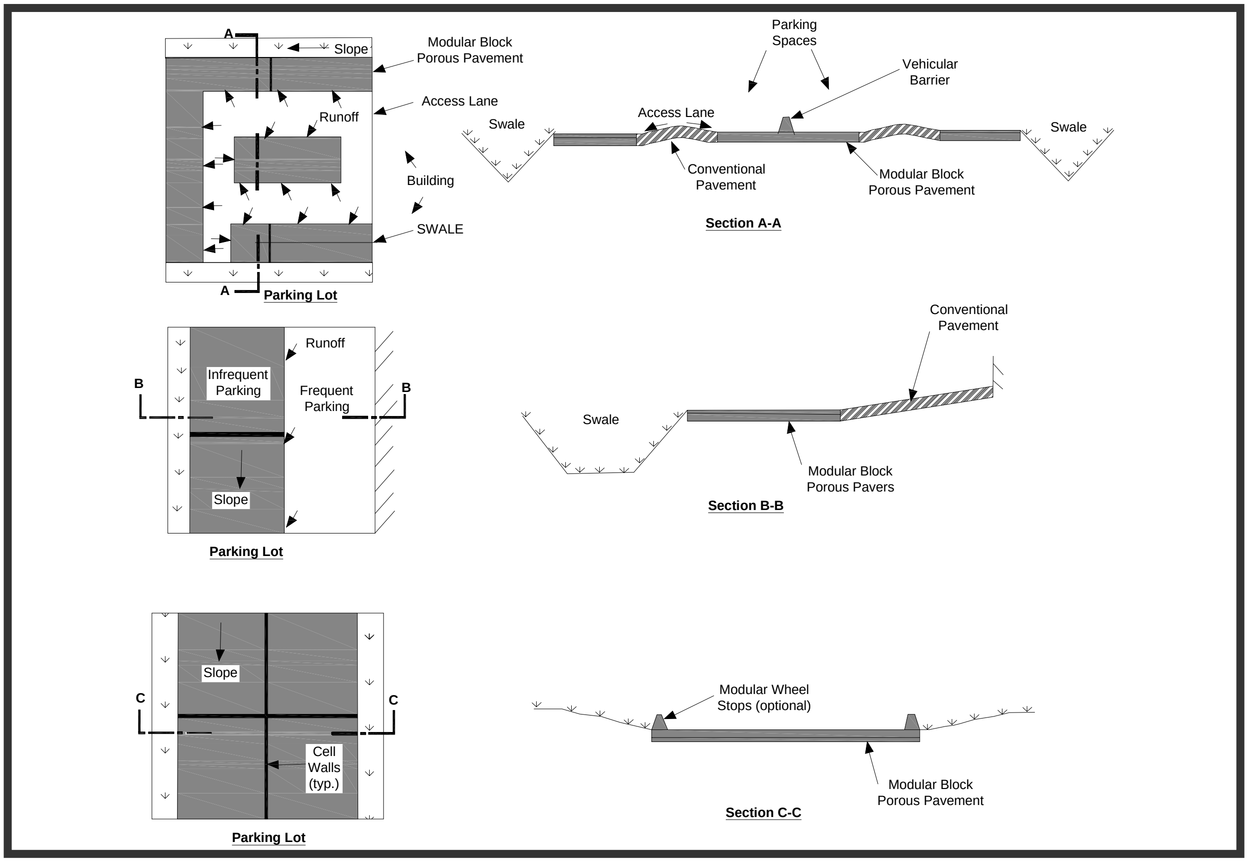
- parking pads in parking lots;
- overflow parking areas;
- residential driveways;
- residential street parking lanes;
- recreational trails;
- golf cart and pedestrian paths; and
- emergency vehicle and fire access lanes.

Porous paver systems shall not be used in high traffic areas due to the potential for cell compaction and failure. Examples of paver systems that have been used for some of the above listed applications are presented in Figures 4-46 and 4-47.

Figure 4-46. Examples of Porous Paver Surfaces

(Sources: Invisible Structures, Inc.; EP Henry Corp.)

Figure 4-47. Typical Modular Porous Paver System Applications
(Source: Urban Drainage and Flood Control District, 2004)



4.3.11.5 Planning and Design Standards

Knox County's design standards for modular porous paver systems are presented below. Design specifications developed by a commercial vendor for prefabricated proprietary systems can also be utilized, but must be approved where such specifications differ and/or are less stringent from the standards presented below. The Director shall have the authority to require additional design conditions if deemed necessary.

<u>A. CONSTRUCTION SEQUENCING</u>

- Care should be taken during construction to minimize the compaction of the soil in the area of the porous paver system and the deposition of sediments from disturbed, unstabilized areas to the system after its installation. This can be minimized or avoided by proper construction sequencing.
- Ideally, the construction of the porous paver system should take place **<u>after</u>** the construction site has been stabilized. In the event that the system is not constructed after site stabilization, diversion of site runoff around the system and installation and maintenance of appropriate erosion prevention and sediment controls prior to site stabilization is required.
- Diversion berms shall be maintained around the paver system area during all phases of construction of the paver system. No runoff shall enter the area prior to completion of construction and the complete stabilization of construction areas. Erosion prevention and sediment controls shall be maintained around the paver system area to prevent runoff and sediment from entering the system during construction.
- Porous paver systems shall not be used as a temporary sediment trap for construction activities.

- During and after excavation of the area where the porous paver system will be located, all excavated materials shall be placed downstream, away from the porous paver location, to prevent redeposition of the material during runoff events.

B. LOCATION AND SITING

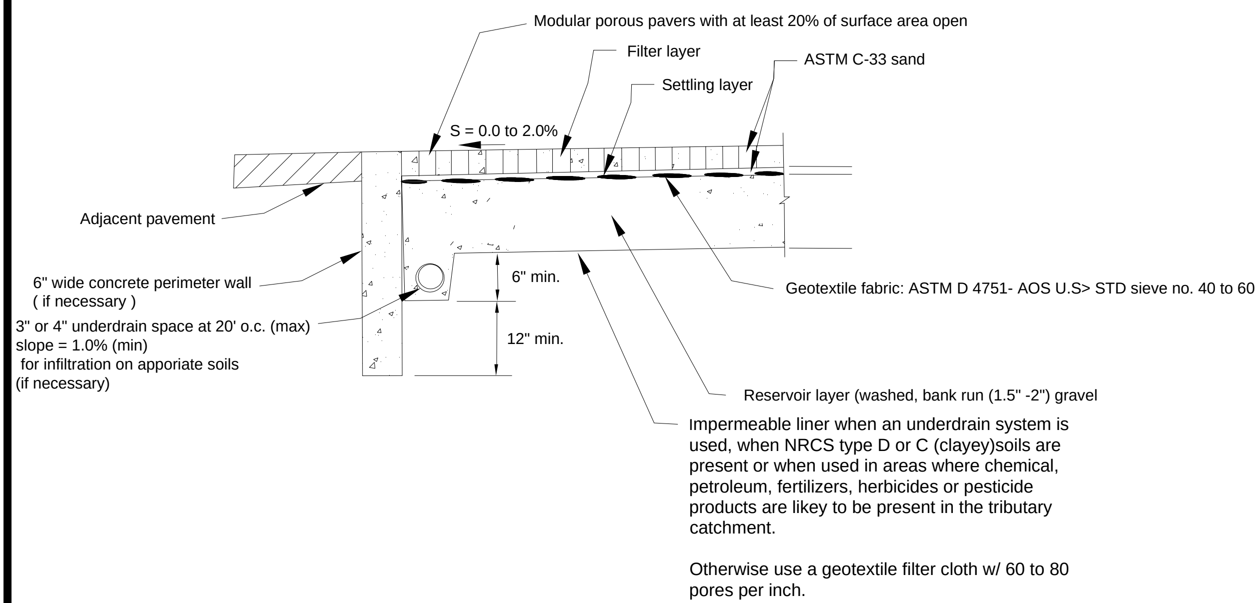
- The use of porous paver systems is limited to low traffic volume areas, such as those identified above, that have a minimum soil infiltration rate of 0.5 in/hr, if an underdrain system is not present.
- Geotechnical testing of potential installation sites is required to verify an acceptable infiltration rate.
- Modular porous paver systems should not be located:
  - Within 4 feet above bedrock or the seasonally high water level,
  - Within 100 feet of a well,
  - Within 10 feet of a building foundation that is above the proposed porous paver area or 100 feet from a building foundation that is below the proposed porous paver location,
  - Within close proximity of sources of contaminants such as gas stations,
  - On slopes greater than 5%.
- Because porous paver systems are not stormwater control devices, ideally, the area where the porous paver system is located should not receive stormwater runoff discharges from other areas. However, if that situation cannot be avoided, pretreatment of the discharges must be performed to remove sediment and other solids that can clog the porous paver system. Further, stormwater runoff discharging to the paver system area must flow into the area in a manner that will not cause damage to, or undermine, the porous paver system. Low velocity, unchannelized discharges are most favorable.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

Several options exist for the top layer or surface of modular porous paver systems. The top layer should be chosen depending on strength required due to traffic loads, infiltration needs, and the manufacturer's recommendations. However, the sub-layers are generally similar, consisting of four to five layers as shown in Figure 4-48. Descriptions of each layer shown in Figure 4-48 are presented below:

- The *Modular Porous Paver Layer* shall consist of a modular pavement grid of plastic, concrete, or brick and an aggregate or a vegetation medium. The depth of this layer shall be 2 to 8 inches deep depending on required bearing strength, pavement design requirements, and manufacturer's specifications.
- The *Settling Layer* shall consist of a 0.5-inch diameter crushed stone to a depth of 1 to 2 inches. This layer serves to stabilize the porous cell layer and can be combined with the reservoir layer using suitable stone.
- The *Reservoir Layer* (or Open Graded Base Material) shall consist of washed, bank-run gravel, 1.5 to 2.5 inches in diameter with a void space of about 40%. The depth of this layer depends on the desired storage volume, which is a function of the soil infiltration rate and void spaces, but typically ranges from two to four feet. The layer must have a minimum depth of 9 inches. The layer shall be designed to drain completely in 48 hours and to store, at a minimum, the WQv. Aggregate contaminated with soil is prohibited for use in this layer. The aggregate reservoir layer can be avoided or minimized if the subgrade is sandy and there is adequate time to infiltrate the necessary runoff volume into the sandy soil without by-passing the water quality volume. Consult the manufacturer's specifications to determine the appropriate layer design.

Figure 4-48. Modular Porous Pavement Layers
(Source: Urban Drainage and Flood Control District, 2004)



- The *Bottom Filter Layer* (not shown in diagram) is not always required. In cases where infiltration needs to be increased, a 6 inch layer of sand or a 2 inch thick layer of 0.5 inch crushed stone can be installed. The layer shall be graded to be completely flat to promote infiltration across the entire surface. This layer serves to stabilize the reservoir layer, to protect the underlying soil from compaction, and act as the interface between the reservoir layer and the filter fabric covering the underlying soil.
- A *Lateral Flow Barrier* as shown in Figure 4-49 is recommended around the modular porous paver area to prevent flow of water downstream and then surfacing at the toe of the porous paver installation. If the porous paver system is large enough, it may be divided into cells with cut-off barriers (also called cell walls) having a maximum distance (L_{max}) between them that shall not exceed:

Equation 4.3.11.1

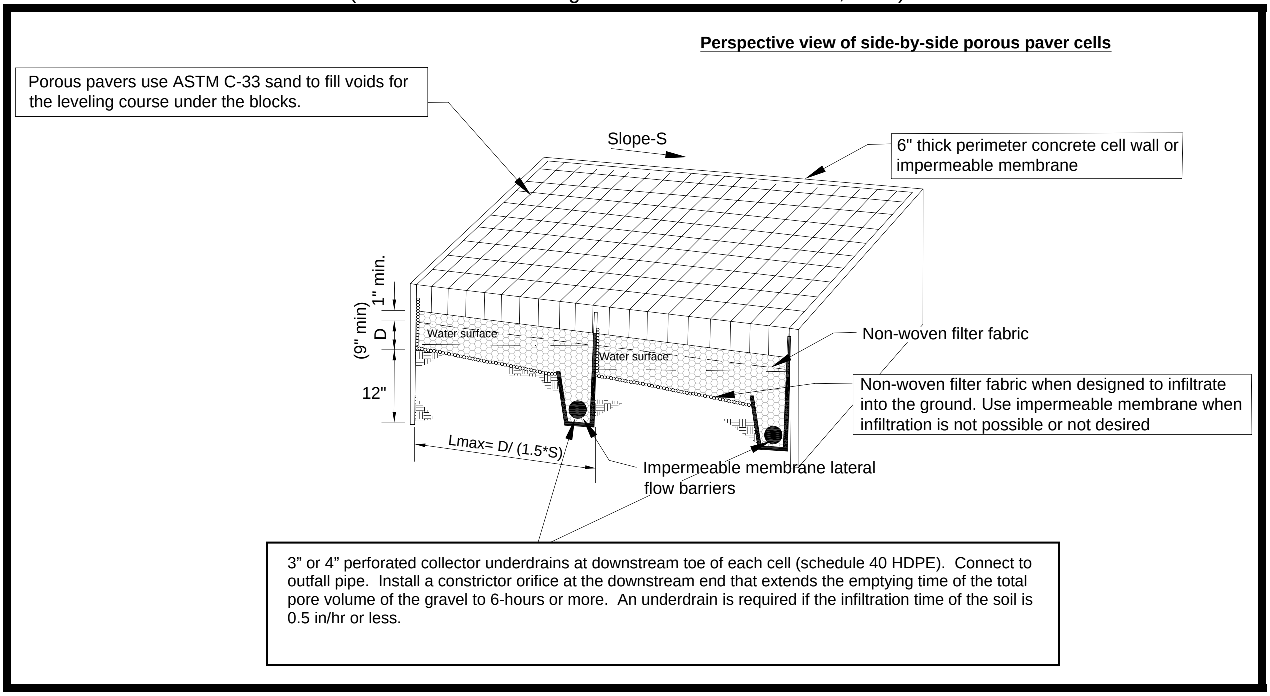
$$L_{max} = \frac{D}{1.5S}$$

where: L_{max} = Maximum distance between cut-off membrane normal to the flow (ft)
S = Slope of the reservoir layer (ft/ft)
D = Depth of reservoir layer (ft)

- *Filter Fabric* serves to inhibit soil from migrating into the reservoir and reducing storage capacity. The entire trench area, including the sides, shall be lined with filter fabric prior to placement of the aggregate. The filter fabric shall be replaced by an impermeable liner where the infiltration rate is less than 0.5”/hr.
- The *Underlying Soil* shall have an infiltration capacity of at least 0.5-inches/hour, but preferably greater than 0.5-inches/hour when an underdrain system is not present. Soils at the lower end of this range may not be suited for a full infiltration system or may require additional infiltration measures such as a perforated pipe or additional sand layer. Test borings are recommended to determine the soil classification, seasonal high ground water table elevation, and impervious substrata, and an initial estimate of permeability.

Figure 4-49. Schematic of Lateral Flow Barriers

(Source: Urban Drainage and Flood Control District, 2004)



- The *Underdrain System* (if required) shall be designed per the modular porous paver system manufacturers' recommendation or a typical underdrain schematic (taken from the Urban Storm Drainage Criteria Manual-Volume 3 - Best Management Practices of the Urban Drainage and Flood Control District in Denver, Colorado). An underdrain system is shown in Figure 4-50.

D. PRETREATMENT

- Stormwater runoff that discharges to the modular porous paver system from surrounding areas require pretreatment to remove sediment and debris. Pretreatment can be provided by a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the stormwater flow prior to run-on to the area covered by porous pavers.

  If a sediment forebay is used, it shall be sized to contain 0.1 inch per impervious acre (363 ft^3) of contributing drainage and shall be no more than 4 to 6 feet deep. The pretreatment storage volume is part of the total WQv design requirement and may be subtracted from the WQv calculated for the site.

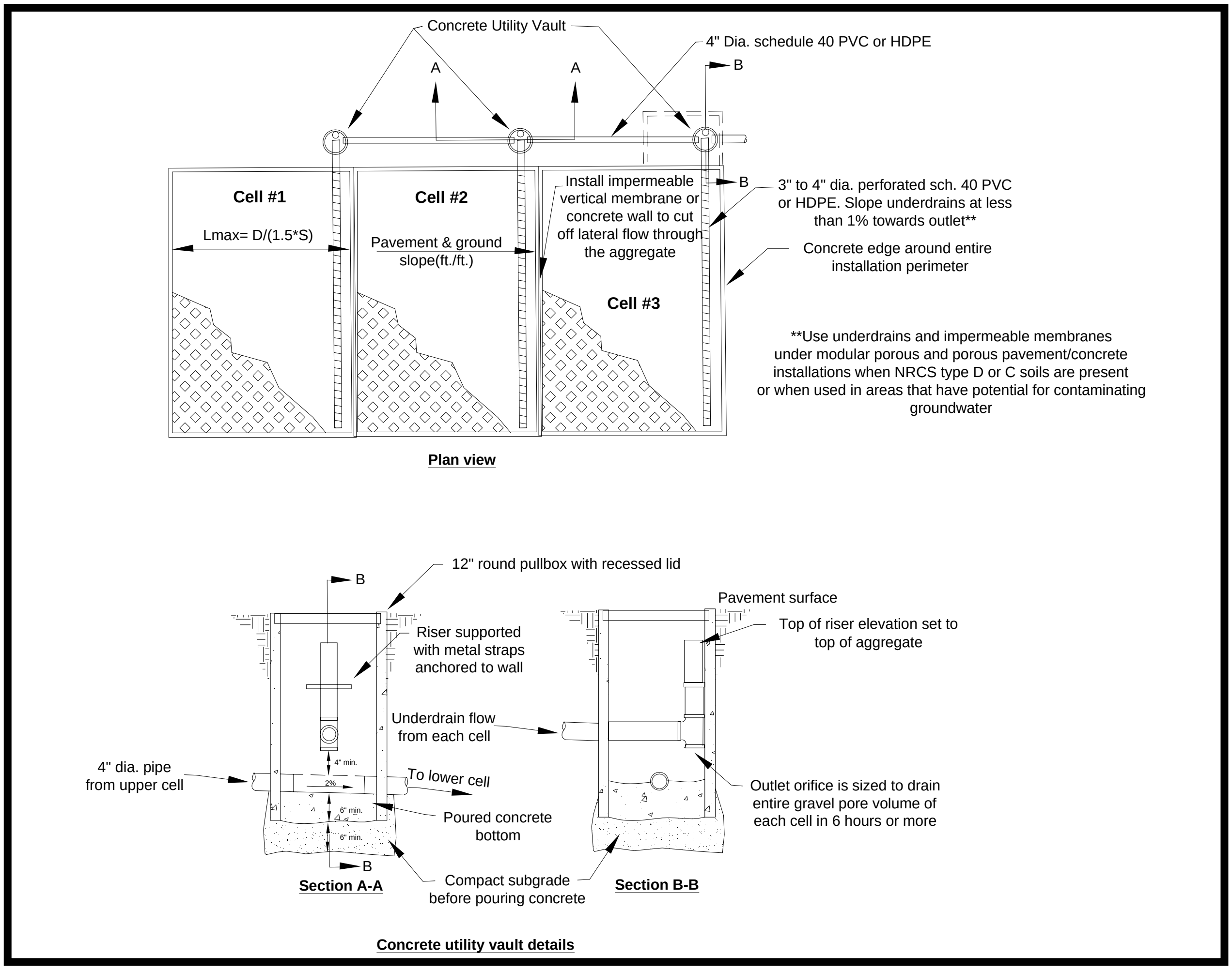
E. OUTLET STRUCTURES

- If an underdrain is incorporated into the design, an outlet pipe shall be provided from the underdrain system to the local stormwater conveyance system. Discharges shall not exit the outlet pipe in an erosive manner. Due to the slow rate of discharge, outlet erosion protection is generally unnecessary for modular porous pavement systems.

F. MAINTENANCE ACCESS

- A minimum 20' wide maintenance right of way or easement shall be provided from a driveway, public or private road. The maintenance access easement shall have a minimum unobstructed drive path width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles.

Figure 4-50. Schematic of an Underdrain System
(Source: Urban Drainage and Flood Control District, 2004)



G. LANDSCAPING

- Porous paver systems can be designed with a grass cover to aid in pollutant removal and prevent clogging. The grass should be capable of withstanding traffic and parking requirements, and frequent periods of inundation and drought.
- Ideally, landscaped areas that may discharge, or are adjacent to, the porous paver system should consist largely of grassy vegetation and have no exposed soil. Mulch, sticks, and leaves are debris that can clog the surface of the paver system, reducing its ability to infiltrate stormwater runoff and potentially affecting the structural integrity of the system. If such landscaped areas are utilized near the paver system, care should be taken to design and maintain the landscaped area in a manner and frequency that prevents such debris from entering the paver area, or ensures frequent removal of such debris from the area. For example, maintenance practices should increase during the fall to remove leaves from the paver system if deciduous trees are located near the system. Mowing on and around the system should be performed using a bagging mower.

H. SPECIAL CONSIDERATIONS FOR THE AS-BUILT CERTIFICATION AND PLANS/PLATS

- Because the use of the modular porous paver area reduces the WQv for the site and provides for stormwater treatment of some pollutants, the area must be shown on the as-built certification and the final plat specifically as a water quality BMP. The following components must be addressed in the as-built certification and final plat:

1. The boundaries of the porous pavement area, clearly identified with a note that states “Do not pave over this area. Only porous pavement is allowed in this area”.
2. Clear identification of the manufacturer and type of paver system used.
3. A copy of the manufacturer’s specifications for the design and installation of the system.
4. The underdrain design and specifications (if an underdrain is utilized).

4.3.11.6 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.11.6 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective use of porous paver systems as stormwater best management practices. It is the responsibility of the property owner to maintain all stormwater facilities in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for porous paver systems, along with a suggested frequency for each activity. Individual porous paver systems may have more, or less, frequent maintenance needs, depending upon a variety of factors including traffic volume, overly wet or dry (i.e., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain porous paver systems properly at all times.

Inspection Activities	Suggested Schedule
• Determine if the porous paver surface is free of sediment and debris (e.g., mulch, leaves, trash, etc.). • Determine if standing water exists for long periods of time after a storm event.	As needed
• Check that stormwater is not stored in the paver system longer than 48 hours after a storm. • Inspect vegetated areas that drain to the paver system and the paver system itself for evidence of erosion.	Monthly
• Inspect the surface of the paver system for structural integrity, deterioration, compaction, or spalling.	Annually
Maintenance Activities	**Suggested Schedule**
• Ensure that contributing area and porous paver surface are clear of debris (e.g., mulch, leaves, trash, etc.). • Stabilize (i.e., cover exposed soil) vegetated areas that discharge, or are adjacent to, the porous paver system. Grassy areas should be fully vegetated and mowed, with grass clippings removed. Landscaped areas should be designed and/or maintained such that they will not discharge debris (e.g., mulch, leaves) to the paver system, or that such debris is removed often.	As needed
• Vacuum sweep porous paver surface to keep free of sediment.	Quarterly
• Repair or reinstall the porous paver system, including the top and base course.	As needed

Additional Maintenance Considerations and Requirements

- Additional maintenance requirements for a porous paver system should be obtained from the manufacturer of the system and included in the Operations and Maintenance Plan for the site.
- Knox County encourages the use of the inspection checklist presented below for guidance in the inspection and maintenance of the porous paver system. Additional items should be added to the list, based on the inspection and maintenance information provided by the manufacturer of the pavers. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the unit. Questions regarding inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST – POROUS PAVEMENT SYSTEMS

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ____

Date: __________ Time: ____________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Signs of clogging (e.g., standing water)?		
Debris (e.g., mulch, trash) accumulation?		
Sediment accumulation?		
Standing water?		
Erosion from paver system underdrain?		
Exposed soil in areas discharging or adjacent to the paver system?		
Other (describe)?		
Other (describe)?		
Other (describe)?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature:______________________________ Inspector Name (printed)______________________________

4.3.11.7 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual, Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

City of Knoxville. *Knoxville Best Management Practices Manual.* City of Knoxville Stormwater Engineering Division, March 2003.

Metropolitan Council. *Minnesota Urban Small Sites BMP Manual.* Metropolitan Council Services, St. Paul Minnesota, 2001.

Urban Drainage and Flood Control District, Denver, Colorado. *Urban Storm Drainage Criteria Manual – Volume 3 – Best Management Practices – Stormwater Quality.* 2004

4.3.11.8 Suggested Reading

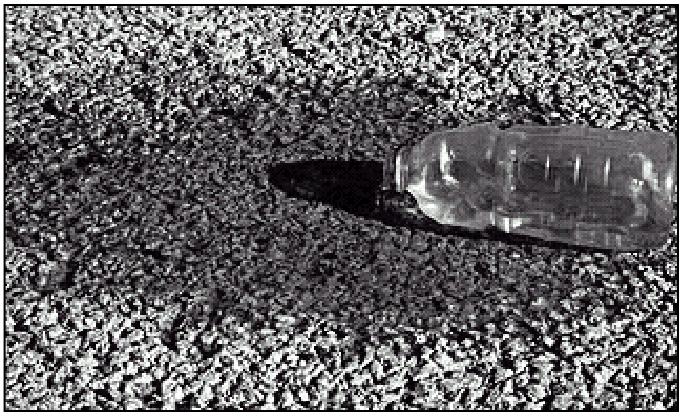
California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

US EPA. *Storm Water Technology Fact Sheet: Modular Treatment Systems.* EPA 832-F-99-044, Office of Water, 1999.

This page left intentionally blank

4.3.12 Porous Pavement

General Application Stormwater BMP



Description: Infiltration practices that are alternatives to traditional asphalt and concrete surfaces. Stormwater runoff is infiltrated into the ground through a permeable layer of pavement and is naturally filtered.

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Design considerations are similar to any paved area (soil properties, load-bearing design, hydrologic design of pavement and subgrade).
- Soil infiltration rate of 0.5 in/hr or greater is required if no underdrain is present.
- The infiltration rate of native soil determines appropriateness and need for an underdrain.
- Not appropriate for heavy or high traffic areas.
- Not appropriate as a water quality treatment BMP for drainage discharged from other areas.

ADVANTAGES / BENEFITS:

- Reduces runoff volume, attenuates peak runoff rate and outflow.
- Can be used as pretreatment for other BMPs for pollutants other than TSS.
- High level of pollutant removal for pollutants other than TSS.

DISADVANTAGES / LIMITATIONS:

- Sediment-laden runoff can clog porous pavement causing failure.
- Subgrade cannot be overly compacted.
- Construction must be sequenced to avoid compaction and clogging of pavement.

MAINTENANCE REQUIREMENTS:

- Vacuum to increase porous pavement life and avoid clogging.
- Ensure that contributing area is clear of debris and areas of erosion.

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality**
- [ ] **Channel/Flood Protection**
- [ ] **Overbank Flood Protection**
- [ ] **Extreme Flood Protection**

Accepts runoff from SPAP land uses: *Yes, but does not provide stormwater treatment.*

FEASIBILITY CONSIDERATIONS

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

Residential/Subdivision Use: *Yes*

Soils: *Not appropriate for use with hydrologic soil groups "D" and "C" without underdrain.*

POLLUTANT REMOVAL

L	**Total Suspended Solids**
H	**Nutrients**: Total Phosphorus / Total Nitrogen
H	**Metals**: Cadmium, Copper, Lead, and Zinc
No Data	**Pathogens**: Coliform, Streptococci, E.Coli

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

4.3.12.1 General Description

Porous pavement is a paved concrete or asphalt driving surface that permits the infiltration of water through the pavement and into the underlying soil. When considering the post-development stormwater runoff from a site, porous pavement is a best management practice (BMP) that allows a developed land surface to "appear" more like undeveloped land – runoff volumes and peak discharges of stormwater runoff from a developed site with porous pavement will be less than on a site without porous pavement. Porous pavement is an excellent application to reduce the effective impervious area on a site, therefore, reducing the design volumes and peak discharges that must be controlled. This will allow a reduction in the cost of other stormwater infrastructure, a fact that may offset the greater placement cost somewhat. Porous pavement can also eliminate problems with standing water, provide for groundwater recharge, control erosion of streambeds and riverbanks, facilitate pollutant removal, reduce thermal pollution of receiving waters, and provide for a more aesthetically pleasing site. **Porous pavement is not a BMP that can be used to remove total suspended solids (TSS).**

There are two types of porous pavement: porous asphalt and pervious concrete. Porous asphalt pavement consists of open-graded coarse aggregate, bonded together by asphalt cement, with sufficient interconnected voids to make it highly permeable to water. Pervious concrete consists of a specially formulated mixture of Portland cement, uniform, open-graded coarse aggregate, and water. Pervious concrete has enough void space to allow rapid percolation of water through the pavement. The void space in pervious concrete is in the 15%-22% range compared to 3%-5% for conventional pavements. The permeable surface is placed over a layer of open-graded gravel and crushed stone. The void spaces in the stone act as a storage reservoir for runoff. Pervious concrete is considered to be more durable than porous asphalt and is thought to have a greater ability than pervious asphalt to maintain its porosity in hot weather.

Porous pavements are best applied in areas that experience low vehicular traffic including parking lots and overflow parking areas; portions of streets such as residential parking lanes; driveways; plazas; and pedestrian or golf cart paths. Porous pavements are not recommended, and will not be approved, for use on driving surfaces that experience high traffic volume, heavy loads, and sediment-laden traffic (e.g., construction areas, dump sites).

A drawback to porous pavement is the cost and complexity of it compared to conventional pavements. Porous pavement requires a very high level of construction workmanship to ensure that it functions as designed. Like any BMP, porous pavement can fail, either for use as a driving/parking surface or an impervious area reduction measure, when improperly designed, constructed, or used. Past failures of porous pavement have been attributed to poor design, inadequate construction techniques, soils with low permeability, heavy vehicular traffic and poor maintenance (USEPA, 1999). This measure, if used, should be monitored and maintained over the life of the development.

Porous pavement is designed primarily for impervious area reduction and the subsequent reduction in stormwater treatment volumes and peak discharges, particularly for smaller storm events. For some smaller sites, trenches can be designed to capture and infiltrate the water quality volume (WQv), and in some cases, the channel protection volume (CPv). Modifications or additions to the standard design presented in this section have been used to pass flows and volumes in excess of the WQv, or to increase storage capacity or treatment. These include:

- placing a perforated pipe near the top of the crushed stone reservoir to pass excess flows after the reservoir is filled;
- providing surface detention storage in a parking lot, adjacent swale, or detention pond with suitable overflow conveyance;
- connecting the stone reservoir layer to a stone filled trench;
- adding a sand layer and perforated pipe beneath the stone layer for filtration of the water quality volume; or,
- placing an underground detention tank or vault system beneath the layers.

Porous pavement has the positive characteristics of volume reduction due to infiltration, groundwater recharge, and an ability to blend into the normal urban landscape relatively unnoticed.

4.3.12.2 Stormwater Management Suitability

Water Quality (WQv)
Porous pavement is designed solely for impervious area reduction and stormwater quality treatment of pollutants other than TSS. Porous pavements shall not be used for TSS removal. These pavements require some pretreatment BMP such as a filter strip for runoff entering the pavement to prevent clogging from sediment.

Channel Protection (CPv), Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})
Porous pavement will not provide for channel, overbank or extreme flood protection. Another structural BMP, such as an extended dry detention pond, that is designed to handle flood control, must be used in conjunction with the porous pavement to achieve the Qp_2, Qp_{10}, Qp_{25} and Qp_{100} design criteria.

4.3.12.3 Pollutant Removal Capabilities

Porous pavement has a high removal of soluble pollutants, where they become trapped, absorbed or broken down in the underlying soil layers. Due to the potential for clogging, porous pavement surfaces shall **not** be used for the removal of sediment or other particulate pollutants.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- Total Suspended Solids – not applicable
- Total Phosphorus – 80%
- Total Nitrogen – 80%
- Pathogens – Insufficient data to provide a pollutant removal value
- Heavy Metals – 90%

4.3.12.4 Application and Feasibility Criteria

Porous pavement is applicable only for use in low-traffic areas that do not encounter heavy loads and/or sediment-laden traffic or runoff, such as:

- parking pads in parking lots;
- overflow parking areas;
- residential driveways;
- residential street parking lanes;
- recreational trails;
- golf cart and pedestrian paths; and,
- emergency vehicle and fire access lanes.

4.3.12.5 Planning and Design Standards

Knox County's design standards for porous pavement are presented below. Design specifications developed by a commercial vendor for prefabricated proprietary systems can also be utilized, but must be approved where such specifications differ and/or are less stringent from the standards presented below. The Director shall have the authority to require additional design conditions if deemed necessary.

A. CONSTRUCTION SEQUENCING

- Care should be taken during construction to minimize the compaction of the soil in the area of the porous pavement and the deposition of sediments from disturbed, unstabilized areas to the system after its installation. This can be minimized or avoided by proper construction sequencing.
- Ideally, the construction of the porous pavement should take place **after** the construction site has been stabilized. In the event that the system is not constructed after site stabilization, diversion of site runoff around the system and installation and maintenance of appropriate erosion prevention and sediment controls prior to site stabilization is required.
- Diversion berms shall be maintained around the porous pavement area during all phases of construction. No runoff shall enter the area prior to completion of construction and the complete stabilization of construction areas. Erosion prevention and sediment controls shall be maintained around the porous pavement area to prevent runoff and sediment from entering the area during construction.
- Porous pavement shall not be used as a temporary sediment trap for construction activities.
- During and after excavation of the porous pavement area, all excavated materials shall be placed downstream, to prevent redeposition of the material during runoff events.

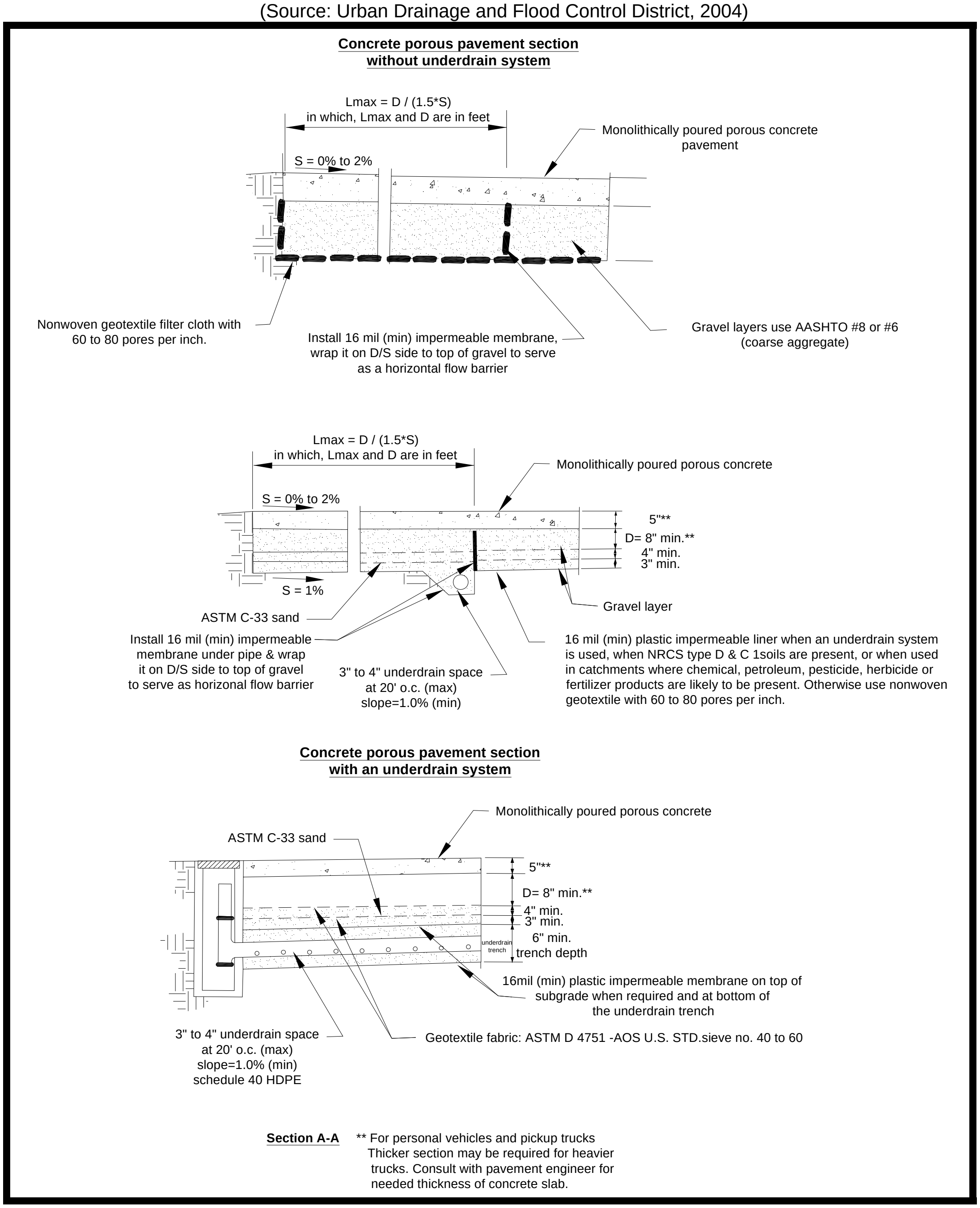
B. LOCATION AND SITING

- Suitable sites for porous pavement are limited to low traffic volume areas with a minimum soil infiltration rate of 0.5 in/hr without an underdrain system. Ideally, the soil should allow the entire runoff capture volume to be discharged from the porous pavement within 24 to 48 hours.
- Geotechnical testing of the proposed installation site is required to verify an acceptable infiltration rate.
- Porous pavement shall **not** be located:
  - within two (2) feet above bedrock or the seasonally high water level,
  - within 100 feet of a well,
  - within ten (10) feet of a building foundation that is above the proposed porous pavement area or 100 feet from a building foundation that is below the proposed porous pavement location,
  - within close proximity of sources of contaminants such as gas stations,
  - on slopes greater than 5%.
- Ideally, slopes should be flat or nearly flat to facilitate infiltration as opposed to runoff.
- The seasonally high water table or bedrock should be at least two feet below the bottom of the gravel layer if infiltration is to be relied on to remove the stored volume.
- Because porous pavement is not a stormwater control device, the area where the porous pavement is located should not receive stormwater runoff discharges from other areas. However, if that situation cannot be avoided, pretreatment of the discharges must be performed to remove sediment and other solids that can clog the porous pavement. Further, stormwater runoff discharging to the porous pavement area must flow into the area in a manner that will not cause damage to, or undermine, the porous pavement. Low velocity, unchannelized discharges are most favorable.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

Porous asphalt or pervious concrete for the top layer or surface of the porous pavement should be chosen depending on strength required due to traffic loads, infiltration needs, and other site constraints. However, the sub-layers are generally similar, consisting of four to five layers as shown in Figure 4-51. The aggregate reservoir layer can sometimes be avoided or minimized if the subgrade is sandy and if there is adequate time to infiltrate the water quality volume into the sandy soil without bypassing any of the water quality volume. Descriptions of each of the layers is presented below.

Figure 4-51. Porous Pavement Layers
(Source: Urban Drainage and Flood Control District, 2004)



- *Porous Pavement Layer* – This layer consists of a porous mixture of concrete or asphalt or a modular pavement grid of plastic, concrete, or brick and an aggregate or a vegetation medium. This layer is usually 4 to 8 inches deep depending on required bearing strength, pavement design requirements, and manufacturer's specifications.

- *Reservoir Layer or Open Graded Base Material* – The reservoir gravel base layer consists of washed, bank-run gravel, 1.5 to 2.5-inches in diameter with a void space of about 40%. The depth of this layer depends on the desired storage volume, which is a function of the soil infiltration rate and void spaces, but typically ranges from two to four feet. The layer must have a minimum depth of nine inches. The layer shall be designed to drain completely in 48 hours. If the porous pavement area is being utilized for stormwater quality treatment (for pollutants other than sediment/TSS), then the area must be designed to store, at a minimum, the WQv. Aggregate contaminated with soil shall not be used for the reservoir layer.

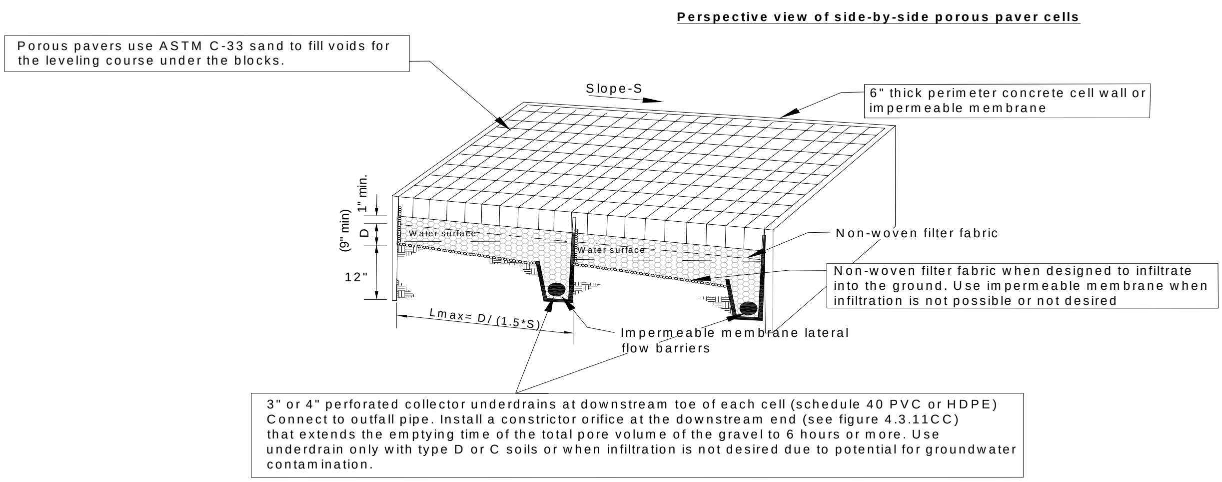
- *Bottom Filter Layer* – In cases where infiltration needs to be increased, a 6-inch layer of sand or a 2-inch thick layer of 0.5-inch crushed stone can be installed. The layer must be completely flat to promote infiltration across the entire surface. This layer serves to stabilize the reservoir layer, to protect the underlying soil from compaction, and act as the interface between the reservoir layer and the filter fabric covering the underlying soil.
- A *Lateral Flow Barrier -* as shown in Figure 4-52 is recommended around the porous pavement area to prevent flow of water downstream and then surfacing at the toe of the porous pavement installation. If the porous pavement area is large enough, it may be divided into cells with cut-off barriers having a maximum distance (L_{max}) between them that shall not exceed:

Equation 4.3.12.1

$$L_{\max} = \frac{D}{1.5S}$$

where: L_{max} = Maximum distance between cut-off membrane normal to the flow (ft)
S = Slope of the reservoir layer (ft/ft)
D = Depth of reservoir layer (ft)

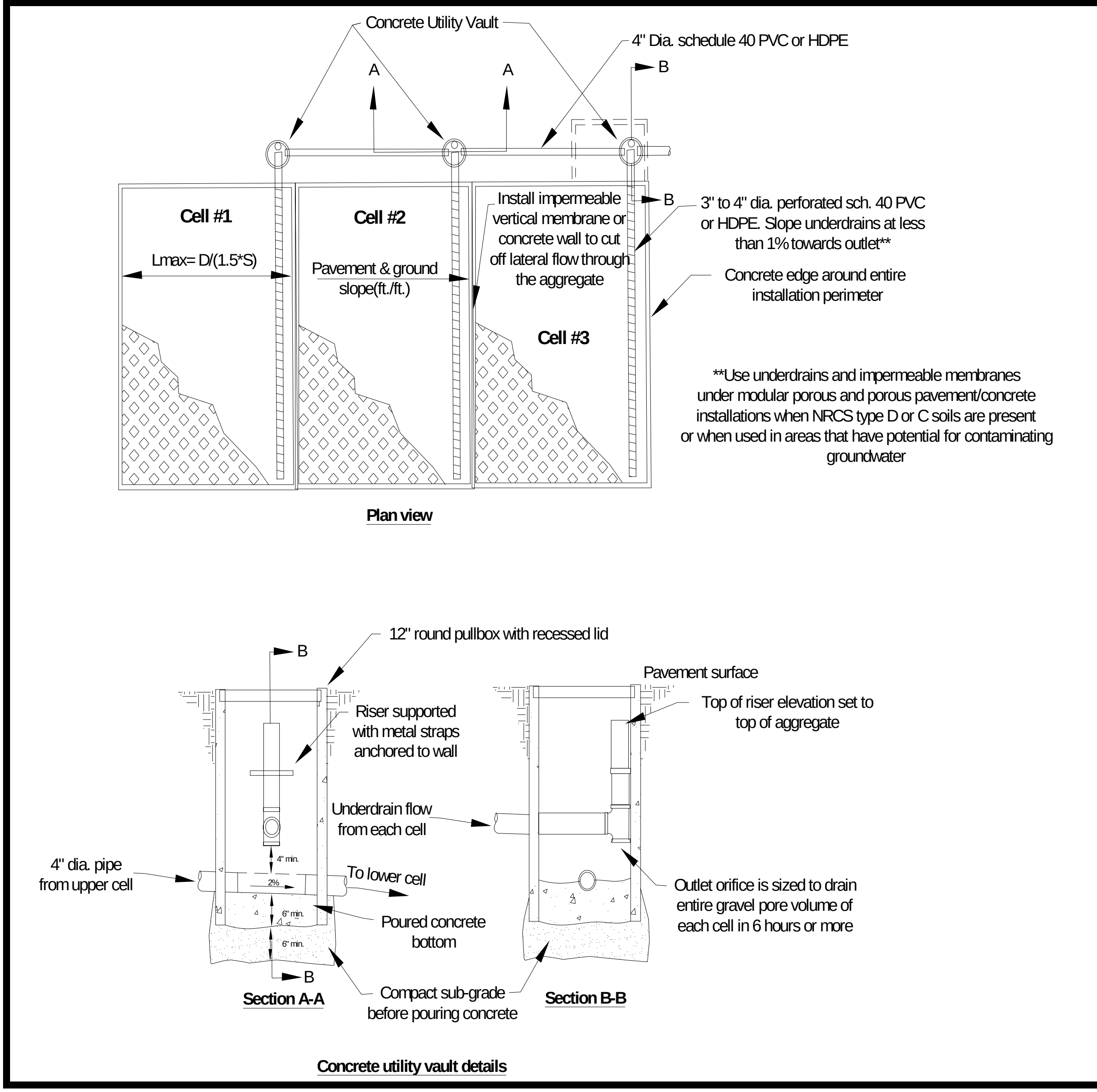
Figure 4-52. Schematic of Lateral Flow Barriers
(Source: Urban Drainage and Flood Control District, 2004)



- *Filter Fabric* – It is very important to line the entire trench area, including the sides, with filter fabric prior to placement of the aggregate. The filter fabric serves to inhibit soil from migrating into the reservoir and reducing storage capacity.
- *Underlying Soil* – The underlying soil should have an infiltration capacity of at least 0.5-inches/hour but preferably greater than 0.5-inches/hour. Soils at the lower end of this range may not be suited for a full infiltration system or may require additional infiltration measures such as a perforated pipe or additional sand layer. Test borings are recommended to determine the soil classification, seasonal high ground water table elevation, and impervious substrata, and an initial estimate of permeability.

- The *Underdrain System* (if required) shall be designed per the porous pavement manufacturers' recommendation or through the use of another reference. A typical underdrain schematic is shown in Figure 4-53.

Figure 4-53. Schematic of an Underdrain System
(Source: Urban Drainage and Flood Control District, 2004)



D. PRETREATMENT

- Although it is not recommended that runoff from other areas be discharged to the porous pavement area, stormwater runoff that discharges to the porous pavement system from surrounding areas requires pretreatment to remove sediment and debris. Pretreatment can be provided by a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the stormwater flow prior to run-on to the area covered by porous pavement.
- If a sediment forebay is used, it shall be sized to contain 0.1 inch per impervious acre (363 ft^3) of contributing drainage and shall be no more than 4 to 6 feet deep. The pretreatment storage volume is part of the total WQv design requirement and may be subtracted from the WQv calculated for the site.

E. OUTLET STRUCTURES

- If an underdrain system is incorporated into the design, an outlet pipe shall be provided from the underdrain system to the local stormwater conveyance system. Discharges shall not exit the outlet pipe in an erosive manner. Due to the slow rate of discharge, outlet erosion protection is generally unnecessary for modular porous pavement systems.

F. MAINTENANCE ACCESS

- A minimum 20' wide maintenance right-of-way or easement shall be provided from a driveway, public or private road. The maintenance access easement shall have a minimum unobstructed drive path width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles.

G. LANDSCAPING

- Landscaped areas that may discharge, or are adjacent to, the porous pavement should consist largely of grassy vegetation and have no exposed soil. Mulch, sticks, grass clippings, and leaves are debris that can clog the surface of the porous pavement, reducing its ability to infiltrate stormwater runoff. If such landscaped areas are utilized near the porous pavement, care should be taken to design and maintain the landscaped area in a manner and frequency that prevents such debris from entering the porous pavement, or ensures frequent removal of such debris from the area. For example, maintenance practices should increase during the fall to remove leaves from the porous pavement if deciduous trees are located near the system. Areas surrounding the system should be mowed with a bagging mower.

H. SPECIAL CONSIDERATIONS FOR THE AS-BUILT CERTIFICATION AND PLANS/PLATS

- Because the use of porous pavement reduces the WQv for the site and provides for stormwater treatment of some pollutants, the area must be shown on the as-built certification and the final plat specifically as a water quality BMP. The following components must be addressed in the as-built certification and final plat:

  1. The boundaries of the porous pavement area, clearly identified with a note that states "Pervious pavement area. Do not pave with impervious pavement surfaces."
  2. Clear identification of the type of porous pavement used.
  3. The underdrain design and specification (if an underdrain is utilized).

4.3.12.6 Maintenance Requirements and Inspection Checklist

Note: Section 4.3.12.6 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective use of porous pavement as a stormwater best management practice. It is the responsibility of the property owner to maintain all stormwater facilities in accordance with the minimum design standards and other guidance provided in this manual. The Director of Engineering and Public Works has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for porous pavement, along with a suggested frequency for each activity. Individual porous pavement applications may have more, or less, frequent maintenance needs, depending upon a variety of factors including traffic loads, the occurrence of large storm events, overly wet or dry (i.e., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain porous pavement properly at all times.

Inspection Activities	Suggested Schedule
• Ensure that the porous pavement surface is free of sediment and debris (e.g., mulch, leaves, trash, etc.). • Ensure that the contributing area upstream of the porous pavement surface is free of sediment and debris.	As needed
• Check to make sure that the porous pavement dewaters between storms.	Monthly
• Inspect the surface for structural integrity. Inspect for evidence of deterioration or spalling.	Annually
Maintenance Activities	**Suggested Schedule**
• Ensure that contributing area and porous pavement surface are clear of debris (e.g., mulch, leaves, trash, etc.). • Ensure that the contributing and adjacent area is stabilized and mowed, with clippings removed.	As needed, based on inspection
• Vacuum sweep porous pavement surface to keep free of sediment.	Typically three to four times a year
• Replace the porous pavement, including the top and base course, as needed.	Upon failure

Knox County encourages the use of the inspection checklist presented below for guidance in the inspection and maintenance of porous pavement. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the unit. Questions regarding inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST – POROUS PAVEMENT

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ____

Date: ___________ Time: ______________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Signs of clogging (e.g., standing water)?		
Debris (mulch, trash) accumulation?		
Sediment accumulation?		
Standing water?		
Erosion from underdrain (if present)?		
Exposed soil in areas discharging or adjacent to the porous pavement area?		
Runoff discharge from pavement area 24 to 48 hours after the end of a storm event?		
Other (describe)?		
Other (describe)?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature:________________________________ Inspector Name (printed)_________________________________

4.3.12.7 Example Schematics

Figure 4-54. Porous Pavement Installation

Figure 4-55. Typical Porous Pavement Applications
(Photos by Bruce Ferguson, Don Wade)

4.3.12.8 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual, Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

City of Knoxville. *Knoxville Best Management Practices Manual.* City of Knoxville Stormwater Engineering Division, March 2003.

Metropolitan Council. *Minnesota Urban Small Sites BMP Manual.* Metropolitan Council Services, St. Paul Minnesota, 2001.

Urban Drainage and Flood Control District, Denver, Colorado. *Urban Storm Drainage Criteria Manual – Volume 3 – Best Management Practices – Stormwater Quality.* 2004

4.3.12.9 Suggested Reading

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

US EPA. *Storm Water Technology Fact Sheet: Modular Treatment Systems.* EPA 832-F-99-044, Office of Water, 1999.

4.4.1 Organic Filter

Limited Application Stormwater BMP

Description: The organic filter is a design variation of the surface sand filter that uses organic media to filter stormwater, as opposed to sand. An organic filter has two chambers. The first chamber is used for settling of heavy pollutant particles. The second chamber is filled with organic media and used to filter out fine particles.

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Maximum drainage area of 10 acres.
- Minimum head requirement of 5 to 8 feet.
- Requires the use of a peat/sand mixture as the filter media.
- Runoff discharges to an underdrain system.
- Intended for hotspot or space-limited applications, or for areas requiring enhanced pollutant removal capability.

ADVANTAGES / BENEFITS:

- Useful for treatment of small drainage areas and highly impervious areas.
- Good retrofit capability.

DISADVANTAGES / LIMITATIONS:

- High maintenance burden.
- Not recommended for areas that have high sediment content in stormwater or clay/silt runoff areas.
- Relatively costly to install and maintain.
- Possible odor problems.
- Cannot be installed until site construction is complete.

MAINTENANCE REQUIREMENTS:

- Inspect for clogging.
- Remove sediment from forebay/chamber.
- Replace filter media as needed.
- Stabilize, clean and maintain upstream drainage areas.

STORMWATER MANAGEMENT SUITABILITY

✓	**Water Quality**
✷	**Channel/Flood Protection**
	Overbank Flood Protection
	Extreme Flood Protection

✷ in certain situations

Accepts runoff from SPAP land uses: *Yes*

FEASIBILITY CONSIDERATIONS

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

Residential/Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *10 acres maximum*

Soils: *Not recommended for clay/silt drainage areas that are not stabilized.*

POLLUTANT REMOVAL

H	**Total Suspended Solids**
M	**Nutrients**: Total Phosphorus / Total Nitrogen
M-H	**Metals**: Cadmium, Copper, Lead, and Zinc
M	**Pathogens**: Coliform, Streptococci, E.Coli

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

4.4.1.1 General Description

The organic filter is a design variant of the surface sand filter with the unique characteristic of using organic materials such as leaf compost or a peat/sand mixture as the filter media. The organic material enhances pollutant removal by providing adsorption of contaminants such as soluble metals, hydrocarbons, and other organic chemicals.

As with the surface sand filter, an organic filter consists of a pretreatment chamber, and one or more filter cells. Each filter bed contains a layer of leaf compost or the peat/sand mixture, followed by filter fabric and a gravel/perforated pipe underdrain system. The filter bed and subsoils can be separated by an impermeable polyliner or concrete structure to prevent movement into groundwater.

Organic filters are typically used in high-density applications, or for areas requiring enhanced pollutant removal ability. Maintenance is typically higher than the surface sand filter facility due to the need to reduce the potential for debris and sediment clogging the organic filter. In addition, organic filter systems have a higher head requirement than sand filters.

4.4.1.2 Stormwater Management Suitability

Organic filter systems are designed primarily as off-line systems for treatment of the water quality volume and will typically need to be used in conjunction with another structural BMP that can provide downstream channel protection, overbank flood protection, and extreme flood protection. However, under certain circumstances, organic filters can provide limited runoff quantity control, particularly for smaller storm events.

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

Channel Protection (CPv)
For smaller sites, an organic filter may be designed to capture the entire channel protection volume (CPv) in either an off- or on-line configuration. Given that an organic filter system is typically designed to completely drain over 40 hours, the channel protection design requirement for extended detention of the 1-year, 24-hour storm runoff volume can be met. For larger sites or where only the WQv is diverted to the organic filter facility, another structural control must be used to provide extended detention of the CPv.

Overbank Flood Protection (up to Qp25) and Extreme Flood Protection (Qp100)
Organic filters are not useful for flood protection. Another structural control, such as a conventional detention pond must be used in conjunction with an organic filter system to control stormwater peak discharges. Further, organic filter facilities must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the filter bed and facility.

4.4.1.3 Pollutant Removal Capabilities

Peat/sand filter systems provide good removal of bacteria and organic waste metals. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- Total Suspended Solids – 80%
- Total Phosphorus – 60%
- Total Nitrogen – 40%
- Heavy Metals – 75%
- Pathogens – 50%

For additional information and data on pollutant removal capabilities for organic filters, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the International Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

4.4.1.4 Application and Site Feasibility Criteria

Organic filter systems are well-suited for highly impervious areas where land available for structural BMPs is limited. Organic filters should primarily be considered for new construction or retrofit opportunities for commercial, industrial, and institutional areas where the sediment load is relatively low, such as: parking lots, driveways, loading docks, gas stations, garages, airport runways/taxiways, and storage yards. Organic filters may also be feasible and appropriate in some multi-family residential developments where maintenance is performed by a landscaping (or other suitably capable) company.

To avoid rapid clogging and failure of the filter media, the use of organic filters should be avoided in areas with less than 50% impervious cover, or high sediment yield sites with clay/silt soils.

The following basic criteria should be evaluated to ensure the suitability of an organic filter facility for meeting stormwater management objectives on a site or development.

General Feasibility

- Not generally suitable for use in a residential subdivision.
- Suitable for use in high density/ultra urban areas.
- Not suitable for use as a regional stormwater control. On-site applications are typically most feasible.

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – Ten (10) acres maximum
- Space Required – Function of available head at site
- Minimum Head – The surface slope across the filter location should be no greater than 6%. The elevation difference needed at a site from the inflow to the outflow is 5 to 8 feet.
- Minimum Depth to Water Table – If used on a site with an underlying water supply aquifer, a separation distance of 2 feet required between the bottom of the organic filter and the elevation of the seasonally high water table to prevent groundwater contamination.
- Soils – Not recommended for drainage areas with exposed soil. Karst areas may require a liner.

Other Constraints / Considerations

- Aquifer Protection – Do not allow infiltration of filtered hotspot runoff into groundwater

4.4.1.5 Planning and Design Standards

The following standards shall be considered **minimum** design standards for the design of organic filters. Organic filters that are not designed to these standards will not be approved. The Director of Engineering and Public Works (the Director) shall have the authority to require additional design conditions if deemed necessary.

A. CONSTRUCTION SEQUENCING

- Care shall be taken during construction to minimize the risk of premature failure of the organic filter due to deposition of sediments from disturbed, unstabilized areas. This can be minimized or avoided by proper construction sequencing.
- Ideally, the construction of an organic filter shall take place **after** the construction site has been stabilized. In the event that the organic filter is not constructed after site stabilization, diversion of site runoff around the organic filter and installation and maintenance of appropriate erosion prevention and sediment controls prior to site stabilization is required.

- Diversion berms shall be maintained around an organic filter during all phases of construction. No runoff shall enter the organic filter area prior to completion of construction and the complete stabilization of construction areas. Erosion prevention and sediment controls shall be maintained around the organic filter to prevent runoff and sediment from entering the organic filter during construction.
- Organic filters shall not be used as a temporary sediment trap for construction activities.
- During and after excavation of the organic filter, all excavated materials shall be placed downstream, away from the organic filters, to prevent redeposit of the material during runoff events.

B. LOCATION AND SITING

- Organic filters shall have a contributing drainage area of 10 acres or less.
- Organic filter systems are generally applied to land uses with a high percentage of impervious surfaces. Organic filters shall not be utilized for sites that have less than 50% impervious cover. Pretreatment must be provided as described in part E below, due to the potential for high clay/silt sediment loads that could result in clogging and failure of the filter bed. Any disturbed or denuded areas located within the area draining to and treated by the organic filter shall be stabilized prior to construction and use of the organic filter. The organic filter shall only be constructed after the construction site is stabilized.
- It is preferred that organic filters only be used in an off-line configuration where the WQv (and CPv if used for this purpose) is diverted to the filter facility through the use of a flow diversion structure and flow splitter. Stormwater flows greater than the WQv (and CPv if used for this purpose) are then diverted to other controls or downstream using a diversion structure or flow splitter.
- Organic filter systems shall be designed for intermittent flow and must be allowed to drain and re-aerate between rainfall events. They shall not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

C. GENERAL DESIGN

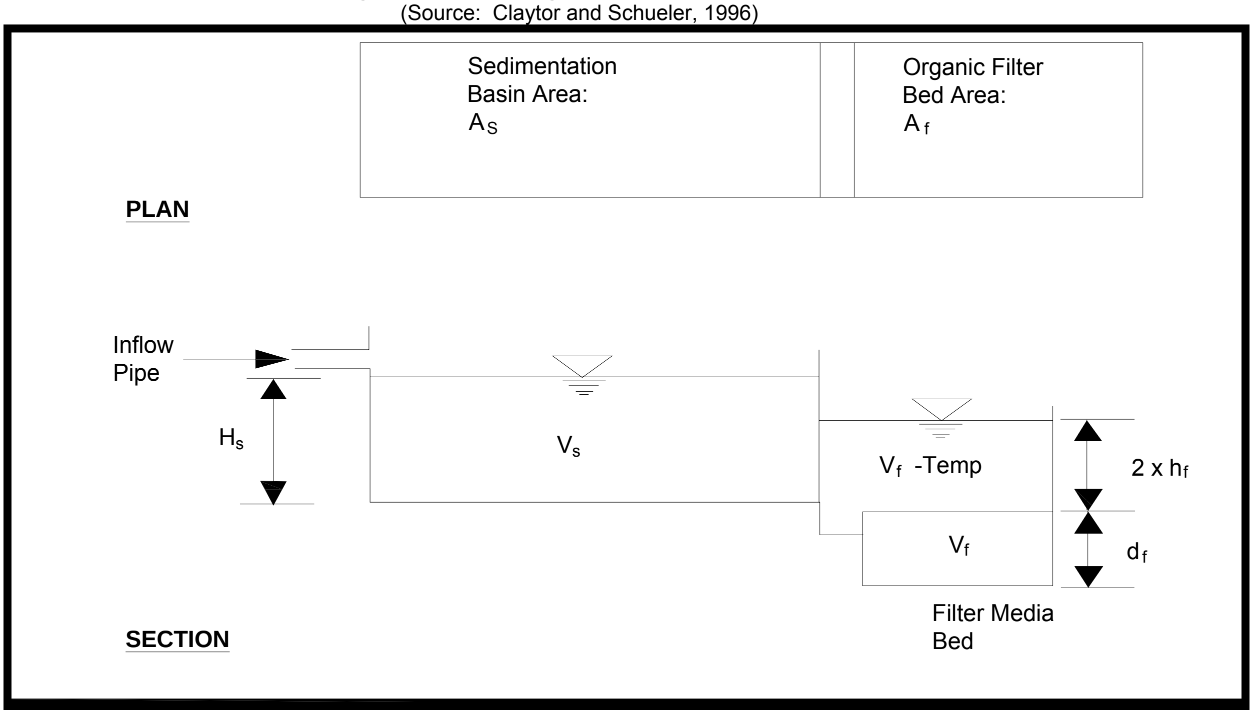
- An organic filter facility shall consist of a two-chamber open-air structure, which is located at ground-level. The first chamber is the sediment forebay (commonly referred to as the sedimentation chamber) while the second chamber houses the filtration chamber (organic filter bed). Flow enters the sedimentation chamber where settling of larger sediment particles occurs. Runoff is then discharged from the sedimentation chamber through a perforated standpipe into the filtration chamber. After passing though the filter bed, runoff is collected by a perforated pipe and gravel underdrain system. Figure 4-56 provides a plan view and profile schematic of an organic filter.
- Organic filters can utilize a variety of organic materials as the filtering media. Two typical media bed configurations are the peat/sand filter and compost filter (see Figure 4-56). The peat filter includes an 18-inch 50/50 peat/sand mix over a 6-inch sand layer and can be optionally covered by 3 inches of topsoil and vegetation. The compost filter has an 18-inch compost layer.
- The type of peat used in a peat/sand filter is critically important. Fibric peat in which undecomposed fibrous organic material is readily identifiable is the preferred type. Hemic peat containing more decomposed material may also be used. Sapric peat made up of largely decomposed matter should *not* be used in an organic filter.

D. PHYSICAL SPECIFICATIONS / GEOMETRY

- The entire organic filter treatment system (including the sedimentation chamber) shall be designed to temporarily hold at least 75% of the WQv prior to filtration. Figure 4-56 illustrates the distribution of the treatment volume (0.75 WQv) among the various components of the surface sand filter, including:
  - V_s – volume within the sedimentation basin
  - V_f – volume within the voids in the filter bed
  - $V_{f\text{-}temp}$ – temporary volume stored above the filter bed

- A_s – the surface area of the sedimentation basin
- A_f – surface area of the filter media
- h_s – height of water in the sedimentation basin
- h_f – average height of water above the filter media
- d_f – depth of filter media

Figure 4-56. Organic Filter Volumes
(Source: Claytor and Schueler, 1996)

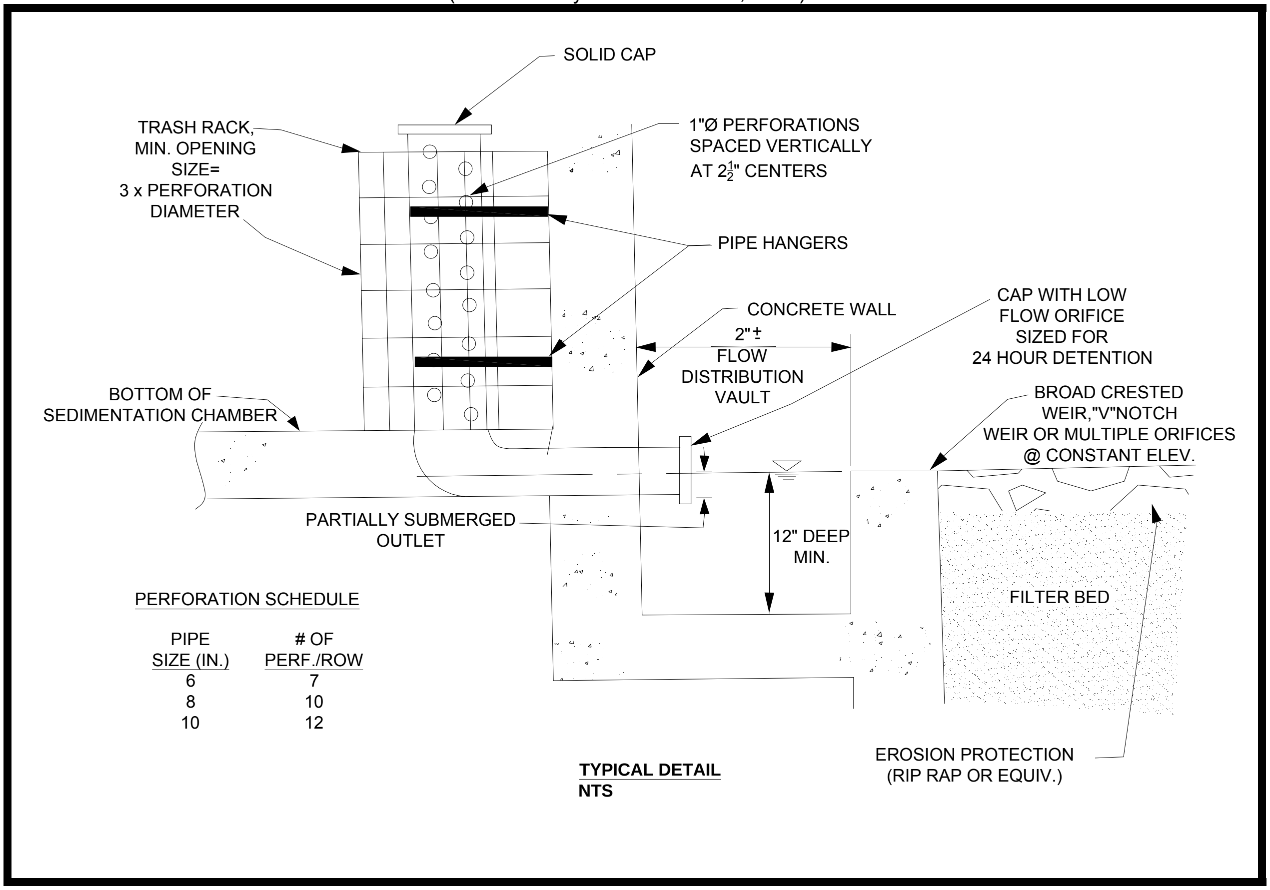


- The sedimentation chamber shall be sized to hold at least 25% of the computed WQv and have a length-to-width ratio of at least 2:1. Inlet and outlet structures should be located at opposite ends of the chamber.
- The filter area shall be sized based on the principles of Darcy's Law. A coefficient of permeability (k) of 3.5 ft/day for sand, 2.0 ft/day for peat and 8.7 ft/day for leaf compost shall be used. The filter bed shall be designed to completely drain in 40 hours or less.
- The filter media for an organic filter shall consist of either an 18" layer of peat/sand mixture on top of a 6" sand layer or an 18" layer of leaf compost. Both types of media are placed on top of the underdrain system. Three inches of topsoil shall be placed over the sand bed. Permeable filter fabric shall be placed both above and below the filter bed to prevent clogging of the filter media and the underdrain system. Figure 4-56 illustrates a typical media cross section.
- The filter bed shall be equipped with a 6-inch perforated pipe underdrain (PVC AASHTO M 252, HDPE, or other suitable pipe material) in a gravel layer (see figure 4-58). The underdrain shall have a minimum grade of 1/8-inch per foot (1% slope). Holes shall be 3/8-inch diameter and spaced approximately 6 inches on center. Gravel shall be clean-washed aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches with a void space of about 40%. Aggregate contaminated with soil shall not be used.
- The structure of the organic filter may be constructed of impermeable media such as concrete, or through the use of excavations and earthen embankments. When constructed with earthen walls/embankments, filter fabric shall be used to line the bottom and side slopes of the structures before installation of the underdrain system and filter media.

E. PRETREATMENT / INLETS

- Pretreatment of runoff in an organic filter system shall be by a sedimentation chamber, designed in accordance with the criteria stated above.
- Energy dissipators shall be used at the inlets to organic filters. Figure 4-57 shows a typical inlet pipe from the sedimentation basin to the filter media basin.
- The organic filter shall be designed such that runoff exits the sedimentation chamber at a non-erosive velocity.

Figure 4-57. Surface Sand Filter Perforated Stand-Pipe
(Source: Claytor and Schueler, 1996)



F. OUTLET STRUCTURES

- An outlet pipe shall be provided from the underdrain system to the facility discharge. Due to the slow rate of filtration, outlet protection is generally unnecessary (except for emergency overflows and spillways). However, the design shall ensure that the discharges from the underdrain system occur in a non-erosive manner.

G. EMERGENCY SPILLWAY

- An emergency or bypass spillway must be included in the organic filter design to safely pass flows that exceed the WQv (and CPv if the filter is utilized for channel protection purposes). The spillway prevents filter water levels from overtopping the embankment and causing structural damage. The emergency spillway shall be located so that embankments, downstream buildings and structures will not be impacted by spillway discharges.

H. MAINTENANCE ACCESS

- A minimum 20' wide maintenance right-of-way or drainage easement shall be provided for the organic filter from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles. Adequate access must be provided to the filter bed. Facility designs must enable maintenance personnel to easily remove and replace upper layers of the filter media.

I. SAFETY FEATURES

- Where necessary, surface organic filter facilities can be fenced to prevent access.
- Inlets and outlets shall be designed and maintained so as not to permit access by children.

J. LANDSCAPING

- Organic filters can be designed with a grass cover to aid in pollutant removal and prevent clogging. The grass should be capable of withstanding frequent periods of inundation and drought.

K. ADDITIONAL SITE-SPECIFIC DESIGN CRITERIA AND ISSUES

Physiographic Factors - Local terrain design constraints

- Low Relief – Use of an organic filter may be limited by low head
- High Relief – Filter bed surface must be level
- Karst – Use liner or impermeable membrane to seal bottom earthen surface of the organic filter or use watertight structure

Special Downstream Watershed Considerations

- Wellhead Protection – Reduce potential groundwater contamination (in required wellhead protection areas) by preventing infiltration of hotspot runoff. May require liner for type "A" and "B" soils; Pretreat hotspots; provide 2 to 4 foot separation distance from water table.

4.4.1.6 Design Procedures

Step 1. Compute runoff control volumes

Calculate WQv, CPv, in accordance with the guidance presented in Volume 2, Chapter 3.

Step 2. Determine if the development site and conditions are appropriate for the use of organic filter.

Consider the Application and Site Feasibility Criteria, and the Additional Site Specific Design Criteria and Issues noted above. Check with Knox County Engineering and other agencies as appropriate to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 3. Compute WQv peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion structures (see Volume 2, Chapter 3 for more information on this calculation).

(a) Using WQv, compute CN

(b) Compute time of concentration using TR-55 method

(c) Determine appropriate unit peak discharge from time of concentration

(d) Compute Q_{wq} (in inches) from unit peak discharge, drainage area, and WQv

Step 4. Size flow diversion structure, if needed

A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQv to the organic filter facility. Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 5. Size filtration basin chamber

The filter area is sized using the following equation (based on Darcy's Law):

$$A_f = (WQv)\ (d_f)\ /\ [(k)\ (h_f + d_f)\ (t_f)]$$

where:

- WQv = water quality volume (ft^3)
- A_f = surface area of filter bed (ft^2)
- d_f = filter bed depth
  (at least 1.5 feet, no more than 2 feet)
- k = coefficient of permeability of filter media (ft/day)
  (use 3.5 ft/day for sand)
  (use 2.0 ft/day for peat)
  (use 8.7 ft/day for leaf compost)
- h_f = average height of water above filter bed (ft)
  (1/2 h_{max}, which varies based on site but h_{max} is typically $\leq$ 6 feet)
- t_f = design filter bed drain time (days)
  (1.67 days or 40 hours is required maximum time)

Set preliminary dimensions of filtration basin chamber.

Step 6. Size sedimentation chamber

The sedimentation chamber shall be sized to at least 25% of the computed WQv and have a length-to-width ratio of 2:1. The Camp-Hazen equation is used to compute the required surface area:

$$A_s = -(Q_o/w) * \mathrm{Ln}\ (1-E)$$

where:

- A_s = sedimentation basin surface area (ft^2)
- Q_o = rate of outflow = the WQv (ft^3) / 86400 seconds
- w = particle settling velocity (ft/sec)
- E = trap efficiency

Assuming:

- E = 90% sediment trap efficiency (0.9)
- w = particle settling velocity (ft/sec) = 0.0033 ft/sec for imperviousness (I) ≥ 75%
- w = particle settling velocity (ft/sec) = 0.0004 ft/sec for imperviousness (I) < 75%
- average of 24 hour holding period

Then:

A_s = (0.0081) (WQv) ft^2 for I ≥ 75%
A_s = (0.066) (WQv) ft^2 for I < 75%

Set preliminary dimensions of sedimentation chamber.

Step 7. Compute V_{min}

$$V_{min} = 0.75 * WQv$$

Step 8. Compute storage volumes within entire facility and sedimentation chamber orifice size

Use the following equation:

$$Vol_{min} = 0.75\ WQv = V_s + V_f + V_{f\text{-}temp}$$

(1) Compute V_f = water volume within filter bed/gravel/pipe = A_f * d_f * n
Where: n = porosity = 0.4 for most applications

(2) Compute $V_{f\text{-}temp}$ = temporary storage volume above the filter bed = 2 * h_f * A_f

(3) Compute V_s = volume within sediment chamber = Vol_{min} - V_f - $V_{f\text{-}temp}$

(4) Compute h_s = height in sedimentation chamber = V_s/A_s

(5) Ensure h_s and h_f fit available head and other dimensions still fit – change as necessary in design iterations until all site dimensions fit.

(6) Size orifice from sediment chamber to filter chamber to release V_s within 24-hours at average release rate with 0.5 h_s as average head.

(7) Design outlet structure with perforations allowing for a safety factor of 10.

(8) Size distribution chamber to spread flow over filtration media – level spreader weir.

Step 9. Design inlets, pretreatment facilities, underdrain system, and outlet structures

See design criteria above for more details.

Step 10. Compute overflow weir sizes

1. Size overflow weir at elevation h_s in sedimentation chamber (above perforated stand pipe) to handle surcharge of flow through filter system from 25-year storm.

2. Plan inlet protection for overflow from sedimentation chamber and size overflow weir at elevation h_f in filtration chamber (above perforated stand pipe) to handle surcharge of flow through filter system from 25-year storm (see example).

This page left intentionally blank

4.4.1.7 Maintenance Requirements and Inspection Checklist

Note: Section 4.4.1.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of an organic filter as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for organic filters, along with a suggested frequency for each activity. Individual organic filters may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the organic filter in proper operating condition at all times.

Inspection Activities	**Suggested Schedule**
• A record should be kept of the dewatering time (i.e., the time required to drain the filter bed completely after a storm event) for an organic filter to determine if maintenance is necessary. The filter bed should drain completely in about 40 hours after the end of the rainfall. • Check to ensure that the filter surface does not clog after storm events.	After Rain Events
• Check the contributing drainage area, facility, inlets and outlets for debris. • Check to ensure that the filter surface is not clogging.	Monthly
• Check to see that the filter bed is clean of sediment, and the sediment chamber is not more than 50% full of sediment or the sediment accumulation is not more than 6 inches, whichever is less sediment. Remove sediment as necessary. • Make sure that there is no evidence of deterioration, spalling, bulging, or cracking of concrete. • Inspect inlets, outlets and overflow spillway to ensure good condition and no evidence of erosion. • Check to see if stormwater flow is bypassing the facility. • Ensure that no noticeable odors are detected outside the facility.	Annually
Maintenance Activities	**Suggested Schedule**
• Mow and stabilize (prevent erosion, vegetate denuded areas) the area draining to the organic filter. Collect and remove grass clippings. Remove trash and debris. • Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system.	Monthly
• Check to see that the filter bed is clean of sediment, and the sediment chamber is not more than 50% full of sediment or the sediment accumulation is not more than 6 inches, whichever is less sediment. Remove sediment as necessary. • Repair or replace any damaged structural parts. • Stabilize any eroded areas.	Annually
• If filter bed is clogged or partially clogged, manual manipulation of the surface layer of filter media may be required. Remove the top few inches of filter media, roto-till or otherwise cultivate the surface, and replace with media meeting the design specifications. • Replace any filter fabric that has become clogged.	As needed

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of organic filters. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the organic filter. Questions regarding stormwater facility inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
ORGANIC FILTER INSPECTION CHECKLIST

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ____

Date: __________ Time: _____________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Organic Filter Inspection List		
Complete drainage of the filter in about 40 hours after a rain event?		
Clogging of filter surface?		
Clogging of inlet/outlet structures?		
Clogging of filter fabric?		
Filter clear of debris and functional?		
Leaks or seeps in filter?		
Obstructions of spillway(s)?		
Animal burrows in filter?		
Sediment accumulation in filter bed (less than 50% is acceptable)?		
Cracking, spalling, bulging or deterioration of concrete?		
Erosion in area draining to organic filter?		
Erosion around inlets, filter bed, or outlets?		
Pipes and other structures in good condition?		
Undesirable vegetation growth?		
Other (describe)?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

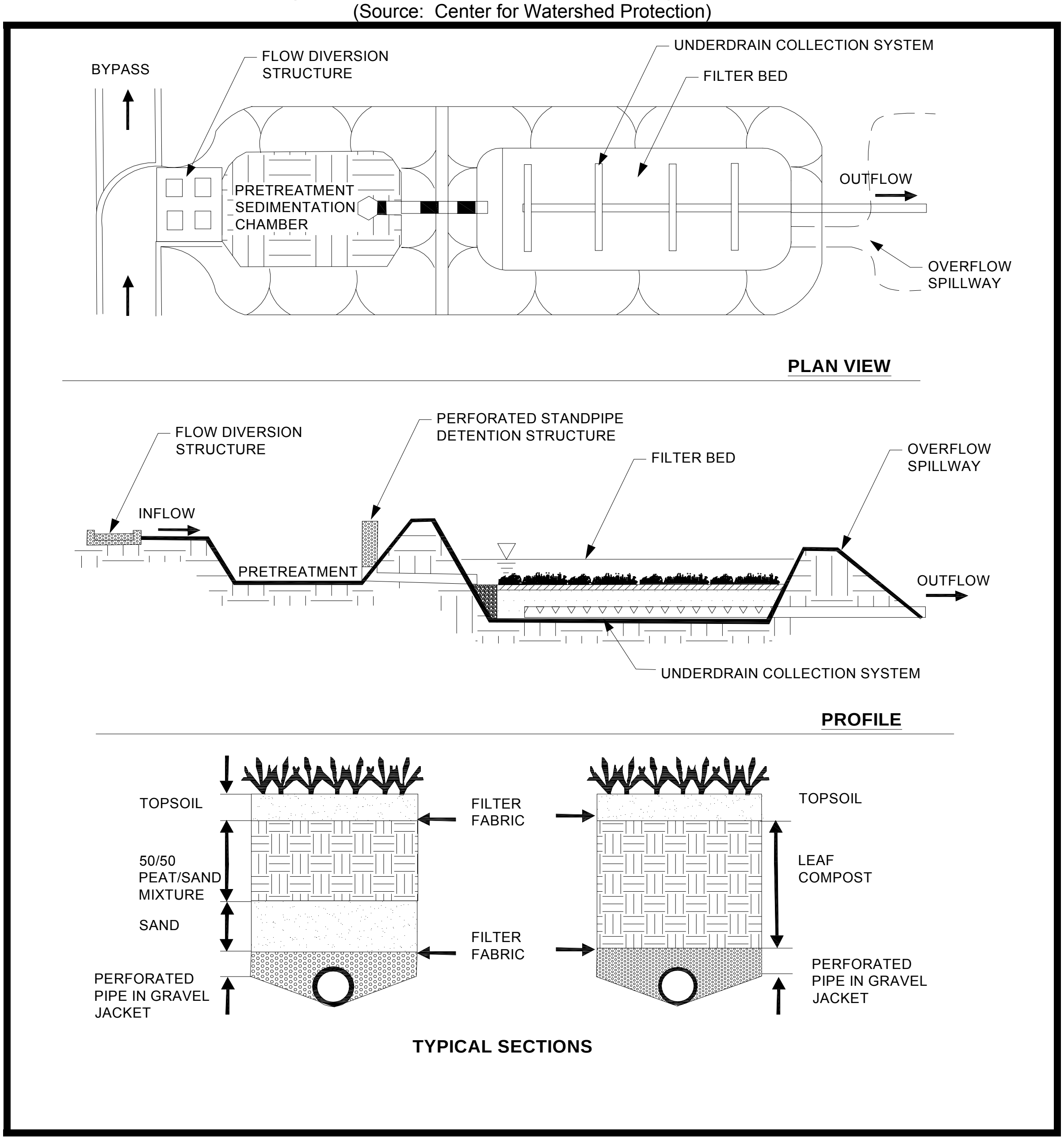
If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature:_______________________________ Inspector Name (printed)___________________________

4.4.1.8 Example Schematic

Figure 4-58. Schematic of an Organic Filter
(Source: Center for Watershed Protection)



4.4.1.9 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual, Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

Maryland Department of the Environment. *Maryland Stormwater Design Manual, Volumes I and II.* Prepared by Center for Watershed Protection (CWP). 2000.

Minnesota Pollution Control Agency. *Minnesota Stormwater Manual.* Accessed January 2006. http://www.pca.state.mn.us/water/stormwater/stormwater-manual.html

Washington State Department of Ecology. *Stormwater Management Manual for Western Washington.* 2000.

4.4.1.10 Suggested Reading

Bell, W., L. Stokes, L.J. Gavan, and T. Nguyen. *Assessment of the Pollutant Removal Efficiencies of Delaware Sand Filter BMPs.* City of Alexandria, Department of Transportation and Environmental Services, Alexandria, VA, 1995.

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

City of Austin, TX. *Water Quality Management.* Environmental Criteria Manual, Environmental and Conservation Services, 1988.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

Claytor, R.A., and T.R. Schueler. *Design of Stormwater Filtering Systems.* The Center for Watershed Protection, Silver Spring, MD, 1996.

Horner, R.R., and C.R. Horner. *Design, Construction, and Evaluation of a Sand Filter Stormwater Treatment System, Part II: Performance Monitoring.* Report to Alaska Marine Lines, Seattle, WA, 1995.

Metropolitan Washington Council of Governments (MWCOG). *A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone.* March, 1992.

Northern Virginia Regional Commission (NVRC). *The Northern Virginia BMP Handbook.* Annandale, VA, 1992.

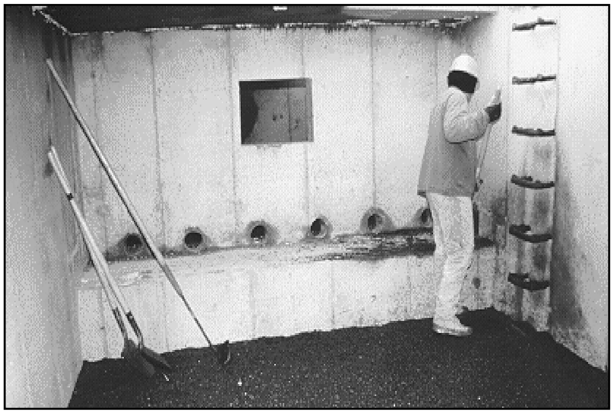
Schueler, T.R. *Developments in Sand Filter Technology to Improve Stormwater Runoff Quality.* Watershed Protection Techniques 1(2):47-54, 1994.

US EPA. *Storm Water Technology Fact Sheet: Sand Filters.* EPA 832-F-99-007, Office of Water. 1999.

Young, G.K., S. Stein, P. Cole, T. Kammer, F. Graziano, and F. Bank. *Evaluation and Management of Highway Runoff Water Quality.* FHWA-PD-96-032. Federal Highway Administration, Office of Environment and Planning, 1996.

4.4.2 Underground Sand Filter

Limited Application Stormwater BMP



Description: The underground sand filter is a design variation of the surface sand filter, where the sand filter chambers and media are located in an underground vault.

KEY DESIGN CONSIDERATIONS

DESIGN GUIDELINES:

- Maximum contributing drainage area of 2 acres
- Typically requires 2 to 6 feet of head
- Precast concrete shells available, which decrease construction costs
- Underdrain required

ADVANTAGES / BENEFITS:

- High pollutant removal
- Applicable to small drainage areas
- Good for highly impervious areas
- Good retrofit capability

DISADVANTAGES / LIMITATIONS:

- High maintenance burden
- Not recommended for areas with high stormwater sediment or clay/silt runoff areas
- Possible odor problems
- Cannot be installed until site work is complete

MAINTENANCE REQUIREMENTS:

- Inspect for clogging – rake first inch of sand
- Remove sediment from forebay/chamber
- Replace sand filter media as needed

STORMWATER MANAGEMENT SUITABILITY

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

Accepts Hotspot Runoff: *Yes (requires impermeable liner)*

✷ in certain situations

FEASIBILITY CONSIDERATIONS

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

Residential Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *2 acres maximum*

Soils: *Not recommended for clay/silt drainage areas that are not stabilized*

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

POLLUTANT REMOVAL

H	**Total Suspended Solids**
M	**Nutrients** - Total Phosphorus / Total Nitrogen
M	**Metals** - Cadmium, Copper, Lead, and Zinc
M	**Pathogens** - Coliform, Streptococci, E.Coli

OTHER CONSIDERATIONS:

- Must be combined with other controls to provide water quantity control

4.4.2.1 General Description

The underground sand filter is a design variant of the surface sand filter. The underground sand filter is an enclosed filter system typically constructed just below grade in a vault along the edge of an impervious area such as a parking lot. The system consists of a sedimentation chamber and a sand bed filter. Runoff flows into the structure through a series of inlet grates located along the top of the control. Underground sand filters are best used for high-density land uses or ultra-urban applications where space for surface stormwater controls is limited. Figure 4-59 presents an example of an underground sand filter.

Figure 4-59. Example of an Underground Sand Filter

Multiple configurations have been developed for underground filters including the DC filter and the Delaware filter. The DC filter is intended to treat stormwater that is conveyed by a storm drain system. The Delaware filter (also known as the perimeter sand filter) is designed to collect flow directly from impervious surfaces and is well suited for installation along parking areas. Both systems operate in the same manner.

The underground sand filter is a three-chamber system. The initial chamber is a sedimentation (pretreatment) chamber that temporarily stores runoff and utilizes a wet pool to capture sediment. The sedimentation chamber is connected to the sand filter chamber by a submerged wall that protects the filter bed from oil and trash. The filter bed is 18 to 24 inches deep and may have a protective screen of gravel or permeable geotextile to limit clogging. The sand filter chamber also includes an underdrain system with inspection and clean out wells. Perforated drain pipes under the sand filter bed extend into a third chamber that collects filtered runoff. Flows beyond the filter capacity are diverted through an overflow weir.

Due to its location below the surface, underground sand filters have a high maintenance burden and should only be used where adequate inspection and maintenance can be ensured.

4.4.2.2 Stormwater Management Suitability

Underground sand filter systems are designed primarily as off-line systems for treatment of the water quality volume and will typically need to be used in conjunction with another structural BMP that can provide downstream channel protection, overbank flood protection, and extreme flood protection. However, under certain circumstances, underground sand filters can provide limited runoff quantity control, particularly for smaller storm events.

Water Quality (WQv)
In underground sand filter systems, stormwater pollutants are removed through a combination of gravitational settling, filtration and adsorption. The filtration process effectively removes suspended solids and particulates, biochemical oxygen demand (BOD), fecal coliform bacteria, and other pollutants.

Channel Protection (CPv)
For smaller sites, an underground sand filter may be designed to capture the entire channel protection volume (CPv) in either an off- or on-line configuration. Given that an underground sand filter system is typically designed to completely drain over 40 hours, the channel protection design requirement for extended detention of the 1-year, 24-hour storm runoff volume can be met. For larger sites or where only the WQv is diverted to the underground sand filter facility, another structural control must be used to provide extended detention of the CPv.

Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})
Underground sand filters are not useful for flood protection. Another structural control, such as a conventional detention pond, must be used in conjunction with an underground sand filter system to control stormwater peak discharges. Further, underground sand filter facilities utilized on-line must provide flow diversion and/or be designed to safely pass extreme storm flows and protect the filter bed and facility.

4.4.2.3 Pollutant Removal Capabilities

Underground sand filters are presumed to be able to remove 80% of the total suspended solids (TSS) load in typical urban post-development runoff when sized, designed, constructed and maintained in accordance with the recommended specifications. Undersized or poorly designed underground sand filters can reduce TSS removal performance.

Additionally, research has shown that use of underground sand filters will have benefits beyond the removal of TSS, such as the removal of other pollutants (i.e. phosphorous, nitrogen, fecal coliform and heavy metals), as well, which is useful information should the pollutant removal criteria change in the future. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data.

- Total Suspended Solids – 80%
- Total Phosphorus – 50%
- Total Nitrogen – 30%
- Heavy Metals – 50%
- Pathogens – 40%

For additional information and data on pollutant removal capabilities for underground sand filters, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the International Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

4.4.2.4 Application and Site Feasibility Criteria

Underground sand filter systems are well-suited for highly impervious areas where land available for structural BMPs is limited. Underground sand filters should primarily be considered for new construction or retrofit opportunities for commercial, industrial, and institutional areas where the sediment load is relatively low, such as: parking lots, driveways, loading docks, gas stations, garages, airport runways/taxiways, and storage yards.

To avoid rapid clogging and failure of the filter media, the use of underground sand filters should be avoided in areas with less than 50% impervious cover, or high sediment yield sites with clay/silt soils.

The following basic criteria should be evaluated to ensure the suitability of an underground sand filter facility for meeting stormwater management objectives on a site or development.

General Feasibility

- Not suitable for use in a residential subdivision
- Suitable for use in high density/ultra-urban areas
- Not suitable for use as a regional stormwater control. On-site applications are typically most feasible.

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – 2 acres maximum for an underground sand filter
- Space Required – Function of available head at site
- Minimum Head – The surface slope across the filter location should be no greater than 6%. The elevation difference needed at a site from the inflow to the outflow is 2-6 feet.
- Minimum Depth to Water Table – If used on a site with an underlying water supply aquifer, a separation distance of 2 feet is required between the bottom of the sand filter and the elevation of the seasonally high water table to prevent groundwater contamination.
- Soils – Not recommended for clay/silt drainage areas that are not stabilized.

Other Constraints / Considerations

- Aquifer Protection – Do not allow infiltration of filtered hotspot runoff into groundwater

4.4.2.5 Planning and Design Standards

The following standards shall be considered **minimum** design standards for the design of underground sand filters. Underground sand filters that are not designed to these standards will not be approved. The Director of Engineering and Public Works (the Director) shall have the authority to require additional design conditions if deemed necessary.

A. CONSTRUCTION SEQUENCING

- Care shall be taken during construction to minimize the risk of premature failure of the underground sand filter due to deposition of sediments from disturbed, unstabilized areas. This can be minimized or avoided by proper construction sequencing.
- Ideally, the construction of an underground filter shall take place **after** the construction site has been stabilized. In the event that the underground sand filter is not constructed after site stabilization, diversion of site runoff around the sand filter and installation and maintenance of appropriate erosion prevention and sediment controls prior to site stabilization is required.
- Diversion berms shall be maintained around an underground sand filter during all phases of construction. No runoff shall enter the sand filter area prior to completion of construction and the complete stabilization of construction areas. Erosion prevention and sediment controls shall be maintained around the filter to prevent runoff and sediment from entering the sand filter during construction.
- Underground sand filters shall not be used as a temporary sediment trap for construction activities.
- During and after excavation of the underground sand filter area, all excavated materials shall be placed downstream, away from the sand filter, to prevent redeposit of the material during runoff events.

B. LOCATION AND SITING

- Underground sand filters shall have a contributing drainage area of 2 acres or less.
- Underground sand filter systems are generally applied to land uses with a high percentage of impervious surfaces. Sand filters shall not be utilized for sites that have less than 50% impervious cover. Any disturbed or denuded areas located within the area draining to and treated by the underground sand filter shall be stabilized prior to construction and use of the sand filter.
- Delaware underground sand filters are typically sited along the edge, or perimeter, of an impervious area such as a parking lot.
- DC underground sand filters are installed within the storm drain system.
- Underground sand filter systems shall be designed for intermittent flow and must be allowed to drain and re-aerate between rainfall events. They shall not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

- The entire treatment system (including the sedimentation chamber) must temporarily hold at least 75% of the WQv prior to filtration. Figures 4-60 and 4-61 illustrate the distribution of the treatment volume (0.75 WQv) among the various components of the underground sand filters, including:
    - V_w – wet pool volume within the sedimentation basin
    - V_f – volume within the voids in the filter bed
    - V_{temp} – temporary volume stored above the filter bed
    - A_s – the surface area of the sedimentation basin
    - A_f – surface area of the filter media
    - h_f – average height of water above the filter media (1/2 h_{temp})
    - d_f – depth of filter media
- The sedimentation chamber shall be sized to at least 50% of the computed WQv.
- The filter area shall be sized based on the principles of Darcy's Law. A coefficient of permeability (k) of 3.5 ft/day for sand shall be used. The filter bed shall be designed to completely drain in 40 hours or less.
- The filter media shall consist of an 18-inch to 24-inch layer of clean washed medium aggregate concrete sand (ASTM C-33) on top of the underdrain system. Figure 4-62 illustrates a typical media cross section.
- The filter bed shall be equipped with a 6-inch perforated pipe underdrain (PVC AASHTO M 252, HDPE, or other suitable pipe material) in a gravel layer. The underdrain shall have a minimum grade of 1/8-inch per foot (1% slope). Holes shall be 3/8-inch diameter and spaced approximately 6 inches on center. Gravel shall be clean-washed aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches with a void space of about 40%. Aggregate contaminated with soil shall not be used.

Figure 4-60. Underground (DC) Sand Filter Volumes

(Source: Center for Watershed Protection)

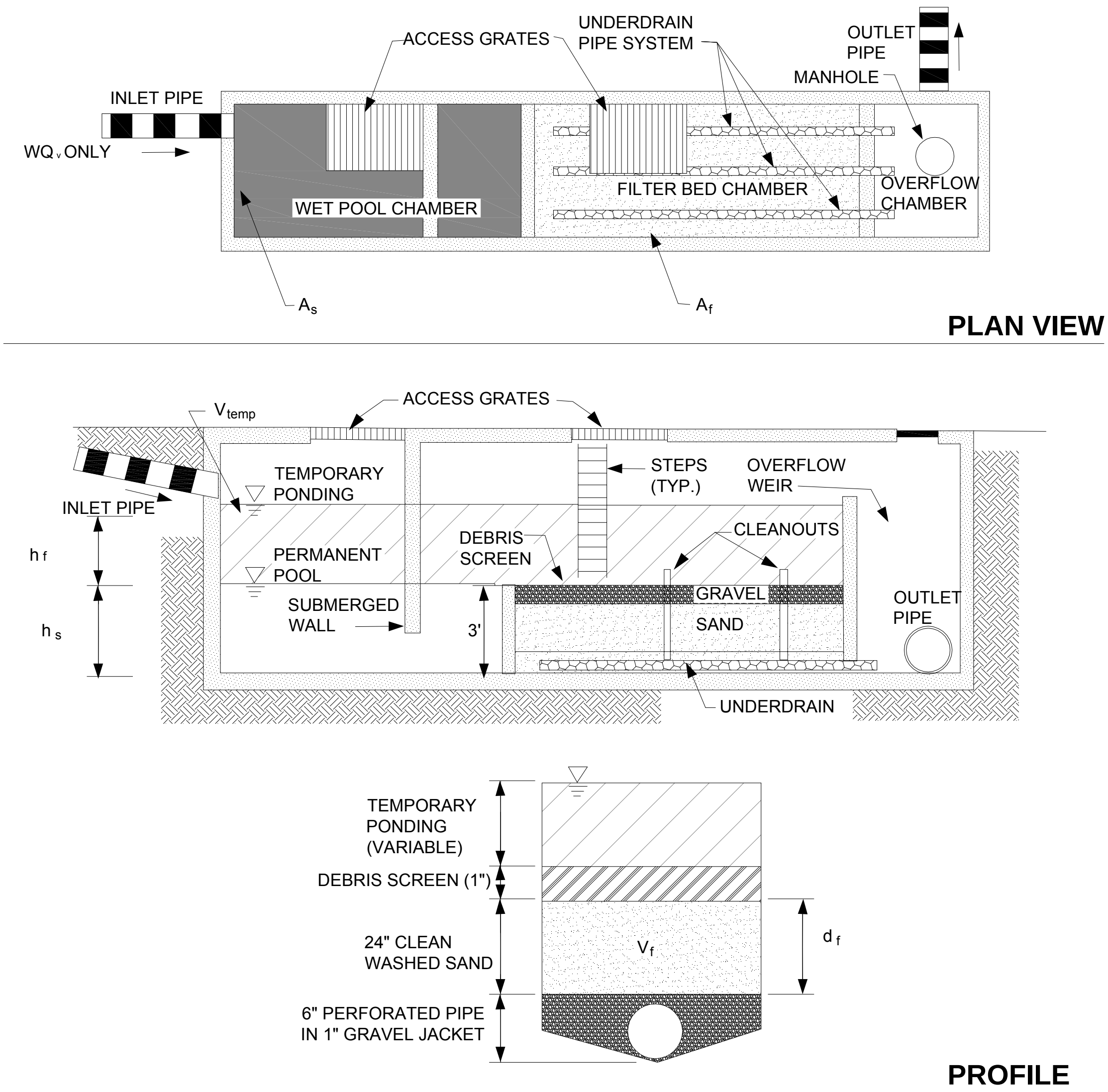


Figure 4-61. Perimeter Sand Filter Volumes
(Source: Claytor and Schueler, 1996)

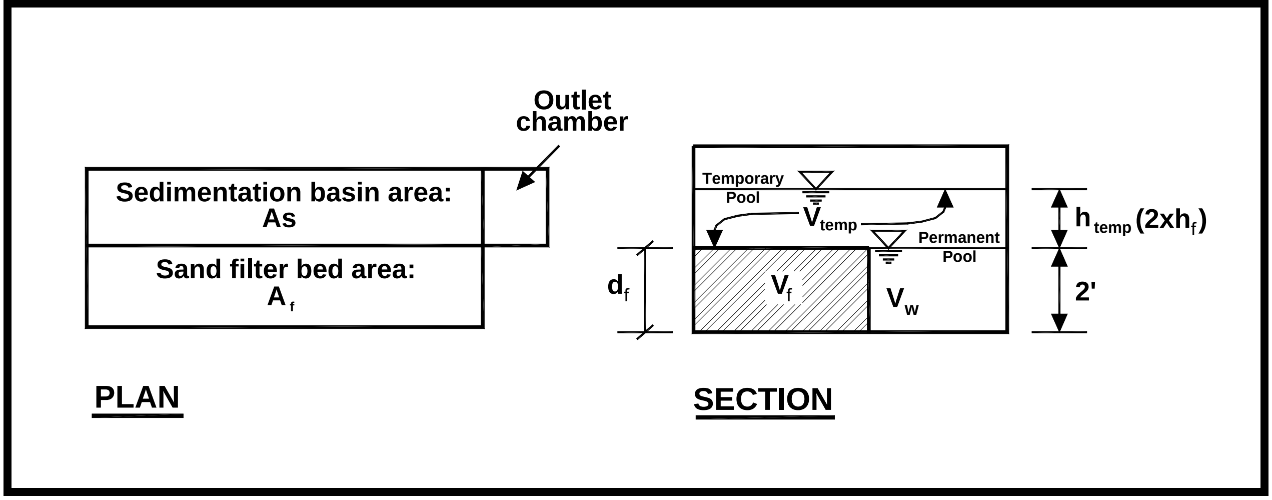
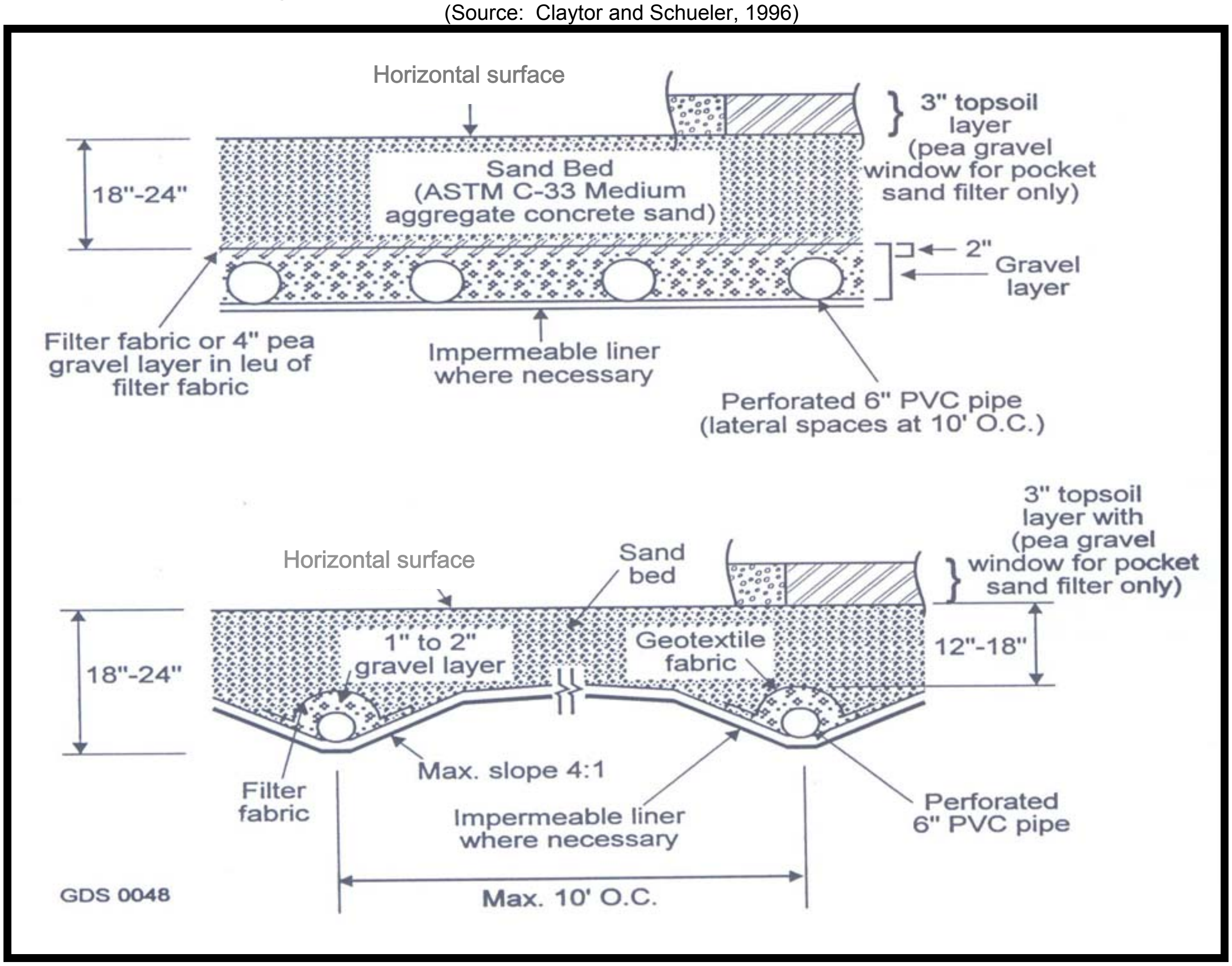


Figure 4-62. Typical Sand Filter Media Cross Sections
(Source: Claytor and Schueler, 1996)



D. OUTLET STRUCTURES

- An outlet pipe shall be provided from the underdrain system to the facility discharge. Due to the slow rate of filtration, outlet protection is generally unnecessary (except for emergency overflows and spillways). However, the design shall ensure that the discharges from the underdrain system occur in a non-erosive manner.

E. EMERGENCY SPILLWAY

- An emergency bypass spillway or weir must be included in the underground sand filter design to safely pass flows that exceed the WQv (and CPv if the filter is utilized for channel protection purposes

F. MAINTENANCE ACCESS

- A minimum 20' wide maintenance right-of-way or drainage easement shall be provided for an underground sand filter from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles. Adequate access must be provided to the grates of the filter bed. Facility designs must enable maintenance personnel to easily remove and replace upper layers of the filter media.

G. SAFETY FEATURES

- Inlets, access grates and outlets shall be designed and maintained so as not to permit access by children. Inlet and access grates to the underground sand filters may be locked.

4.4.2.6 Design Procedures

Step 1. Compute runoff control volumes

Calculate WQv, CPv in accordance with the guidance presented in Volume 2, Chapter 3.

Step 2. Determine if the development site and conditions are appropriate for the use of an underground sand filter.

Consider the Application and Site Feasibility Criteria Check with Knox County Engineering and other agencies as appropriate to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 3. Compute WQv peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion structures (see Volume 2, Chapter 3 for more information on this calculation).

(a) Using WQv, compute CN
(b) Compute time of concentration using TR-55 method
(c) Determine appropriate unit peak discharge from time of concentration
(d) Compute Q_{wq} in inches from unit peak discharge, drainage area, and WQv.

Step 4. Size flow diversion structure, (if needed)

If a diversion structure is utilized, a flow regulator should be supplied to divert the WQv to the underground sand filter facility. Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 5. Size filtration basin chamber

The filter area is sized using the following equation (based on Darcy's Law):

$$A_f = (WQv)\,(d_f) / [(k)\,(h_f + d_f)\,(t_f)]$$

where:

A_f = surface area of filter bed (ft^2)
WQv = water quality volume (ft^3)

d_f = filter bed depth (1.5 ft)
(at least 18 inches, no more than 24 inches)
k = coefficient of permeability of filter media (ft/day)
(use 3.5 ft/day for sand)
h_f = average height of water above filter bed (ft)
(1/2 h_{max}, which varies based on site but h_{max} is typically ≤ 6 feet)
t_f = design filter bed drain time (days)
(1.67 days or 40 hours is required maximum time)

Set preliminary dimensions of filtration basin chamber.

Step 6. Size sedimentation chamber

Depending on the type of underground sand filter system utilized, the sedimentation chamber shall be sized to at least 50% of the computed WQv and have a length-to-width ratio of 2:1. The Camp-Hazen equation is used to compute the required surface area:

$$A_s = -(Q_o/w) * \text{Ln}(1-E)$$

where:
A_s = sedimentation basin surface area (ft^2)
Q_o = rate of outflow = the WQv (ft^3) / 86400 seconds
w = particle settling velocity (ft/sec)
E = trap efficiency

Assuming:
- E = 90% sediment trap efficiency (0.9)
- w = particle settling velocity (ft/sec) = 0.0033 ft/sec for imperviousness ≥ 75%
- w = particle settling velocity (ft/sec) = 0.0004 ft/sec for imperviousness < 75%
- average of 24 hour holding period

Then:
A_s = (0.0081) (WQv) ft^2 for I ≥ 75%
A_s = (0.066) (WQv) ft^2 for I < 75%

Set preliminary dimensions of sedimentation chamber.

Step 7. Compute V_{min}

$$V_{min} = 0.75 * WQv$$

Step 8. Compute storage volumes within entire facility and sedimentation chamber orifice size

Underground (D.C.) sand filter:

$$V_{min} = 0.75\ WQv = V_s + V_f + V_{f\text{-}temp}$$

(1) Compute V_f = water volume within filter bed/gravel/pipe = $A_f * d_f * n$
Where: n = porosity = 0.4 for most applications

(2) Compute $V_{f\text{-}temp}$ = temporary storage volume above the filter bed = $2 * h_f * A_f$

(3) Compute V_s = volume within sediment chamber = $V_{min} - V_f - V_{f\text{-}temp}$

(4) Compute h_s = height in sedimentation chamber = V_s/A_s

(5) Ensure h_s and h_f fit available head and other dimensions still fit – change as necessary in design iterations until all site dimensions fit.

(6) Size weir from sediment chamber to filter chamber to release V_s within 24-hours at average release rate with 0.5 h_s as average head.

(7) Design outlet structure with perforations allowing for a safety factor of safety ten (10) times the orifice capacity.

(8) Size distribution chamber to spread flow over filtration media – level spreader weir.

***Underground perimeter (Delaware) sand filter*:**

(1) Compute V_f = water volume within filter bed/gravel/pipe = $A_f * d_f * n$
where: A_f = surface area of filter bed (ft^2)
d_f = filter bed depth (1.5 ft)
(at least 18 inches, no more than 24 inches)
n = porosity = 0.4 for most applications

(2) Compute V_w = wet pool storage volume A_s * 2 feet minimum

(3) Compute V_{temp} = temporary storage volume = $V_{min} – (V_f + V_w)$

(4) Compute h_{temp} = temporary storage height = $V_{temp} / (A_f + A_s)$

(5) Ensure $h_{temp} \geq 2 * h_f$, otherwise decrease h_f and re-compute. Ensure dimensions fit available head and area – change as necessary in design iterations until all site dimensions fit.

(6) Size distribution weirs from sediment chamber to filter chamber.

<u>Step 9. Design inlets, underdrain system, and outlet structures</u>

See design criteria above for more details.

<u>Step 10. Compute overflow weir sizes</u>

***Underground (D.C.) sand filter*:**

$$V_{min} = 0.75\ WQv = V_s + V_f + V_{f\text{-}temp}$$

(1) Compute V_f = water volume within filter bed/gravel/pipe = $A_f * d_f * n$
where: n = porosity = 0.4 for most applications

(2) Compute $V_{f\text{-}temp}$ = temporary storage volume above the filter bed = $2 * h_f * A_f$

(3) Compute V_s = volume within sediment chamber = $V_{min} - V_f - V_{f\text{-}temp}$

(4) Compute hs = height in sedimentation chamber = Vs/As

(5) Ensure hs and hf fit available head and other dimensions still fit – change as necessary in design iterations until all site dimensions fit.

(6) Size orifice from sediment chamber to filter chamber to release Vs within 24-hours at average release rate with 0.5 hs as average head.

(7) Design outlet structure with perforations allowing for a safety factor of ten (10) times the orifice capacity.

(8) Size distribution chamber to spread flow over filtration media – level spreader weir or orifices.

Underground perimeter (Delaware) sand filter: Size overflow weir at end of sedimentation chamber to handle excess inflow, set at WQv elevation.

4.4.2.7 Maintenance Requirements and Inspection Checklist

Note: Section 4.2.2.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of an underground sand filter as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for underground sand filters, along with a suggested frequency for each activity. Individual filters may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the sand filter in proper operating condition at all times.

Inspection Activities	Suggested Schedule
• A record should be kept of the dewatering time (i.e., the time required to drain the filter bed completely after a storm event) for a sand filter to determine if maintenance is necessary. The filter bed should drain completely in about 40 hours after the end of the rainfall. • Check to ensure that the filter surface does not clog after storm events.	After Rain Events
• Check the contributing drainage area, facility, inlets and outlets for debris. • Check to ensure that the filter surface is not clogging.	Monthly
• Check to see that the filter bed is clean of sediment, and the sediment chamber is not more than 50% full or 6 inches, whichever is less, of sediment. Remove sediment as necessary. • Make sure that there is no evidence of deterioration, spalling, bulging, or cracking of concrete. • Inspect grates of sand filter (perimeter and Delaware). • Inspect inlets, outlets and overflow spillway to ensure good condition and no evidence of erosion. • Check to see if stormwater flow is bypassing the facility (if so designed). • Ensure that no noticeable odors are detected outside the facility.	Annually
Maintenance Activities	**Suggested Schedule**
• Mow and stabilize (prevent erosion, vegetate denuded areas) the area draining to the underground sand filter. Collect and remove grass clippings. Remove trash and debris. • Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system. • If permanent water level is present (perimeter and Delaware).in sand filter, ensure that the chamber does not leak, and normal pool level is retained.	Monthly
• Check to see that the filter bed is clean of sediment, and the sediment chamber is not more than 50% full or 6 inches, whichever is less, of sediment. Remove sediment as necessary. • Repair or replace any damaged structural parts. • Stabilize any eroded areas.	Annually
• If filter bed is clogged or partially clogged, manual manipulation of the surface layer of sand may be required. Remove the top few inches of sand, roto-till or otherwise cultivate the surface, and replace media with sand meeting the design specifications. • Replace any filter fabric that has become clogged.	As needed

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of underground sand filters. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the underground sand filter. Questions regarding stormwater facility inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
UNDERGROUND SAND FILTER INSPECTION CHECKLIST

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ____

Date: __________ Time: ____________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Underground Sand Filter Inspection List		
Complete drainage of the filter in about 40 hours after a rain event?		
Clogging of filter surface?		
Clogging of inlet/outlet structures?		
Clogging of filter fabric?		
Filter clear of debris and functional?		
Leaks or seeps in filter?		
Obstructions of spillway(s)?		
Animal burrows in filter?		
Sediment accumulation in filter bed (less than 50% is acceptable)?		
Cracking, spalling, bulging or deterioration of concrete?		
Erosion in area draining to sand filter?		
Erosion around inlets, filter bed, or outlets?		
Pipes and other structures in good condition?		
Undesirable vegetation growth?		
Other (describe)?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature:______________________________ Inspector Name (printed)________________________

4.4.2.8 Example Schematics

Figure 4-63. Schematic of an Underground (D.C.) Sand Filter
(Source: Center for Watershed Protection)

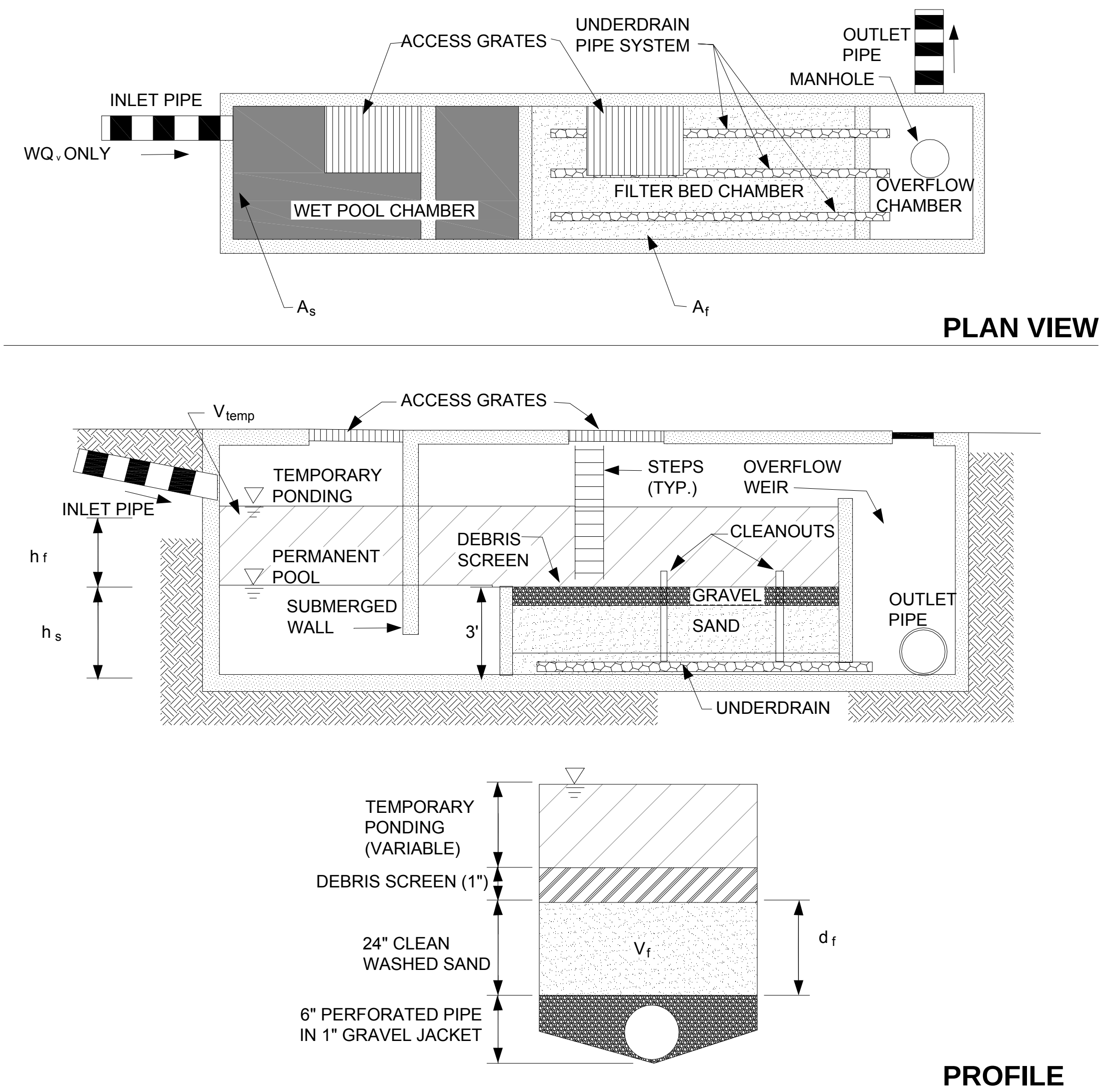
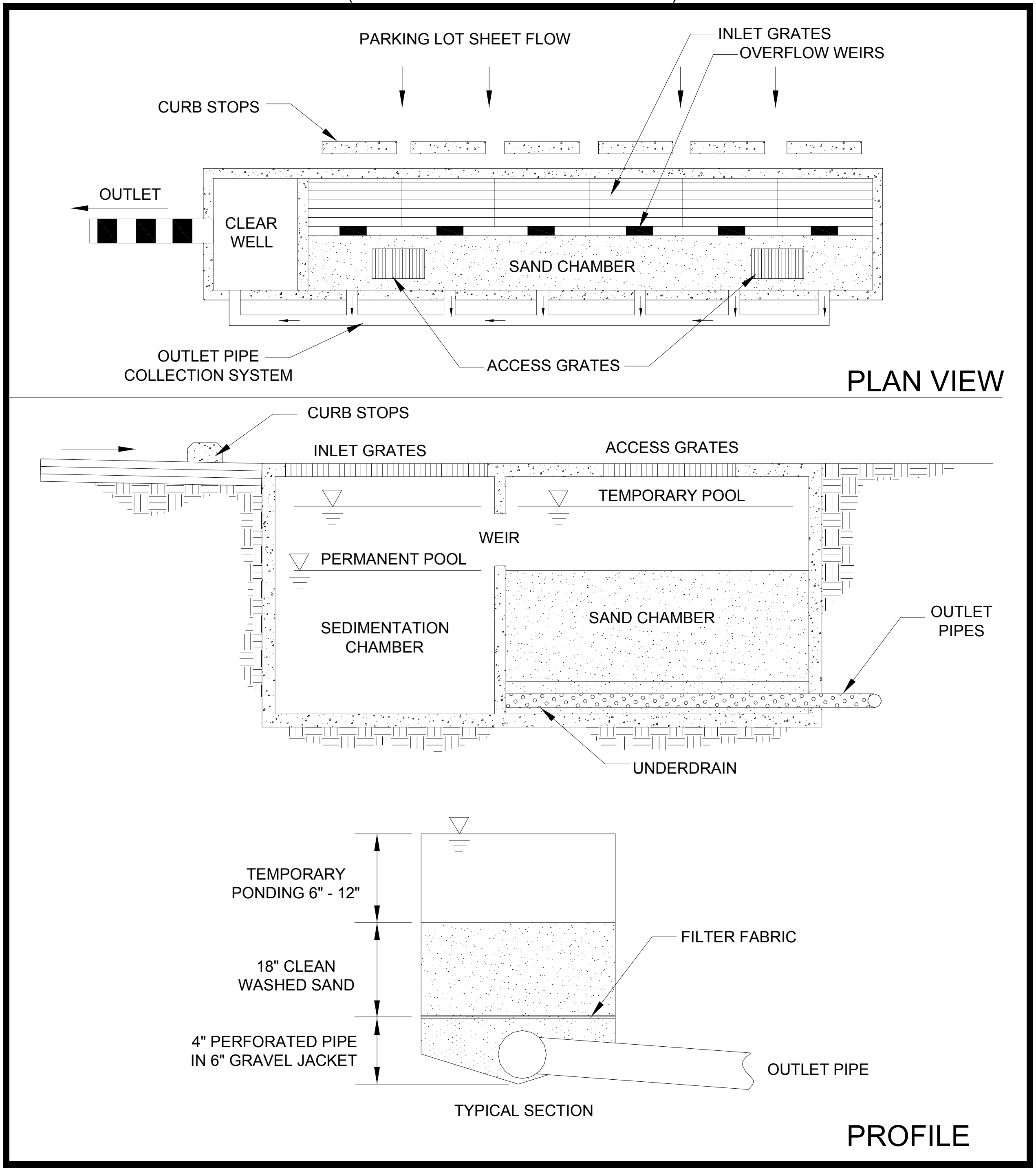


Figure 4-64. Schematic of Perimeter (Delaware) Sand Filter
(Source: Center for Watershed Protection)



4.4.2.9 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook*. 2001.

Brown, W., and T. Schueler. *The Economics of Stormwater BMPs in the Mid-Atlantic Region.* Prepared for Chesapeake Research Consortium, Edgewater, MD, by the Center for Watershed Protection, Ellicott City, MD, 1997.

Center for Watershed Protection (CWP). *Design of Stormwater Filtering Systems.* Prepared for the Chesapeake Research Consortium, Solomons, MD, and U.S. EPA Region 5, Chicago, IL, by the Center for Watershed Protection, Ellicott City, MD, 1996.

Center for Watershed Protection (CWP). *Multi-Chamber Treatment Train developed for stormwater hot spots.* Watershed Protection Techniques 2(3):445-449, 1997.

Maryland Department of the Environment (MDE). *Maryland Stormwater Design Manual.* 2000. Available at: http://www.mde.state.md.us/programs/waterprograms/sedimentandstormwater/stormwater_design/index.asp.

Minnesota Pollution Control Agency. *Minnesota Stormwater Manual.* Accessed January 2006. http://www.pca.state.mn.us/water/stormwater/stormwater-manual.html

New Jersey Department of Environmental Protection. *Stormwater Best Management Practices Manual.* 2004.

Schueler, T. *Developments in Sand Filter Technology to Improve Stormwater Runoff Quality.* Watershed Protection Techniques 1(2):47-54, 1994.

StormwaterAuthority.com. *Sand and Organic Filters.* Accessed January 2006. www.stormwaterauthority.com

4.4.2.10 Suggested Reading

Bell, W., L. Stokes, L.J. Gavan, and T. Nguyen. *Assessment of the Pollutant Removal Efficiencies of Delaware Sand Filter BMPs.* City of Alexandria, Department of Transportation and Environmental Services, Alexandria, VA, 1995.

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks,* 1993.

City of Austin, TX. *Water Quality Management.* Environmental Criteria Manual, Environmental and Conservation Services, 1988.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

Claytor, R.A., and T.R. Schueler. *Design of Stormwater Filtering Systems.* The Center for Watershed Protection, Silver Spring, MD, 1996.

Horner, R.R., and C.R. Horner. *Design, Construction, and Evaluation of a Sand Filter Stormwater Treatment System. Part II: Performance Monitoring.* Report to Alaska Marine Lines, Seattle, WA, 1995.

Metropolitan Washington Council of Governments (MWCOG). *A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone,* March, 1992.

Northern Virginia Regional Commission (NVRC). *The Northern Virginia BMP Handbook.* Annandale, VA, 1992.

US EPA. *Storm Water Technology Fact Sheet: Sand Filters.* EPA 832-F-99-007, Office of Water, 1999.

Young, G.K., S. Stein, P. Cole, T. Kammer, F. Graziano, and F. Bank. *Evaluation and Management of Highway Runoff Water Quality.* FHWA-PD-96-032, Federal Highway Administration, Office of Environment and Planning, 1996.

4.4.3 Submerged Gravel Wetland

Limited Application Stormwater BMP

Description: Submerged gravel wetlands are one or more cells filled with crushed rock designed to support wetland plants. Stormwater flows subsurface through the root zone of the constructed wetland where pollutant removal takes place.

KEY DESIGN CONSIDERATIONS

DESIGN GUIDELINES:

- Intended for space-limited applications
- High removal rates for sediment, Biochemical Oxygen Demand, and fecal coliform bacteria
- Max drainage area ≤ 5 acres

ADVANTAGES / BENEFITS:

- High TSS removal
- Generally requires low land consumption, and can fit within an area that is typically devoted to landscaping
- High pollutant removal capabilities are expected; however, limited performance data exist
- Can be located in low-permeability soils with a high water table

DISADVANTAGES / LIMITATIONS:

- High maintenance burden
- Not recommended for areas with high sediment content in stormwater or clay/silt runoff areas
- Cannot be installed until site construction is complete

MAINTENANCE REQUIREMENTS:

- Periodic sediment removal required to prevent clogging of gravel base

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality**
- [ ] **Channel Protection**
- [ ] **Overbank Flood Protection**
- [ ] **Extreme Flood Protection**

Accepts Hotspot Runoff: *Yes (requires impermeable liner)*

FEASABILITY CONSIDERATIONS

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

Residential Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *5 acres max.*

Soils: *No restrictions*

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

POLLUTANT REMOVAL

H	**Total Suspended Solids**
M-L	**Nutrients** - Total Phosphorus / Total Nitrogen
M	**Metals** - Cadmium, Copper, Lead, and Zinc
M-H	**Pathogens** - Coliform, Streptococci, E.Coli

OTHER CONSIDERATIONS:

- Needs to be combined with other controls to provide water quantity control

4.4.3.1 General Description

The submerged gravel wetland system consists of one or more treatment cells that are filled with crushed rock or gravel and is designed to allow stormwater to flow subsurface through the root zone of the constructed wetland. The outlet from each cell is set at an elevation to keep the rock or gravel submerged. Wetland plants are rooted in the media, where they can directly take up pollutants. In addition, algae and microbes thrive on the surface area of the rocks. In particular, the anaerobic conditions on the bottom of the filter can foster the denitrification process. Although widely used for wastewater treatment in recent years, only a handful of submerged gravel wetland systems have been designed to treat stormwater. Mimicking the pollutant removal ability of nature, this structural control relies on the pollutant-stripping ability of plants and soils to remove pollutants from runoff.

4.4.3.2 Stormwater Management Suitability

Submerged gravel wetlands are designed as off-line systems for treatment of the water quality volume and will need to be used in conjunction with another structural BMP that can provide downstream channel protection, overbank flood protection, and extreme flood protection.

Water Quality (WQv)
In submerged gravel wetlands, stormwater runoff flows through a gravel filter with wetland plants at the surface. Pollutants are removed through biological activity on the surface of the gravel and pollutant uptake by the plants. This practice is fundamentally different from other wetland designs because while most wetland designs behave like wet ponds, with differences in grading and landscaping, gravel wetlands are similar to filtering practices. The filtration process effectively removes suspended solids and particulates, biochemical oxygen demand (BOD), fecal coliform bacteria, and other pollutants.

Channel Protection (CPv)
The WQv is diverted to the submerged gravel wetland, therefore, it requires the use of another structural BMP to provide CPv extended detention.

Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})
Submerged gravel wetlands are not useful for flood protection. Another structural BMP, such as a conventional detention pond, must be used in conjunction with a submerged gravel wetland to provide flood protection. All submerged gravel wetlands must provide flow diversion to protect the gravel bed.

4.4.3.3 Pollutant Removal Capabilities

The pollution removal efficiency of the submerged gravel wetland is similar to a typical wetland. Recent data show a TSS removal rate in excess of the 80% goal. This reflects the settling environment of the gravel media. These systems also exhibit removals of about 60% Total Phosphorous and 20% Total Nitrogen. The growth of algae and microbes among the gravel media has been determined to be the primary removal mechanism of the submerged gravel wetland.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- Total Suspended Solids – 80%
- Total Phosphorus – 60%
- Total Nitrogen – 20%
- Heavy Metals – 50%
- Pathogens – 70%

For additional information and data on pollutant removal capabilities for submerged gravel wetlands, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the International Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

4.4.3.4 Application and Site Feasibility Criteria

Submerged gravel wetlands are well-suited for highly impervious areas where land available for structural BMPs is limited. Submerged gravel wetlands should primarily be considered for new construction or retrofit opportunities for commercial, industrial, and institutional areas where the sediment load is relatively low, such as: parking lots, driveways, loading docks, gas stations, garages, airport runways/taxiways, and storage yards.

To avoid clogging, the use of submerged gravel wetlands should be avoided in areas with less than 50% impervious cover, or high sediment yield sites with clay/silt soils.

The following basic criteria should be evaluated to ensure the suitability of a submerged gravel wetland for meeting stormwater management objectives on a site or development.

<u>General Feasibility</u>

- Suitable for use in high density/ultra-urban areas
- Not suitable for use in a residential subdivision
- Not suitable for use as a regional stormwater control. On-site applications are typically most feasible.

<u>Physical Feasibility - Physical Constraints at Project Site</u>

- <u>Drainage Area</u> – 5 acres maximum for a submerged gravel wetland. Submerged gravel wetland systems need sufficient drainage area to maintain vegetation. See subsection 3.1.8 for guidance on performing a water balance calculation.
- <u>Space Required</u> – Function of drainage area and available head at site.
- <u>Minimum Head</u> – The local slope should be relatively flat. While there is no minimum slope requirement, there does need to be enough elevation drop from the inlet to the outlet to ensure that hydraulic conveyance by gravity is feasible (generally about 3 to 5 feet).
- <u>Pretreatment</u> – Submerged gravel wetland designs shall include a sediment forebay or other equivalent pretreatment measures to prevent sediment or debris from entering and clogging the gravel bed.
- <u>Minimum Depth to Water Table</u> – Unless they receive hotspot runoff, submerged gravel wetland systems can be allowed to intersect the groundwater table. If a submerged gravel wetland receives hotspot runoff and has an underlying water supply aquifer, a separation distance of 2 feet is required between the bottom of the gravel and the elevation of the seasonally high water table to prevent groundwater contamination.
- <u>Soils</u> – Not recommended for clay/silt drainage areas that are not stabilized.

<u>Other Constraints / Considerations</u>

- See subsection 4.3.4 (*Stormwater Wetlands*) for additional planning and design guidance.

4.4.3.5 Planning and Design Standards

The following standards shall be considered **minimum** design standards for the design of submerged gravel wetlands. Submerged gravel wetlands that are not designed to these standards will not be approved. The Director of Engineering and Public Works (the Director) shall have the authority to require additional design conditions if deemed necessary.

<u>A. LOCATION AND SITING</u>

- Submerged gravel wetlands shall have a contributing drainage area of 5 acres or less.
- Submerged gravel wetlands are generally applied to land uses with a high percentage of impervious surfaces and should not be utilized for sites that have less than 50% impervious cover. Any disturbed or denuded areas located within the area draining to and treated by the submerged gravel wetland shall be stabilized prior to construction and use of the submerged gravel wetland.

Manufactured (i.e., Proprietary) Submerged Gravel Wetlands:

- A manufacturer of a treatment system utilizing a submerged gravel wetland is identified below. Manufactured submerged gravel wetlands should be selected on the basis of good design, suitability for the desired pollution control goals, durabilty, ease of installation, ease of maintenance, and reliability. The product listed below is not the only product available, nor should its presence in this manual be construed as an endorsement of this product. It is merely shown as a manufactured submerged gravel wetland that is known to operate in the southeast.

StormTreat	www.stormtreat.com

B. PRETREATMENT / INLETS

- Sediment regulation and removal is critical to sustain submerged gravel wetlands. A gravel wetland facility shall have a sediment forebay or equivalent upstream pretreatment.
- A sediment forebay is designed to remove incoming sediment from the stormwater flow prior to dispersal into the wetland. The forebay shall consist of a separate cell, formed by an acceptable barrier. A forebay shall be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the gravel wetland facility.
- The forebay shall be sized to contain 0.1 inches per impervious acre (363 ft^3) of contributing drainage and shall be no more than 4 to 6 feet deep. The pretreatment storage volume is part of the total WQv design requirement and may be subtracted from the WQv for wetland storage sizing.
- Inflow channels shall be stabilized with flared riprap aprons, or the equivalent. Exit velocities from the forebay to the wetland shall be non-erosive.

C. OUTLET STRUCTURES

- An outlet pipe shall be provided from the submerged gravel wetland to the facility discharge. The design shall ensure that the discharges occur in a non-erosive manner.

D. MAINTENANCE ACCESS

- A minimum 20' wide maintenance right-of-way or drainage easement shall be provided to a submerged gravel wetland from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles.

E. LANDSCAPING

- A landscaping plan shall be developed that indicates the methods used to establish and maintain wetland coverage. Minimum considerations of the plan include: selection of plant species, planting plan and sources of plant material. More information on wetland plants can be found at the following websites:
  - http://wetlands.fws.gov/
  - http://www.npwrc.usgs.gov/resource/plants/floraso/species.htm
  - http://www.tva.gov/river/landandshore/stabilization/plantsearch.htm

4.4.3.6 Design Procedures

Step 1. Compute runoff control volumes

Calculate WQv, CPv in accordance with the guidance presented in Volume 2, Chapter 2.

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

Consider the Application and Site Feasibility Criteria in subsections 4.4.3.4 and 4.4.3.5-A (Location and Siting).

Step 3. Confirm design criteria and applicability

Check with Knox County, TDEC, or other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4. Compute WQv peak discharge (Q_{wq})

The peak rate of discharge for the water quality design storm is needed for sizing the off-line diversion structures (see Volume 2, Chapter 3 for more information on this calculation).

(a) Using WQv, compute CN

(b) Compute time of concentration using TR-55 method

(c) Determine appropriate unit peak discharge from time of concentration

(d) Compute Q_{wq} in inches from unit peak discharge, drainage area, and WQv.

Step 5. Size flow diversion structure

A flow regulator should be supplied to divert the WQv to the submerged gravel wetland. Size low flow orifice, weir, or other device to pass Q_{wq}.

Step 6. Determine pretreatment volume

A sediment forebay is provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the gravel wetland. The forebay shall be sized to contain 0.1 inches per impervious acre (363 ft^3) of contributing drainage and shall be 4 to 6 feet deep. The forebay storage volume counts toward the total WQv requirement and may be subtracted from the WQv for subsequent calculations.

Step 7. Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

See subsection 4.4.3.5 for more details.

Step 8. Design landscape plan

A landscape plan for a stormwater wetland shall be prepared to indicate how it will be stabilized and established with vegetation. See subsection 4.4.3.5-E (Landscaping) for more details.

This page left intentionally blank

4.4.3.7 Inspection and Maintenance Requirements

Note: Section 4.4.3.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of a submerged gravel wetland as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for submerged gravel wetlands, along with a suggested frequency for each activity. Individual gravel wetlands may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e.., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the gravel wetlands in proper operating condition at all times.

Inspection Activities	**Suggested Schedule**
• Ensure that inlets and outlets to each submerged gravel wetland cell are free from debris and not clogged.	Monthly
• Check for sediment buildup in gravel bed.	Annually
Maintenance Activities	**Suggested Schedule**
• If sediment buildup is preventing flow through the wetland, remove gravel and sediment from cell. Replace with clean gravel and replant vegetation.	As needed
• Ensure that inlets and outlets to each submerged gravel wetland cell are free from debris and not clogged.	Monthly
• Check for sediment buildup in gravel bed.	Annually

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of submerged gravel wetlands. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the submerged gravel wetland. Questions regarding stormwater facility inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
SUBMERGED GRAVEL WETLAND INSPECTION CHECKLIST

Location: ____ Owner Change since last inspection? Y N
Owner Name, Address, Phone: ____
Date: __________ Time: ____________ Site conditions:______________________________________

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Wetland Area		
Healthy vegetation?		
Animal burrows present?		
Erosion in drainage area feeding wetland?		
Other (describe)?		
Inlet/Outlet Structures and Channels		
Clear of debris and functional?		
Trash rack clear of debris and functional?		
Sediment accumulation?		
Condition of concrete/masonry?		
Metal pipes in good condition?		
Control valve operation?		
Drain valve operation?		
Outfall channels function, not eroding?		
Other (describe)?		
Sediment Forebays		
Evidence of sediment accumulation?		
Wetland Vegetation Areas		
Vegetation adequate?		
Undesirable vegetation growth?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

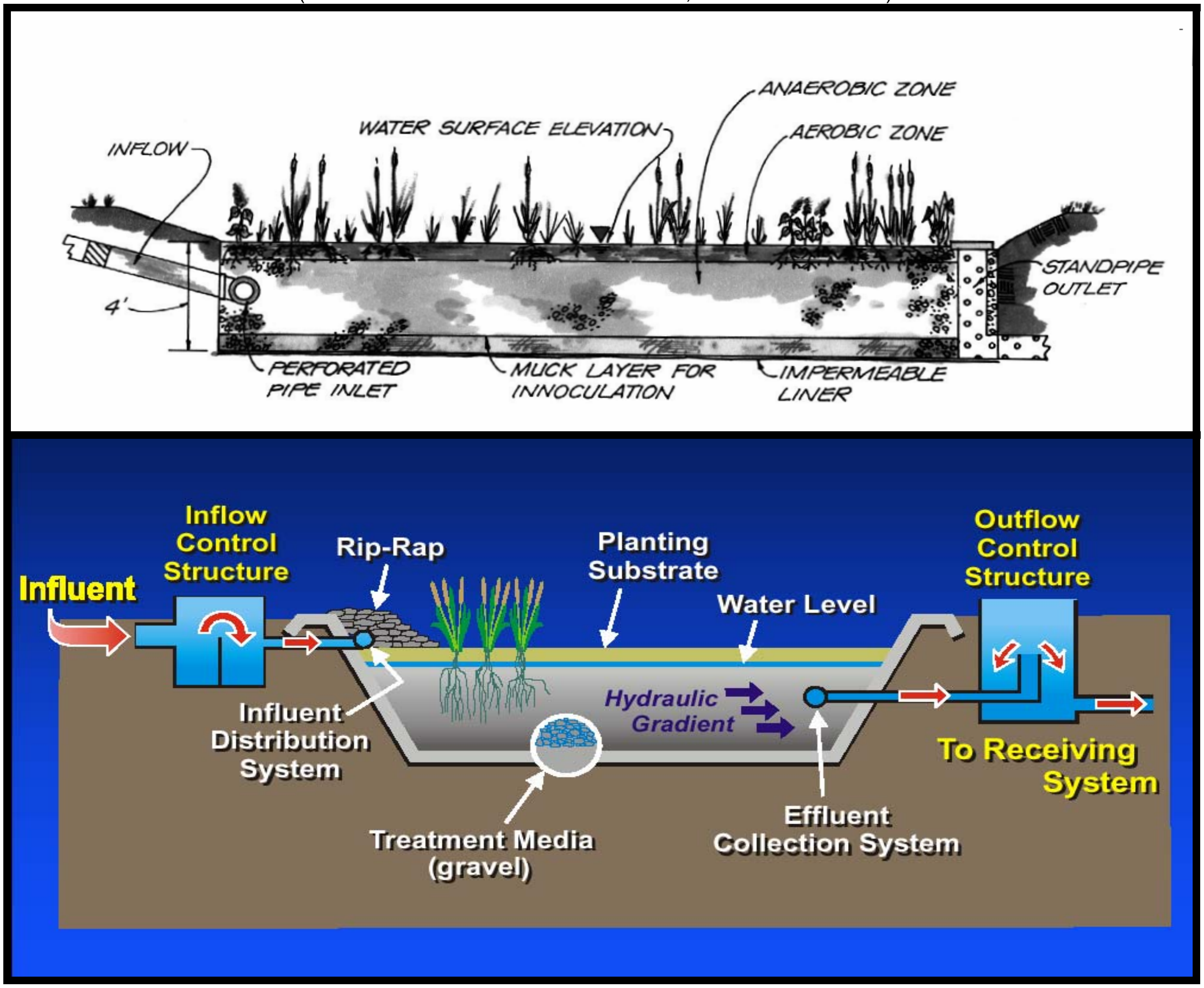
Corrective Action Needed	Due Date

Inspector Signature:______________________________ Inspector Name (printed)__________________________

4.4.3.8 Example Schematics

Figure 4-65. Schematics of Submerged Gravel Wetland Systems

(Sources: Center for Watershed Protection; Roux Associates Inc.)



4.4.3.9 References

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

Center for Watershed Protection (CWP). *Design of Stormwater Filtering Systems.* Prepared for the Chesapeake Research Consortium, Solomons, MD, and U.S. EPA Region 5, Chicago, IL, by the Center for Watershed Protection, Ellicott City, MD, 1996.

Center for Watershed Protection. *Manual Builder.* Stormwater Manager's Resource Center, Accessed July 2005. www.stormwatercenter.net

Maryland Department of the Environment (MDE). *Maryland Stormwater Design Manual.* 2000. Available at: http://www.mde.state.md.us/programs/waterprograms/sedimentandstormwater/stormwater_design/index.asp.

New Jersey Department of Environmental Protection. *Stormwater Best Management Practices Manual.* 2004.

Northern Virginia Planning District Commission (NVPDC). Northern Virginia BMP Handbook. Annadale, Virginia: November 1992.

Schueler, T.R. Design of Stormwater Wetland Systems: Guidelines for Creating Diverse and Effective Stormwater Wetland Systems in the Mid-Atlantic Region. Washington, D.C.: Metrolpolitan Washington Council of Governments (MWCOG), October, 1992.

4.4.3.10 Suggested Reading

Adams, L., Dove L.E., D.L. Leedy, and T. Franklin. *Urban Wetlands for Stormwater Control and Wildlife Enhancement – Analysis and Evaluation.* Urban Wildlife Research Center, Columbia, Maryland, 1983.

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks.* 1993.

City of Austin, TX. *Water Quality Management.* Environmental Criteria Manual, Environmental and Conservation Services, 1988.

City of Sacramento, CA. *Guidance Manual for On-Site Stormwater Quality Control Measures.* Department of Utilities, 2000.

Claytor, R.A., and T.R. Schueler. *Design of Stormwater Filtering Systems.* The Center for Watershed Protection, Silver Spring, MD, 1996.

US EPA. *Storm Water Technology Fact Sheet: Storm Water Wetlands.* EPA 832-F-99-025, Office of Water, 1999.

Faulkner, S. and C. Richardson. *Physical and Chemical Characteristics of Freshwater Wetland Soils.* Constructed Wetlands for Wastewater Treatment, ed. D. Hammer, Lewis Publishers, 831 pp, 1991.

Guntenspergen, G.R., F. Stearns, and J. A. Kadlec. *Wetland Vegetation.* Constructed Wetlands for Wastewater Treatment, ed. D. A. Hammer, Lewis Publishers, 1991.

Metropolitan Washington Council of Governments (MWCOG). A Current Assessment of Urban Best Management Practices: Techniques for Reducing Nonpoint Source Pollution in the Coastal Zone. March, 1992.

4.4.4 Alum Treatment System

Limited Application Stormwater BMP

Description: Alum treatment systems provide chemical treatment of stormwater runoff by means of adding liquid aluminum sulfate (alum) to sediment-laden runoff. The alum combines with phosphorus, heavy metals and suspended solids, causing them to settle-out of suspension.

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Intended for areas requiring regional stormwater treatment from a piped stormwater drainage system where general application BMPs are not feasible.
- Typical drainage area > 50 acres.
- Typically consists of mechanical/electrical dosing system, chemical storage facilities, a downstream settling pond and floc drying beds.

ADVANTAGES / BENEFITS:

- High pollutant removal capability.
- Can be used as a regional stormwater treatment BMP.

DISADVANTAGES / LIMITATIONS:

- High capital, operations and maintenance costs.
- Requires more frequent maintenance than most other stormwater treatment controls.
- Generally, not cost effective for small sites.
- Potential for stormwater quality impacts must be evaluated prior to design/use of the system.

MAINTENANCE REQUIREMENTS:

- Requires trained system operator.
- Restock chemicals frequently.
- Inspect and maintain all components on a frequent, routine basis.
- Remove floc build-up from settling pond.

STORMWATER MANAGEMENT SUITABILITY

- ☑ **Water Quality**
- ☐ **Channel/Flood Protection**
- ☐ **Overbank Flood Protection**
- ☐ **Extreme Flood Protection**

FEASIBILITY CONSIDERATIONS

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

Residential/Subdivision Use: *No*

High Density/Ultra-Urban: *Yes*

Drainage Area: *50 acres minimum*

POLLUTANT REMOVAL

H	**Total Suspended Solids**
M	**Nutrients**: Total Phosphorus / Total Nitrogen
H	**Metals**: Cadmium, Copper, Lead, and Zinc
H	**Pathogens**: Coliform, Streptococci, E.Coli

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

4.4.4.1 General Description

The process of alum (aluminum sulfate) treatment provides treatment of stormwater runoff from a piped stormwater drainage system entering a wet pond by injecting liquid alum into storm sewer lines on a flow-weighted basis during rain events. When added to runoff, liquid alum forms the harmless precipitates of aluminum hydroxide [$Al(OH)_3$] and aluminum phosphate [$AlPO_4$]. These precipitates combine with phosphorus, suspended solids and heavy metals, which then settle-out in a downstream capture pond.

An alum treatment system generally consists of three parts: a flow-weighted dosing system that fits inside a storm sewer manhole; remotely located alum storage tanks; and a downstream settling pond that allows the alum, pollutants and sediments to settle out. (Kurz, 1998). Disposal of the floc that settles in the downstream pond is critical, because of the concentration of dissolved chemicals, and also because bacteria and viruses remain viable in the floc layer (Kurz, 1998). In addition to the settling pond, a separate floc collection pump-out facility should be installed to further reduce the chance of resuspension and transport of floc to receiving waterbodies. The pump disposes the floc into the sanitary sewer system or onto nearby upland areas or sludge drying beds. Permits (from the local utility) will be required to pump to the sanitary sewer, however. The quantity of sludge produced at a site can be as much as 0.5 percent of the volume of water treated (Gibb et al., 1991). Figures 4-66 and 4-67 provide photographs of an alum treatment system settling pond and dosing/injection system, respectively.

Figure 4-66. Settling Pond for Alum Treatment System
(Source: Georgia Stormwater Management Manual)

Figure 4-67. Dosing/Injection Components of an Alum Treatment System
(Source: Georgia Stormwater Management Manual)

The precipitate that is formed when alum is injected into the stormwater system is stable in sediments and will not re-dissolve due to changes in redox potential or pH under conditions normally found in surface water bodies. Laboratory or field testing may be necessary to verify feasibility and to establish design, maintenance, and operational parameters, such as the optimum coagulant dose required to achieve the desired water quality goals, chemical pumping rates and pump sizes.

Alum treatment systems can be expensive to construct and maintain. Capital construction costs depend primarily on the number of outfall locations treated rather than the size of the area draining to the system. Operations and maintenance expenses include costs for chemicals, power to the system, manpower for routine inspections and maintenance, and equipment renewal and replacement costs. In addition, regulatory agencies or wastewater utilities may require long-term monitoring of water quality downstream of alum treatment systems, which further increases maintenance costs.

4.4.4.2 Stormwater Management Suitability

Alum treatment systems are designed primarily for large watersheds. They are designed solely for the purpose of treating stormwater quality and do not have the ability to provide channel or flood protection.

4.4.4.3 Pollutant Removal Capabilities

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment.

- Total Suspended Solids – 90%
- Total Phosphorus – 80%
- Total Nitrogen – 60%

- Heavy Metals – 75%
- Pathogens – 90%

For additional information and data on pollutant removal capabilities for alum treatment systems, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the International Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org.

4.4.4.4 Application and Site Feasibility Criteria

The following basic criteria should be evaluated to ensure the suitability of an alum treatment system for meeting stormwater management objectives on a site or development.

General Feasibility

- Well-suited for large drainage areas that discharge into a closed body of water (e.g., a lake or pond).
- Suitable for use in high density/ultra-urban areas.
- Suitable for use as a regional stormwater control.

Physical Feasibility - Physical Constraints at Project Site

- Drainage Area – Typically 50 acres minimum for an alum treatment system.

4.4.4.5 Planning and Design Standards

Alum treatment systems are fairly complex, and design details are beyond the scope of this manual. Further information can be obtained from the Internet and by contacting engineers who have designed and implemented successful systems. The Director of Engineering and Public Works (the Director) shall have the authority to set the design conditions for alum treatment systems on a case-by-case basis.

The following information is provided as guidance for the design of alum treatment systems.

- Injection points should be 100 feet upstream of discharge points.
- Alum concentration is typically 10 μg/l.
- Alum treatment systems may need to control pH.
- For new pond design, the required size is approximately 1% of the drainage basin size, as opposed to 10 to 15% of the drainage basin area for a standard detention pond.
- No volume requirement is required when discharging to existing lakes.

4.4.4.6 Inspection and Maintenance Requirements

Note: Section 4.4.4.6 and the operation and maintenance document supplied by the alum system designer must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of an alum treatment system as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for alum treatment systems, along with a suggested frequency for each activity. Individual alum treatment systems may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the alum treatment system in proper operating condition at all times. Inspections and maintenance of the alum treatment system must be performed by a trained system operator..

Inspection Activities	Suggested Schedule
• Dosing equipment – monitor dosage of alum and other chemicals. Also monitor the expected flows through the system. • Perform routine inspection of dosing equipment and pump-out facility to ensure that all equipment is in proper operating condition. • Inspect dosing equipment and storage facility for signs of leaks or spills. • Inspect chemical amounts and restock if needed. • Monitor pH and other parameters in the settling pond to determine potential negative impacts to receiving waters. • Inspect settling pond for signs of damage, impending failure, poor water quality. • Inspect storage capacity of settling pond and floc drying beds (if used).	Monthly or more frequently
Maintenance Activities	**Suggested Schedule**
• Adjust the dosage of alum and other chemicals and possibly regulate flows through the basin to ensure proper dosage and delivery of runoff to the settling pond. • Perform maintenance and repair of pump equipment, chemical supplies and delivery system. • Dredge settling pond and properly dispose of accumulated floc.	As Needed

Knox County encourages the use of the inspection checklist presented below for guidance in the inspection and maintenance of the alum treatment system. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the treatment system. Owners of alum treatment systems are encouraged to provide additional inspection/maintenance items to ensure the long-term proper operation of the treatment system. Questions regarding inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST FOR ALUM TREATMENT SYSTEMS

Location: ____ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ____

Date: ___________ Time: ______________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Dosing System		
Dispensing proper dose?		
Signs of leaks or spills?		
In proper operating condition?		
Chemical Storage Facility		
Signs of leaks or spills?		
Proper delivery of chemicals to dosing system?		
In proper operating condition?		
Settling Pond		
pH and water quality condition?		
Erosion on embankment?		
Animal burrows in embankment?		
Cracking, sliding, bulging of dam?		
Blocked or malfunctioning drains?		
Leaks or seeps on embankment?		
Obstructions of spillway(s)?		
Clear of debris and functional?		
Sediment/floc accumulation?		
Condition of concrete/masonry?		
Metal pipes in good condition?		
Control valve operation?		
Pond drain valve operation?		
Channels/spillways function, not eroding?		
Other (describe)?		
Other (describe)?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature:________________________________ Inspector Name (printed)____________________________

4.4.4.7 References

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

Gibb, A., B. Bennet, and A. Birkbeck. *Urban Runoff Quality and Treatment: A Comprehensive Review*. Prepared for the Vancouver Regional District, the Municipality of Surrey, British Columbia, Ministry of Transportation and Highways, and British Columbia Ministry of Advanced Education and Trainiing. Document No. 2-51-246(242), 1991.

Harper, H.H. and J.L. Kerr. *Design, Alum Treatment of Stormwater Runoff: The First Ten Years.* Environmental Research and Design, Orlando, Florida, 1996.

Kurz, R. *Removal of Microbial Indicators from Stormwater Using Sand Filtration, Wet Detention, and Alum Treatment Best Management Practices.* Southwest Florida Water Management District, Brooksville, Florida, 1998.

www.stormwaterauthority.org/assets/alum%20injection.pdf

4.4.4.8 Suggested Reading

Center for Watershed Protection. *Manual Builder.* Stormwater Manager's Resource Center. *www.stormwatercenter.net*

Maryland Department of the Environment. *Maryland Stormwater Design Manual, Volumes I and II.* Prepared by Center for Watershed Protection (CWP), 2000.

US EPA. *Storm Water Technology Fact Sheet: Sand Filters.* EPA 832-F-99-007. Office of Water. 1999.

Walker, W. *Phosphorus Removal by Urban Runoff Detention Basins.* Lake and Reservoir Management, North American Society for Lake Management, 314, 1987.

This page left intentionally blank

4.4.5 Proprietary Structural BMPs

Limited Application Stormwater BMP

Description: Proprietary systems are defined as manufactured stormwater treatment systems that are available from commercial vendors.

REASONS FOR LIMITED USE	STORMWATER MANAGEMENT SUITABILITY
Depending on the proprietary system, there may be: • Limited performance data • Application constraints • High maintenance requirements • Higher costs than other structural control alternatives	* **Water Quality** * **Channel/Flood Protection**
KEY CONSIDERATIONS	**SPECIAL APPLICATIONS**
• Independent performance data must be available to prove a demonstrated capability of meeting stormwater management goal(s) • System or device must be appropriate for use in Knox County conditions • Installation and operations/maintenance requirements must be understood by all parties approving and using the system or device in question	* **Pretreatment** * **High Density/Ultra-Urban** * **Other:** **Residential Subdivision Use**: * * Depends on the specific proprietary structural control

Note: Knox County does not recommend any specific commercial vendors for proprietary systems. This subsection is being included in order to provide Knox County with a rationale for approving the use of a proprietary system or practice.

4.4.5.1 General Description

There are many types of commercially-available proprietary stormwater structural controls for both water quality treatment and quantity control. These systems include:

- Hydrodynamic systems such as gravity and vortex separators
- Filtration systems
- Catch basin media inserts
- Chemical treatment systems
- Package treatment plants
- Prefabricated detention structures

Many proprietary systems are useful on small sites and space-limited areas where there is not enough land or room for other structural BMP alternatives. Proprietary systems can often be used in pretreatment applications in a treatment train. However, proprietary systems are often more costly than other alternatives and may have high maintenance requirements. Perhaps the largest difficulty in using a proprietary system is the lack of adequate independent performance data, particularly for use in Knox County conditions. Below are general guidelines that should be followed before considering the use of a proprietary commercial system.

4.4.5.2 Guidelines for Using Proprietary Systems

In order for use as a limited application control, a proprietary system must have a demonstrated capability of meeting the stormwater management goals for which it is being intended. This means that the system must provide:

(1) Independent third-party scientific verification of the ability of the proprietary system to meet water quality treatment objectives and/or to provide water quantity control (channel or flood protection);

(2) Proven record of longevity in the field; and,

(3) Proven ability to function in Knox County conditions (e.g., climate, rainfall patterns, soil types, etc.).

For a propriety system to meet item (1) listed above, the following monitoring criteria should be met for supporting studies:

- At least 15 storm events must be sampled
- The study must be independent or independently verified (i.e., may not be conducted by the vendor or designer without third-party verification)
- The study must be conducted in the field, as opposed to laboratory testing
- Field monitoring must be conducted using standard protocols which require proportional sampling both upstream and downstream of the device
- Concentrations reported in the study must be flow-weighted
- The propriety system or device must have been in place for at least one year at the time of monitoring

Although local data is preferred, data from other regions can be accepted as long as the design accounts for the local conditions.

Knox County may submit a proprietary system to further scrutiny based on the performance of similar practices. A poor performance record or high failure rate is valid justification for not allowing the use of a proprietary system or device. Consult Knox County Engineering & Public Works Stormwater Management Section for more information in regards to the use of proprietary structural stormwater controls.

4.4.6 Underground Detention

Limited Application Stormwater BMP

Description: Detention storage located in underground pipe systems or vaults designed to provide water quantity control through detention and/or extended detention of storm-water runoff.

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Maximum drainage area = 25 acres
- Maximum drainage area = 1 acre for structures passing the Qp_{100}
- Access point for maintenance required.
- Used downstream of a water quality BMP.

ADVANTAGES / BENEFITS:

- To be used for space-limited applications only.
- Good for retrofitting small urbanized lots.
- Concrete vaults or pipe systems can be used.
- Longevity is high, with proper maintenance.

DISADVANTAGES / LIMITATIONS:

- Discouraged unless other options unfeasible.
- Controls for stormwater quantity only – not intended to provide water quality treatment.
- Dissolved pollutants are not removed.
- Frequent maintenance required.

MAINTENANCE REQUIREMENTS:

- Remove debris from inlet and outlet structures.
- Monitor sediment accumulation.
- Clean out sediment and floatable debris using catch basin cleaning equipment (vacuum pumps).

STORMWATER MANAGEMENT SUITABILITY

☐	**Water Quality**
✓	**Channel/Flood Protection**
✓	**Overbank Flood Protection**
✷	**Extreme Flood Protection**

✷ in certain situations

FEASIBILITY CONSIDERATIONS

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

Residential/Subdivision Use: *No*

Drainage Area: *25 acres maximum, 1 acre maximum for Qp_{100}*

Soils: *Not dependent upon soil type*

POLLUTANT REMOVAL

L	**Total Suspended Solids**
L	**Nutrients**: Total Phosphorus / Total Nitrogen
L	**Metals**: Cadmium, Copper, Lead, and Zinc
L	**Pathogens**: Coliform, Streptococci, E.Coli

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

4.4.6.1 General Description

Underground detention is typically utilized on sites where developable surface area is at a minimum. Underground detention facilities can be either box-shaped facilities constructed with reinforced concrete, facilities constructed with large diameter metal or plastic pipe or commercially-available proprietary underground systems. All methods serve as alternatives to surface dry detention for stormwater quantity control where there is not adequate land for a dry detention basin or multi-purpose detention area.

Underground detention can provide channel protection through extended detention of the channel protection volume and overbank flood control (and in some cases extreme flood) through normal detention. Basic storage design and routing methods are the same as for dry detention basins except that the bypass for high flows must be included in the design.

Due to the potential problems that local conditions present, the Knox County Engineering & Public Works Department does not support the use of underground detention unless other peak discharge control options are deemed physically infeasible.

4.4.6.2 Pollutant Removal Capabilities

Underground detention facilities are not capable of significant pollutant removal. Therefore, because underground detention is not intended for water quality treatment, it must be used in a treatment train approach with other structural BMPs that provide treatment of the WQv. This will prevent the underground pipe systems or vaults from becoming clogged with trash or sediment and significantly reducing the maintenance requirements.

4.4.6.3 Planning and Design Standards

If underground detention is allowed by the Director, the following standards shall be considered **minimum** design standards for the design of underground detention. Underground detention that is not designed to these standards will not be approved. The Director of Engineering and Public Works (the Director) shall have the authority to require additional design conditions if deemed necessary.

A. LOCATION AND SITING

- The maximum contributing drainage area to be served by a single underground detention vault or tank is 25 acres.
- Flood protection controls for peak discharge control (Qp_2, Qp_{10}, Qp_{25} and Qp_{100}) should be designed as final controls for on-site stormwater. Therefore, underground detention will typically be located downstream of structural stormwater BMPs that are designed to provide treatment of the water quality volume (WQv) and channel protection volume (CPv).
- Underground detention shall be placed in a drainage easement that is recorded with the deed and shown on the plan. The drainage easement shall be located 15 feet from the outside limits of the underground detention structure. Minimum setback requirements for the easement shall be as follows unless otherwise specified by the Director:
  - From a public water system well – TDEC specified distance per designated category
  - From a private well – 50 feet; if the well is down gradient from a land use that must obtain a Special Pollution Abatement Permit, then the minimum setback is 250 feet
  - From a septic system tank/leach field – 50 feet
- The drainage easement shall be located 15 feet from the outside limits of the underground detention structure. The first floor elevation (FFE) for any structure adjacent to underground detention shall have an elevation no lower than 1 foot above the emergency spillway elevation.

B. GENERAL DESIGN

- Underground detention shall consist of the following elements, designed in accordance with the specifications provided in this section.
  (1) An outlet structure;
  (2) An emergency spillway; and
  (3) Maintenance access.
- Underground detention systems are sized to provide extended detention of the channel protection volume over 24 hours and temporarily store the volume of runoff required to provide overbank flood (Qp_{25}) protection. Some systems will also provide extreme flood (Qp_{100}) protection.
- Routing calculations must be used to demonstrate that the storage volume is adequate. See Chapter 3 for procedures on the design of detention storage.
- Adequate maintenance access must be provided for all underground detention systems. Access must be provided over the inlet pipe and outflow structure. Access openings can consist of a standard frame, grate and solid cover, or a removable panel. Vaults with widths of 10 feet or less should have removable lids.

C. PHYSICAL SPECIFICATIONS / GEOMETRY

- Underground detention vaults and tanks must meet structural requirements for overburden support and traffic loading if appropriate.
- Detention Vaults: Minimum 3,000 psi structural reinforced concrete may be used for underground detention vaults. All construction joints must be provided with water stops. Cast-in-place wall sections must be designed as retaining walls. The maximum depth from finished grade to the vault invert should be 20 feet.
- Detention Pipes: The minimum pipe diameter for underground detention is 36 inches.

Inlet and Outlet Structures

- A separate sediment sump or vault chamber sized to contain 0.1 inch per impervious acre (363 ft^3/acre) of contributing drainage should be provided at the inlet for underground detention systems that are in a treatment train with off-line water quality treatment structural BMPs.
- For CPv control, a low flow orifice capable of releasing the channel protection volume over 24-72 hours must be provided. The channel protection should be adequately protected from clogging by an acceptable external trash rack. Orifices smaller than 3” require internal orifice protection (i.e., an over perforated vertical stand pipe with 0.5-inch orifices or slots that are protected by wire cloth and a stone filtering jacket). Adjustable gate valves can also be used to achieve this equivalent diameter.
- For overbank flood protection, an additional outlet is sized for Qp25 control (based upon hydrologic routing calculations) and can consist of a weir, orifice, outlet pipe, combination outlet, or other acceptable control structure. See Chapter 3 for more information on the design of outlet works.
- Water shall not be discharged from underground detention in an erosive manner. Riprap, plunge pools or pads, or other energy dissipators are to be placed at the end of the outlet to prevent scouring and erosion. If a pond outlet discharges immediately to a channel that carries dry weather flow, care should be taken to minimize disturbance along the downstream channel and streambanks, and to reestablish a forested riparian zone in the shortest possible distance (if the downstream area is located in a water quality buffer). See Chapter 7 for more guidance.

D. EMERGENCY SPILLWAY

- A high flow bypass shall be included in the underground detention design to safely pass Qp_{100} in the event of outlet structure blockage or mechanical failure. The bypass shall be located so that downstream structures will not be impacted by emergency discharges.

E. MAINTENANCE ACCESS

- A maintenance right-of-way or easement having a minimum width of 20 feet shall be provided from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles.
- The maintenance access shall extend to the forebay (if included) and outlet works, and, to the extent feasible, be designed to allow vehicles to turn around.

4.4.6.4 Design Procedures

In general, site designers should perform the following design procedures when designing underground detention.

Step 1. Compute runoff control volumes.

Calculate Qp_2, Qp_{10}, Qp_{25} and Qp_{100}, in accordance with the guidance presented in Volume 2, Chapter 2.

Step 2. Confirm design criteria and applicability.

Consider any special site-specific design conditions/criteria from subsection 4.4.6.3. Check with Knox County Engineering, TDEC, or other agencies to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply to the site.

Step 3. Calculate Qp_2, Qp_{10}, Qp_{25} and Qp_{100} release rates and water surface elevations.

Set up stage-storage-discharge relationships for the control structure for the 2, 10, 25 and 100-year storms.

Step 4. Design spillway(s)

Size emergency spillway (bypass) and analyze safe passage of the Qp_{100}. Set the emergency spillway elevation a minimum of 0.1 feet above the 100-year water surface elevation.

Step 5. Design inlets, outlet structures and maintenance access.

See subsection 4.4.6.3 for more details.

4.4.6.5 Maintenance Requirements and Inspection Checklist

Note: Section 4.4.6.5 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective operation of underground detention as designed. It is the responsibility of the property owner to maintain all stormwater BMPs in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for underground detention, along with a suggested frequency for each activity. Individual underground detention locations may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain the pond in proper operating condition at all times.

Inspection Activities	Suggested Schedule
• After several storm events or an extreme storm event, inspect for: signs of clogging of the inlet or outlet structures and sediment accumulation.	As Needed
• Inspect for: trash and debris; clogging of the outlet structures and any pilot channels; excessive erosion; sediment accumulation in the basin and inlet/outlet structures; tree growth on dam or embankment; the presence of burrowing animals; standing water where there should be none; vigor and density of the grass turf on the basin side slopes and floor; differential settlement; cracking; leakage; and slope stability.	Semi-annually
• Inspect that the outlet structures, pipes, and downstream and pilot channels are free of debris and are operational. • Note signs of pollution, such as oil sheens, discolored water, or unpleasant odors. • Check for sediment accumulation in the facility. • Check for proper operation of control gates, valves or other mechanical devices.	Annually
Maintenance Activities	**Suggested Schedule**
• Perform structural repairs to inlet and outlets • Clean and remove debris from inlet and outlet structures.	Monthly or as needed
• Repair damage to inlet or outlet structures, control gates, valves, or other mechanical devices; repair undercut or eroded areas.	As Needed
• Monitor sediment accumulations, and remove sediment when the pond volume has become reduced significantly.	As Needed

Knox County encourages the use of the inspection checklist that is presented on the next page to guide the property owner in the inspection and maintenance of underground detention facilities. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the underground detention facilities. Questions regarding stormwater facility inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST AND MAINTENANCE GUIDANCE (continued)
UNDERGROUND DETENTION INSPECTION CHECKLIST

Location: ______________________________ Owner Change since last inspection? Y N

Owner Name, Address, Phone: ______________________________

Date: __________ Time: __________ Site conditions: ______________________________

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Inlet/Outlet Structures		
Clear of debris and functional?		
Trash rack clear of debris and functional?		
Sediment accumulation?		
Condition of concrete/masonry?		
Metal pipes in good condition?		
Control valve operational?		
Pond drain valve operational?		
Outfall channels function, not eroding?		
Other (describe)?		
Pond Bottom		
Excessive sedimentation?		

If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

Corrective Action Needed	Due Date

Inspector Signature: ______________________________ Inspector Name (printed) ______________________________

4.4.6.6 Example Schematics

Figure 4-68. Example Underground Detention Pipe System

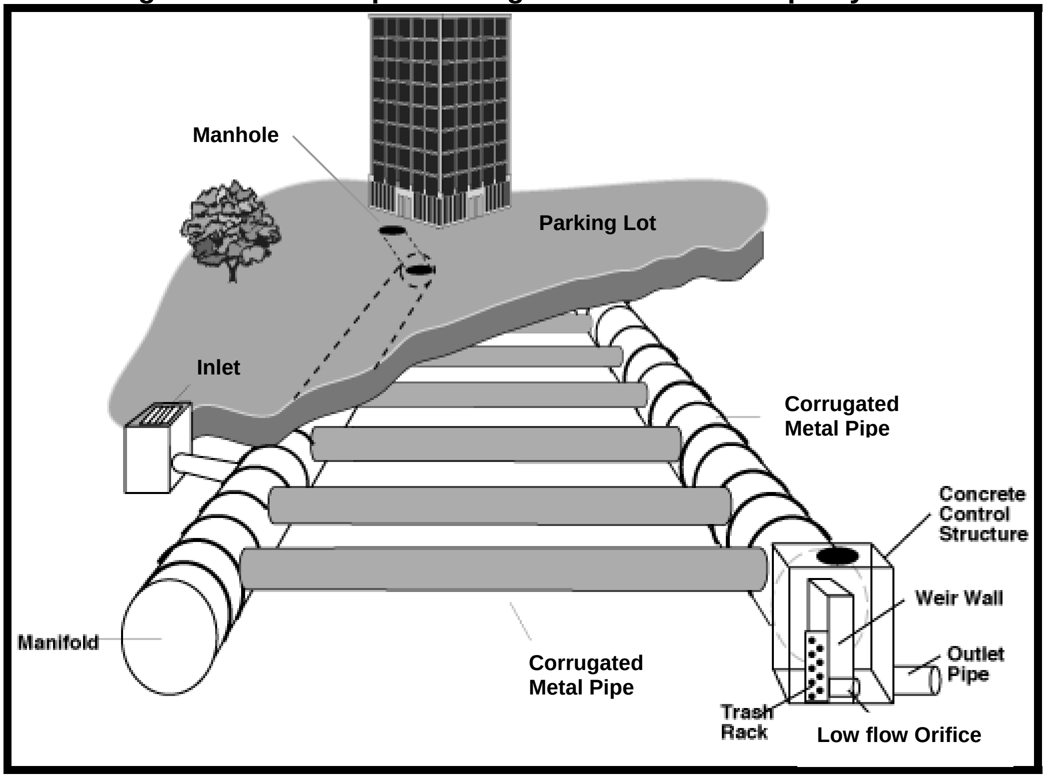
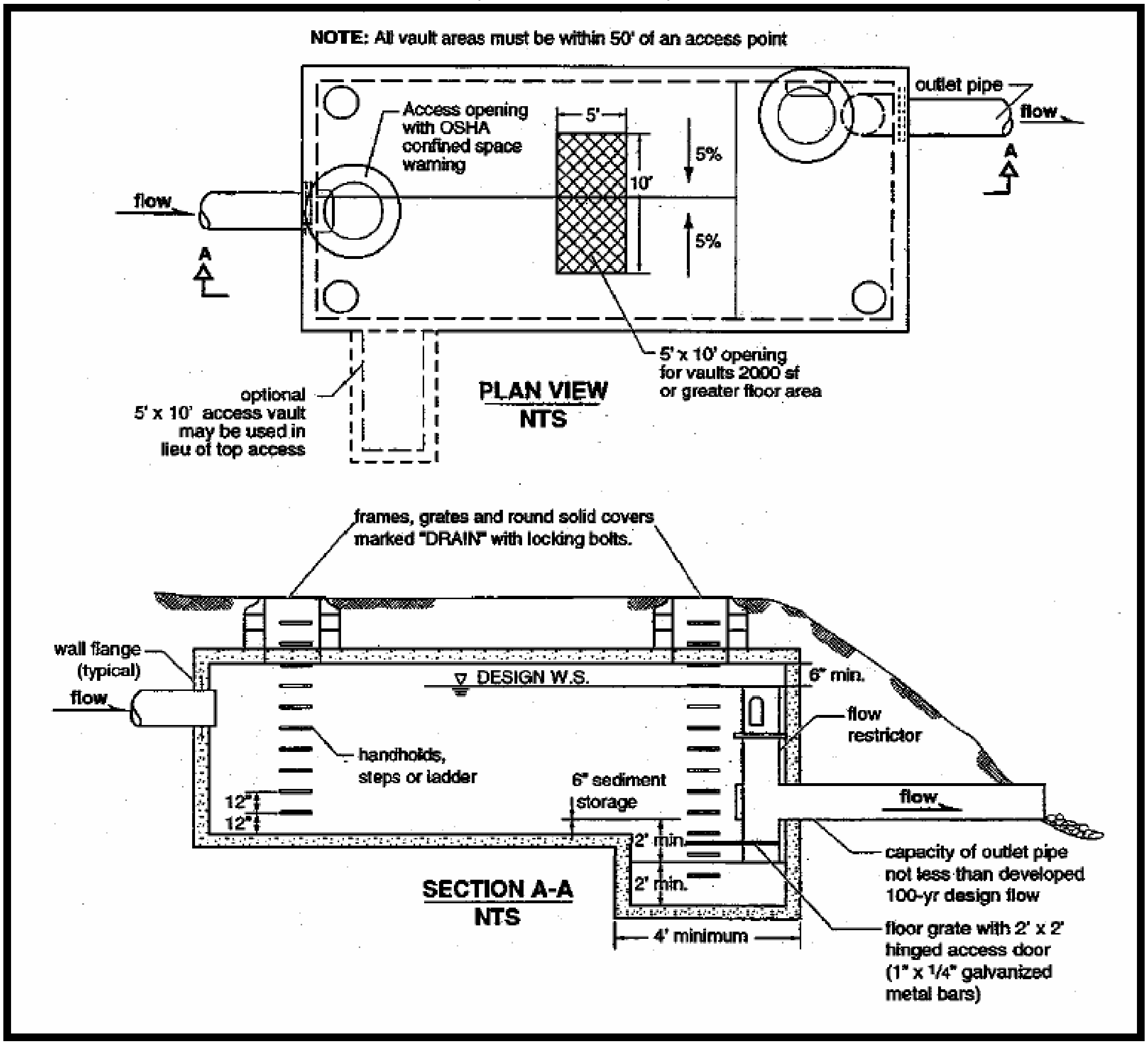


Figure 4-69. Schematic of a Typical Underground Detention Vault
(Source: WDE, 2000)



4.4.6.7 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual, Volume 4 Best Management Practices.* 2006.

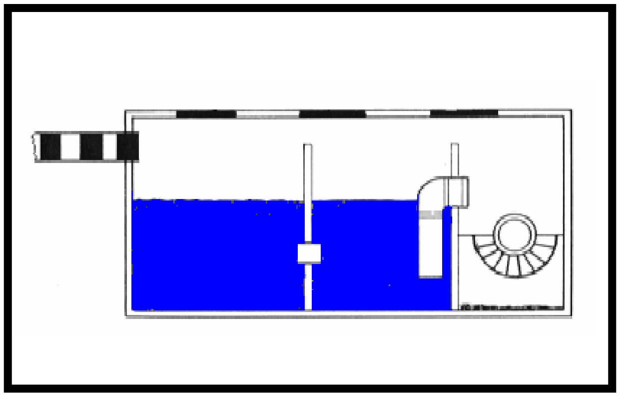
Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

Washington State Department of Ecology. *Stormwater Management Manual for Western Washington.* 2000.

This page left intentionally blank

4.4.7 Oil-Grit (Gravity) Separator

Limited Application Stormwater BMP



Description: The oil/grit separator is a device designed to remove settleable solids, oil and grease, debris and floatables from stormwater runoff. This is done through gravitational settling and trapping of pollutants. Oil-grit separators are also called gravity separators or oil/water separators.

KEY CONSIDERATIONS

DESIGN GUIDELINES:

- Maximum drainage area of 1 acre.
- Access for maintenance is required.
- Performance dependent on design and frequency of inspection and cleanout of unit.
- Openings to device must be 1/16 inch or less to prevent mosquito intrusion and breeding.
- Install as an off-line device unless separator can be sized to handle a small drainage area.
- Install inspection/collection manhole on downstream side to provide easy access for sampling of effluent.

ADVANTAGES / BENEFITS:

- Good for sites where larger, or above-ground BMPs are not an option, or for retrofitting small urbanized lots.
- Can be used as pretreatment for other BMPs.
- Longevity is high, with proper maintenance.
- Standardized designs allow for easy installation.

DISADVANTAGES / LIMITATIONS:

- Cannot alone achieve the 80% TSS removal target.
- Limited performance data.
- Dissolved pollutants are not removed.
- Frequent maintenance required.

MAINTENANCE REQUIREMENTS:

- Inspect the gravity separator unit.
- Clean out sediment, oil and grease, and floatable debris using catch basin cleaning equipment (vacuum pumps).

STORMWATER MANAGEMENT SUITABILITY

- [x] **Water Quality**
- [ ] **Channel/Flood Protection**
- [ ] **Overbank Flood Protection**
- [ ] **Extreme Flood Protection**

Accepts runoff from SPAP land uses: *Yes*

FEASIBILITY CONSIDERATIONS

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

Residential/Subdivision Use: *No*

Drainage Area: *1 acre maximum*

Soils: *Not dependent upon soil type*

POLLUTANT REMOVAL

M	**Total Suspended Solids**
L	**Nutrients**: Total Phosphorus / Total Nitrogen
No Data	**Metals**: Cadmium, Copper, Lead, and Zinc
No Data	**Pathogens**: Coliform, Streptococci, E.Coli

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

4.4.7.1 General Description

Oil-grit separators (also called oil/water separators or gravity separators) are hydrodynamic separation devices that are designed to remove grit and heavy sediments, oil and grease, debris and floatable matter (e.g., litter) from stormwater runoff through gravitational settling and trapping. There are two basic types of oil-grit separators, as displayed in Figure 4-71, in Section 4.4.7.8. Conventional separators rely upon gravity, physical characteristics of oil and sediments, and good design parameters to achieve pollutant removal. Coalescing plate interceptor (CPI) separators contain closely-spaced plates which greatly enhance the removal efficiency for oils and greases. In addition, a wide variety of separator systems are commercially-available in a variety of layouts, for which vendors have design data and procedures. Example schematics for both types of oil-water separators are displayed in Section 4.4.7.8.

Conventional oil-grit separator units contain a permanent pool of water and typically consist of an inlet chamber, separation/storage chamber, a bypass chamber, and an access port for maintenance purposes. Runoff enters the inlet chamber where heavy sediments and solids drop out. Then the flow moves into the main separation chamber, where further settling of suspended solids takes place. Oil and grease are skimmed and stored in a waste oil storage compartment for future removal. After moving into the outlet chamber, the clarified runoff is then discharged to the site's stormwater conveyance system. Oil-grit separators are sized based on a design flow rate, the Water Quality Peak Flow Rate (Q_{wq}). This contrasts with most other stormwater structural controls, which are sized based on capturing and treating a specific volume.

CPI separators include coalescing tubes or plates that provide an additional media in which minute oil globules can agglomerate to aid in the separation process. Oil that agglomerates around the coalescing tubes/plates can easily be skimmed through the gravity process. CPI separators **must** be utilized in "hotspot" areas where oil, grease, or other petroleum products are potential pollutants (e.g., fueling areas, gas stations, etc.).

The performance of oil-grit separator systems is based primarily on the relatively low solubility of petroleum products in water and the difference between the specific gravity of water and the specific gravities of petroleum compounds. Separators are not designed to separate other products such as solvents, detergents, or dissolved pollutants. The typical oil-grit separator unit may be enhanced with a pretreatment swirl concentrator chamber, oil draw-off devices that continuously remove the accumulated light liquids, and flow control valves that regulate the flow rate into the unit. Separators are available as prefabricated proprietary systems from a number of different commercial vendors. Some of the enhancements added by commercial vendors are presented in the example schematics presented in section 4.4.7.8.

4.4.7.2 Stormwater Management Suitability

Oil-grit separators are designed solely as stormwater quality treatment and do not have the ability to provide channel protection or flood protection. An important consideration when designing an oil-grit separator system for a site is how to bypass large storm events that exceed the design flow capacity around the separator without damaging the unit, exceeding the design flow capacity, or resuspending collected pollutants. Since resuspension of accumulated sediments and oil droplets is possible during heavy storm events, oil-grit separator units are typically installed off-line with a bypass installed to minimize pollutant wash-out or resuspension.

<u>Water Quality (WQv)</u>
To treat stormwater runoff, oil-grit separators rely on gravity and trapping to filter pollutants. An oil-grit separator cannot alone achieve the 80% TSS removal criteria. Therefore, separators are frequently used as the upstream pretreatment measure in a series of BMPs, ahead of a detention basin or wetland.

<u>Channel Protection (CPv), Overbank Flood Protection (up to Qp_{25}) and Extreme Flood Protection (Qp_{100})</u>
Oil-grit separators will not provide for channel protection, overbank or extreme flood protection. Another structural BMP, such as an extended dry detention pond, that is designed to handle flood control, must be used in conjunction with the oil-grit separator to achieve the CPv, Qp_2, $Qp_{10,}$ Qp_{25} and Qp_{100} design criteria.

4.4.7.3 Pollutant Removal Capabilities

Testing of gravity separators has shown that they can remove between 40% and 50% of the TSS loading when used in an off-line configuration (Curran, 1996 and Henry, 1999). Gravity separators also provide removal of debris, hydrocarbons, trash and other floatables. They provide only minimal removal of nutrients and organic matter.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling and professional judgment. Actual field testing data and pollutant removal rates from an independent source should be obtained before using a proprietary gravity separator system.

- Total Suspended Solids – 40%
- Total Phosphorus – 5%
- Total Nitrogen – 5%
- Pathogens – Insufficient data to provide a pollutant removal value
- Heavy Metals – Insufficient data to provide a pollutant removal vale

4.4.7.4 Application and Feasibility Criteria

One of the most important selection criteria when considering an oil-grit separator is the long-term maintenance and operation costs, and the need for regular inspections and cleanout. Inspection and maintenance needs for such systems can be considered high relative to other stormwater BMPs. Therefore, the oil-grit separator system should only be constructed if the property owner or tenant of the site has both the physical and fiscal ability to perform regular inspection and maintenance of the system on a long-term basis. This is one of the constraints that will be considered by Knox County Engineering when oil-grit separators are proposed as a BMP for a development or redevelopment site.

Oil-grit separators are best used in commercial, industrial and transportation land uses and are intended primarily as a pretreatment measure for high-density or ultra-urban sites, or for use in hydrocarbon hotspots, such as gas stations and areas with high vehicular traffic. However, **separators cannot be used for the removal of dissolved or emulsified oils or for pollutants such as coolants, soluble lubricants, glycols and alcohols**. Suitable applications of an oil-grit separator include:

- pretreatment for other structural controls;
- parking lots, streets, driveways, truck loading areas;
- runways, marinas, loading wharves;
- gasoline stations, refueling areas;
- automotive repair facilities, oil-change businesses, fleet maintenance yards;
- recycling or salvage yards which accept automotive equipment; and,
- commercial vehicle washing facilities.

4.4.7.5 Planning and Design Standards

Knox County's design standards for oil-grit (gravity) separators are presented below. Design specifications developed by a commercial vendor for prefabricated proprietary systems can also be utilized, but must be approved where such specifications differ and/or are less stringent from the standards presented below. The Director shall have the authority to require additional design conditions if deemed necessary.

A. LOCATION AND SITING

- Any individual oil-grit separator shall have a contributing drainage area no greater than 1 acre.
- It is desirable to maintain reasonable dimensions by bypassing larger storm flows in excess of the design flow rates. Thus, it is preferred that oil-grit separators be located off-line. An off-line separator can be an

existing or proposed manhole with a baffle or other control (shown in Figure 4-71).

- Oil-grit separator systems can be installed in almost any soil or terrain. Since these devices are underground, appearance is not an issue and public safety risks are low.
- The design loading rate for oil-grit separators is low; therefore, they can only be cost-effectively sized to detain and treat the water quality volume, or other low flows if required by Knox County Engineering. It is usually not economical or feasible to size an oil-grit separator to treat large design storms. Oil-grit separators require frequent maintenance for the life of the separator unit. Maintenance can be minimized (and performance can be increased) by careful planning and design, particularly upstream and downstream of the separator unit.

B. PHYSICAL SPECIFICATIONS / GEOMETRY

- Design procedures for commercially available oil-grit separators are usually given by the manufacturer in simplified tables or graphs based on field testing and observed pollutant removal rates. Pollutant removal rates higher than those indicated in Section 4.4.7.3 must be proven using the criteria for proprietary BMPs presented in Volume 2, Chapter 2 of this manual.
- Oil-grit separators must be constructed with watertight joints and seals.
- The separation chamber shall provide for three separate storage volumes, as follows:
  (1) A volume for separated oil storage at the top of the chamber;
  (2) A volume for settleable solids accumulation at the bottom of the chamber; and,
  (3) A volume required to give adequate flow-through detention time for separation of oil and sediment from the stormwater flow.
- Ideally, a gravity separator design will provide an oil draw-off mechanism to a separate chamber or storage area. This design is required where a gravity separator is utilized to treat oil, grease and/or petroleum hotspots.
- Oil-grit separators are typically designed to bypass runoff flows in excess of the water quality volume peak flow. Some designs have built-in high flow bypass mechanisms, whereas others require a diversion structure or flow splitter located upstream of the device in the drainage system. Bypass mechanisms must minimize potential for captured pollutants from being washed-out or resuspended by large flows. Regardless of the bypass mechanism, an adequate outfall/outlet must be provided for both the discharge from the separator itself, and the bypassed discharge. Runoff shall be discharged in a non-erosive manner.
- The device shall be designed such that the velocity through the separation chamber does not exceed the entrance velocity.
- A trash rack shall be included in the design to capture floating debris, preferably near the inlet chamber to prevent debris from becoming oil impregnated.
- The total wet storage of the gravity separator unit shall be no less than 400 cubic feet per contributing impervious acre.
- The theoretical sizing of a conventional oil-grit separator requires the use of Stokes Law for the computation of rise velocity of oil droplets:

Equation 4.4.7.1

$$V_p = \frac{1.79 \times 10^{-8} (S_w - S_p) D_p^{\ 2}}{N}$$

where: V_p = upward rise velocity of petroleum droplet (ft/s)
S_w = specific gravity of water (0.998 to 1.000)
S_p = specific gravity of the petroleum droplet (typically 0.85 to 0.95)
D_p = diameter of petroleum droplet to be removed (microns)
N = absolute viscosity of water (poises)

The expected temperature is generally chosen for cold winter months. Typical values for the specific gravity and absolute viscosity of water at various temperatures are shown in the following table:

Temperature	S_w	N
32° F	0.999	0.01794
40° F	1.000	0.01546
50° F	0.999	0.01310
60° F	0.999	0.01129
70° F	0.998	0.00982

Sizing a Conventional Oil-Grit Separator:

- Using V_p above, a conventional oil-grit separator can be sized as follows:

Equation 4.4.7.2

$$D = \left(\frac{Q}{RV_H} \right)^{0.5}$$

Equation 4.4.7.3

$$W = RD$$

Equation 4.4.7.4

$$L = \frac{V_H D}{V_P}$$

Equation 4.4.7.5

$$V_H = \frac{V_P D}{L} = 15(V_P)$$

where: D = depth of unit (feet), generally between 3 and 8 feet
W = width of unit (feet), usually twice the depth
L = length of unit (feet), usually fifteen times the depth
Q = design flow rate (cfs), i.e., the water quality peak flow rate, Q_{wq}
R = width to depth ratio, generally a value of 2 is recommended
V_H = allowable horizontal velocity (ft/s), maximum 0.05 ft/s
V_p = upward rise velocity of oil droplet (ft/s)

- The total depth shall be adjusted by adding 1 foot of freeboard to the depth calculated using the equations above, or equations provided by a manufacturer.
- Top baffles should extend downward by 0.85D, and bottom baffles should extend upward by 0.15D, where D is the depth of the unit (in feet). The distribution baffle should be located at a distance of 0.10L from the inlet of the unity, where L is the length of the unit (in feet).

Sizing a Coalescing Plate Interceptor (CPI) Separator:

- CPI separators require considerably less space than a conventional separator to obtain the same effluent quality. A CPI separator is able to process smaller oil droplets by collecting them upon polyurethane plates or other materials. It is recommended that the design engineer consult vendors for a plate package that will meet site and flow criteria. Manufacturers will typically identify the capacity of various standard units.
- Using V_p above, a CPI separator can be sized as follows:

Equation 4.4.7.6

$$A_P = \frac{Q}{EV_P \cos(H)}$$

where: A_p = total surface area of coalescing plates (square ft)
Q = design flow rate (cfs), i.e., the water quality peak flow rate, Q_{wq}
E = efficiency of coalescent plates (typically 0.35 to 0.95)
V_p = upward rise velocity of oil droplet (ft/s)
H = angle of coalescing plates measured from horizontal (degrees), from 0° to 60°

- A plate angle of 45° to 60° is optimal, allowing sediment to slide off the plate and settle at the bottom of the chamber. At an angle of 0°, the plates would be horizontal and sediment will settle on the plates, reducing its effectiveness.
- Select a likely plate length and width, and then compute the number of plates needed using the following equation.

Equation 4.4.7.7

$$N = \frac{A_P}{W_P L_P}$$

where: N = number of plates required
A_p = total surface area of coalescing plates (ft^2)
W_p = width of plates (ft)
L_p = length of plates (ft)

- The space between the plates is usually about 1-inch. Placing plates closer together reduces the total required volume, but may instead allow debris such as twigs, plastics or paper to clog the plates.
- Calculate the chamber geometry and volume to contain the coalescing plates. Add a minimum of 1 foot below the plates to account for sediment storage. Add 6 to 12-inches above the plates for oil accumulation. Finally, add 1 foot above the oil accumulation allowance for freeboard.
- The CPI separator shall include a forebay chamber to collect floatable debris and evenly distribute flow if more than one plate is needed. Larger units may have a device to remove and store oil from the water surface, such as a skimmer or vacuum.

Manufactured (i.e., Proprietary) Oil-Grit Separators:

- Several manufacturers of oil-grit separators are identified in the references for this section. Manufactured separators should be selected on the basis of good design, suitability for the desired pollution control goals, durabilty, ease of installation, ease of maintenance, and reliability. The products listed in the reference section and/or shown in schematics are not the only products available, nor should their presence in this manual be construed as an endorsement of these products. They are merely shown as manufactured separators that are known to operate in Tennessee.
- Manufacturers generally provide design methods, installation guidelines, and proof of effectiveness for each application where used. These structures tend to include innovative methods of providing high flow bypass. However, it is incumbent upon the landowner to carefully investigate the suitability and overall trustworthiness of each manufacturer and/or subcontractor.

C. MAINTENANCE ACCESS

- A minimum 20 foot wide maintenance right-of-way or drainage easement shall be provided for the oil-grit separator from a driveway, public or private road. The maintenance access easement shall have a maximum slope of no more than 15% and shall have a minimum unobstructed drive path having a width of 12 feet, appropriately stabilized to withstand maintenance equipment and vehicles. The right-of-way shall be located such that maintenance vehicles and equipment can access the oil-grit separator.

4.4.7.6 Design Example

Basic Data

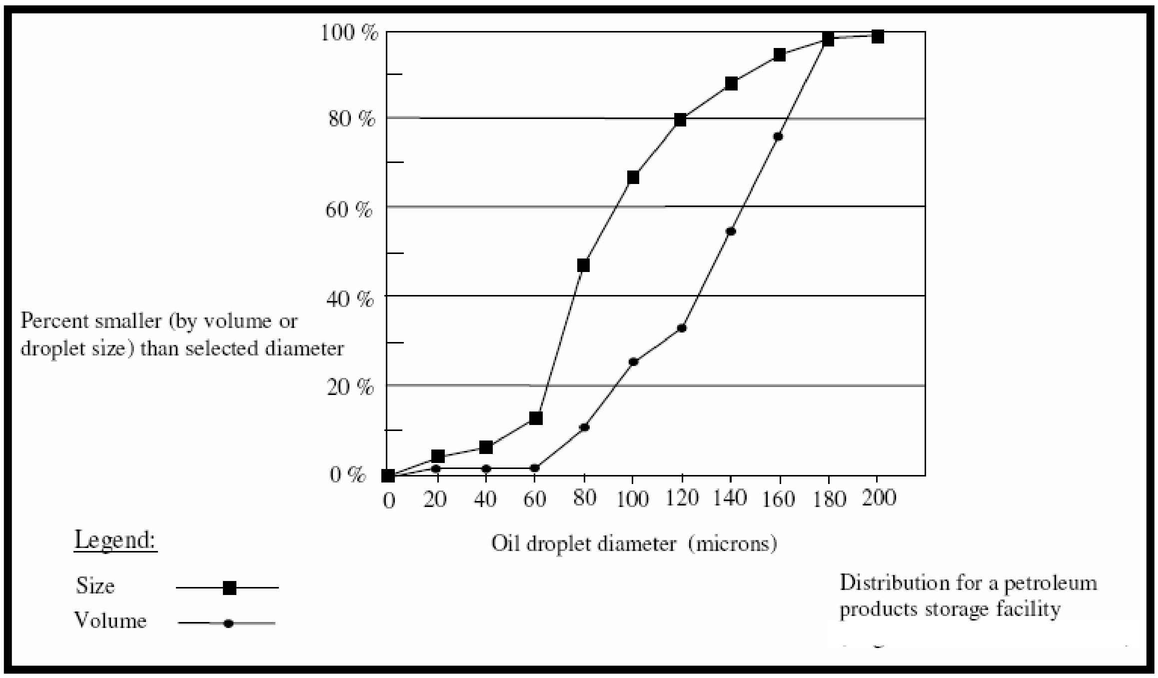
A conventional oil-grit separator unit is desired for use on a 1 acre parking lot.

- S_p = specific gravity of the petroleum droplet = 0.90
- V_p = 0.00080 ft/s for water temperature of 32°F (1 foot in 21 minutes)
- V_p = 0.00127 ft/s for water temperature of 60°F (1 foot in 13 minutes)
- Impervious percentage (I) = 90%
- Area (A) = 1 acre
- Time of concentration (t_c) = 6 minutes

Consider the effluent goal as 10 parts per million (ppm) and the design influent concentration is estimated to be 50 ppm (or equivalent to 50 mg/l), so that an oil removal efficiency of 80% is the desired target. From Figure 4-70 below, this can be achieved by removing all oil droplets with diameters of 90 microns or larger.

Figure 4-70. Typical Size and Volume Distribution of Oil Droplets

(Source: City of Knoxville, 2003)



Step 1: Calculate the Water Quality Peak Flow Rate (Q_{wq}):

(See Chapter 3 for equation information)

Compute the Runoff Peak Volume (Q_{wv}) in inches for 1.1-inch rainfall (P = 1.1):

$$Q_{wv} = PRv = 1.1Rv = 1.1(0.015 + (0.0092)(90)) = 0.93 \text{ inches}$$

Compute modified CN:

$$\begin{aligned} CN &= 1000/[10+5P+10\,Q_{wv} - 10(Q_{wv}^{\ 2}+1.25Q_{wv}P)^{½}] \\ &= 1000/[10+5(1.1)+10(0.93)-10(0.93^2+1.25(0.93)1.1)^{½}] \\ &= 98.4 \quad \text{(Use CN = 98)} \end{aligned}$$

For CN = 98 and an estimated time of concentration (t_c) of 6 minutes (0.1 hours), compute the Q_{wq} for a 1.1 inch storm.

I_a = 0.041 (from Table 3-13 in Chapter 3), therefore $I_a/P = 0.041/1.1 = 0.037$.

Using Figure 3-6 in Chapter 3, q_u can be estimated for a Type II storm as approximately 1000 csm/in or more (use 1000 csm/in because of limits in Figure 3-6).

q_u = 1000 csm/in, and therefore:

$Q_{wq} = q_u A\ Q_{wv}$ = (1000 csm/in) (1.0ac/640ac/mi2) (0.93in) = 1.45 cfs

Step 2: Size the oil-grit separator:

The allowable horizontal velocity (V_H) is:

$V_H = 15V_p = 15(0.00080) = 0.012$ ft/s

Compute the required depth (D), width (W) and length (L) of the unit (R = 2):

$D = (Q_{wq}/RV_H)^{0.5} = (1.45/(2)(0.012))^{0.5} = 7.8$ ft
$W = RD = 2(7.8) = 14.8$ ft
$L = (V_H D)/V_p = (0.012x7.8)/0.00080 = 117$ ft

The very large size separator size (8' x 16' x 111') computed above is an indication of the fact that oil and water do not separate easily. By careful design of upstream and downstream reaches, it is possible to reduce turbulent flows, drop heights, mixing or swirling stormwater runoff, and excessive velocities. The large unit sized above also indicates the importance of subbasin size to unit size. It is important to keep drainage areas small (i.e., less than 1 acre); this will keep oil-grit separators to manageable sizes.

4.4.7.7 Maintenance Requirements and Inspection Checklist

Note: Section 4.4.7.7 must be included in the Operations and Maintenance Plan that is recorded with the deed.

Regular inspection and maintenance is critical to the effective use of oil-grit (gravity) separators as stormwater best management practices. It is the responsibility of the property owner to maintain all stormwater facilities in accordance with the minimum design standards and other guidance provided in this manual. The Director has the authority to impose additional maintenance requirements where deemed necessary.

This page provides guidance on maintenance activities that are typically required for oil-grit (gravity) separators, along with a suggested frequency for each activity. Individual gravity separators may have more, or less, frequent maintenance needs, depending upon a variety of factors including the occurrence of large storm events, overly wet or dry (i.e., drought) regional hydrologic conditions, and any changes or redevelopment in the upstream land use. Each property owner shall perform the activities identified below at the frequency needed to maintain oil-grit separators properly at all times.

Inspection Activities	Suggested Schedule
• Inspect the gravity separator unit for clogging, accumulated debris, sediment, and/or oil and grease.	Regularly (at least every three months)
Maintenance Activities	**Suggested Schedule**
• Clean out sediment, oil and grease, and floatables, using catch basin cleaning equipment (vacuum pumps). Manual removal of pollutants may be necessary.	As Needed

Additional Maintenance Considerations and Requirements

- Additional maintenance requirements for a proprietary system should be obtained from the manufacturer and included in the Operations and Maintenance Plan for the site.
- Consider using a licensed commercial subcontractor, who may have special equipment and abilities to perform periodic cleanout on oil-grit separators.
- Cleanout may require the implementation of confined-space procedures and equipment as required by OSHA regulations, such as non-sparking electrical equipment, oxygen meter, flammable gas meter, etc.
- Proper disposal of oil, solids and floatables removed from the gravity separator must be ensured. Floating oil, grease and petroleum substances removed using special vacuum hoses; should be treated as hazardous waste. Sediments may also contain heavy metals or other toxic substances and should be handled as hazardous waste.
- Removal of sediment depends upon accumulation rate, available storage, watershed size, nearby construction, industrial or commercial activities upstream, etc. The sediment composition should be identified by testing prior to disposal. Some sediment may contain contaminants for which the Tennessee Department of Environment and Conservation (TDEC) requires special disposal procedures. Consult TDEC's Division of Water Pollution Control if uncertain about what the sediments contain or if it is known to contain contaminants. Generally, give special attention or sampling to sediments accumulated in industrial or manufacturing facilities, fueling centers or automotive maintenance areas, large parking areas, or other areas where pollutants are suspected to accumulate.
- There is usually uncertainty about what types of oil or petroleum products may be encountered. A significant percentage of petroleum products are attached to fine suspended solids, and therefore, are not easily removed by settling.
- Knox County encourages the use of the inspection checklist presented below for guidance in the inspection and maintenance of the oil-grit separator. Additional items should be added to the list, based on the inspection and maintenance information provided by the manufacturer of the separator unit. The Director can require the use of this checklist or other form(s) of maintenance documentation when and where deemed necessary in order to ensure the long-term proper operation of the unit. Questions regarding inspection and maintenance should be referred to the Knox County Department of Engineering and Public Works, Stormwater Management Division.

INSPECTION CHECKLIST: OIL-GRIT (OIL/WATER OR GRAVITY) SEPARATOR

Location: ____ Owner Change since last inspection? Y N

Type of Separator Unit (provide Manufacturer and Unit Name/ID if known): ______________________________

Owner Name, Address, Phone: ____

Date: __________ Time: ____________ Site conditions:____

Inspection Items	Satisfactory (S) or Unsatisfactory (U)	Comments/Corrective Action
Signs of clogging?		
Debris (trash) accumulation?		
Oil accumulation?		
Sediment accumulation?		
Standing water upstream of unit?		
Erosion downstream of unit?		
Other (describe)?		
Other (describe)?		
Other (describe)?		
Other (describe)?		
Hazards		
Have there been complaints from residents?		
Public hazards noted?		

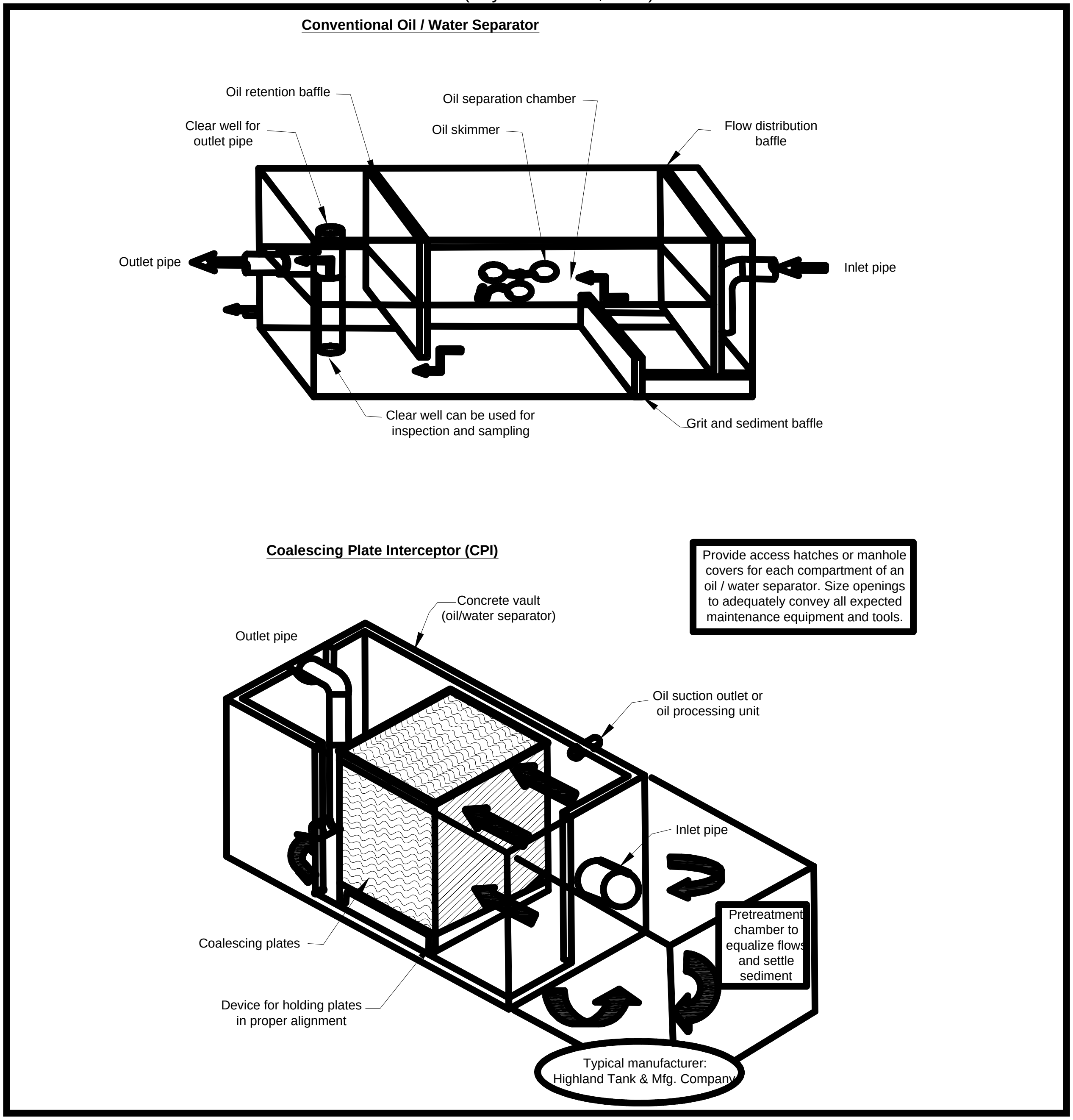
If any of the above inspection items are **UNSATISFACTORY**, list corrective actions and the corresponding completion dates below:

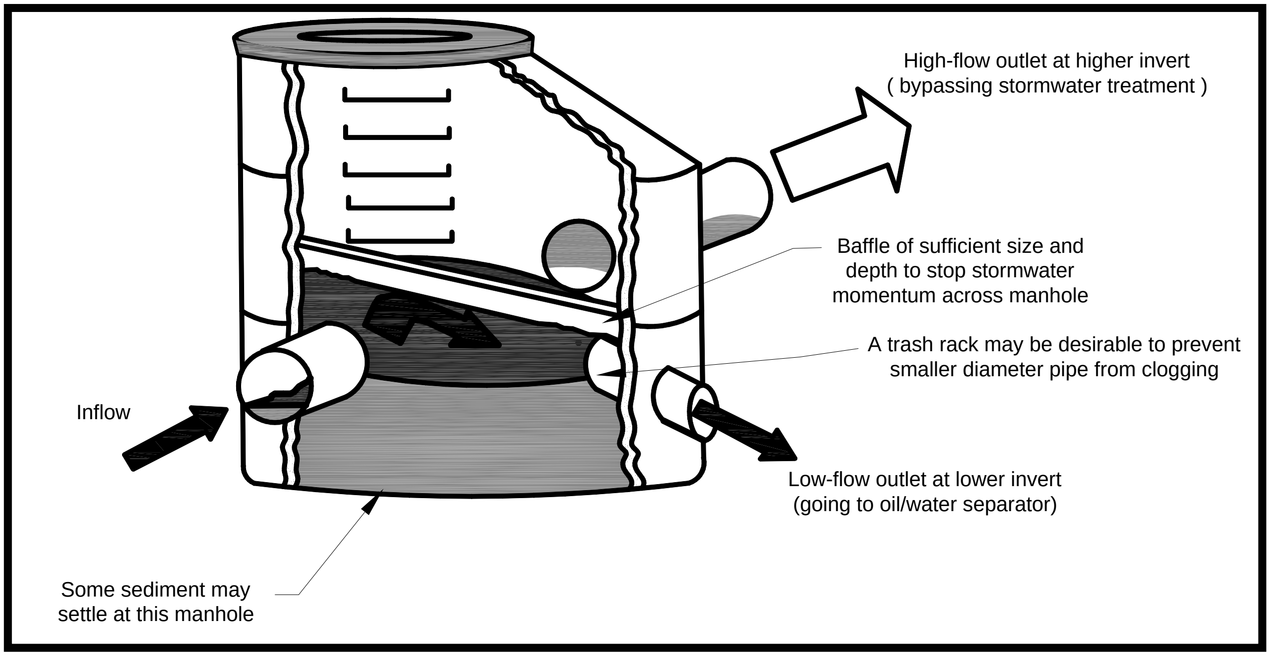
Corrective Action Needed	Due Date

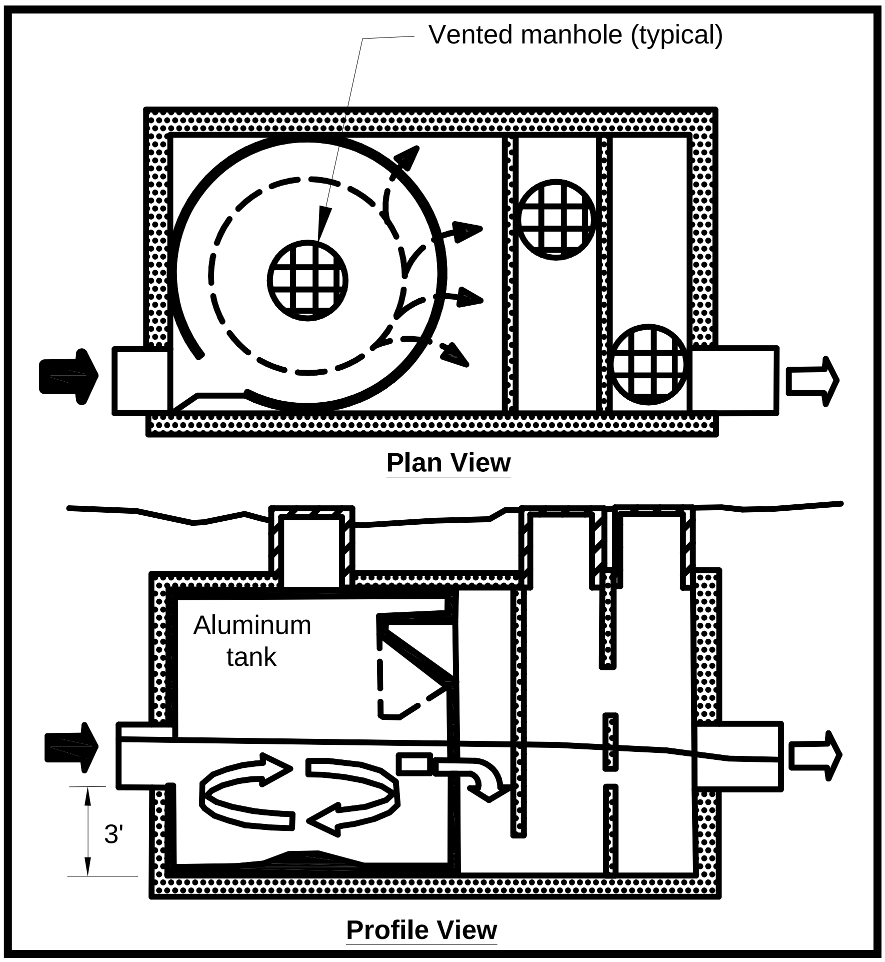
Inspector Signature:______________________________ Inspector Name (printed)______________________________

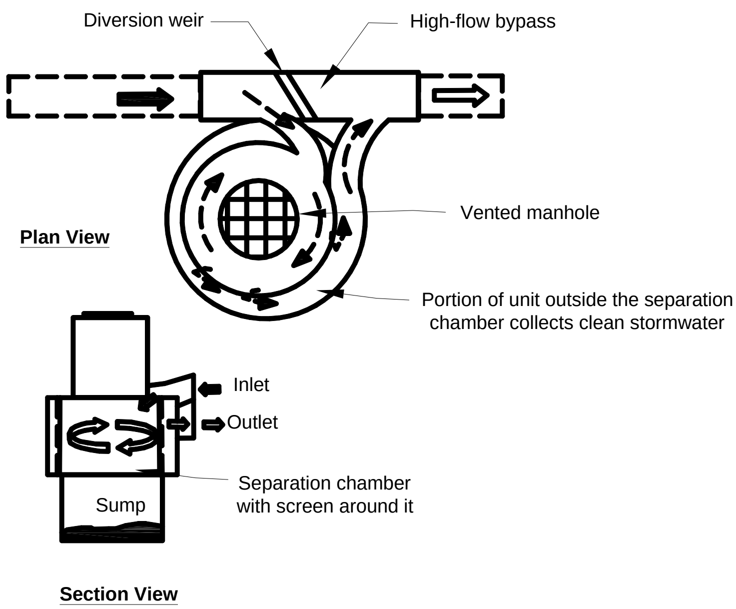
4.4.7.8 Example Schematics

Figure 4-71. Typical Oil-Grit (Oil/Water) Separators
(City of Knoxville, 2003)



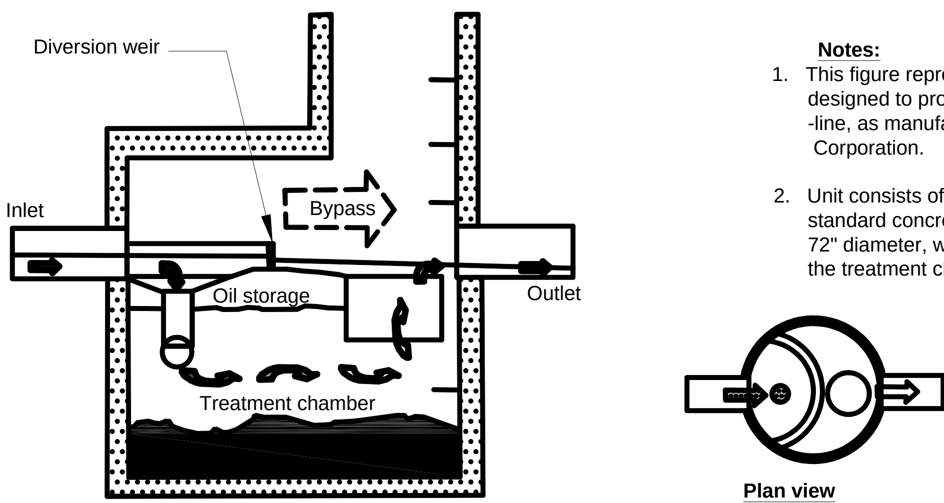

High-flow outlet at higher invert
(bypassing stormwater treatment)
Baffle of sufficient size and depth to stop stormwater momentum across manhole
A trash rack may be desirable to prevent smaller diameter pipe from clogging
Inflow
Low-flow outlet at lower invert
(going to oil/water separator)
Some sediment may settle at this manhole


Vented manhole (typical)
Plan View
Aluminum tank
3'
Profile View



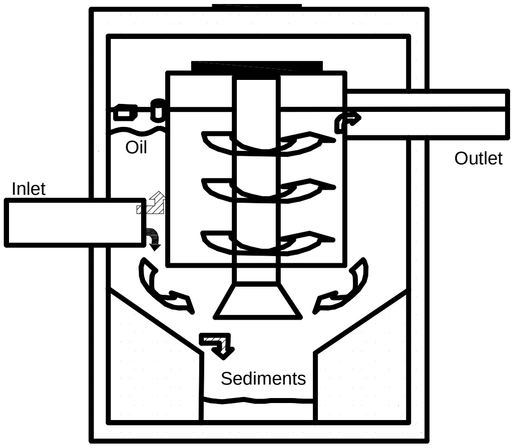
Notes:

1. This figure represents the continuous deflection stormwater treatment as manufactured by CDS Technologies. Units can also be retrofitted onto existing storm drains.
2. Units are manufactured from either fiberglass or precast concrete.
3. Manufacturer recommends the use of sorbent material within CDS separtion chamber to improve capture of oil and grease. Usage rate is typically several pounds of sorbent per acre per year.



Notes:

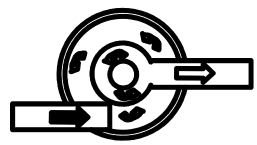
1. This figure represents a single-unit system designed to process stormwater runoff on -line, as manufactured by Stormceptor Corporation.
2. Unit consists of an insert placed into a standard concrete manhole. Basic size is 72" diameter, with larger sections used for the treatment chambers as needed.



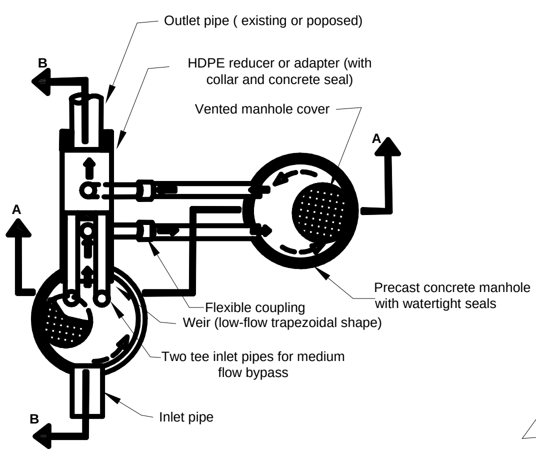
Notes:

1. This figure shows a single unit to treat stormwater runoff, manufactured by H.I.L Technologies, Inc.

2. Unit consists of polyethylene components supported by a stainless frame, inserted into a standard concrete manhole. Concrete manhole sizes vary from 4' to 10'.



Plan view

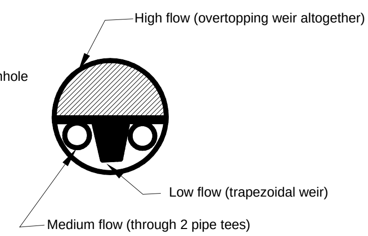


Plan View (with low flow)

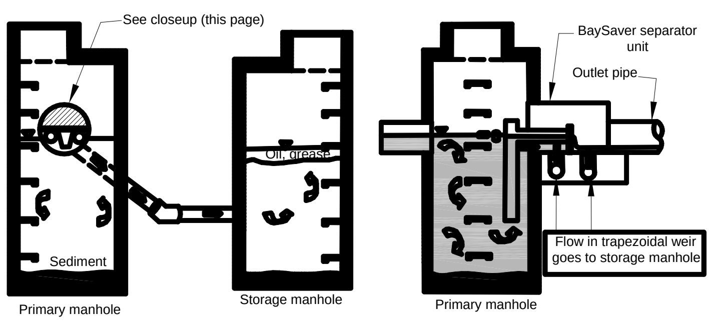
Notes:

1. This figure represents the BaySaver separation system, an off-line unit that divides flow into low, medium and high regimes.

2. The unit can be retrofitted onto existing storm drain system drain system or installed as part of a new storm drain system, using two standard precast concrete manholes.



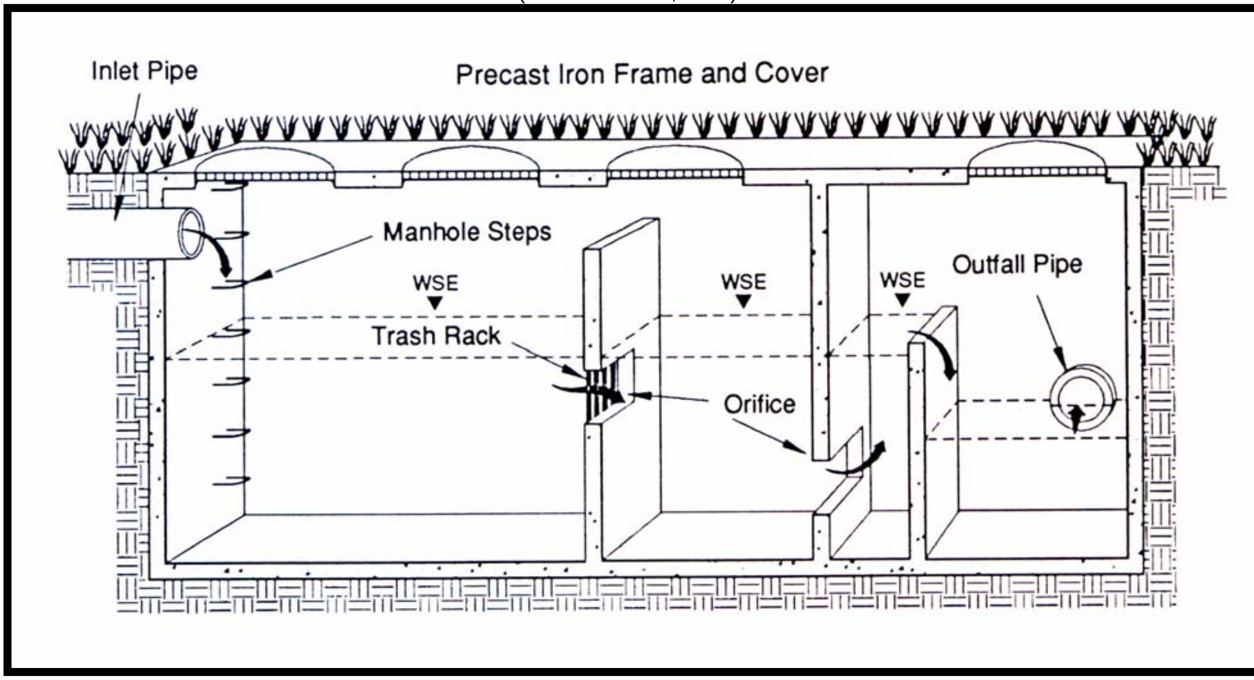
Closeup of Section A-A (flow control)



Section A-A (with low flow)

Section B-B (withmedium flow)

Figure 4-72. Schematic of an Example Gravity (Oil-Grit) Separator
(Source: NVRC, 1992)



4.4.7.9 References

AMEC. *Metropolitan Nashville and Davidson County Stormwater Management Manual, Volume 4 Best Management Practices.* 2006.

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 2 Technical Handbook.* 2001.

City of Knoxville. *Knoxville Best Management Practices Manual.* City of Knoxville Stormwater Engineering Division, March 2003.

Metropolitan Council. *Minnesota Urban Small Sites BMP Manual.* Metropolitan Council Services, St. Paul Minnesota, 2001.

4.4.7.10 Suggested Reading

California Storm Water Quality Task Force. *California Storm Water Best Management Practice Handbooks,* 1993.

4.4.7.11 Oil/Grit Separator Manufacturers

Highland Tank (CPI unit)	www.highlandtank.com
Vortechnics, Inc.	www.vortechnics.com
CDS Technologies	www.cdstech.com.au/us/
Stormceptor Corporation	www.stormceptor.com
H.I.L. Technology, Inc.	www.hil-tech.com
BaySaver, Inc.	www.baysaver.com
Aquashield, Inc.	www.squashieldinc.com
Environment 21, LLC	www.env21.com

Table of Contents

CHAPTER 5 — STORMWATER CREDITS AND BETTER SITE DESIGN PRACTICES

Chapter 5 - List of Figures

Chapter 5 - List of Tables

This page left intentionally blank

CHAPTER

5

STORMWATER CREDITS AND BETTER SITE DESIGN PRACTICES

5.1 Introduction

Stormwater management begins with the site planning and design process. Development projects can be designed to reduce their impact on watersheds when careful efforts are made to preserve natural areas, reduce impervious cover and better integrate stormwater treatment throughout the proposed development. By implementing a combination of these nonstructural approaches, collectively known as stormwater "better site design" practices, it is possible to reduce the amount of runoff and pollutants that are generated from a site and provide for some nonstructural on-site treatment and control of runoff.

The goals of better site design practices include:

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

Better site design for stormwater management includes a number of site design techniques that lay-out the natural and proposed man-made site elements in a way that reduces the stormwater impact. This is achieved primarily by reducing the amount of impervious surfaces and utilizing natural features on the site for stormwater management. The aim is to reduce the environmental impact "footprint" of the site while retaining and enhancing the owner/developer's purpose and vision for the site. Many of the better site design concepts can reduce the cost of infrastructure while maintaining, and in some cases increasing, the value of the property.

Reduction of adverse stormwater runoff impacts through the use of better site design should be the **first** consideration of the design engineer. Operationally, economically, and aesthetically, the use of better site design practices offers significant benefits over treating and controlling runoff downstream. Therefore, it is often advantageous to explore and exhaust all options for better site design before considering structural stormwater controls.

The reduction in runoff and pollutants using better site design techniques can reduce peak discharges and runoff volume that need to be conveyed and controlled on a site and, therefore, the size and cost of necessary drainage infrastructure and structural stormwater controls. In some cases, the use of better site design practices may eliminate the need for structural controls entirely.

Hence, better site design concepts can be viewed as both a water quantity and water quality management tool. The use of stormwater better site design can also have a number of other ancillary benefits including:

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

Several of the site design practices described in this section provide a calculable reduction in the volume of water required for treatment. Such a volume reduction is henceforth called a “credit”. Section 5.2 discusses each credit in detail, provides the credit criteria and calculation rules, and presents examples of their application. A more general discussion of better site design practices is presented in Section 5.3.

5.2 Water Quality Volume (WQv) Credits

5.2.1 Introduction

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

The basic premise of the credit system is to recognize the water quality benefits of certain site design practices by allowing for a reduction in the water quality treatment volume (WQv). If a developer incorporates one or more of the credited practices in the design of the site, the requirement for capture and treatment of the WQv will be reduced. Site designers are encouraged to utilize as many credits as they can on a site. Greater reductions in stormwater storage volumes can be achieved when many credits are combined (e.g., disconnecting rooftops and protecting natural conservation areas).

5.2.2 Site Planning Using Stormwater Site Design Credits

During the site planning process described in Vol. 1 Chapter 4 of this manual, there are several steps involved in site layout and design, each more clearly defining the location and function of the various components of the stormwater management system. The integration of WQv credits can be integrated with this process, as generally shown in Table 5-1.

Table 5-1. Integration of Site Design Credits with Site Development Process

Site Development Phase	Site Design Credit Activity
Initial Site Reconnaissance	• Identify and delineate natural features and potential preservation areas (natural areas, stream buffers, steep slopes, wetlands, springs and sinkholes, etc.). • Identify potential areas for better site design and WQv credits.

Site Development Phase	Site Design Credit Activity
Concept Plan	• Reduce impervious surface area through various better site design techniques. ✓ Preserve natural areas, stream buffers, steep slopes, wetlands, springs, sinkholes and other sensitive areas during site layout. ✓ Identify locations for use of vegetated channels. ✓ Look for areas to disconnect impervious surfaces. • Document the use of any WQv credits.
Design Plan	• Perform layout and design of credit areas – integrating them into treatment trains. • Ensure that WQv and channel protection volume (CPv) are satisfied. • Ensure appropriate documentation of WQv credits in accordance with credit criteria specified in this manual. • Develop maintenance requirements and documents for stormwater facilities and credited areas.
Construction	• Ensure proper protection of preservation areas and buffers. • Ensure correct final construction of areas required to achieve credits.
Final Inspection	• Prepare and submit As-Built certification, including credited areas as appropriate. • Make any necessary corrections to easements on final plats. • Ensure credit areas are identified on final plan and plat if applicable.

5.2.3 General Policies for WQv Credits

The WQv credits that are available in Knox County are listed in Table 5-2, and discussed in detail in the following sections.

Table 5-2. Summary of WQv Credits

Credit	Description
Credit 1: Natural area preservation	Undisturbed natural areas are conserved on a site, thereby retaining their pre-development hydrologic and water quality characteristics.
Credit 2: Managed area preservation	Managed areas of open space are preserved on a site, reducing total site runoff and retaining near pre-development hydrologic and water quality characteristics.
Credit 3: Stream and vegetated buffers	Stormwater runoff is treated by directing sheet flow runoff through a naturally vegetated or forested buffer as overland flow.
Credit 4: Vegetated channels	Vegetated channels are used to provide stormwater treatment.

Credit	Description
Credit 5: Impervious area disconnection	Overland flow filtration/infiltration zones are incorporated into the site design to receive runoff from rooftops and other small impervious areas.
Credit 6: Environmentally sensitive large lot neighborhood	A group of site design techniques are applied to low and very low density residential development.

General requirements and policies applicable to all the WQv credits are as follows.

1. WQv credit can only be claimed if the area or practice for which credit is requested conforms to all of the required minimum criteria and conditions stated in Section 5.2 of the Knox County Stormwater Management Manual. Credit will not be given to areas or practices that do not conform to such criteria and conditions. The intent of this policy is to avoid situations that could lead to a credit being granted without the corresponding reduction in pollution attributable to an effective better site design practice.
2. WQv credits cannot be claimed twice for an identical area of the site (e.g. claiming credit for stream buffers and disconnecting rooftops over the same site area).
3. General better site design practices and techniques performed without regard to the criteria and conditions stated herein, many of which are discussed in Section 5.3, will not be awarded WQv credits. However, it is important to remember that these practices, which reduce the overall impervious area on a site, already implicitly reduce the total amount of stormwater runoff generated by a site, and thus reduce the required WQv.

5.2.4 Credit #1: Natural Area Preservation

Description

A credit may be granted when undisturbed, natural areas are preserved on a site, thereby retaining their pre-development hydrologic and water quality characteristics. Under this credit, a site designer can subtract preserved areas from total site area when computing water quality volume requirements. The area can be used as an undisturbed buffer for sheet flow discharge for site design Credit #3, or for sheet flow of disconnected impervious areas under Credit #5. An added benefit of the use of the natural area preservation credit will be that the post-development peak discharges will be smaller for all design events, and hence other required control volumes and peak discharges (i.e., CPv, Qp_{25}, and Qf) will be reduced due to lower post-development curve numbers or rational formula "C" values.

Rule

Subtract preserved natural areas from the total site area (A) when computing the water quality volume (WQv). The percent impervious (I) is held constant when calculating WQv. Areas qualifying for this credit receive a one-hundred percent (100%) TSS reduction value in pollutant reduction computations.

Design/Implementation Criteria

1. The vegetative target for the preserved natural area is undisturbed, mature forest (i.e., trees) with woody shrubs and understory vegetation. Areas that can be characterized as an early successional (i.e., immature) forest, consisting of a combination of grasses, vines, shrubs, tree saplings and possibly even a few mature trees will qualify for the credit.
2. It is preferable that vegetation in the preserved natural area be native and non-invasive.
3. The Director may require (or allow if requested by the property owner) restoration or enhancement of preserved natural areas that do not conform to the vegetative requirements stated in item 1 above at the time of development, in order to receive credit.

4. Areas that do not conform to the vegetative target defined in item 1 can be planted with vegetation as appropriate to achieve the vegetative requirements. For these areas, a restoration plan must be submitted that is developed in accordance with the provisions and guidance presented in this manual. Guidance on the natural area restoration plan is provided below. Credit will be granted only after approval of the plan.
5. The preserved natural area cannot be disturbed during project construction without prior approval by the Director. If it is already disturbed prior to development or redevelopment, it can be restored as a natural area to receive credit.
6. The limits of disturbance on the site surrounding the preserved natural area shall be clearly shown on all construction drawings. The area must be staked in the field prior to issuance of a grading permit.
7. The preserved natural area shall be protected in perpetuity by deed restrictions, and/or a permanent preservation easement or conservation easement that is recorded with the deed.
8. If the area is not publicly owned, the easement must be held by a viable third party such as a land trust, land management company or utility. The purpose of the third party is to provide monitoring and oversight to ensure the perpetual protection of the area in accordance with the requirements of a conservation or preservation area. The organization shall:
    a. have the legal authority to accept and maintain such easements;
    b. be bona fide and in perpetual existence;
    c. have conveyance instruments that contain an appropriate provision for re-transfer in the event the organization becomes unable to carry-out it functions.
9. The easement and/or deed restriction must give Knox County the authority to enforce the easement or deed restriction terms.
10. The easement must clearly specify how the natural area vegetation shall be managed and how the boundaries of the area will be marked. (Note: managed turf areas, such as playgrounds and regularly maintained open areas, are not an acceptable form of vegetation management.)
11. The preserved natural area shall have a minimum contiguous area requirement of 10,000 square feet.

<u>Natural Area Restoration Plan Requirements</u>

When vegetative restoration or enhancement of a preserved natural area is desired or required to receive the WQv credit, the Director may approve, or require, a natural area restoration plan. The plan must be submitted and approved by the Director in order to receive the credit. Natural area restoration plans must include the following information:

1. A drawing or plan that shows the location of the preserved natural area in relation to the existing or planned development. The plan should display the area proposed for restoration and the limits of disturbance, grubbing and grading (if permitted);
2. Best management practices for erosion prevention and sediment control during the vegetation restoration or enhancement (if not already submitted with a stormwater management plan for site development or redevelopment);
3. Verbiage and/or drawings indicating the species and density of proposed vegetation. Mortality must be accounted for in the initial planted density of all vegetation;
4. Verbiage, guidance, and/or drawings indicating the planting practices that will be utilized;
5. A maintenance and monitoring plan for one full growing season; and,
6. An implementation schedule.

Example 5-1 presents an example calculation of the preserved natural area credit. The example utilizes the WQv equation presented in Volume 2 Chapter 3 of this manual.

Example 5-1. Nature Area Preservation Credit Calculation
Proposed site: residential subdivision
Site area = 38 acres
Preserved natural area = 7 acres
Site impervious area = 13.8 acres
Rainfall depth for 85% storm event = 1.1 inches

Credit Rule: Subtract the preserved natural area from the total site area when computing WQv. The percent impervious (I) is held constant.

I = site percent imperviousness = (13.8 acres)/(38 acres) * 100% = 36.3%

Runoff coefficient = Rv = 0.015 + 0.0092*(I) = 0.015 + 0.0092*(36.3%) = 0.35
Drainage area = total site area – preserved natural area = 38 acres – 7acres = 31 acres

WQv (acre-ft) = P*Rv*A/12
Before credit:
WQv = (1.1 in)*(0.35)*(38 acres)/12 = 1.22 ac-ft
With credit:
WQv = (1.1 in)*(0.35)*(31 acres)/12 = 0.99 ac-ft

The preserved natural area credit resulted in a 19% reduction in the WQv required for the site. The area will also receive a 100% TSS reduction value in the TSS calculation.

5.2.5 Credit #2: Managed Open Space Preservation

Description
A credit may be granted when areas of managed open space, typically reserved for passive recreation or agricultural practices, are preserved on a site. Under this credit, a site designer can subtract the preserved areas from total site area when computing water quality volume requirements. The area can be used for sheet flow of disconnected impervious areas under Credit #5. An added benefit of the use of the managed preservation area credit will be that the post-development peak discharges will be smaller for all design events, and hence water quantity control volumes (CPv, Qp_{25}, and Qp_{100}) will be reduced due to lower post-development curve numbers or rational formula "C" values.

Rule
Subtract preserved managed areas from the total site area (A) when computing the water quality volume (WQv). The percent impervious (I) is held constant when calculating WQv. No TSS reduction is received through this credit.

Design/Implementation Criteria
1. The managed area must have a passive recreational or agricultural land use.
2. The minimum vegetative target for managed areas that do not have an agricultural land use is grass covering 100% of the site. If the preserved area will have a land-use that requires vegetation that differs from this target, a legally-binding Management Plan must accompany the easement or deed restrictions. The Management Plan must show the vegetation management practices and other BMPs that will be utilized for maintenance of the area.
3. Impervious surfaces are prohibited in the credited area.
4. Practices that have the potential to cause discharges of pollutants and sediment off-site are prohibited in the credited area.
5. If the area is to be actively farmed, the Management Plan must be approved and monitored by the Natural Resource Conservation Service.

6. The preserved area shall be protected in perpetuity by deed restrictions, and/or a permanent preservation easement or conservation easement that is recorded with the deed.
7. If the area is not publicly owned, the easement must be held by a viable third party such as a land trust, land management company or utility. The purpose of the third party is to provide monitoring and oversight to ensure the perpetual protection of the area in accordance with the requirements of a conservation or preservation area. The organization shall:
   a. have the legal authority to accept and maintain such easements;
   b. be bona fide and in perpetual existence;
   c. have conveyance instruments that contain an appropriate provision for re-transfer in the event the organization becomes unable to carry-out it functions.
8. The easement and/or deed restriction must give Knox County the authority to enforce the easement or deed restriction terms.
9. The easement must clearly specify how vegetation shall be managed and how the boundaries of the area will be marked.
10. The managed area shall have a minimum contiguous area requirement of 10,000 square feet.

Example 5-2. Managed Open Space Preservation Credit Calculation

Proposed site: residential subdivision
Site area = 38 acres
Preserved managed area = 7 acres
Site impervious area = 13.8 acres
Rainfall depth for 85% storm event = 1.1 inches

Credit Rule:: Subtract the preserved managed area from the total site area when computing WQv. The percent impervious (I) is held constant.

I = site percent imperviousness = (13.8 acres)/(38 acres) * 100% = 36.3%

Runoff coefficient = Rv = 0.015 + 0.0092*(I) = 0.015 + 0.0092*(36.3%) = 0.35

Drainage area = total site area – preserved managed area = 38 acres–7acres = 31 acres

WQv = P*Rv*A/12
Before credit:
WQv = (1.1 in)*(0.35)*(38 acres)/12 = 1.22 ac-ft
With credit:
WQv = (1.1 in)*(0.35)*(31 acres)/12 = 0.99 ac-ft

The preserved managed area credit resulted in a 19% reduction in the WQv required for the site.

5.2.6 Credit #3: Stream and Vegetated Buffers

Description

This credit may be granted when stormwater runoff is effectively treated by a stream buffer or other vegetated buffer. Effective treatment constitutes treating runoff as overland sheet flow through an appropriately vegetated and forested buffer. Under this credit, a site designer can subtract areas draining via overland flow to the buffer from total site area when computing water quality volume requirements. The area draining to the buffer and the buffer itself qualify for credit. In addition, the volume of runoff draining to the buffer can be subtracted from the channel protection volume (CPv).

Rule

Subtract areas draining via overland flow to the buffer from total site area when computing the water quality volume (WQv). The Rv value derived from impervious percentage is held constant when calculating WQv. For stream buffers with a grassed outer zone that have been established and managed in accordance with the provisions of Chapter 6 of this manual, the buffer and areas draining to the buffer qualify for the credit and receive an eighty percent (80%) TSS reduction credit. For buffers that are at least fifty (50) feet in width and are comprised entirely of undisturbed forest vegetation, the buffer itself can qualify for credit #1 as a natural conservation area or if preserved as managed open space can qualify for credit #2, while the areas draining to the buffer qualify for the buffer credit and receive an eighty percent (80%) TSS reduction value.

Design/Implementation Criteria

1. This credit is not applicable if the impervious area disconnection credit (Credit 5) is already being applied to the same area.
2. The portion of the buffer that is utilized for stormwater treatment must have a minimum buffer width of fifty (50) feet. If buffer averaging is utilized, portions of the buffer that have a width less than fifty (50) feet are not eligible to receive this credit. Increases in buffer width and/or widths of forest vegetation are strongly encouraged.
3. At a minimum, buffers must be designed and managed (in perpetuity) in accordance with the requirements and policies for water quality buffers presented in the Knox County Stormwater Management Ordinance and in Volume 2 Chapter 6 of this manual.
4. Undisturbed, forested buffers that are at least fifty (50) feet wide can qualify for credit #1 as a natural conservation area. Areas preserved as managed open space can qualify for credit #2.
5. Stormwater runoff must enter the buffer as overland sheet flow. A level spreader must be utilized if sheet flow does not occur naturally, or if the average contributing slope is greater than 3%.
6. The minimum contributing length of sheet flow shall be ten (10) feet.
7. The maximum contributing length of sheet flow shall be two-hundred twenty-five (225) feet, with a maximum of one hundred fifty (150) feet for pervious surfaces, and seventy-five (75) feet for impervious surfaces.
8. The design of the buffer treatment system must use appropriate methods for conveying flows above the annual recurrence (1-yr storm) event. Flows encountered for storms greater than the 1-yr can be piped beneath the buffer to the stream, so long as proper outfall protection is employed and channel protection and peak flow control criteria have been met.

Example 5-3. Stream and Vegetated Buffer Credit Calculation

Residential Subdivision
Site Area = 38 acres
Impervious Area = 13.8 acres
Area of undisturbed forested buffer having a 50' width = 2 acres
Area Draining to Buffer = 5 acres
Rainfall depth for 85% storm event = 1.1 inches

Credit Rule: Subtract the area draining to the buffer when computing WQv. Since this is an undisturbed buffer area at least 50' in width, the area of the buffer can be considered a natural area under credit #1, and therefore can also be subtracted from the total site area when computing WQv. The percent impervious (I) is held constant.

I = site percent imperviousness = (13.8 acres)/(38 acres) * 100% = 36.3%
Runoff coefficient = Rv = 0.015 + 0.0092*(I) = 0.015 + 0.0092*(36.3%) = 0.35
Drainage area = Total site area – buffer area – area draining to buffer
Drainage area = 38 acres – 2 acre – 5 acres = 31 acres

Before credit:
WQv = P*Rv*A/12
WQv = (1.1)(0.35)(38 acres)/12 = 1.22 ac-ft
With credit:
WQv = (1.1)(0.35)(31 acres)/12 = 0.99 ac-ft

The buffer credit resulted in a 19% reduction in the WQv required for the site. The buffer area will also receive a 100% TSS reduction value in the TSS calculation. The area draining to the buffer will receive an 80% TSS reduction value.

5.2.7 Credit #4: Use of Vegetated Channels

This credit may be granted when vegetated (grass) channels are used for water quality treatment. Site designers will be able to subtract the areas draining to a grass channel and the channel area itself from the total site area when computing water quality volume requirements. A vegetated channel may be able to fully meet the water quality volume requirements for certain kinds of low density residential development (see Credit #6). An added benefit will be that the post-development peak discharges will likely be lower due to a longer time of concentration for the site.

Rule
Subtract the areas draining to a vegetated (grass) channel from total site area when computing the water quality volume (WQv). The percent impervious (I) shall be held constant when calculating WQv. Areas qualifying for this credit receive an eighty percent (80%) TSS reduction value.

Design/Implementation Criteria

1. The vegetated channels must be located within a drainage, water quality or preservation easement.
2. The credit shall only be applied to residential land uses that have a maximum density of three (3) dwelling units per acre for the total development.
3. The maximum flow velocity in the channel for the WQv design storm shall be less than or equal to one (1.0) feet per second.
4. The minimum travel time for the water quality storm in the channel shall be five (5) minutes.
5. The bottom width shall be a maximum of six (6) feet. If a larger channel is needed, use of a compound cross-section (i.e., a benched channel) is required.
6. The side slopes shall be 3:1 (horizontal:vertical) or flatter.
7. This credit will not be granted if engineered grass channels are being used in order to meet the 80% TSS removal goal for WQv treatment.

Example 5-4. Vegetated Channels Credit Calculation
Residential Subdivision
Site Area = 38 acres
Impervious Area = 13.8 acres
Area draining to vegetated channels = 12.5 acres
Rainfall depth for 85% storm event = 1.1 inches

Credit Rule: Subtract the area draining to the vegetated channels when computing WQv. The percent impervious (I) is held constant.

I = site percent imperviousness = (13.8 acres)/(38 acres) * 100% = 36.3%
Runoff coefficient = Rv = 0.015 + 0.0092*(I) = 0.015 + 0.0092*(36.3%) = 0.35

Drainage area = Total site area – area draining to vegetated channels
Drainage area = 38 acres – 12.5 acres = 25.5acres
Before credit:
WQv = P*Rv*A/12
WQv = (1.1)(0.35)(38 acres)/12 = 1.22 ac-ft
With credit:
WQv = (1.1)(0.35)(25.5 acres)/12 = 0.82 ac-ft

The credit resulted in a 33% reduction in the WQv required for the site. The area draining to the channel will also receive an 80% TSS reduction value in the TSS calculation.

5.2.8 Credit #5: Impervious Area Disconnection

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

Rule

If impervious areas are adequately disconnected, they can be deducted from the total site area when computing the water quality volume (WQv). The percent impervious area (I) shall be held constant when calculating WQv. Areas qualifying for this credit receive an 80% TSS reduction value in pollutant reduction computations.

Design/Implementation Criteria

1. For those areas draining directly to a buffer, either the impervious area disconnection credit or the stream buffer credit can be used, but not both.
2. Relatively permeable soils, soil amendments, or placed topsoil (hydrologic soil groups A and B) should be present in the pervious areas that receive discharges from disconnected impervious areas.
3. Impervious area disconnection credits will not be given for areas that have, or will have after development, the land uses listed below:
    a. Developments or facilities that include on-site sewage disposal and treatment systems (i.e., septic systems), raised septic systems, subsurface discharges from a wastewater treatment plant, or land application of biosolids or animal waste;
    b. Landfills (demolition landfills, permitted landfills, closed-in-place landfills);
    c. Junkyards;
    d. Commercial or industrial facilities that store and/or service motor vehicles;
    e. Commercial greenhouses or landscape supply facilities;
    f. Agricultural facilities, farms, feedlots, and confined animal feed operations;
    g. Animal care facilities, kennels, and commercial/business developments or facilities that provide short-term or long-term care of animals; or,
    h. Other land uses deemed by the Director to have the potential to generate higher than normal pollutant loadings.

4. The maximum contributing impervious flow path length shall be 75 feet.
5. Downspouts shall be at least 10 feet away from the nearest accessible impervious surface (including off site impervious areas) to discourage "re-connections" or flow concentration along a paved edge. In general, downspout discharges should be directed away from foundations of habitable structures.
6. The disconnection shall drain continuously through a vegetated channel, swale, or filter strip to the property line or to a structural stormwater control.
7. The length of the "disconnection" shall be equal to or greater than the contributing length.
8. The entire vegetative disconnection shall be on a slope less than or equal to 3 percent.
9. The impervious surface area to any one point discharge location shall not exceed 5,000 square feet.
10. There must be a note in the final plat that indicates the locations of the disconnected downspouts, and states that such downspouts "shall remain disconnected from the impervious surfaces and shall forever be discharged onto pervious surfaces".

Example 5-5. Impervious Area Disconnection Credit Calculation
Office Building
Site Area = 3.0 acres
Impervious Area = 1.9 acres
Disconnected impervious area = 0.5 acres
Rainfall depth for 85% storm event = 1.1 inches

Credit Rule: Subtract the area disconnected impervious areas when computing WQv. The percent impervious (I) is held constant.

I = site percent imperviousness = (1.9 acres)/(3 acres) * 100% = 63.3%
Runoff coefficient = Rv = 0.015 + 0.0092*(I) = 0.015 + 0.0092*(63.3%) = 0.60

Drainage area = Total site area – disconnected impervious area
Drainage area = 3 acres – 0.5 acres = 2.5 acres

Before credit:
WQv = P*Rv*A/12
WQv = (1.1)(0.60)(3 acres)/12 = 0.17 ac-ft
With credit:
WQv = (1.1)(0.60)(2.5 acres)/12 = 0.14 ac-ft
The credit resulted in an 18% reduction in the WQv required for the site. The disconnected impervious areas will also receive an 80% TSS reduction value in the TSS calculation.

5.2.9 Credit #6: Environmentally Sensitive Large Lot Neighborhoods

This credit is targeted toward large lot residential developments that implement a number of Better Site Design practices to reduce stormwater discharges from the development as a whole. This credit may be granted when a group of environmental site design techniques are applied to low and very low density residential development (e.g., 1 dwelling unit per acre [du/ac] or lower). The credit can eliminate the need for structural stormwater controls to treat water quality volume requirements. This credit will likely have limited application.

Rule

The requirement for structural controls necessary to achieve the water quality volume treatment design criteria shall be waived.

Design/Implementation Criteria

1. There are two development density options:
   a. The maximum density of the residential development shall be one (1) dwelling unit per acre, and shall have a total impervious cover footprint (including streets, sidewalks, and other impervious infrastructure) no greater than twelve percent (12%); or,
   b. The maximum density of the residential development shall be one (1) dwelling unit per two (2) acres, and shall have a total impervious cover footprint (including streets, sidewalks and other impervious infrastructure) no greater than fifteen percent (15%).
2. To verify the amount of development in an impervious area, the developer must provide one of the following with the stormwater management plan:
   a. The impervious footprints for roadways and lots, and the calculated percent imperviousness for the site. This option requires the developer to know the housing footprints and the general locations of each house on each lot so that driveway areas can be measured.
   b. The impervious footprint for roadways, the maximum expected impervious footprint per lot, and the calculated percent imperviousness for the development. The developer simply needs to know the size range of housing .to be constructed in the development and to justify the per lot imperviousness based upon the housing size range anticipated.
3. Restrictive covenants, easements or other legal instrument must be used to limit imperviousness for each lot or development, or to set open space aside as perpetually undeveloped. The legal instrument must be conveyed to each property within the development, and must transfer accordingly with any subsequent property transfers.
4. Grass channels shall be used to convey runoff versus curb and gutter.
5. Impervious areas shall be disconnected, in accordance with the criteria set forth in Credit #5, to the maximum extent practicable.

5.3 Better Site Design Practices

5.3.1 Overview

The remainder of this chapter has been developed to provide the developer and/or site designer detailed guidance on the use of a number of better site design practices. While the better site design practices presented herein are not required by Knox County, they are strongly encouraged. A number of these practices can be utilized to gain WQv credits, as discussed previously in this chapter. However, beyond the credits, there is strong incentive to utilize better site design practices that is provided by the use of the WQv approach. That is – the goal of many better site design practices is the reduction of imperviousness which, in the WQv approach, will reduce the volume of stormwater runoff required for treatment.

5.3.2 Incorporating Better Site Design Practices into Site Design Process

Better site design should be done in unison with the design and layout of stormwater infrastructure in attaining the stormwater management goals and criteria discussed in Chapters 1 and 3 of Volume 2 of this manual. Figure 5-1 illustrates the four major steps of the site design process.

The first step in stormwater better site design involves identifying significant natural features and resources on a site such as undisturbed forest areas, stream buffers and steep slopes that should be preserved to retain some of the original hydrologic function of the site. Next, the site layout is designed such that these conservation areas are preserved and the impact of the development is minimized. A number of techniques can then be used to reduce the overall imperviousness of the development site. Finally, natural features and conservation areas can be utilized to serve stormwater quantity and quality management purposes.

Figure 5-1. Stormwater Better Site Design Process

Identify Natural Features and Resources – Delineate Site Conservation Areas

↓

Design Site Layout to Preserve Conservation Areas and Minimize Stormwater Impacts

↓

Use Various Techniques to Reduce Impervious Cover in the Site Design

↓

Utilize Natural Features and Conservation Areas to Manage Stormwater Quantity and Quality

5.3.3 Discussion of Better Site Design Practices

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

- **Conservation of Natural Features and Resources**
  - Preserve undisturbed natural areas
  - Preserve riparian (i.e., stream) buffers
  - Avoid development and grading in floodplains
  - Avoid steep slopes
  - Minimize development on porous or erodible soils

- **Lower Impact Site Design Techniques**
  - Fit design to the terrain
  - Locate development in less sensitive areas
  - Reduce limits of clearing and grading
  - Utilize open space development
  - Consider creative development design

- **Reduction of Impervious Cover**
  - Reduce roadway lengths and widths
  - Reduce building footprints
  - Reduce the parking footprint
  - Reduce setbacks and frontages
  - Use fewer or alternative cul-de-sacs
  - Create parking lot stormwater "islands"

- **Utilization of Natural Features for Stormwater Management**
  - Use buffers and undisturbed areas
  - Use natural drainageways instead of storm sewers
  - Use vegetated swales instead of curb and gutter
  - Drain rooftop runoff to pervious areas

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

5.3.4 Conservation of Natural Features and Resources

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

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

Some of the ways used to conserve natural features and resources described over the next several pages include the following methods, which correspond to the fact sheets that follow:

1. Preserve undisturbed natural areas

2. Preserve riparian buffers

3. Avoid floodplains

4. Avoid steep slopes

5. Minimize development on porous or erodible soils

Delineation of natural features is typically done very early in the development process, through an analysis of the features and resources on the site. From this site analysis, the preservation and protection of natural features can be integrated into the vision prior to development of the concept plan.

This space left intentionally blank.

Better Site Design Practice #1:
Preserve Undisturbed Natural Areas

Conservation of Natural Features and Resources

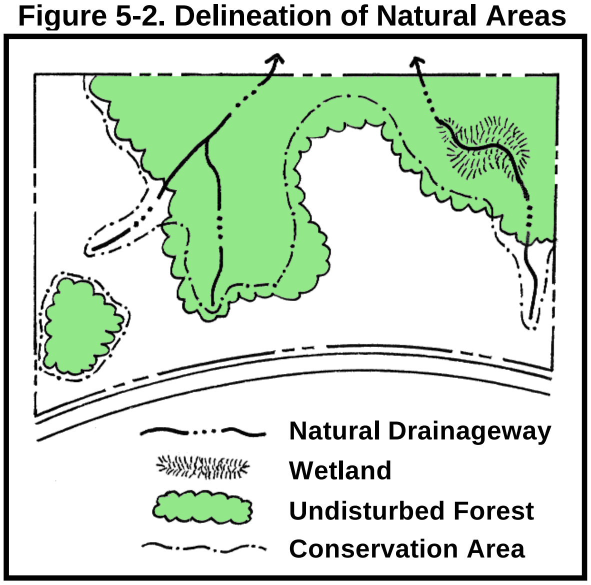
Description: Important natural features and areas such as undisturbed forested and vegetated areas, natural drainageways, steep slopes, stream corridors, wetlands, erodible soils and other important site features should be delineated and placed into conservation areas.

KEY BENEFITS	USING THIS PRACTICE
• Preserves a portion of the site's natural hydrology prior to development. • Can be used as filtering and infiltration zones for stormwater runoff from developed areas. • Preserves some of the site's natural character and aesthetic features. • May increase the value of the developed property. • A stormwater site design credit can be taken if the area complies with the criteria listed in section 5.2.	☑ Delineate natural areas before performing site layout and design. ☑ Ensure that conservation areas and native vegetation are protected in an *undisturbed state* throughout construction and occupancy.

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

Natural conservation areas are typically identified through a site analysis using maps and aerial/satellite photography, or by conducting a site visit. These areas should be delineated before any site design, clearing or construction begins. When done before the concept plan phase, the planned conservation areas can be used to guide the layout of the site. Figure 5-2 shows a site map with undisturbed natural areas delineated.

Figure 5-2. Delineation of Natural Areas



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

Once established, natural areas should be managed by a responsible party that is able to maintain the areas in a natural state in perpetuity. Typically, conservation areas are protected by legally enforceable deed restrictions, conservation easements, and maintenance agreements. If the natural area is utilized for WQv credits, Knox County requires the use of a responsible third party to achieve the perpetual preservation of the area. Refer to Credit #1 for more information on the Natural Area Preservation Credit.

Better Site Design Practice #2:
Preserve Riparian Buffers

Conservation of Natural Features and Resources

Description: Preserve naturally vegetated buffers along perennial streams, rivers, lakes, and wetlands.

KEY BENEFITS	USING THIS PRACTICE
• Can be used as nonstructural stormwater filtering and infiltration zones. • Keeps structures out of the floodplain and provides a right-of-way for large flood events. • Helps to preserve riparian ecosystems and habitats. • A stormwater site design credit can be taken if it fulfills the criteria listed in section 5.2.	☑ Delineate and preserve naturally vegetated riparian buffers. ☑ Ensure that buffers are established, maintained and protected in accordance with guidance set forth in Volume 2 Chapter 6.

As discussed previously in Chapter 6, buffers are a special type of natural conservation area located along a stream, wetland or pond/lake where development is restricted or prohibited. In Knox County, water quality buffers are required on all waterbodies that can be designated as "community waters" as defined in this manual and in the Knox County Stormwater Management Ordinance. Such buffers must be established, maintained and protected in accordance with the provisions of the ordinance and Chapter 6 of this manual. This section provides some general information about buffers.

Figure 5-3. Riparian Stream Buffer

The primary function of buffers is to protect and physically separate a waterbody from future disturbance or encroachment. If properly designed, a buffer can provide stormwater management functions, can act as a right-of-way during floods, and can sustain the integrity of stream ecosystems and habitats. An example of a riparian stream buffer is shown in Figure 5-3. Knox County's buffer requirements include provisions for a minimum fifty (50) foot dual-zone, forested and grassed buffer along streams, and a minimum twenty-five (25) foot one-zone forested or grass buffer around wetlands and ponds/lakes, respectively.

In general, forested zones of buffers should be maintained and reforestation should be encouraged where no wooded buffer exists. Chapter 6 of this manual contains provisions and guidance for buffer reforestation (herein called "enhancement"). Proper enhancement of forested areas should include all layers of the forest plant community, including understory shrubs and groundcover, not just trees. Native vegetation is required in forested zones. Impervious areas are prohibited in all areas of the buffer.

Better Site Design Practice #3:
Avoid Floodplains

Conservation of Natural Features and Resources

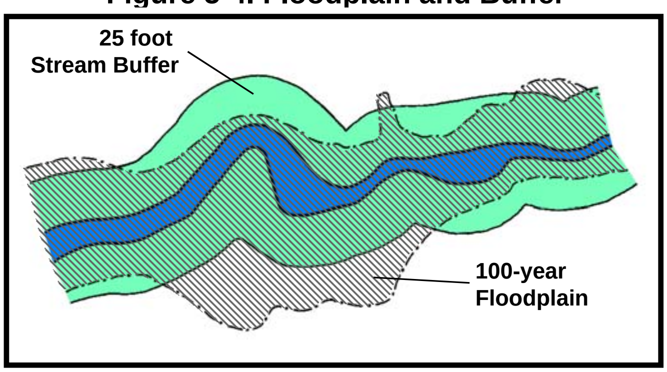
Description: Development in floodplain areas should be avoided to minimize potential property damages and safety risks, and to allow the natural stream corridor to accommodate flood flows.

KEY BENEFITS	USING THIS PRACTICE
• Provides a natural right-of-way and temporary storage for large flood events. • Keeps people and structures out of potentially flooded areas. • Helps to preserve riparian ecosystems and habitats. • Can be combined with riparian buffer protection to create linear greenways.	☑ Be aware of Knox County's floodplain development requirements. ☑ Do not encroach into designated floodplain areas. Maps are available from the Knox County Department of Engineering and Public Works.

Floodplains are the low-lying flat lands that border streams and rivers. When a stream reaches its capacity and overflows its channel after storm events, the floodplain provides the natural storage and conveyance areas for these excess flows. When left in a naturally vegetated state with forest, shrubs and other woody vegetation, floodplains can provide a reduction in discharge velocities and peak discharge rates during flood events. Floodplains also play an important role in reducing sedimentation and filtering runoff, and provide habitat for both aquatic and terrestrial life. Development in floodplain areas can reduce the ability of the floodplain to convey stormwater, potentially causing safety problems or significant damage to the site in question, as well as to both upstream and downstream properties. As a participant community of the National Flood Insurance Program (NFIP), Knox County regulates the use of floodplain areas to minimize the risk to human life as well as to avoid flood damage to structures and property.

As such, all floodplain areas should be avoided on a development site. Ideally, the entire 100-year future floodplain should be avoided for clearing or building activities, and should be preserved in a natural undisturbed state where possible. At a minimum, developers should also ensure that their site design complies with Knox County's floodplain requirements. Among other requirements, Knox County prohibits construction fill that alters the conveyance and storage capacity of the natural floodplain; fill is prohibited in the flood fringe one-half the linear distance between the floodway line and the 100-year floodplain line. Floodplain maps and flood elevation profiles can be obtained from the Knox County Department of Engineering and Public Works.

Figure 5-4. Floodplain and Buffer



Floodplain protection is complementary to riparian buffer preservation. Both practices preserve stream corridors in a natural state and allow for the protection of vegetation and habitat. Depending on the site topography, 100-year floodplain boundaries may lie inside or outside of a preserved riparian buffer corridor, as shown in Figure 5-4.

Better Site Design Practice #4:
Avoid Steep Slopes

Conservation of Natural Features and Resources

Description: Development on steep slopes should be avoided due to the potential for soil erosion and increased sediment loading to nearby streams. Excessive grading and flattening of hills and ridges should be minimized.

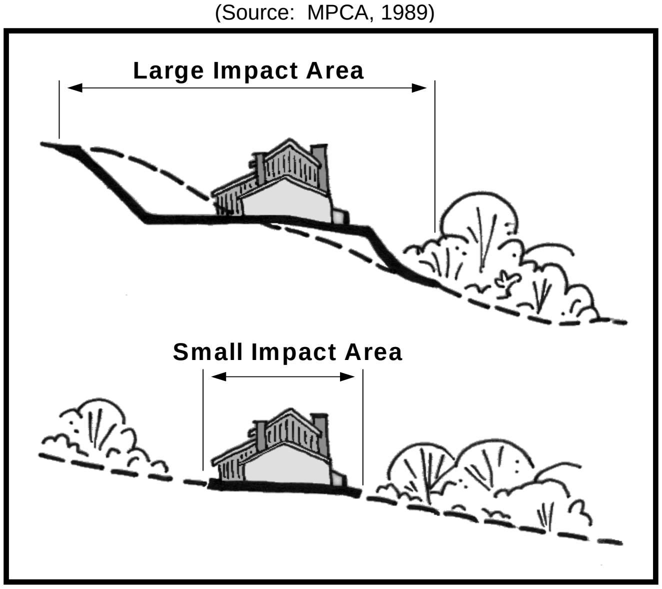
KEY BENEFITS	USING THIS PRACTICE
• Preserving steep slopes helps to prevent soil erosion and degradation of stormwater runoff. • Steep slopes can be kept in an undisturbed natural condition to help stabilize hillsides and soils. • Building on flatter areas will reduce the need for cut-and-fill and grading.	☑ Avoid development on steep slope areas, especially those with a grade of 15% or greater. ☑ Fit the development into the natural terrain, as opposed to modifying the terrain to fit the development. ☑ Minimize grading and flattening of hills and ridges.

Development in steep slope areas has the potential to cause excessive soil erosion and stormwater runoff during and after construction. Past studies by the Soil Conservation Service (now Natural Resource Conservation Service) and others have shown that soil erosion is significantly increased on slopes of 15 percent or greater. In addition, greater areas of soil and land area are disturbed when development is located on steep slopes as compared to flatter slopes. This is demonstrated in Figure 5-5.

Therefore, development on slopes with a grade of 15 percent or greater should be avoided to limit soil loss, erosion, excessive stormwater runoff, and the degradation of surface water. Excessive grading should be avoided on all slopes, as should the flattening of hills and ridges. Steep slopes should be kept in an undisturbed natural condition to help stabilize hillsides and soils.

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

Figure 5-5. Impacts of Development on Slopes
(Source: MPCA, 1989)



Better Site Design Practice #5:
Minimize Development on Porous and Erodible Soils

Conservation of Natural Features and Resources

Description: Porous soils such as sand and gravels provide an opportunity for groundwater recharge of stormwater runoff and should be preserved as a potential stormwater management option. Unstable or easily erodible soils should be avoided because they are more likely to erode.

KEY BENEFITS	USING THIS PRACTICE
• Areas with highly permeable soils can be used for infiltration of stormwater runoff. WQv credits can be taken if the area complies with the criteria listed in section 5.2, potentially for a Natural Area Preservation credit or Impervious Area Disconnection credit. • Avoiding high erodible or unstable soils can prevent erosion and sedimentation problems and water quality degradation.	☑ Use soil surveys to determine site soil types. ☑ Leave areas of porous or highly erodible soils as undisturbed preservation areas.

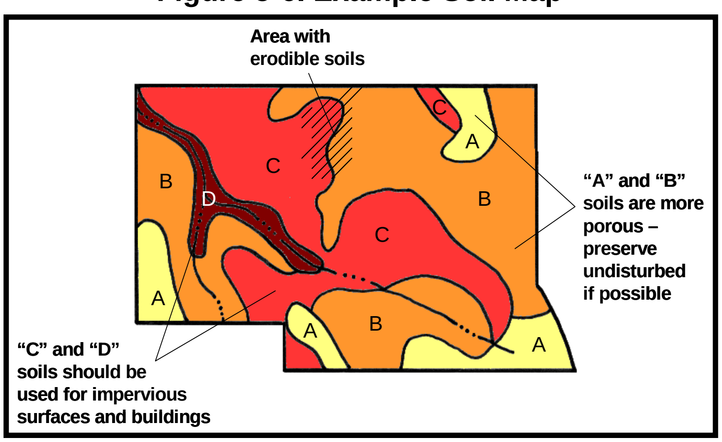
Infiltration of stormwater into the soil reduces both the volume and peak discharge of runoff from a given rainfall event, and also provides for water quality treatment and groundwater recharge. Soils with maximum permeability (hydrologic soil group A and B soils, such as sands and sandy loams) allow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

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

Soils on a development site should be mapped in order to preserve areas with porous soils, and to identify those areas with unstable or erodible soils as shown in Figure 5-6. The Knox County soil surveys can provide a considerable amount of information relating to all relevant aspects of soils. Soil surveys are available from the local National Resource Conservation Service office.

General soil types should be delineated on concept site plans to guide site layout and the placement of buildings and impervious surfaces.

Figure 5-6. Example Soil Map



5.3.5 Low Impact Site Design Techniques

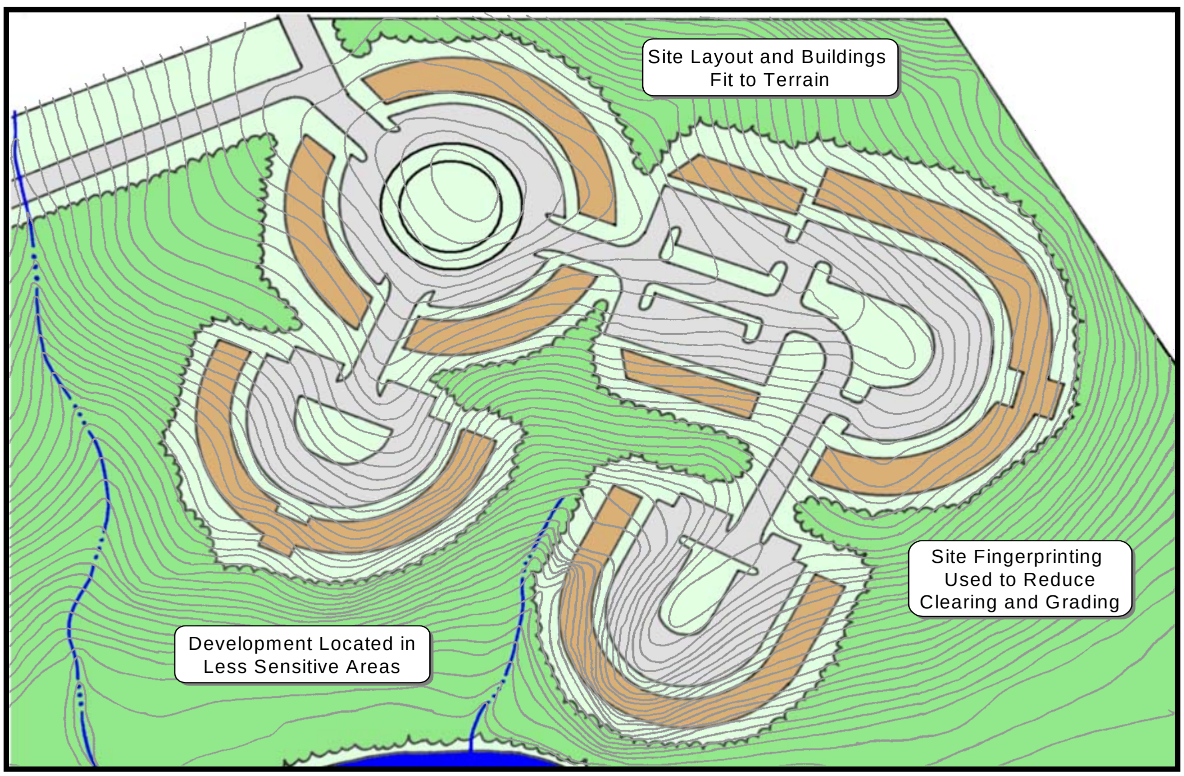
After a site analysis has been performed and conservation areas have been delineated, there are numerous opportunities in the site design and layout phase to reduce both water quantity and quality impacts of stormwater runoff. These primarily deal with the location and configuration of impervious surfaces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

- # 6. Fit the Design to the Terrain
- # 7. Locate Development in Less Sensitive Areas
- # 8. Reduce Limits of Clearing and Grading
- # 9. Utilize Open Space Development
- # 10. Consider Creative Development Design

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

Figure 5-7 shows a development that has utilized several low impact site design techniques in its overall layout and design.

Figure 5-7. Example of Low Impact Site Design Techniques



Better Site Design Practice #6:
Fit Design to the Terrain

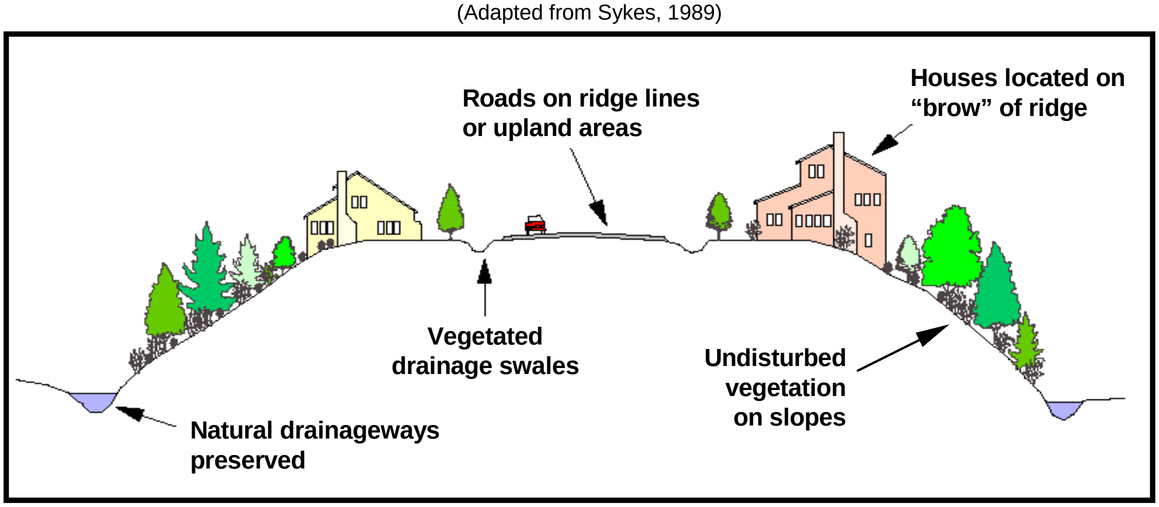
Low Impact Site Design Techniques

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

KEY BENEFITS	USING THIS PRACTICE
• Helps to preserve the natural hydrology and drainageways of a site. • Reduces the need for grading and land disturbance. • Provides a framework for site design and layout.	☑ Develop roadway patterns to fit the site terrain. ☑ Locate buildings and impervious surfaces away from steep slopes, drainageways, and floodplains.

All site layouts should be designed to conform with or "fit" the natural landforms and topography of a site. This helps to preserve the natural hydrology and drainageways on the site, as well as reduces the need for grading and disturbance of vegetation and soils. Figure 5-8 illustrates the placement of roads and homes in a residential development.

Figure 5-8. Preserving the Natural Topography of the Site
(Adapted from Sykes, 1989)



Roadway patterns on a site should match the terrain. In rolling or hilly terrain, streets should be designed to follow natural contours to reduce clearing and grading. Street hierarchies with local streets branching from collectors in short loops, and cul-de-sacs located along ridgelines help to prevent the crossing of streams and drainageways as shown in Figure 5-9. In flatter areas, a traditional grid pattern of streets or "fluid" grids which bend and may be interrupted by natural drainageways may be more appropriate (see Figure 5-10). In either case, buildings and impervious surfaces should be kept off of steep slopes, away from natural drainageways, and out of floodplains and other lower lying areas. In addition, the major axis of buildings should be oriented parallel to existing contours.

Figure 5-9. Example Subdivision Design for Hilly or Steep Terrain

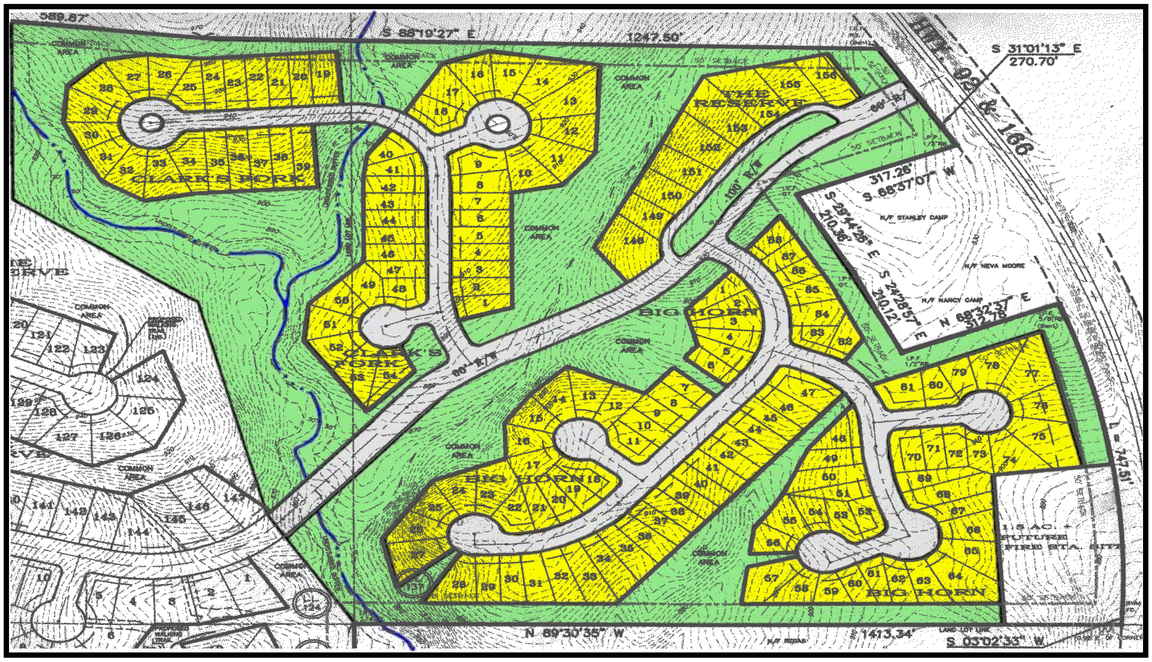
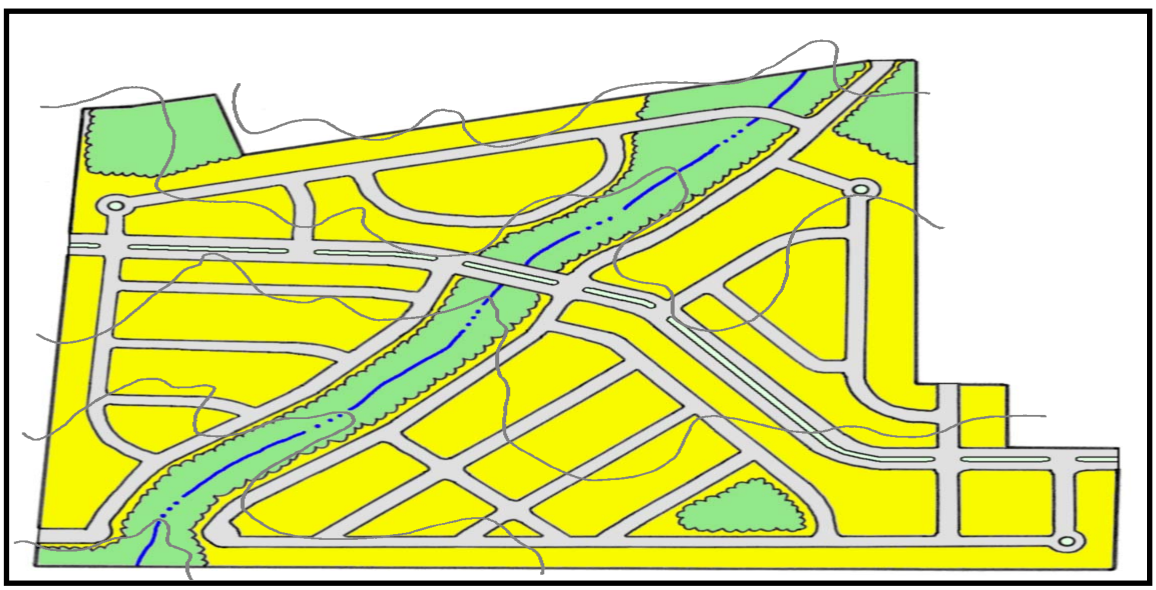


Figure 5-10. Example Subdivision Design for Flat Terrain



Better Site Design Practice #7:

Locate Development in Less Sensitive Areas

**Low Impact
Site Design Techniques**

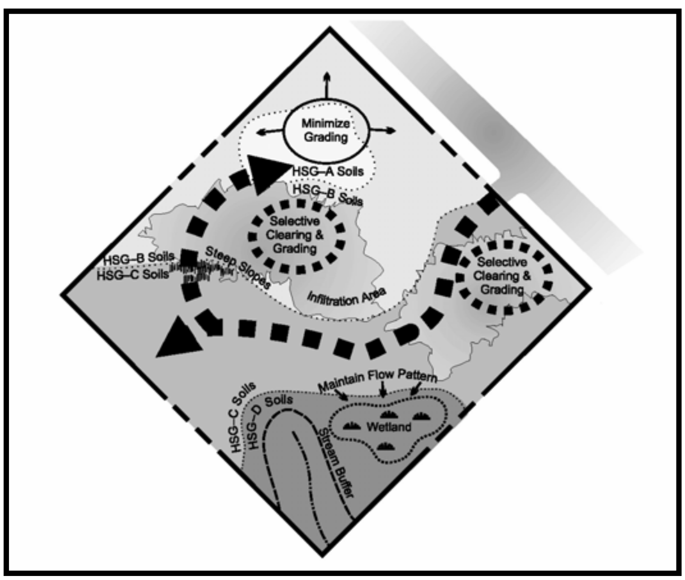
Description: To minimize the hydrologic impacts on the existing site land cover, the area of development should be located in areas of the site that are less sensitive to disturbance or have a lower value in terms of hydrologic function.

KEY BENEFITS	USING THIS PRACTICE
• Helps to preserve the natural hydrology and drainageways of a site. • Makes the most efficient use of natural site features for preventing and mitigating stormwater impacts. • Provides a framework for site design and layout.	☑ Lay out the site design to minimize the hydrologic impact of structures and impervious surfaces.

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

- Locate buildings and impervious surfaces away from stream corridors, wetlands and natural drainageways. Use buffers to preserve and protect riparian areas and corridors.
- Areas of the site with porous soils should be left in an undisturbed condition and/or used as stormwater runoff infiltration zones. Buildings and impervious surfaces should be located in areas with less permeable soils.

Figure 5-11. Guiding Development to Less Sensitive Areas of a Site
(Source: Prince George's County, MD, 1999)



- Avoid construction on areas with steep slopes or unstable soils.
- Minimize the clearing of areas with dense tree canopy or thick vegetation. Ideally, preserve these as natural conservation areas.
- Ensure that natural drainageways and flow paths are preserved, where possible. Avoid the filling or grading of natural depressions and ponding areas.

Figure 5-11 shows a development site where the natural features have been mapped in order to delineate the sensitive areas. Through careful site planning, sensitive areas can be set aside as natural open space areas In many cases, such areas can be used as buffer spaces between land uses on the site or between adjacent sites.

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

Low Impact Site Design Techniques

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

KEY BENEFITS	USING THIS PRACTICE
• Preserves more undisturbed natural areas on a development site. • Techniques can be used to help protect natural conservation areas and other site features.	☑ Establish the limits of disturbance for all development activities. ☑ Use site footprinting to minimize clearing and land disturbance.

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

- Establishing a limit of disturbance (LOD) based on maximum disturbance zone radii/lengths. These maximum distances should reflect reasonable construction techniques and equipment needs together with the physical situation of the development site such as slopes or soils. LOD distances may vary by type of development, size of lot or site, and by the specific development feature involved.
- Using site "footprinting" which maps all of the limits of disturbance to identify the smallest possible land area on a site which requires clearing or land disturbance. Examples of site footprinting are illustrated in Figures 5-12 and 5-13.
- Fitting the site design to the terrain.
- Using special procedures and equipment which reduce land disturbance.

Figure 5-12. Example of Limits of Clearing

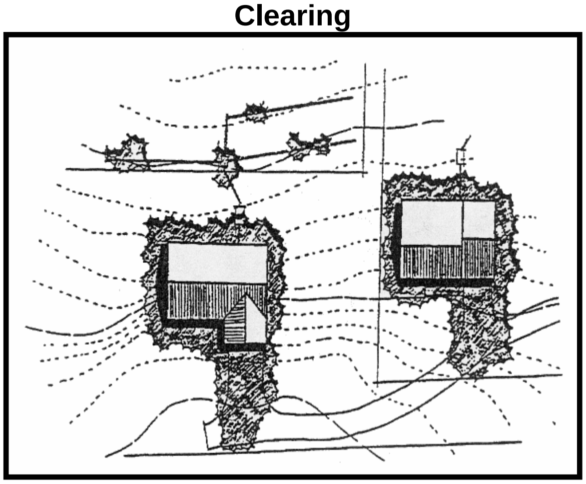
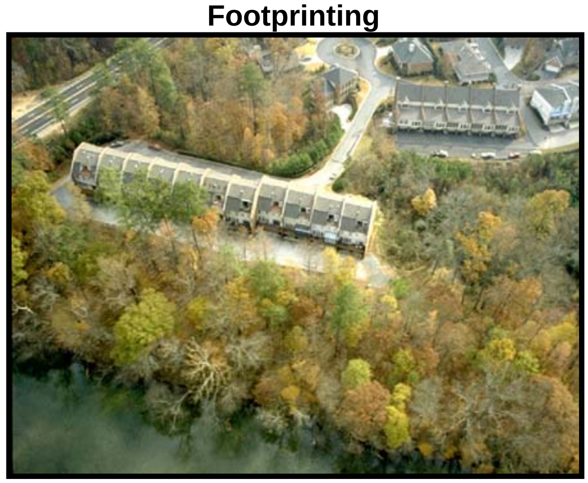


Figure 5-13. Example of Site Footprinting



Better Site Design Practice #9:
Utilize Open Space Development

Low Impact
Site Design Techniques

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

KEY BENEFITS	USING THIS PRACTICE
• Preserves conservation areas. • Can be used to preserve natural hydrology and drainagways. • Can be used to help protect natural conservation areas and other site features. • Reduces the need for grading and land disturbance. • Reduces infrastructure needs and overall development costs.	☑ Use a site design which concentrates development and preserves open space and natural areas of the site.

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

Open space developments have many benefits compared with conventional commercial developments or residential subdivisions: they can reduce impervious cover, stormwater pollution, construction costs, and the need for grading and landscaping, while providing for the conservation of natural areas. Figures 5-14 presents an example of the concept of open space site design for a residential area. Figure 5-15 provides an example of an existing open space development.

Along with reduced imperviousness, open space designs provide a host of other environmental benefits that most conventional designs lack. These developments reduce potential pressure to encroach on conservation and buffer areas because enough open space is usually reserved to accommodate these protection areas. Since less land is cleared during the construction process, alteration of the natural hydrology and the potential for soil erosion are also greatly diminished. Perhaps most importantly, open space design reserves 25 to 50 percent of the development site in conservation areas that would not otherwise be protected.

Open space developments can also be significantly less expensive to build than conventional projects. Most of the cost savings are due to reduced infrastructure cost for roads and stormwater management controls and conveyances. While open space developments are frequently less expensive to build, developers find that these properties often command higher prices than those in more conventional developments. Several studies estimate that residential properties in open space developments garner premiums that are higher than conventional subdivisions and moreover, sell or lease at an increased rate.

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

Figure 5-14. Open Space Subdivision Site Design Example

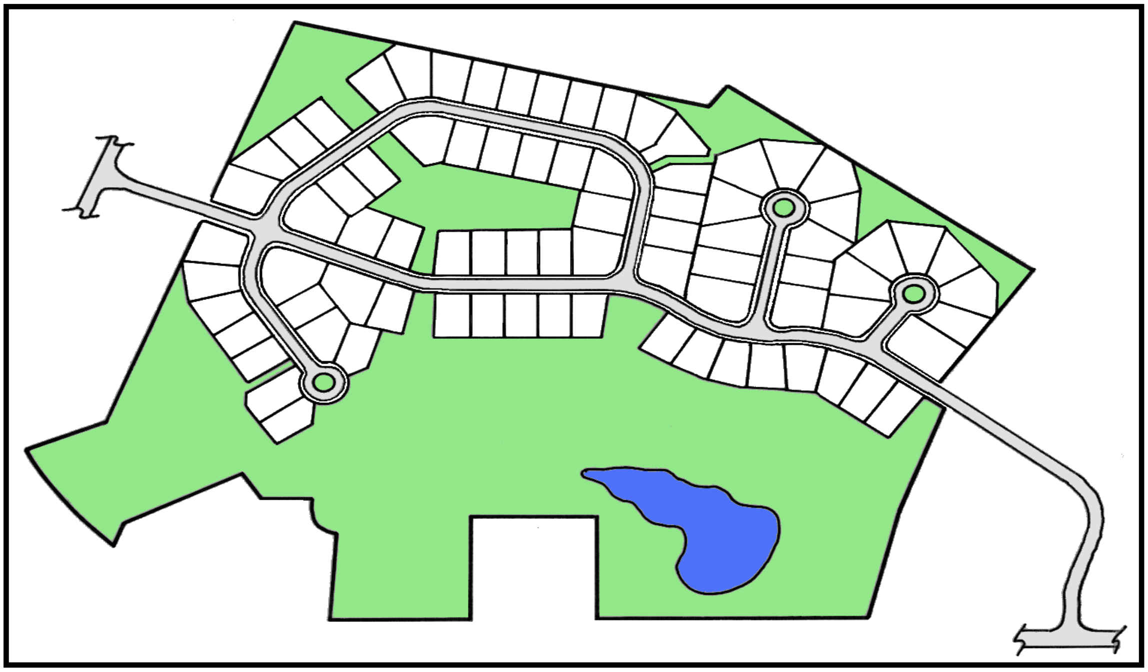


Figure 5-15. Aerial View of an Open Space Subdivision

Better Site Design Practice #10:
Consider Creative Development Design

Low Impact
Site Design Techniques

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

KEY BENEFITS	USING THIS PRACTICE
• Allows flexibility to developers to implement creative site designs which include stormwater better site design practices. • May be useful for implementing an open space development.	☑ Check with the Metropolitan Planning Commission to determine the type and nature of deviations allowed and other criteria for receiving PUD approval.

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

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

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

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

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

5.3.6 Reduction of Impervious Cover

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

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

- # 11. Reduce Roadway Lengths and Widths
- # 12. Reduce Building Footprints
- # 13. Reduce the Parking Footprint
- # 14. Reduce Setbacks and Frontages
- # 15. Use Fewer or Alternative Cul-de-Sacs
- # 16. Create Parking Lot Stormwater Islands

Figure 5-16. shows an example of a residential subdivision that employed several of these principles to reduce the overall imperviousness of the development. The next several pages cover these methods in more detail.

Figure 5-16. Examples (clockwise from upper left): (a) Cul-de-sac with Landscaped Island; (b) Narrower Residential Street; (c) Landscape Median in Roadway; and (d) "Green" Parking Lot with Landscaped Islands

Better Site Design Practice #11:
Reduce Roadway Lengths and Widths

Reduction of Impervious Cover

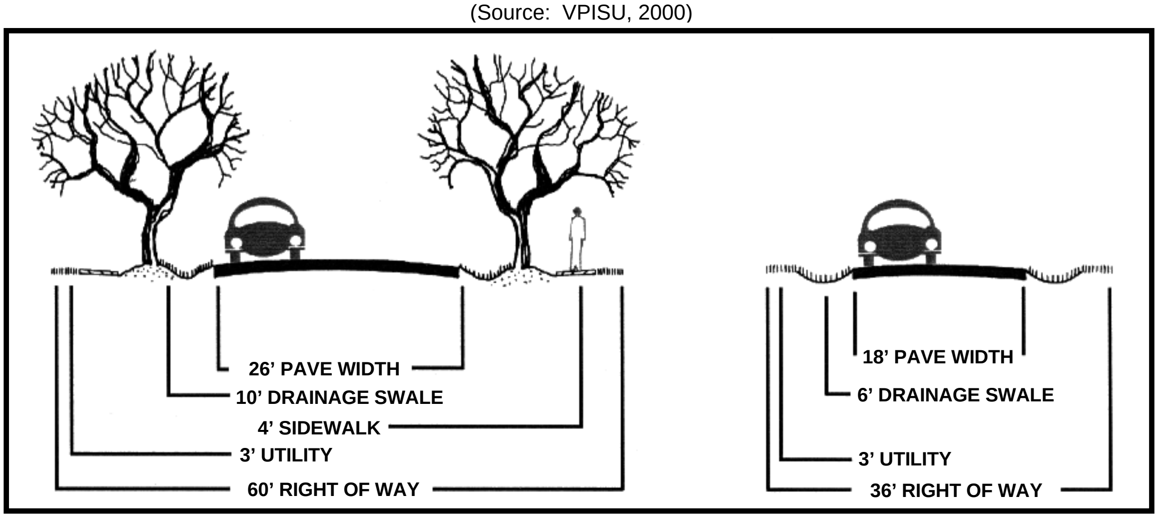
Description: Roadway lengths and widths should be minimized on a development site where possible to reduce overall imperviousness.

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

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

In addition, residential streets and private streets within commercial and other development should be designed for the minimum required pavement width needed to support travel lanes, on-street parking, and emergency access. Figure 5-17 shows one option for narrower street designs. Many times on-street parking can be reduced to one lane or eliminated on local access roads with less than 200 ADT on cul-de-sac streets and 400 ADT on two-way loops. One-way single-lane loop roads are another way to reduce the width of lower traffic streets.

Figure 5-17. Potential Design Options for Narrower Roadway Widths
(Source: VPISU, 2000)



Better Site Design Practice #12:
Reduce Building Footprints

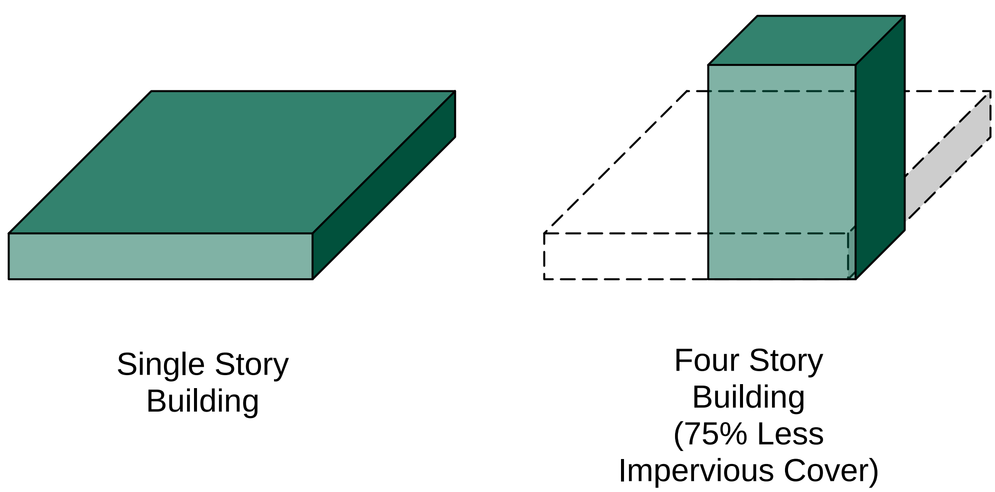
Reduction of Impervious Cover

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

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

In order to reduce the imperviousness associated with the footprint and rooftops of buildings and other structures, alternative and/or vertical (taller) building designs should be considered. Consolidate functions and buildings, as required, or segment facilities to reduce the footprint of individual structures. Figure 5-18 shows the reduction in impervious footprint from a taller building as opposed to a single story building.

Figure 5-18. Impervious Cover Comparison



Better Site Design Practice #13:
Reduce the Parking Footprint

Reduction of Impervious Cover

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

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

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

Many parking lot designs result in far more spaces than actually required. This problem is exacerbated by a common practice of setting parking ratios to accommodate the highest hourly parking during the peak season. By determining average parking demand instead, a lower maximum number of parking spaces can be set to accommodate most of the demand. Table 5-3 provides examples of conventional parking requirements and compares them to average parking demand.

Table 5-3. Conventional Minimum Parking Ratios
(Source: www.stormwatercenter.net)

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

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

Structured parking decks are one method to significantly reduce the overall parking footprint by minimizing surface parking. Figure 5-19 shows a parking deck used for a commercial development.

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

Figure 5-19. Structured Parking at an Office Park

Utilizing alternative surfaces such as porous pavers or porous concrete is an effective way to reduce the amount of runoff generated by parking lots. They can replace conventional asphalt or concrete in both new developments and redevelopment projects. Figure 5-20 is an example of porous pavers used at an overflow lot. Such pavers can also capture and treat runoff from other site areas. However, porous pavement surfaces generally require precise installation and more maintenance than conventional asphalt or concrete. For more specific information using these alternative surfaces, see Section 4.3 of this manual.

Figure 5-20. Grass Paver Surface Used for Parking

Better Site Design Practice #14:
Reduce Setbacks and Frontages

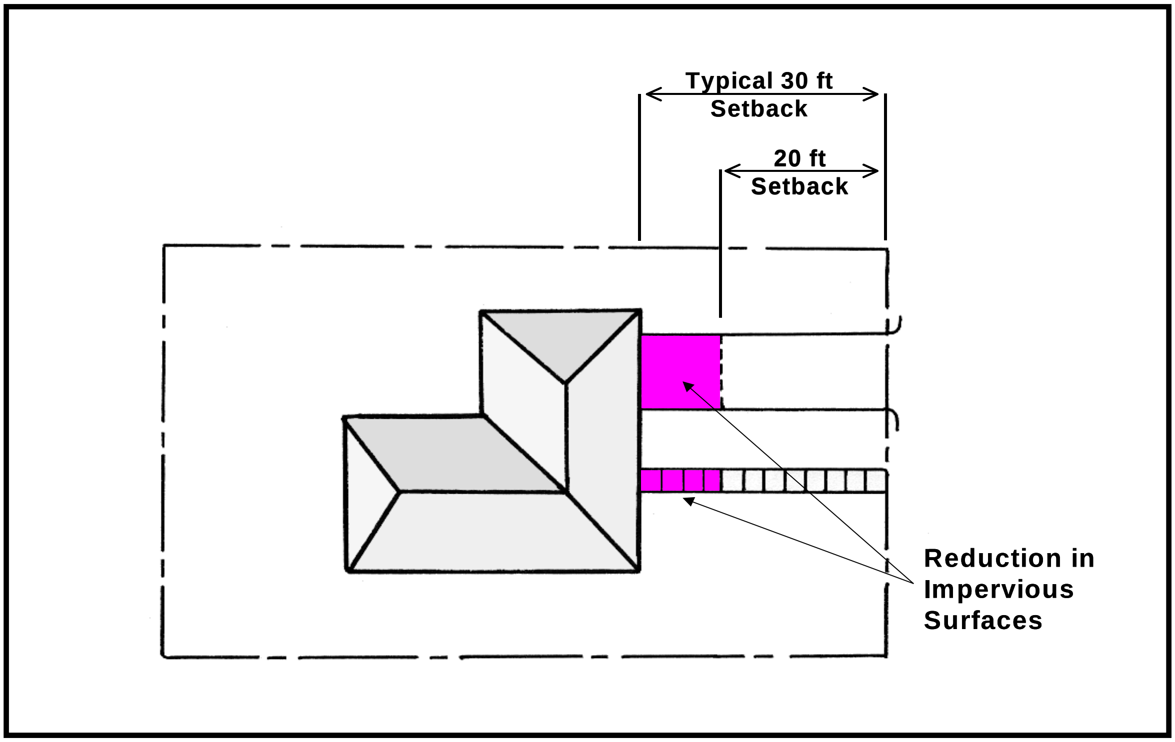
Reduction of Impervious Cover

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

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

Building and home setbacks should be shortened to reduce the amount of impervious cover from driveways and entry walks. A setback of 20 feet is more than sufficient to allow a car to park in a driveway without encroaching into the public right of way, and reduces driveway and walk pavement by more than 30 percent compared with a setback of 30 feet (see Figure 5-21).

Figure 5-21. Reduced Impervious Cover by Using Smaller Setbacks
(Adapted from: MPCA, 1989)

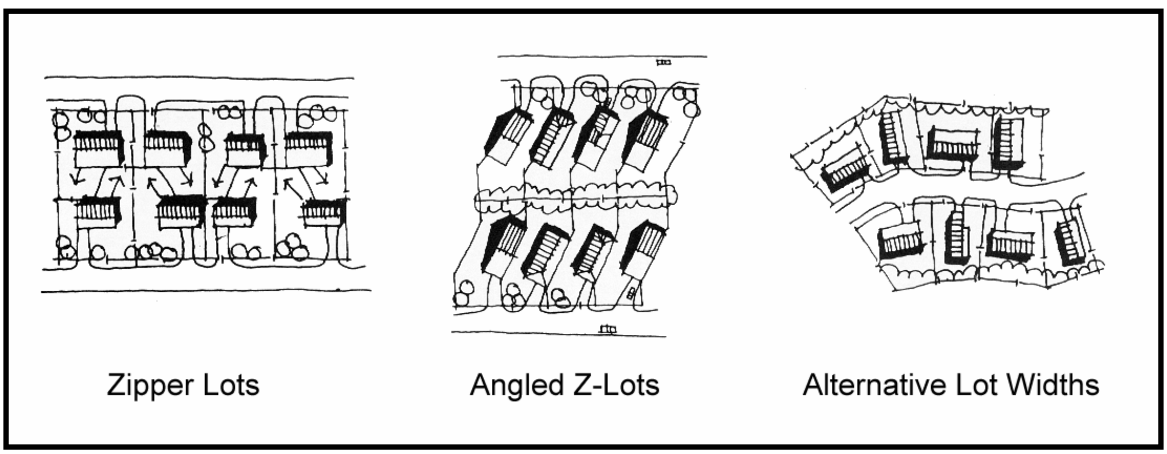


Further, reducing side yard setbacks and using narrower frontages can reduce total street length, which is especially important in cluster and open space designs. Figure 5-22 shows residential examples of reduced front and side yard setbacks and narrow frontages.

Flexible lot shapes and setback and frontage distances allow site designers to create attractive and unique lots that provide homeowners with enough space while allowing for the preservation of natural areas in a residential subdivision. Figure 5-23 illustrates various nontraditional lot designs.

Figure 5-22. Examples of Reduced Frontages and Side Yard

Figure 5-23. Nontraditional Lot Designs
(Source: ULI, 1992)



Better Site Design Practice #15:
Use Fewer or Alternative Cul-de-Sacs

Reduction of Impervious Cover

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

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

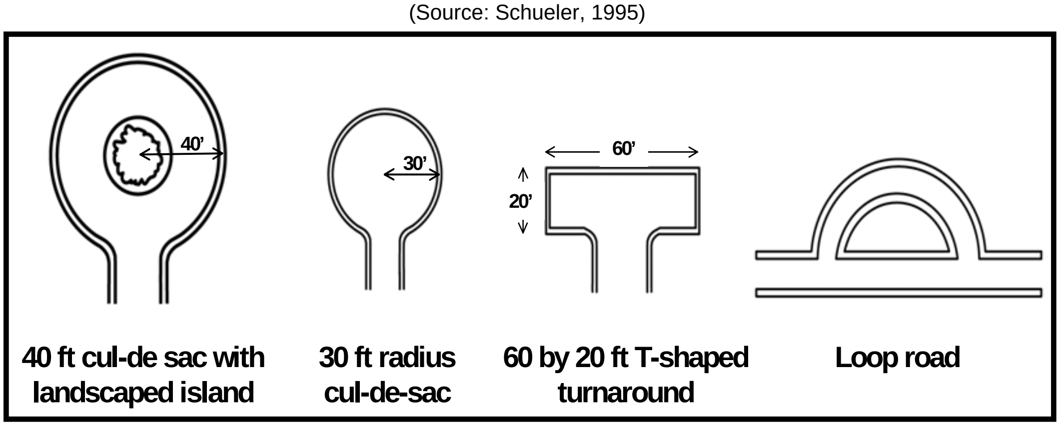
Alternative turnarounds are designs for end-of-street vehicle turnarounds that replace cul-de-sacs and reduce the amount of impervious cover created in developments. Cul-de-sacs are local access streets with a closed circular end that allows for vehicle turnarounds. Many of these cul-de-sacs can have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

Numerous alternatives create less impervious cover than the traditional 40-foot cul-de-sac. These alternatives include reducing cul-de-sacs to a 30-foot radius and creating hammerheads, loop roads, and pervious islands in the cul-de-sac center (see Figure 5-24).

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

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

Figure 5-24. Four Turnaround Options for Residential Streets
(Source: Schueler, 1995)



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

Reduction of Impervious Cover

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

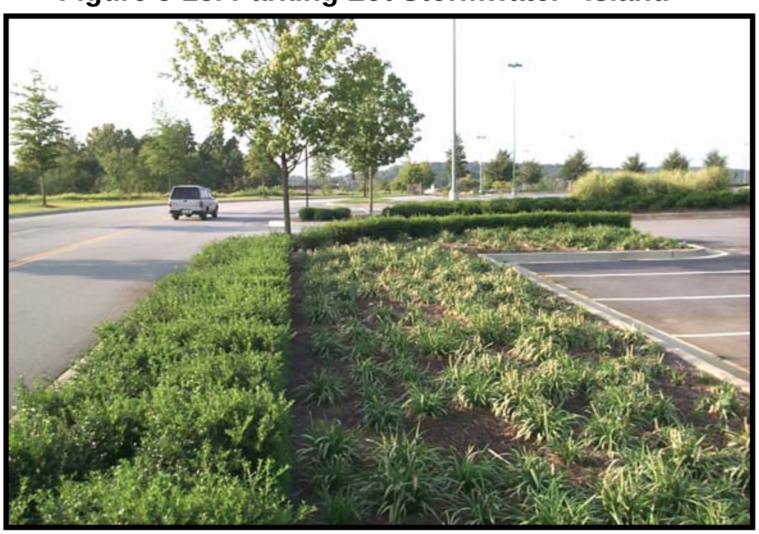
KEY BENEFITS	USING THIS PRACTICE
• Reduces the amount of impervious cover and associated runoff and pollutants generated. • Provides an opportunity for the siting of structural control facilities. • Trees in parking lots provide shading for cars and are more visually appealing.	☑ Integrate porous areas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

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

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

Structural control facilities such as filter strips, dry swales and bioretention areas can be incorporated into parking lot islands. Stormwater is directed into these landscaped areas and temporarily detained. The runoff then flows through or filters down through the bed of the facility and is infiltrated into the subsurface or collected for discharge into a stream or another stormwater facility. These facilities can be attractively integrated into landscaped areas and can be maintained by commercial landscaping firms. For detailed design specifications of filter strips, enhanced swales and bioretention areas, refer to Chapter 4. An example of a parking lot stormwater "island" is presented in Figure 5-25.

Figure 5-25. Parking Lot Stormwater "Island"



5.3.7 Using Natural Site Features for Stormwater Management

Traditional stormwater drainage design usually ignores and replaces natural drainage patterns, which often results in overly efficient hydraulic conveyance systems. These conveyance systems are overly efficient in that they quickly collect and carry water away from sites rather than allowing water to infiltrate naturally. Conveyance systems are composed of structural stormwater controls that are costly and often require high levels of maintenance to operate properly. The use of natural site features and drainage systems through careful site design can reduce the need and size of structural conveyance systems and controls.

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

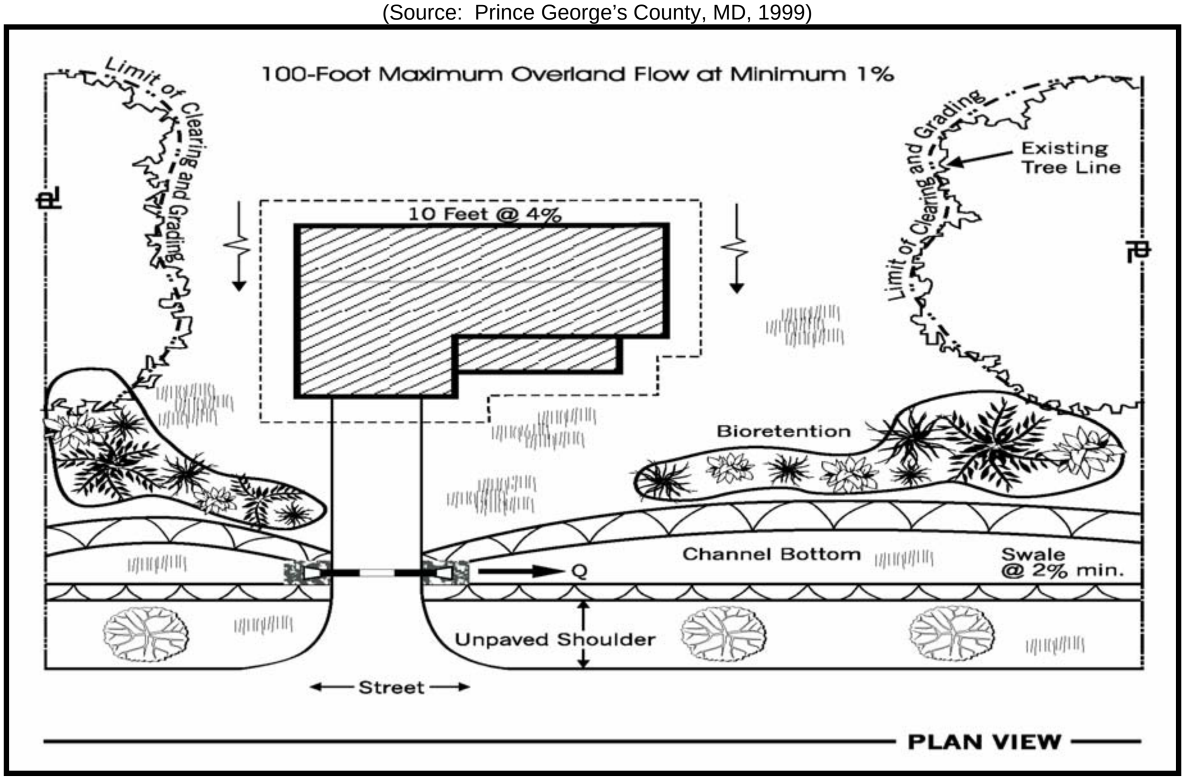
Some of the methods of incorporating natural features into an overall stormwater management site plan include the following practices:

17. Use Buffers and Undisturbed Areas
18. Use Natural Drainageways Instead of Storm Sewers
19. Use Vegetated Swales Instead of Curb and Gutter
20. Drain Runoff to Pervious Areas

Figure 5-26 presents an example of these better site design practices on a residential lot. The following pages cover each practice in more detail.

Figure 5-26. Residential Site Design Using Natural Features for Stormwater Management

(Source: Prince George's County, MD, 1999)



Better Site Design Practice #17:
Use Buffers and Undisturbed Areas

Utilization of Natural Features for Stormwater Management

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

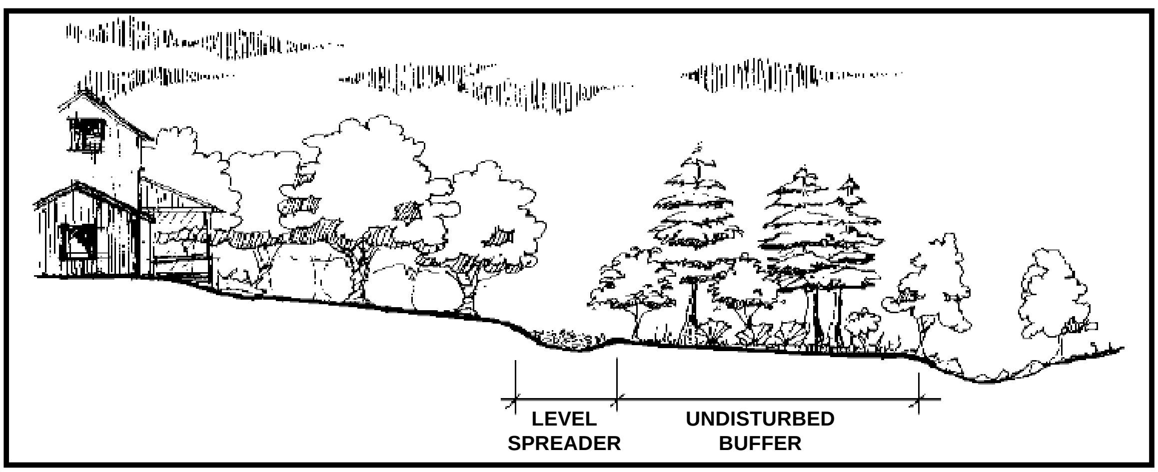
KEY BENEFITS	USING THIS PRACTICE
• Riparian buffers and undisturbed vegetated areas can be used to filter and infiltrate stormwater runoff. • Natural depressions can provide inexpensive storage and detention of stormwater flows. • A stormwater site design credit can be taken for the areas that comply with the criteria listed in Section 5.2.	☑ Direct runoff towards buffers and undisturbed areas using a level spreader to ensure sheet flow. ☑ Utilize natural depressions for runoff storage.

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

The objective of using natural areas for stormwater infiltration is to intercept runoff before it has become substantially concentrated and then distribute this flow evenly (as sheet flow) to the buffer or natural area. This can typically be accomplished using a level spreader, as seen in Figure 5-27. A mechanism for the bypass of higher flow events should be provided to reduce erosion or damage to a buffer or undisturbed natural area.

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

Figure 5-27. Use of a Level Spreader with a Riparian Buffer
(Adapted from NCDENR, 1998)



Better Site Design Practice #18:

Use Natural Drainageways Instead of Storm Sewers

Utilization of Natural Features for Stormwater Management

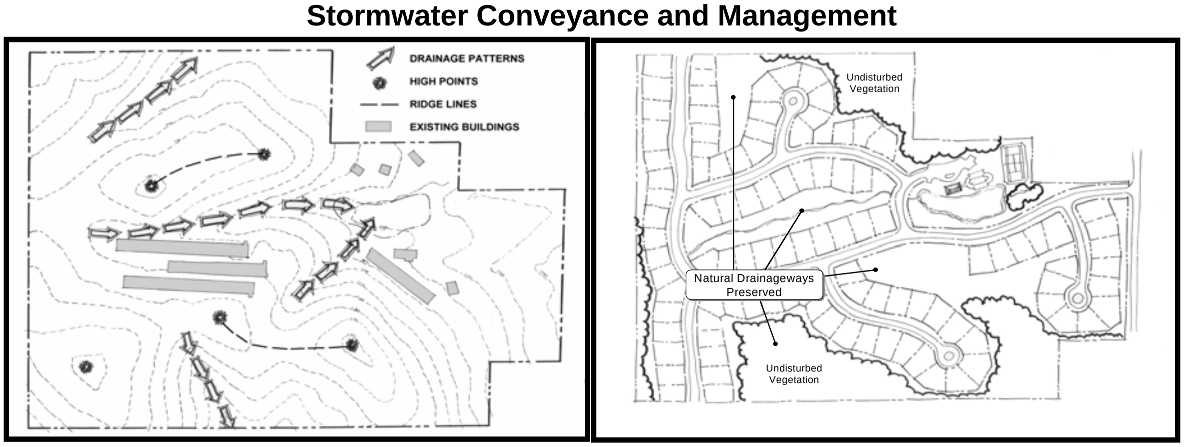
Description: The natural drainage paths of a site can be used instead of constructed underground storm sewers or concrete open channels.

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

Structural drainage systems and storm sewers are designed to be hydraulically efficient in removing stormwater from a site. This type of system tends to increase peak runoff discharges, flow velocities, and pollutant loading to downstream waters. Alternatives are natural drainageways and vegetated swales (where slopes and soils permit), which carry stormwater flows to their natural outlets, particularly for low-density development and residential subdivisions.

The use of natural open channels allows for more storage of stormwater flows on-site, lower stormwater peak flows, a reduction in erosive runoff velocities, infiltration of a portion of the runoff volume, and the capture and treatment of stormwater pollutants. It is critical that natural drainageways be protected from higher post-development flows by applying downstream channel protection methods (including the CPv criteria) to prevent erosion and degradation. Figure 5-28 presents an example of the use of natural drainage ways for stormwater conveyance.

Figure 5-28. Example of the Use of Natural Drainageways for Stormwater Conveyance and Management



Better Site Design Practice #19:
Use Vegetated Swales Instead of Curb and Gutter

Utilization of Natural Features for Stormwater Management

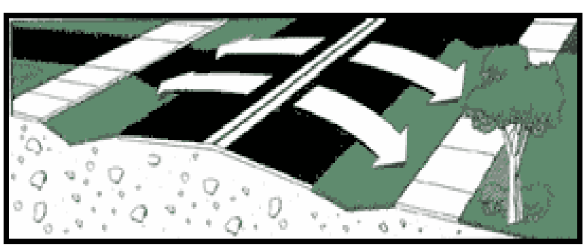
Description: Where density, topography, soils, slope, and safety concerns permit, vegetated open channels can be used in the street right-of-way to convey and treat stormwater runoff from roadways.

KEY BENEFITS	USING THIS PRACTICE
• Reduces the cost of road and storm sewer construction • Provides for some runoff storage and infiltration, as well as treatment of stormwater. • A stormwater site design credit can be taken if the swales comply with the criteria listed in section 5.2.	☑ Use vegetated open channels (enhanced wet or dry swales or grass channels) in place of curb and gutter to convey and treat stormwater runoff.

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

Open vegetated channels along a roadway (see Figure 5-29) remove pollutants by allowing infiltration and filtering to occur, unlike curb and gutter systems which move water with virtually no treatment. Engineering advances prevent past problems with roadside ditches, which suffered from erosion, standing water and break up at the road edge. Grass channels and enhanced dry swales are two such alternatives. If they are properly installed under the right site conditions, they are excellent methods for treating stormwater on-site. In addition, open vegetated channels can be less expensive to install than curb and gutter systems. Further design information and specifications for grass channels and enhanced swales can be found in Chapters 3 and 7 of the Knox County Stormwater Management Manual.

Figure 5-29. Vegetated Swales Instead of Curb and Gutter



Better Site Design Practice #20:
Drain Runoff to Pervious Areas

Utilization of Natural Features for Stormwater Management

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

KEY BENEFITS	USING THIS PRACTICE
• Sending runoff to pervious vegetated areas increases overland flow time and reduces peak flows. • Vegetated areas can often filter and infiltrate stormwater runoff. • A stormwater site design credit can be taken if the area complies with the criteria listed in Section 5.2.	☑ Minimize directly connected impervious areas and drain runoff as sheet flow to pervious vegetated areas.

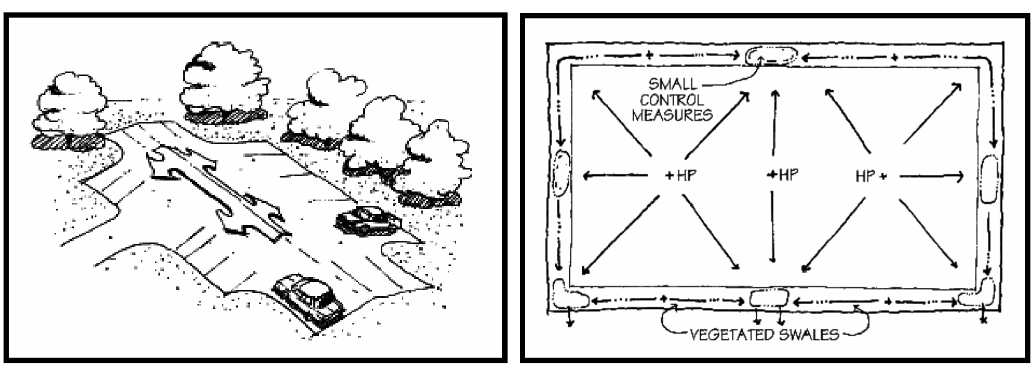
Stormwater quantity and quality benefits can be achieved by routing the runoff from impervious areas to pervious areas such as lawns, landscaping, filter strips and vegetated channels. Much like the use of undisturbed buffers and natural areas (Better Site Design Practice #17), revegetated areas such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

Some of the methods for disconnecting impervious areas include:

- Designing roof drains to flow to vegetated areas.
- Directing flow from paved areas such as driveways to stabilized vegetated areas.
- Breaking up flow directions from large paved surfaces (see Figure 5-30).
- Carefully locating impervious areas and grading landscaped areas to achieve sheet flow runoff to the vegetated pervious areas.

For maximum benefit, runoff from impervious areas to vegetated areas must occur as sheet flow and vegetation must be stabilized. See Chapters 3 and 7 for more design information and specifications on filter strips and vegetated channels.

Figure 5-30. Design Paved Surfaces to Disperse Flow to Vegetated Areas
Source: NCDENR, 1998



5.3.8 Better Site Design Examples (Source: ARC, 2001)

Residential Subdivision Example 1

A typical residential subdivision design on a parcel is shown in Figure 5-31(a). The entire parcel except for the subdivision amenity area (clubhouse and tennis courts) is used for lots. The entire site is cleared and mass graded, and no attempt is made to fit the road layout to the existing topography. Because of the clearing and grading, all of the existing tree cover and vegetation and topsoil are removed dramatically altering both the natural hydrology and drainage of the site. The wide residential streets create unnecessary impervious cover and a curb and gutter system that carries stormwater flows to the storm sewer system. No provision for nonstructural stormwater treatment is provided on the subdivision site.

A residential subdivision employing stormwater better site design practices is presented in Figure 5-31(b). This subdivision configuration preserves a quarter of the property as undisturbed open space and vegetation. The road layout is designed to fit the topography of the parcel, following the high points and ridgelines. The natural drainage patterns of the site are preserved and are utilized to provide natural stormwater treatment and conveyance. Narrower streets reduce impervious cover and grass channels provide for treatment and conveyance of roadway and driveway runoff. Landscaped islands at the ends of cul-de-sacs also reduce impervious cover and provide stormwater treatment functions. When constructing and building homes, only the building envelopes of the individual lots are cleared and graded.

Residential Subdivision Example 2

Another typical residential subdivision design is shown in Figure 5-32(a). Most of this site is cleared and mass graded, with the exception of a small riparian buffer along the large stream at the right boundary of the property. Almost no buffer was provided along the small stream that runs through the middle of the property. In fact, areas within the 100-year floodplain were cleared and filled for home sites. As is typical in many subdivision designs, this one has wide streets that can be used for on-street parking and large cul-de-sacs.

The better site design subdivision can be seen in Figure 5-32(b). This subdivision layout was designed to conform to the natural terrain. The street pattern consists of a wider main thoroughfare that winds through the subdivision along the ridgeline. Narrower loop roads branch off of the main road and utilize landscaped islands. Large riparian buffers are preserved along both the small and large streams. The total undisturbed conservation area is close to one-third of the site.

Commercial Development Example

Figure 4-33(a) shows a typical commercial development containing a supermarket, drugstore, smaller shops and a restaurant on an out-lot. The majority of the parcel is a concentrated parking lot area. The only pervious area is a small replanted vegetation area acting as a buffer between the shopping center and adjacent land uses. Stormwater quality and quantity control are provided by a wet extended detention pond in the corner of the parcel.

A better site design commercial development can be seen in Figure 5-33(b). Here the retail buildings are dispersed on the property, providing more of an “urban village” feel with pedestrian access between the buildings. The parking is broken up, and bioretention areas for stormwater treatment are built into parking lot islands. A large bioretention area, which serves as open green space, is located at the main entrance to the shopping center. A larger undisturbed buffer has been preserved on the site. Because the bioretention areas and buffer provide water quality treatment, only a dry extended detention basin is needed for water quantity control.

Office Park Example

An office park with a conventional design is shown in Figure 5-34(a). Here the site has been graded to fit the building layout and parking area. All of the vegetated areas of this site are replanted areas. The better site design layout, presented in Figure 5-34(b), preserves undisturbed vegetated buffers and open space areas on the site. The layout has been designed to fit the natural terrain of the site. A modular porous paver system is used for the overflow parking areas.

Figure 5-31(a). Example 1 Traditional Residential Subdivision Design

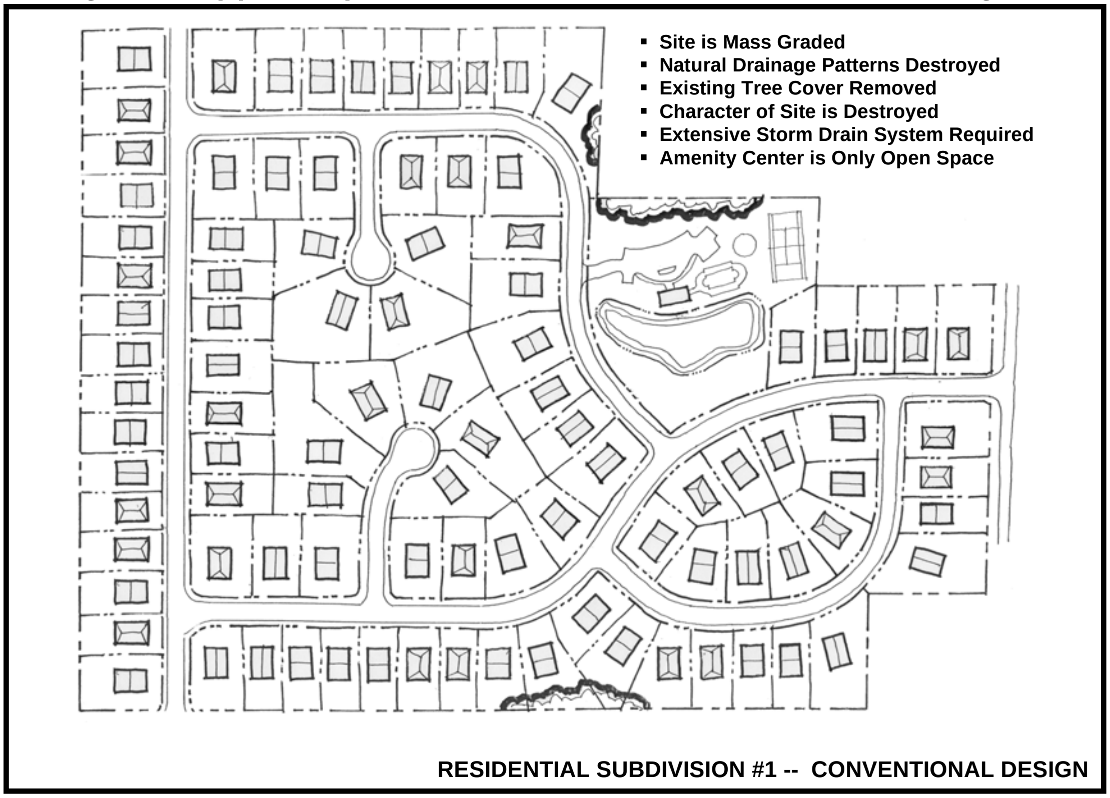


Figure 5-31(b). Example 1 Residential Subdivision Design after Application of Better Site Design Practices

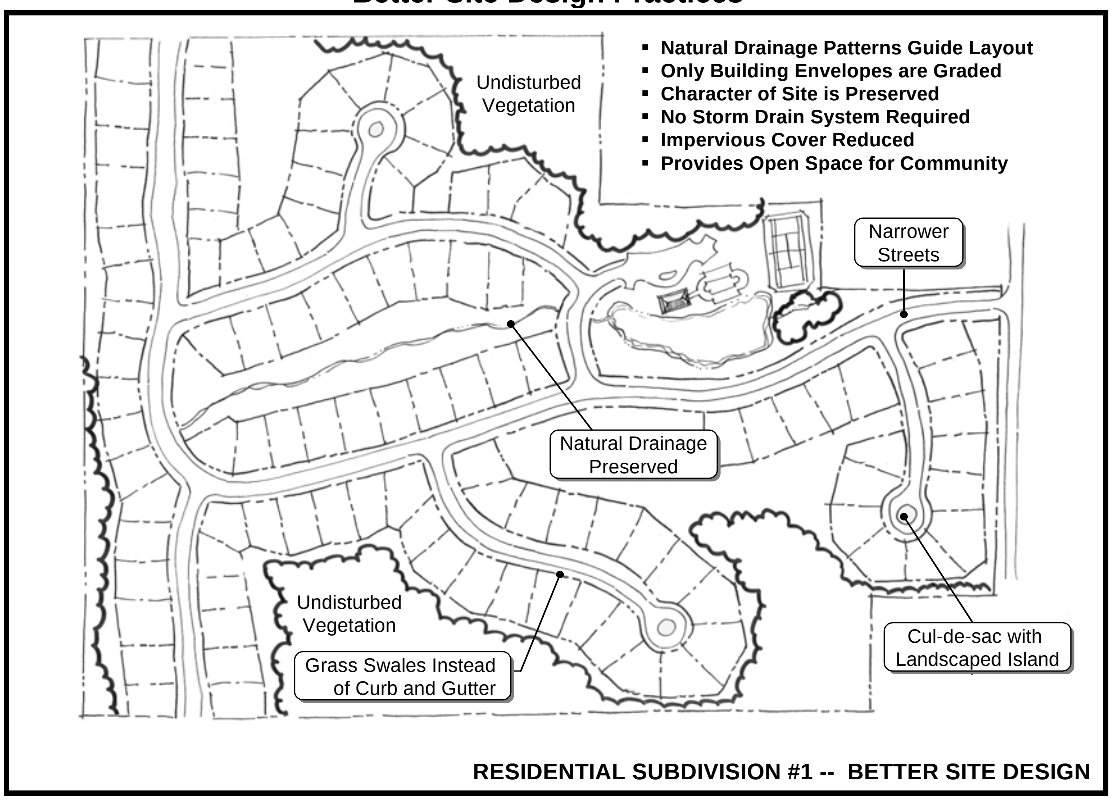


Figure 5-32(a). Example 2 Traditional Residential Subdivision Design

Figure 5-32(b). Example 2 Residential Subdivision Design after Application of Better Site Design Practices

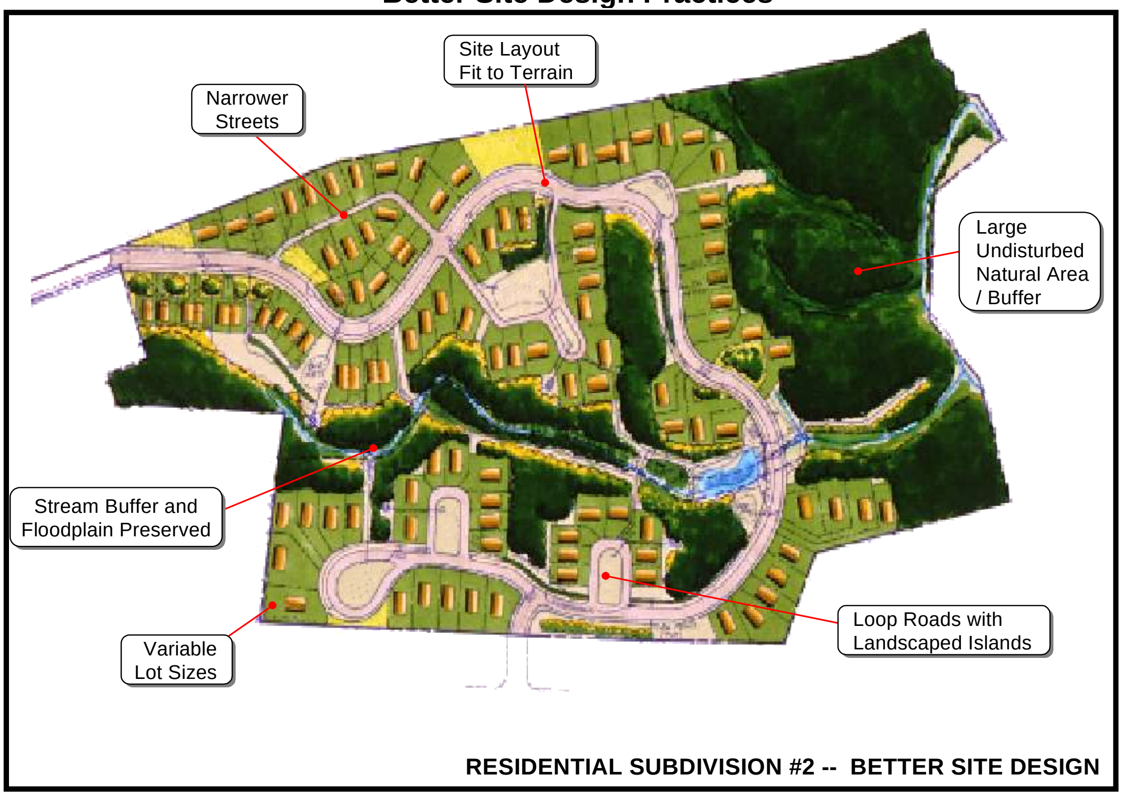


Figure 5-33(a). Example 3 Traditional Commercial Development Design

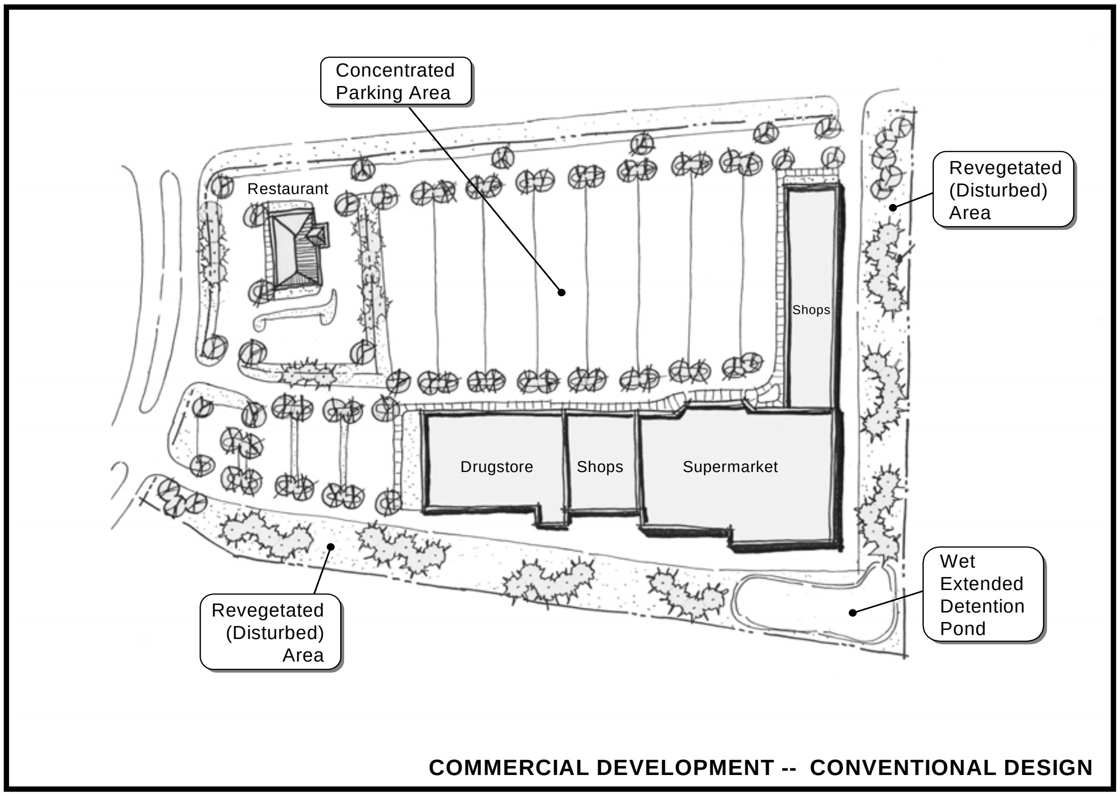


Figure 5-33(b). Example 3 Commercial Development Design after Application of Better Site Design Practices

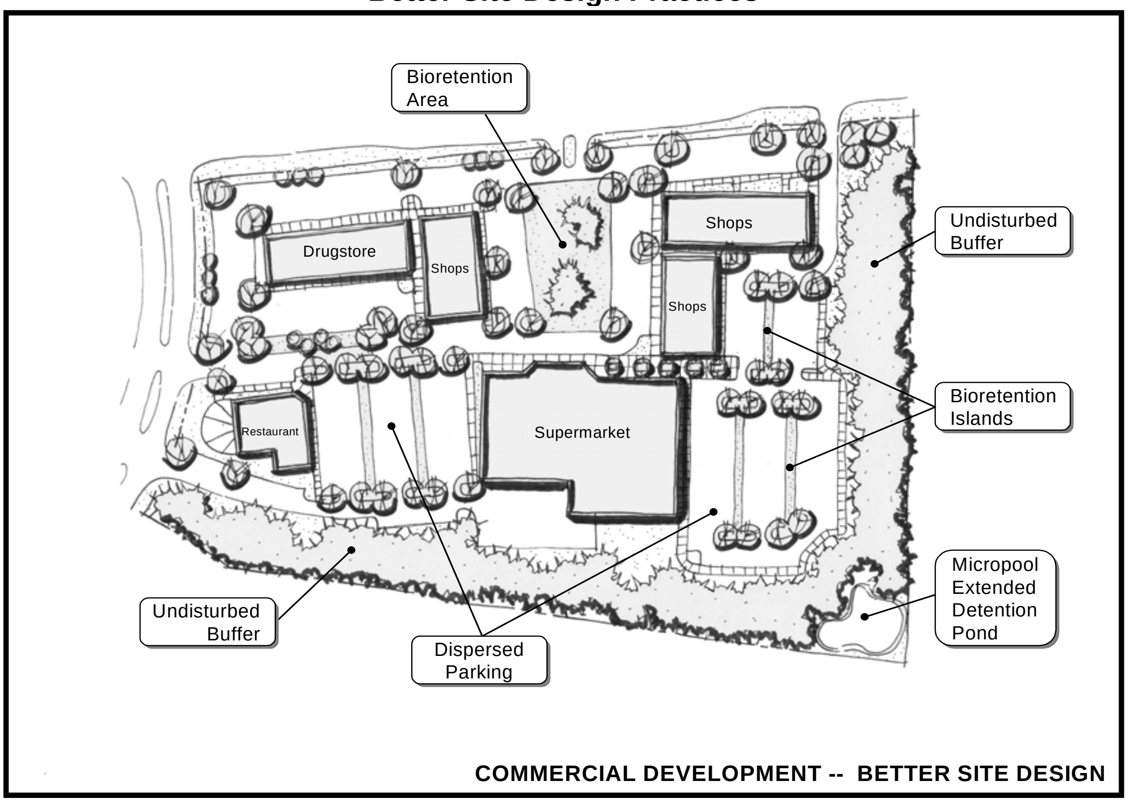


Figure 5-34(a). Example 4 Traditional Office Park Development

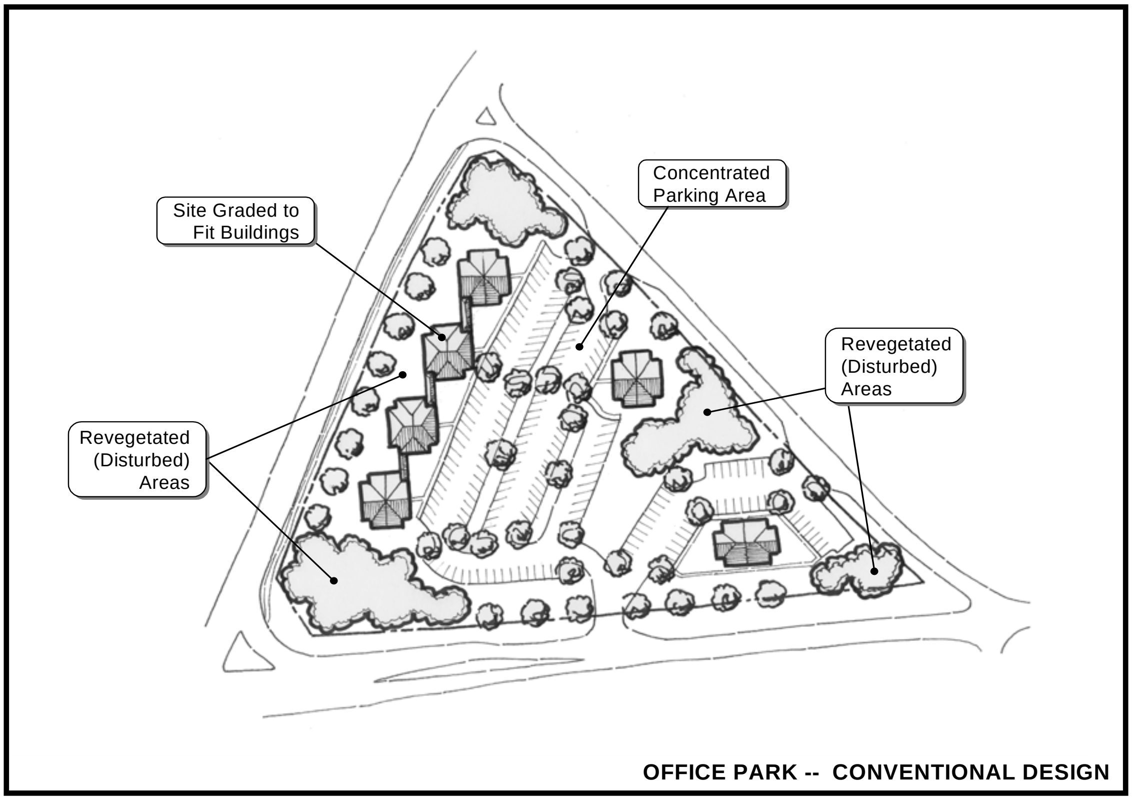
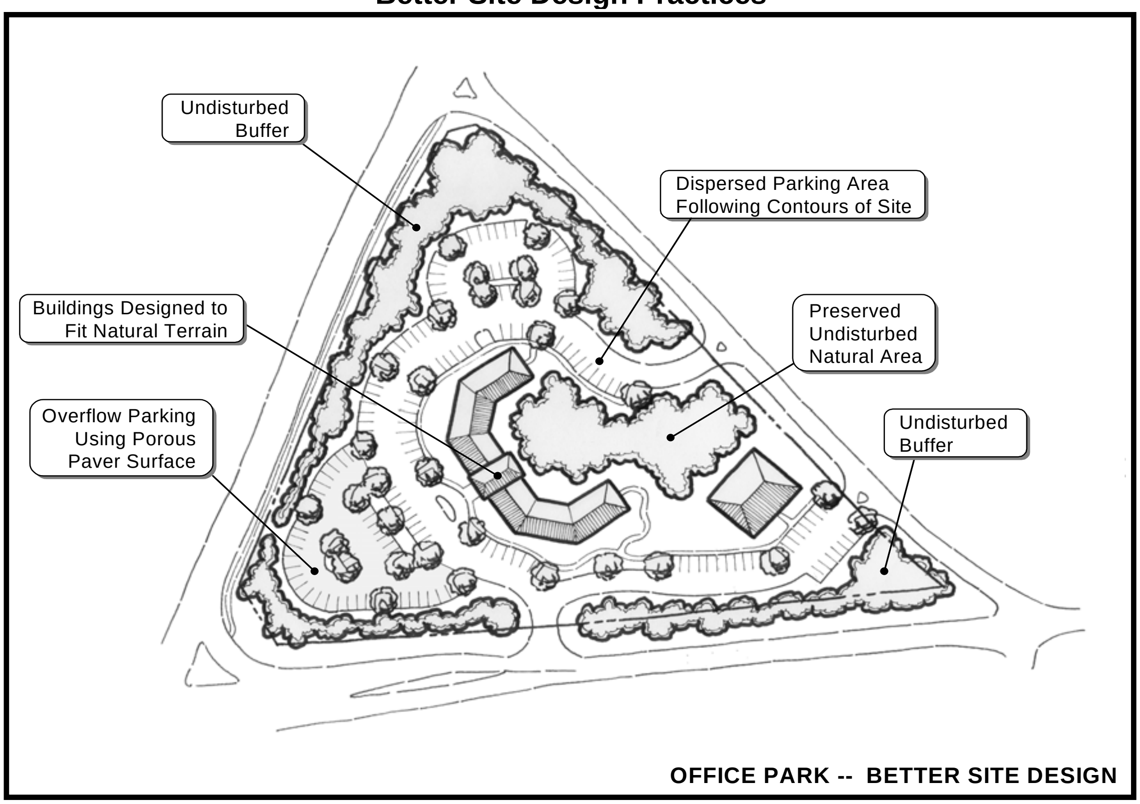


Figure 5-34(b). Example 4 Office Park Development after Application of Better Site Design Practices



References

Atlanta Regional Council (ARC). *Georgia Stormwater Management Manual Volume 1 Stormwater Policy Guidebook.* 2001.

Minnesota Pollution Control Agency (MPCA). *Protecting Water Quality in Urban Areas*. 1989.

North Carolina Department of Environment and Natural Resources (NCDENR). *Stormwater Management Site Planning*. 1998.

Prince George's County, MD. *Low-Impact Development Design Strategies, An Integrated Design Approach*. 1999.

Schueler, T. *Site Planning for Urban Stream Protection*. Prepared by Center for Watershed Protection (CWP). Metropolitan Washington Council of Governments, 1995.

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

Urban Land Institute. *Density by Design*. 1992.

Virginia Polytechnic Institute and State University (VPISU), College of Architecture and Urban Studies. *Water Sensitive Site Planning*. 2000.

www.stormwatercenter.net Better Site Design: Green Parking. Accessed August 2006.

This page left intentionally blank

Table of Contents

CHAPTER 6 — WATER QUALITY BUFFERS

Chapter 6 – List of Figures

CHAPTER

6

WATER QUALITY BUFFERS

6.1 Introduction

Water quality buffers are naturally vegetated areas that are located along the edge or perimeter of streams, lakes, ponds, reservoirs, and wetlands that are defined as "community waters" in unincorporated Knox County. Knox County's stormwater management ordinance requires water quality buffers for all new developments and redevelopments that are bounded by or contain community waters. Community waters are defined in the sections below.

Water quality buffers provide several benefits to water quality in Knox County and are one of a number of best management practices (BMPs) that are available for site developers to incorporate into development plans to meet water quality goals. Buffers provide a tool for the improvement of stream water quality and habitat. A properly designed buffer will slow and spread-out stormwater runoff from upstream impervious areas, and will filter sediment and the chemicals and pollutants that attach to sediment particles via the trees, shrubs, and grasses that comprise the buffer area. Further, the trees and other vegetation in a buffer provide shade for the stream and buffer area, allowing stormwater runoff that has been heated on roofed and paved areas to cool before reaching the stream.

PURPOSES OF A STREAM BUFFER:

- Reduce erosion and stabilize stream banks.
- Assist with infiltration of stormwater runoff.
- Control sedimentation.
- Reduce the effects of flood and drought.
- Provide shade to streams.
- Provide and protect habitat for aquatic species and other wildlife.
- Offer scenic value and recreational opportunities.
- Restore and maintain the chemical, physical and biological integrity of water resources.
- Minimize public investment in waterway restoration, stormwater management, and other public water resource endeavors.

Shueler, WPT Summer, 1995
Vegetated Riparian Buffers and Buffer Ordinances, SCDEC

Vegetated buffers can also act as a flood and erosion management tool. Buffers slow runoff velocities, counter channelization of runoff inflows, and reduce inflow volumes somewhat through infiltration into the soil and capture in vegetation. These effects can reduce the potential for downstream flooding, and the potential for streambank instability and erosion, both in the buffered area and downstream. Because development in buffered areas is limited, buffers can be helpful for floodplain management, preventing development along the stream edge and, in some cases, out of the floodway.

Knox County must incorporate requirements for water quality buffers into its stormwater management program in order to comply with the National Pollutant Discharge Elimination System (NPDES) Phase II Permit. The permit requires that Knox County "develop and implement a set of requirements to establish, protect and maintain water quality buffers in areas of new development and redevelopment." In the permit, water quality buffers are defined as "undisturbed vegetation including trees, shrubs, and herbaceous vegetation; enhanced or restored vegetation; or the re-establishment of vegetation bordering streams, ponds, wetlands, reservoirs, or lakes, which exists or is established to protect those water bodies."

6.2 Applicability

6.2.1 Community Waters Definitions

For new developments and redevelopments, water quality buffers must be established, protected and maintained along or around all water bodies that are designated as community waters that are located in the unincorporated areas of Knox County. Community waters include streams, wetlands, ponds, and lakes as defined below.

6.2.1.1 Streams

Streams are defined in the Knox County Stormwater Management Ordinance as linear surface water conveyances that can be characterized as having either perennial or ephemeral base flow and

a. have published floodplain elevations that have been computed as part of an approved flood study; *or*

b. are identified as a blue line on a 7.5-minute USGS quadrangle, unless otherwise designated by Tennessee Department of Environmental Conservation (TDEC); *or*

c. are identified by the property owner as a stream; *or,*

d. are determined to be streams by Knox County, the United States Army Corps of Engineers (USACE) or Tennessee Department of Environmental Conservation (TDEC).

If the designation of a linear drainage feature as a stream based on the above stated criteria is disputed, and if the feature has an upstream drainage area of one-hundred (100) acres or more, a stream determination can be performed by the property owner. The stream determination must be performed using the stream determination protocol provided by Knox County Engineering. The protocol should be completed by a qualified individual. The determination will be reviewed and approved or denied by Knox County staff.

In the event that the property owner disputes the identification by Knox County or another party of a linear drainage feature as a stream (and therefore community waters), TDEC can be utilized to make the ultimate determination. It is the responsibility of the property owner to contact and coordinate stream determination activities with TDEC. Any determination made by TDEC must be documented in writing and submitted to Knox County with the stormwater management plan prepared for the site. In the event that TDEC determines that a linear drainage feature is not a stream, Knox County will accept TDEC's written determination, and a water quality buffer will not be required.

6.2.1.2 Wetlands

Wetlands are defined in the Knox County Stormwater Management Ordinance as areas that are inundated or saturated by surface water or groundwater at a frequency and duration sufficient to support, and under normal circumstances does support a prevalence of vegetation typically adapted for life in saturated soil conditions.

Knox County staff will not make wetland determinations. Wetland determinations must be performed by one of the following agencies:

- USACE; and/or
- EPA; and/or
- TDEC; and/or
- the Natural Resources Conservation Service (NRCS); and/or
- a qualified professional that has been trained in the identification and delineation of wetland areas.

6.2.1.3 Ponds and Lakes

Ponds and lakes are defined as inland bodies of standing water. Water quality buffers are required for ponds and lakes that are directly connected to a stream that is considered community water.

For a pond or lake to be directly connected to a stream, at least one of the following criteria must be met:

- a stream, as defined in section 6.2.1.1, must have a direct discharge into the pond or lake; or,
- the pond or lake discharges directly into a stream, as defined in section 6.2.1.1, or wetland.

Buffers are not required for isolated ponds or lakes that do not have a direct connection to a stream. Further, buffers are not required for ponds that have been constructed solely for the purpose of water quantity and/or quality control.

6.3 Buffer Zones and Widths

6.3.1 Zones

Knox County has established a two-zoned buffer approach on community waters. The buffer zones and the width of each zone that are required for each type of community water feature are described below. In general, the inner zone of a water quality buffer (i.e., the zone closest to the water) is to remain undisturbed and should consist of forest vegetation and deep-rooted, woody vegetation. The outer zone of a water quality buffer (i.e., the zone farthest from the water) is a transition zone between the inner zone and development. The outer zone can consist of managed vegetation, including grasses. Knox County allows buffer width averaging within the outer zone. Buffer width averaging is discussed in Section 6.5. The outer zone of a buffer may be disturbed, in accordance with Section 6.7, but must be revegetated. Impervious surfaces are not permitted within either zone.

While the goal of the water quality buffer zone is to maintain undisturbed vegetation with no impervious surfaces in either zone, limited disturbances or impacts are permitted. Section 6.7 provides more information on the permitted disturbances or impacts to water quality buffers.

6.3.2 Widths

The following sections identify zone widths for each type of community water feature.

6.3.2.1 Streams

A fifty (50) foot minimum width buffer shall be provided along each side of a stream, unless a narrower width is permitted through buffer width averaging, presented in Section 6.5. The water quality buffer is divided into two zones and shall be maintained as stated below. A graphical depiction of a two-zone buffer is presented in Figure 6-1.

1. The **inner zone** shall have a minimum width of twenty-five (25) feet measured landward, perpendicular from the top-of-bank of the active channel. For streams that do not have a defined top-of-bank, the buffer shall be measured perpendicular from the centerline of the stream. The inner zone must remain undisturbed except for the allowable disturbances listed in Section 6.7.
2. The **outer zone** shall be measured from the outer edge of the inner zone and shall extend the perpendicular distance required to fulfill the required minimum total, two-zoned buffer width of fifty (50) feet. The outer zone can be disturbed, graded and revegetated according to the specifications in Section 6.7.2.

Examples of buffer width measurement, as referenced from the top-of-bank and stream centerline, are shown in Figures 6-2a and 6-2b, respectively.

Figure 6-1. Streamside Water Quality Buffers: Zones and Widths

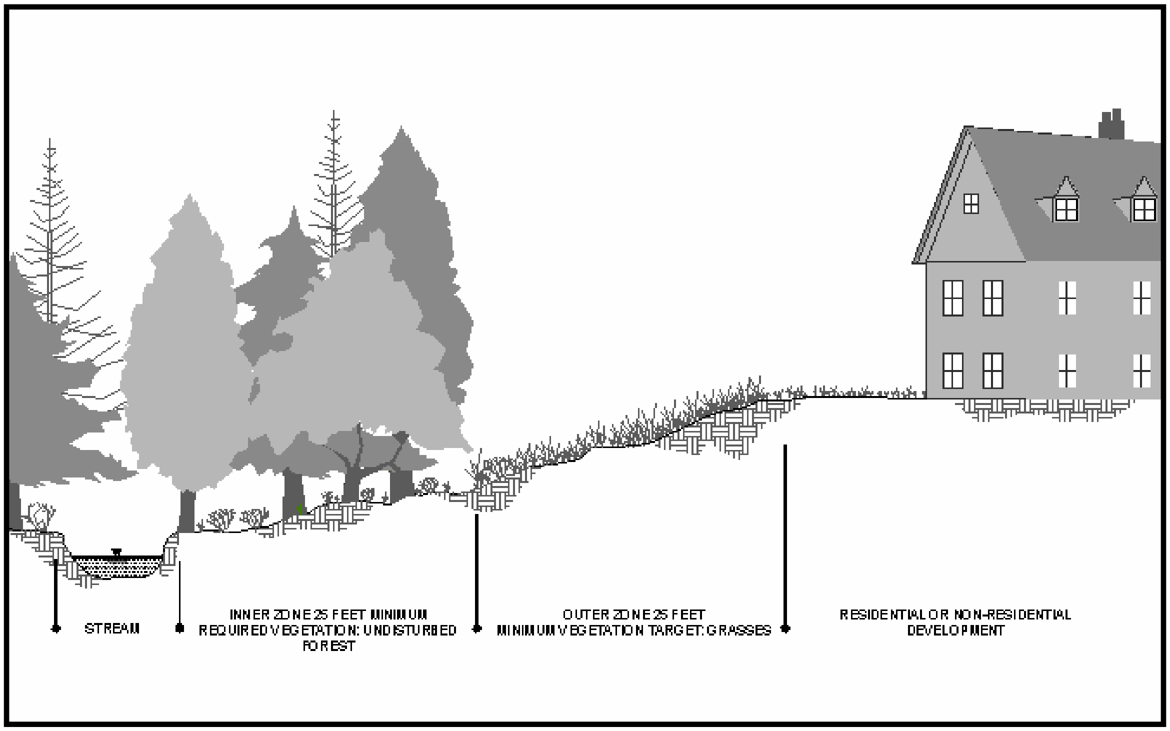


Figure 6-2a. Example Buffer Width Measurement – Top-of-Bank

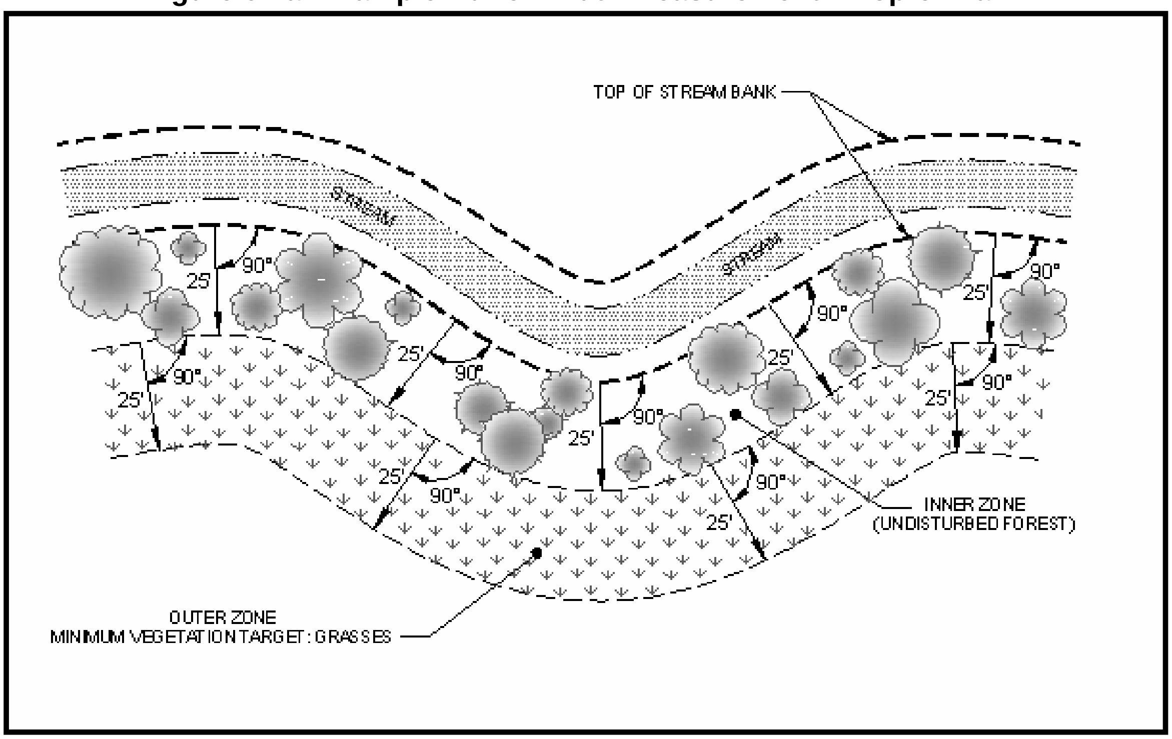
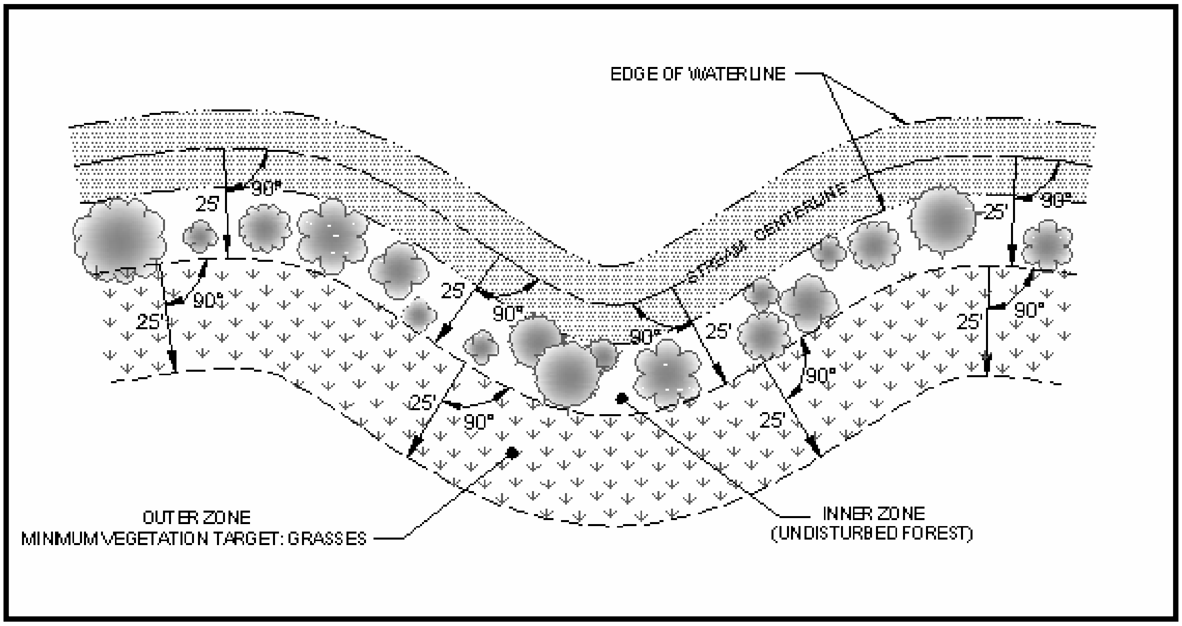


Figure 6-2b. Example Buffer Width Measurement – Stream Centerline



6.3.2.2 Wetlands

A minimum twenty-five (25) foot water quality buffer shall be provided around the perimeter of a wetland, measured perpendicular from the outermost edge of the wetland, as determined by USACE, NRCS, TDEC, or other qualified professional. The water quality buffer on a wetland will consist of one zone, which shall remain undisturbed except for the allowable disturbances listed in Section 6.7.

6.3.2.3 Ponds and Lakes

A minimum twenty-five (25) foot water quality buffer shall be provided around the perimeter of ponds and lakes that are directly connected to other community waters. The water quality buffer width shall be measured perpendicular landward from the topographic contour that defines the normal pool elevation, as shown in Figure 6-3. The water quality buffer shall consist of one zone, which can be disturbed, graded, and revegetated according to the specifications in Section 6.7.

6.3.3 Width Requirements for Steep Slopes

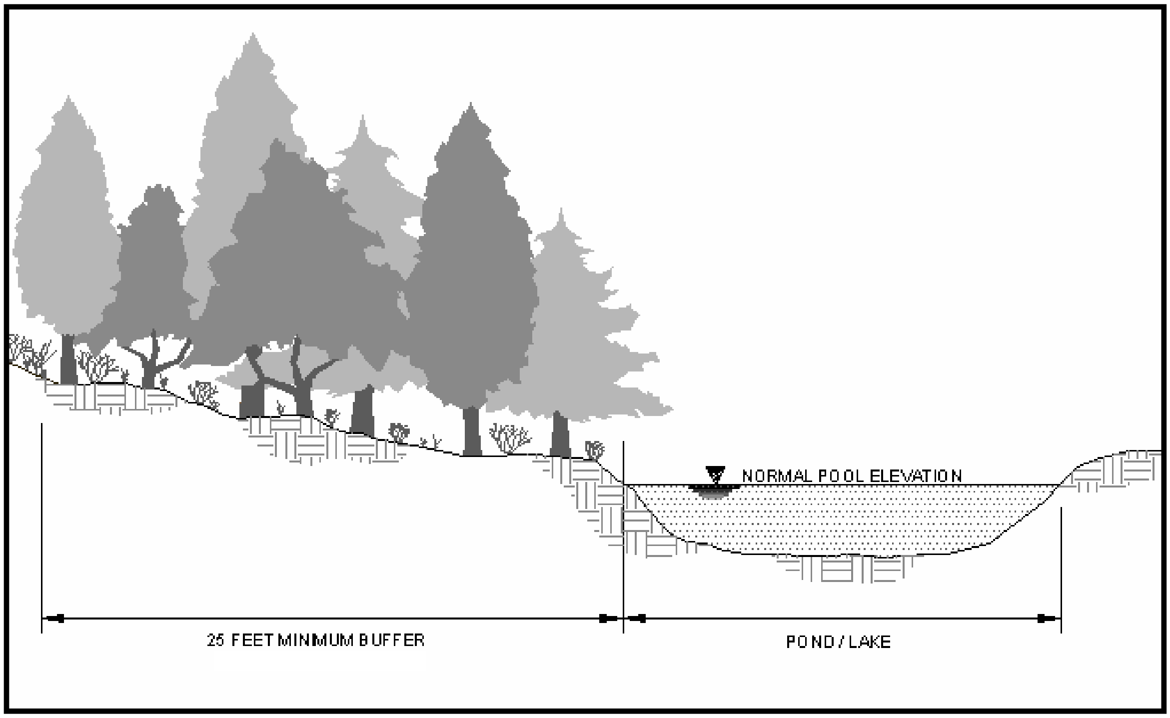
A physical parameter that impacts a buffer's ability to slow stormwater runoff and filter pollutants is the slope of the buffer. A steeper slope will reduce the effectiveness of buffer areas because the runoff moves too quickly through the buffer to allow proper filtration of sediment and infiltration of smaller storm events. This issue can be overcome by requiring buffer width increases when steep slopes are present.

Where steep slopes greater than 15% or where Slope Protection Areas, as identified by the Metropolitan Planning Commission, are located within 50 feet of the community water, one of the two following conditions shall apply to the water quality buffer:

- the buffer width in the steep slope areas shall be adjusted to include an additional 25 feet, giving a total buffer width of 75 feet; or,
- the 50-foot buffer in the steep slope areas shall consist of one-zone, comprised of undisturbed, forested vegetation, as described in section 6.4.1.

Buffer width adjustments using buffer averaging, which is discussed in Section 6.5, are prohibited where steep slopes are present.

Figure 6-3. Example Buffer Width Measurement – Normal Pool Elevation



6.4 Vegetation and Enhancement Requirements

This section provides requirements and guidance on buffer vegetation and buffer enhancement, and describes the contents of Buffer Enhancement Plans. Within the requirements set forth by Knox County, detailed information on streambank and buffer restoration techniques, planting guidelines and native plant species can be found from the following sources:

- Tennessee Valley Authority's Riparian Restoration webpage, located at www.tva.com/river/landandshore/stabilization/index.htm
- Tennessee Valley Authority's Native Plant Finder webpage, located at www.tva.com/river/landandshore/stabilization/plantsearch.htm;
- Banks and Buffers: A guide to selecting native plants for streambanks and shorelines. Contact information to obtain this publication is provided at www.tva.com/river/landandshore/stabilization/websites.htm;
- Knoxville-Knox County Tree Conservation & Planting Plan, published by MPC and available at www.knoxmpc.org
- the Tennessee Exotic Plant Pest Council website, located at www.tneppc.org; and
- the Natural Resource Conservation Service (NRCS).

6.4.1 Inner Zone (Streams and Wetlands)

The vegetative target for the inner zone of stream buffers and for buffers around wetlands is undisturbed, mature, moderately dense forest (i.e., trees) with woody shrubs and understory vegetation that provides a stable forest floor. Native, non-invasive vegetation is preferable, although existing buffer areas that are stable with limited populations of undesirable plant species are acceptable. Native trees, shrubs and grasses that are appropriate for use in a water quality buffer are better-suited for local, long-term hydrologic conditions than non-native vegetation. As such, native vegetation will likely have lower mortality rates and few maintenance requirements, making them ideal for an area that must remain undisturbed.

The Director of Knox County Department of Engineering and Public Works (the Director) may require (or allow if requested by the property owner) limited disturbances of an existing buffer to remove unhealthy, damaged, diseased, dead, and/or non-native or invasive vegetation in order to improve the vegetative condition of the buffer. Such disturbances must be performed in accordance with the provisions of the Knox County Stormwater Management Ordinance and the policies set forth in Section 6.7 of this manual.

The Director may require (or allow if requested by the property owner) enhancement of inner zone areas that do not conform to the vegetative target of a mature forest at the time of development, in accordance with the provisions and guidance presented in this manual. Areas that can be characterized as an early successional forest, consisting of a combination of grasses, vines, shrubs, tree saplings and possibly even a few mature trees, may not require enhancement, provided that the vegetation appears healthy, provides adequate ground coverage, and consists largely of native and non-invasive species.

6.4.2 Outer Zone (Streams and Ponds/Lakes)

The outer zone of the water quality buffer for streams and around ponds and lakes is a transition zone between the inner zone and the developed property. The minimum vegetative target for the outer zone is mowed, dense grass that covers the entire zone. Vegetation in the outer zone can be managed within the allowable uses, activities and disturbances stated in this manual and in the Knox County Stormwater Management Ordinance.

Although native plant species are not required in the outer zone, undisturbed native, shrubs, trees and other woody vegetation are strongly encouraged. Invasive species, such as honeysuckle, privet, ivy, and kudzu are not permitted in the outer zone. Note that leaving the entire buffer area, including the outer zone, as an undisturbed natural area could provide additional water quality treatment benefits and credits against the required water quality treatment volume. Volume II Chapter 5 of this manual provides more information on water quality treatment credits using buffers.

6.4.3 Non-native and Invasive Vegetation

Knox County discourages the introduction or propagation of plants considered as nuisance, non-native (also termed "exotic") and/or invasive plant species, such as honeysuckle, privet, ivys and kudzu. When a Buffer Enhancement Plan is required by the County, non-native and/or invasive plant species will not be permitted. Guidance on non-native species in Tennessee can be found at the Tennessee Exotic Plant Pest Council website, located at www.tneppc.org.

Non-native and/or invasive plant species found in existing buffer areas can be removed. Limited disturbances for vegetation removal and/or maintenance must be performed in accordance with Section 6.7. Large populations of non-native and/or invasive plants may require extensive measures to eliminate the vegetation and will require the submittal of a Buffer Enhancement Plan to Knox County for review and approval.

6.4.4 Buffer Enhancement Requirements

Buffer enhancement can only be performed with prior approval of the Director. The Director has the authority to require enhancement of the water quality buffer when a buffer area:

- does not conform to the vegetative target for the area, or in its existing state does not have the potential to meet the vegetative target through natural vegetative succession;
- has significant populations of non-native and/or invasive plant species; or
- has significant areas of unhealthy, damaged, diseased or dead vegetation.

Property owners can request buffer enhancement if desired. Buffer enhancement activities located on water quality buffers in existing developments may require submittal of a stormwater management plan. Consult the Knox County Stormwater Management Ordinance for guidance. When enhancement of the vegetation within a buffer is desired or required, the Director may approve, or require, a Buffer Enhancement Plan. The plan must be submitted and approved by the Director prior to restoring or enhancing a buffer. Required contents of buffer restoration and/or enhancement plans are presented in Volume I Chapter 4 of this manual.

Enhancement of water quality buffers must be performed in accordance with the requirements of the Knox County Stormwater Management Ordinance and the guidance provided below.

1. All areas/zones of the buffer being enhanced must be planted with vegetation that is appropriate to achieve the vegetative targets stated in Section 6.4 of this manual.
2. All areas/zones of the buffer being enhanced must be stabilized against erosion.
3. If the outer zone of a stream buffer and the buffer around a pond or a lake will consist largely of grasses after enhancement, seeding must be performed at a rate sufficient to provide healthy, dense, permanent vegetative cover for 100% of the buffer area within one growing season. Mulch, pebbles, wood chips and other non-vegetative ground cover is not acceptable for buffer enhancement.
4. Where the removal of such vegetation would cause a reduction in the amount of stream canopy by 50% or more, revegetation with native plants is required to provide the cover of the previous canopy at a minimum. For areas where such vegetation removal would cause a reduction in the amount of streambank vegetation, revegetation specifications with native plants is required to return the amount of vegetative cover to its previous state, at a minimum. To reduce the potential for streambank erosion, revegetation measures along streambanks must include sufficient erosion control measures, such as turf reinforcement mats, erosion control blankets, straw wattles, etc., to stabilize the area in the short- and long-term.
5. To increase the chances for the success and health of the buffer area, the plant species, density, placement, and diversity proposed in Buffer Enhancement Plans must be appropriate for stream, wetland, and pond/lake buffers to achieve the vegetative target that is defined for the buffer through natural succession. Proposed planting and long-term maintenance practices must also be appropriate and properly performed.
6. Vegetation mortality must be accounted for all planting densities that are proposed in Buffer Enhancement Plans.

6.5 Water Quality Buffer Averaging

This section outlines the criteria for buffer averaging on new and redevelopment sites. Buffer averaging can be utilized to adjust the required buffer width, allowing some flexibility for site development. Using buffer averaging, the width of the buffer can be varied with the criteria stated in the Knox County Stormwater Management Ordinance and in this manual, so long as a minimum average width of fifty (50) feet is maintained. Figures 6-4 and 6-5 illustrate the use of buffer averaging for a residential and commercial development.

Figure 6-4. Example of Buffer Averaging in a Residential Development

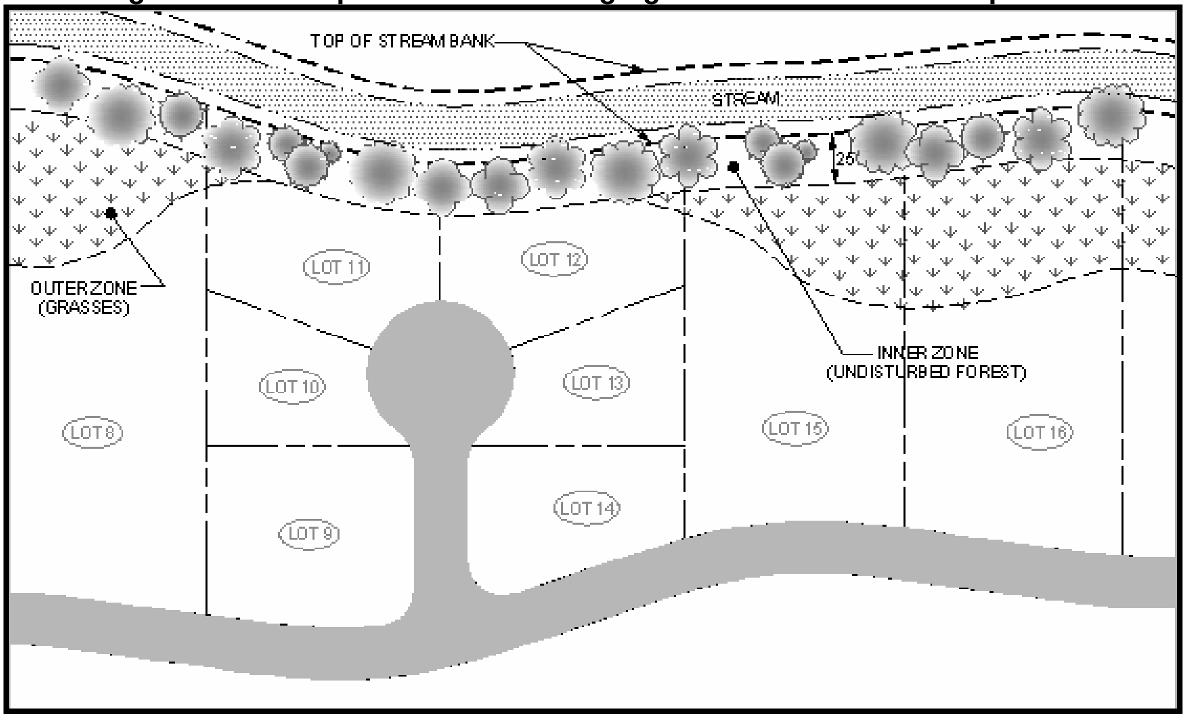
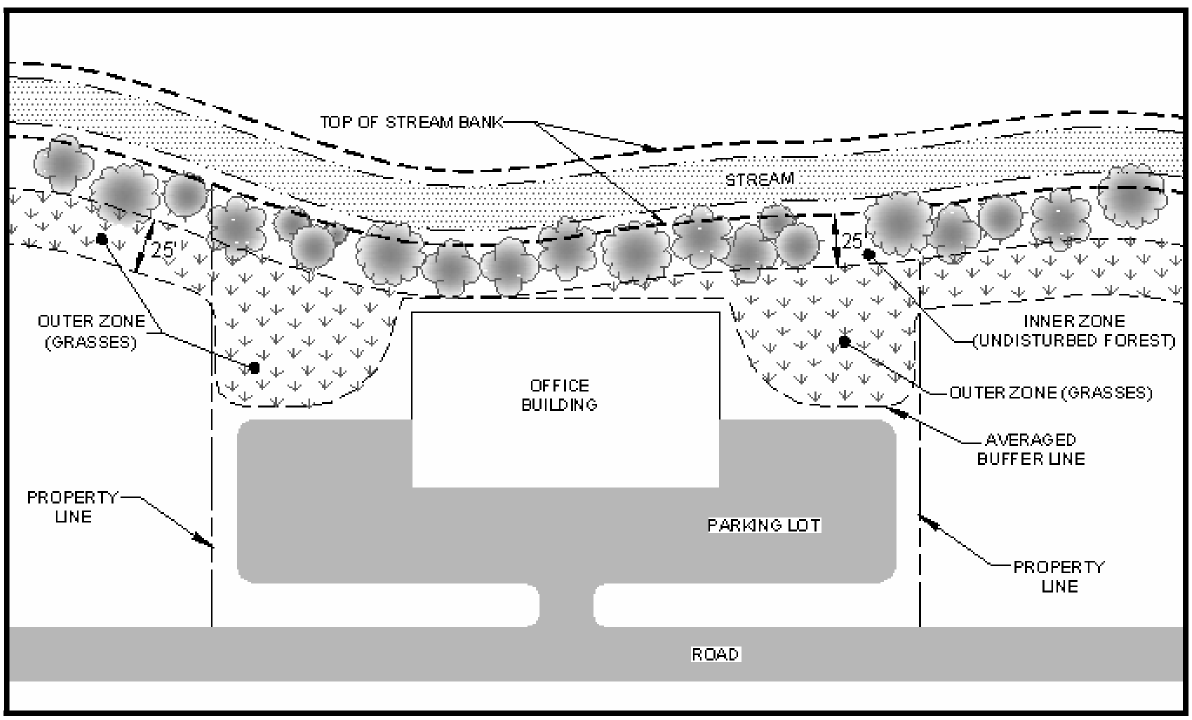


Figure 6-5. Example of Buffer Averaging in a Non-Residential Development



6.5.1 Requirements and Policies

The following criteria must be met in order to utilize buffer averaging on a development site:

1. Buffer averaging is required for water quality buffers that have stream crossings.

2. An overall average buffer width of at least fifty (50) feet must be achieved within the boundaries of the property to be developed. Stream buffer corridors on adjoining properties cannot be included with buffer averaging on a separate property, even if owned by the same property owner.
3. The average width must be calculated based upon the entire length of streambank that is located within the boundaries of the property to be developed. When calculating the buffer length, the natural stream channel should be followed.
4. Stream buffer averaging shall be applied to each side of a stream independently. If the property being developed encompasses both sides of a stream, buffer averaging can be applied to both sides of the stream, but must be applied to both sides of the stream independently.
5. The total width of the buffer shall not be less than twenty-five (25) feet at any location, except at approved stream crossings. Those areas of the buffer having a minimum width of twenty-five (25) feet (or less at approved stream crossings) can comprise no more than fifty (50) percent of the buffer length.
6. The entire length of the buffer shall consist of an inner zone, as defined in Section 6.3.1, that has a minimum width of twenty-five (25) feet, and an outer zone, that has a variable width along its length to achieve the minimum average width of at least fifty (50) feet.

6.5.2 Areas Where Buffer Averaging is Prohibited

Buffer width averaging is prohibited in developments that have, or will have after development, the land uses listed below:

a. Slope protection areas (as identified by MPC), and areas that have slopes greater than 15% that are located within 50 feet of the stream to be buffered;
b. Developments or facilities that include on-site sewage disposal and treatment systems (i.e., septic systems), raised septic systems, subsurface discharges from a wastewater treatment plant, or land application of bio-solids or animal waste;
c. Landfills (demolition landfills, permitted landfills, closed-in-place landfills);
d. Junkyards;
e. Commercial or industrial facilities that store and/or service motor vehicles;
f. Commercial greenhouses or landscape supply facilities;
g. Developments or facilities that have commercial or public pools;
h. Agricultural facilities, farms, feedlots, and confined animal feed operations;
i. Animal care facilities, kennels, and commercial/business developments or facilities that provide short-term or long-term care of animals; or
j. Other land uses deemed by the Director to have the potential to generate higher than normal pollutant loadings.

6.5.3 Example Calculation

This section provides an example calculation of water quality buffer averaging. Consider a development site that is bounded by 500 linear feet of stream, measured following the stream channel. Only one side of the stream is located within the boundaries of the site to be developed. The example site before and after buffer averaging is applied is presented in Figures 6-6a and 6-6b. The example calculation is below.

Constants: Total linear length of buffer = 500 ft

Required width of buffer = 50 ft

Step 1. Calculate the total required area of buffer.

The total linear length of the buffer is 500 feet (ft).
The required width of the buffer (without averaging) is 50 ft.

Total required area of buffer = length of buffer x width of buffer
= 500 ft x 50 ft
= 25,000 ft^2

Step 2. Calculate maximum allowed length of buffer that has the minimum allowed buffer width:

Knox County allows a maximum of 50% of the total length of the buffer to have a width of 25 feet (i.e., comprised only of inner zone). Step 2 will determine the maximum length of buffer that can have the minimum allowed buffer width of 25 feet.

Maximum length of 25 ft buffer = length of buffer x 50%
= 500 ft x 50%
= 250 linear feet

Therefore, 250 linear feet of the buffer can have a minimum width of 25 feet.

Step 3. Calculate total area of buffer that has the minimum allowed buffer width and determine remaining buffer area required.

Total buffer area provided = Length of buffer @ 25 ft x 25 ft width
= 250 ft x 25 ft
= 6250 ft^2

Available buffer area remaining = Total required area of buffer – 6250 ft^2
= 25,000 ft^2 – 6250 ft^2
= 18,750 ft^2

Step 4. Determine the width of remaining buffer.

Length of Remaining buffer = Total width – Length of buffer @ 25 ft width
= 500 ft – 250 ft
= 250 ft

Width of remaining buffer = 18,750 ft^2 / 250 ft
= 75 ft

Therefore, 250 linear feet of buffer will have a minimum 25 ft width and an additional 250 ft of buffer will have a minimum width of 75 ft, with an overall buffer width average of 50 ft.

If more variation in the buffer width is desired, steps 3 and 4 can be repeated using variable buffer widths until an average standard width of 50 feet is achieved keeping the total required area of the buffer constant.

Figure 6-6a. Example Site Before Buffer Averaging

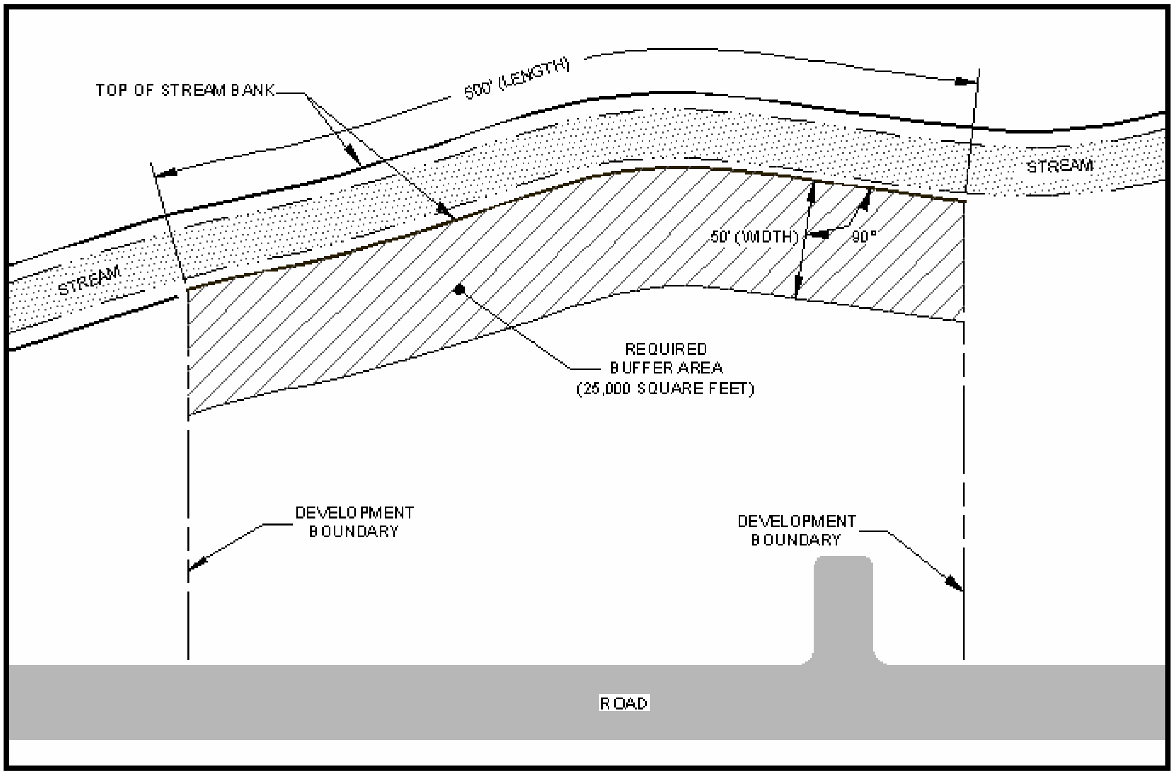
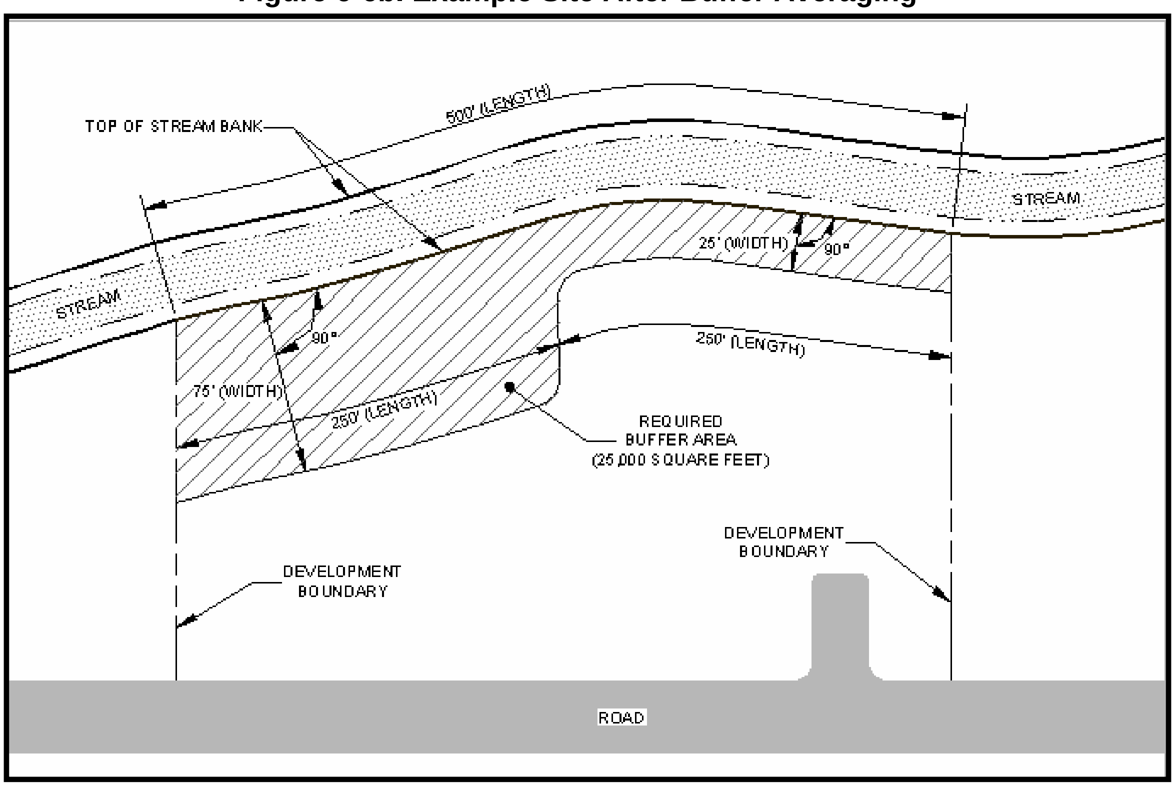


Figure 6-6b. Example Site After Buffer Averaging



6.6 Use of Buffer Areas

As described in previous sections, water quality buffers are use-restricted zones of vegetation that are located along community waters. The intention of the Knox County buffer requirements is to implement buffers that can not only provide stream corridor protection, but serve as water quality best management practices as well. Limiting the use and disturbances within the buffer area serves to protect these intended functions of the buffer. The restrictions on uses of and activities in the buffer address both ongoing activities ("uses") and temporary encroachments ("disturbances"). This section addresses allowed and prohibited uses.

6.6.1 Prohibited Uses

The activities listed below are prohibited within water quality buffers because of their detrimental effects on the water quality buffer functions. These activities may only be performed within the water quality buffer with the express written permission of the Director of Engineering and Public Works.

a. Spraying, filling, dumping, and animal grazing;

b. Use, storage, or application of pesticides, herbicides, fertilizers, or household or commercially-generated wastes;

c. Concentrated animal lots or kennels;

d. Use or storage of motorized vehicles, except for maintenance approved by the Director, or emergency use;

e. Creation of impervious surfaces except for those included in approved stream crossings and other allowed uses;

f. Other uses as deemed by the Director to have the potential to generate higher than normal pollutant loadings.

6.6.2 Allowable Uses in the Inner Zone

The following uses are permitted in the inner zone of stream buffers and the wetland buffer:

a. Conservation uses, wildlife sanctuaries, nature preserves, forest preserves, fishing areas, and passive areas of parklands, provided that no impervious surfaces are created;

b. Recreational trails and greenways that are either unpaved or paved with pervious materials;

c. Education/scientific research that does not require any of the prohibited activities named in Section 6.6.1;

d. Stream restoration projects, facilities and activities, with prior approval of the Director;

e. Infrastructure features such as roads, bridges, storm drainage, stormwater management facilities, utilities, and boat launch ramps, provided that they adhere to the following standards:

   1) The width of the disturbance for the feature is the minimum required to allow for maintenance and access;

   2) The angle of the buffer crossing shall be perpendicular (with up to 15% deviation off perpendicular) to the stream in order to minimize clearing requirements; and,

   3) The number of buffer crossings is minimized, with no more than one crossing every one-thousand (1,000) linear feet. The Director has the authority to approve additional crossings if justified by traffic, safety, or access issues.

   4) Multiple driveway or private roadway crossings of a stream or a wetland within one development shall be prohibited, unless approved by the Director after the property owner has demonstrated that the development has been planned in such a manner that driveway and private roadway crossings have been minimized to the maximum extent possible;

Access areas for utilities (e.g., manholes) that are located in the buffer area are allowed. Access areas must be minimized to the maximum extent possible, and shall be located no less than every 300 feet unless warranted by valid safety, access or service issues. At a minimum, utility access areas shall be vegetated in accordance with the vegetative target defined for the outer zone of a stream buffer in Section 6.4.

6.6.3 Allowable Uses in the Outer Zone

The following uses are allowed in the outer zone of stream buffers and in buffers surrounding lakes and ponds:

a. All activities that are allowable in the inner zone (see above).

b. Yards, trails, greenways, picnic areas, and passive recreation areas as long as they do not have impervious surfaces. Passive recreation areas are defined as recreational activities that do not require hardened, impervious surfaces to be constructed, such as soccer fields without parking and other facilities, walking trails that are either unpaved or paved with permeable materials; bird watching; or hiking. Passive recreation areas do not include golf courses, ball fields that require the construction of impervious surfaces or the maintenance of open soil areas (such as baseball infields), picnic shelters or parking.

6.7 Allowable Disturbances

Disturbances are temporary actions in or encroachments into water quality buffers. From a practical standpoint, it is not always possible to maintain an undisturbed buffer. Disturbances are often necessary for the construction of infrastructure. Roadways, utilities, and other linear forms of development must cross streams, and water dependant structures such as marinas and docks must be built on water features. Allowances for these necessary uses of buffer areas exist (as presented in Section 6.6) as an approval that can be gained from the Department of Engineering and Public Works, rather than through the variance and appeals process. Knox County regulates disturbances for the inner and outer zones of water quality buffers as follows:

6.7.1 Inner Zone

The following disturbances are permitted in the inner zone of stream buffers and the wetland buffer:

a. Limited disturbances to remove and/or plant trees or vegetation, as required to maintain the overall health of vegetation in the buffer area. The pruning of native vegetation is allowed provided that the health and function of the vegetation is not compromised. However, only the individual removal of under-story nuisance vegetation (i.e. honeysuckle, kudzu, privet) and/or non-native vegetation which causes minimal soil disturbance is permitted. On land where the removal of such vegetation would cause a reduction in the amount of stream canopy by 50% or more, revegetation with native plants is required to the cover of the previous canopy at a minimum. For areas where such vegetation removal would cause a reduction in the amount of streambank vegetation, revegetation with native plants is required to return the amount of vegetative cover to its previous state, at a minimum.

b. Removal of individual trees that are in danger of falling, causing damage to dwellings or other structures, are dead or diseased, or have been heavily damaged by storms. The root wad or stump should be left in place, where feasible, to maintain soil stability.

c. Disturbances necessary for the construction of utility access areas and approved stream crossings.

d. Disturbances as required to establish and/or restore buffer areas in accordance with an approved Buffer Enhancement Plan. Section 6.8 provides detailed information on buffer enhancement.

6.7.2 Outer Zone

The following disturbances are allowed in the outer zone of stream buffers and in buffers surrounding lakes and ponds:

a. Clearing, grubbing, grading, and revegetation in accordance with an approved stormwater management plan.

b. Disturbances necessary for the construction of utility access areas and approved stream crossings.

c. Ongoing vegetation maintenance activities such as mowing, bush-hogging, and weed-eating. No chemical applications are allowed in the outer zone.

6.8 Protection of Water Quality Buffers

In order to establish and protect the infiltration, filtration, and stream corridor protection functions of water quality buffers, buffers must be protected prior to, during, and perpetually after construction. Water quality buffer areas must remain protected from land disturbance, vegetation removal, construction of impervious surfaces, and discharges of sediment and other construction-related wastes during development activities. This section discusses methods of buffer protection during construction and after construction activities have been completed.

6.8.1 During Construction

Knox County requires that the following steps be taken during the site plan development and site construction process to protect water quality buffers during construction:

a. Water quality buffers must be clearly identified on all stormwater management plans and construction drawings and marked with the statement "Water Quality Buffer. Do Not Disturb".

b. Water quality buffers cannot be encroached upon or disturbed during project construction, unless they are being established, restored, or enhanced in accordance with an approved Buffer Enhancement Plan.

c. Water quality buffers must be clearly marked or staked at frequent intervals on the property at the time of the pre-construction conference and the marking maintained until completion of construction activities. All contractors and others working on the construction site must be made aware of the existence of the buffer(s) and the restrictions on disturbing the buffer(s).

d. All areas of the water quality buffer, including streambanks, must be left in a stabilized condition upon completion of construction activities. No actively eroding bare or unstable stream banks shall remain, unless approved by the Director. Placement of riprap and other hard armor is only allowed when bioengineering alternatives are not feasible.

6.8.2 After Construction

Once construction has ceased on a project, water quality buffers must still be maintained in accordance with the recorded Covenants for Maintenance of Stormwater Facilities and Best Management Practices. In order to provide for long-term protection and maintenance, Knox County requires that the water quality buffer be protected in perpetuity by placing the buffer in a permanent water quality or other easement that is recorded with the property's deed. If the area is not publicly owned, the easement should be held by one of the following non-governmental entities:

1. A viable third party such as a land trust, land management company or utility. The purpose of the third party is to provide monitoring and oversight to ensure the perpetual protection of the area in accordance with the requirements of a buffer area. The organization shall:

    a) have the legal authority to accept and maintain such easements;

    b) be bona fide and in perpetual existence;

c) have conveyance instruments that contain an appropriate provision for re-transfer in the event the organization becomes unable to carry-out it functions.

2. A homeowners association (HOA), provided that the following criteria are met:

   a) Membership in the HOA is mandatory and automatic for all homeowners of the subdivision and their successors.

   b) The HOA shall have lien authority to ensure the collection of dues from all members.

   c) The HOA assumes the responsibility for protecting, monitoring and maintaining the area as a conservation easement, in perpetuity.

If neither of the above-stated non-governmental entities is able to provide perpetual protection of the buffer area, then the property owner must assume responsibility for the maintenance and protection for the buffer area.

6.8.3 Signage

Permanent boundary marker signs may be required prior to recording the final plat or issuance of a Certificate of Occupancy to ensure that adjacent property owners are aware of the buffer. Further, the Director has the authority to require replacement of such boundary markers that have been removed or destroyed. The following general policies shall apply to buffer boundary markers:

1. Generally, buffer boundary markers must be located on the lot lines at the intersection of the landward edge of the buffer, and at other locations which will approximately delineate the buffer boundary. For single lot site developments, markers, if required, shall be posted every 100 feet along the buffer boundary. For subdivisions where multiple lots are located along the buffer, it is recommended that a buffer boundary marker be located at the intersection of every other lot line with the landward edge of the buffer.

2. Buffer boundary markers shall include the statement “Water Quality Buffer – Do Not Disturb”.

3. Where possible, the markers should be mounted to a tree larger than three (3) inches in diameter. Where it is not possible to mount the marker to a tree, a treated wood, metal, or plastic signpost must be used. The post must extend below the ground surface at least twenty-four (24) inches.

4. The boundary markers must be mounted between four (4) and six (6) feet above the ground surface.

The Knox County Engineering can provide guidance on obtaining the appropriate signage.

6.9 Level Spreaders

Level spreaders are structures that are designed to dissipate energy of concentrated flow and distribute it as sheet flow over a large surface area. For water quality buffers, they are used to maintain the function of buffers by transitioning concentrated flows of stormwater runoff into sheet flow. Water quality buffers are most effective when shallow sheet flow is discharged to them. This creates a shallow flow that has a high surface contact area, increasing infiltration and the effectiveness of filtration. In contrast, concentrated flow can cause erosion in the buffer area, and limits the effectiveness of plants to filter-out pollutants.

Design instructions and schematics for level spreaders are found in Chapter 7 of this manual.

Level spreaders are required where concentrated flows would discharge overland through the buffer. Sheet flow must be ensured through the buffer for the stream and vegetated buffer credit to be granted. Flows encountered for storms greater than the 1-yr can be piped beneath the buffer to the stream, so long as proper outfall protection is employed and channel protection and peak flow control criteria have been met. Level spreaders are simple structures that consist of the following elements:

- A pipe, ditch, or swale through which concentrated flow enters the spreader;
- An energy dissipater that slows the water;
- A level lip provided by the construction of a berm, concrete chute, or other permanent material or a shallow linear trench. The purpose of this component is to distribute runoff perpendicularly over the lip at the same depth for the entire length of the structure.

6.10 Conflicts with State Requirements

The State of Tennessee may require water quality buffers during construction activities via provisions contained in the Tennessee Construction General Permit (CGP) or other regulatory permits and processes. The State's requirements may, or may not, align with Knox County's requirements and policies for water quality buffers. It is the responsibility of the site developer to be informed and educated on any State-level buffer requirements. If a site developer intends to apply Knox County's buffer requirements in lieu of any requirements of the State of Tennessee, the developer must first obtain approval from TDEC and provide Knox County with written documentation of such approval.

References

Shueler, T. *Vegetated Riparian Buffers and Buffer Ordinances.* SCDEC, Summer, 1995.

This page left intentionally blank

Table of Contents

CHAPTER 7 — STORMWATER DRAINAGE DESIGN

Chapter 7 - List of Figures

Chapter 7 - List of Tables

This page left intentionally blank

Table of Contents

CHAPTER 8 — FLOODPLAINS AND SINKHOLES

Chapter 8 - List of Tables

Table of Contents

CHAPTER 8 — FLOODPLAINS AND SINKHOLES

Chapter 8 - List of Tables

CHAPTER

8

FLOODPLAINS AND SINKHOLES

8.1 Introduction

Floodplain management is a decision making process that, when combined with appropriate land use regulations, is implemented in order to determine the most appropriate use(s) of local floodplains. The decision making process for floodplain management hinges on balance between these three concerns: 1) environmental quality, including preservation of natural and beneficial floodplain functions, minimizing stream modifications and protecting stream water quality; 2) economic development where necessary and appropriate; and 3) health and safety including minimizing risk to human life and risk of property damage.

Many natural floodplains have been affected by grading encroachments, bridges, box culverts and restricted channels. Historically, people settled near creeks and streams for uses such as transportation, water supply, agriculture, etc. Acceptable limits for flood risks were then determined mainly by trial and error and personal experience and judgment. Proper use or preservation of the natural floodplain can minimize the extent of flood damage, reduce stream flow velocities, reduce stream bank erosion, improve stormwater quality, provide for wildlife habitat, and provide recreational opportunities.

Knox County's floodplain management program is largely driven by two programs administered by the Federal Emergency Management Agency (FEMA): the National Flood Insurance Program (NFIP), and the Community Rating System (CRS) Program. These programs are described in detail in section 8.2. Knox County's regulations and policies for development in and around sinkholes are discussed in detail in section 8.5. It should be noted that Knox County's participation in the NFIP does not require regulation of stormwater in and around sinkholes.

Definitions for key terms utilized in this chapter (e.g., floodplain, floodway, flood fringe) are included in Volume 1, Appendix B of this manual.

8.2 Applicable Federal Regulations

8.2.1 National Flood Insurance Program

Congress created the NFIP through the National Flood Insurance Act of 1968 to respond to rising costs of taxpayer-funded flood disaster relief and increased flooding damages. The Flood Disaster Protection Act of 1973, the National Flood Insurance Reform Act (NFIRA) of 1994 and other legislation helped broaden the scope of the NFIP. The 1994 NFIRA provided mitigation insurance and a grant program for state and community flood mitigation project planning and established the Community Rating System (CRS), an incentive program for NFIP communities.

The NFIP uses two main mechanisms for achieving its cost-reduction objective: (1) transferring costs for flooding losses to floodplain occupants through insurance policy premiums; and (2) moving future development away from flood hazard areas as appropriate, or requiring flood-resistant design and construction in flood-prone areas. Before the NFIP was established, flood insurance was typically not available from private insurers and most communities did not regulate floodplain development.

Participation in the NFIP is voluntary. Approximately 20,000 communities in the United States and its territories participate in the program. Participation requires the adoption and enforcement of floodplain management ordinances that include minimum floodplain management standards, such as designation of a 100-year floodplain and determination of flood elevations, which have been established by FEMA. Property owners that are located in communities that participate in the NFIP can purchase federally supported flood insurance. Communities that do not participate in the NFIP can be denied access to federal disaster funding, and property owners in such communities may be denied federal home loans, have relatively higher flood insurance rates, or may not even be able to obtain flood insurance.

In keeping with the objectives of the NFIP, Knox County's floodplain management program has the cornerstone objectives of ensuring that flood levels in local (NFIP regulated) streams are not increased, that public and private flood losses are minimized, and that natural and beneficial values of floodplains are preserved. This is accomplished through a combination of:

1. Restricting or prohibiting land uses in floodplains which are dangerous to health, safety and property due to water or erosion hazards, or which result in damaging increases in erosion or in flood heights or velocities;
2. Requiring that land uses vulnerable to floods, including facilities that serve such uses, be protected against flood damage at the time of initial construction;
3. Administering regulations with regards to controlling the alteration of natural floodplains, stream channels and natural protective barriers which are involved in the accommodation of flood waters;
4. Administering regulations with regards to controlling filling, grading, dredging and other development which may increase flood damage erosion;
5. Preventing or regulating the construction of flood barriers which will unnaturally divert flood waters or which may increase flood hazards to other lands; and
6. Seeking ways to reduce loss of natural floodplain areas and enhance natural benefits of floodplains in areas facing development.

Knox County's floodplain management program includes several types of activities undertaken by the County and others to manage floodplain areas. These activities are shown in Table 8-1.

The procedures outlined in this manual and mandated by the Knox County Stormwater Management Ordinance, Zoning Ordinance, and the Knox County Flood Damage Prevention Ordinance are in keeping with these key program goals. In fact, Knox County's floodplain management program performs to a higher standard than that required by FEMA of NFIP participant communities. This higher standard has enabled the County to also participate in the CRS Program.

8.2.2 Community Rating System

Over 1,000 NFIP communities receive flood insurance premium discounts through the CRS Program, a points-based incentive program where communities can receive credit for exceeding the minimum floodplain management methods required for NFIP participation. Each participating community has the opportunity to accrue points to qualify for reduced flood insurance rates for the local flood insurance policyholders. The CRS system is divided into classes, with larger discounts on flood insurance premium rates for communities with higher point totals. There are 18 creditable activities in four categories where communities can earn points within the CRS (shown in Table 8-2). Knox County participates in the CRS, and local flood insurance policyholders in unincorporated portions of the County receive discounted flood insurance premiums.

Table 8-1. Examples of Local Floodplain Management Program Activities

Type of Activity	Who Performs Activity	Activity Examples
Preventative activities	Knox County Engineering Knox County Commission Metropolitan Planning Commission Other agencies including TVA	Floodplain development and stormwater management permitting; stormwater drainage system maintenance; planning and zoning; floodplain regulations; buffer requirement regulations
Property protection	Property Owners (typically performed on a building-by-building or parcel basis)	Relocation; acquisition; building elevation; floodproofing; sewer backup protection; insurance
Natural resource protection	Knox County Engineering Knox County Parks & Recreation Other local agencies and utilities Special interest organizations Property owners and developers	Streambank restoration; wetlands protection; buffer zone implementation or restoration; erosion prevention and sediment control
Emergency services	Knox County Emergency Management Staff and other affiliated agencies	Flood warning; flood response; critical facilities protection; health and safety; maintenance
Structural projects	Knox County Engineering Other agencies including TVA Property owners and developers	Channel modifications; storm sewers; reservoirs; levees; detention ponds
Public information	Knox County Engineering Metropolitan Planning Commission	Outreach projects; technical assistance; environmental education; library

Table 8-2. Summary of CRS Program Credit Activities by Category

Category	Credit Activities
Public Information Activities	1) Maintain new construction elevation certificates 2) Provide and publicize map information service to identify property's FIRM zone 3) Outreach projects sending information to floodprone residents or all community residents 4) Hazard disclosure by real estate agents to potential purchaser of floodprone property 5) Maintaining flood protection information in public library 6) Provide and publicize service for property owner technical advice about protecting their buildings from flooding.

Category	Credit Activities
Mapping and Regulator Activities	1) Develop additional flood data 2) Open space preservation 3) Higher regulatory standards 4) Flood data maintenance 5) Stormwater management regulations to require new development to ensure that post-development runoff is no worse than pre-development runoff
Flood Damage Reduction Activities	1) Comprehensive plan and standard planning process for floodplain management 2) Acquire and/or relocate floodprone buildings to locations outside floodplain 3) Document flood protection (floodproofing or elevated) pre-FIRM buildings 4) Periodic inspections and maintenance as needed for channels and retention basins in drainage system
Flood Preparedness Activities	1) Provide early public flood warnings and have detailed flood response plan 2) Maintain levees not credited with providing base flood protection 3) Communities in a state with an approved dam safety program receive dam safety credit

8.3 Local Floodplain Management Tools

8.3.1 FEMA Flood Insurance Studies

The Knox County FIS and associated FBFMs and FIRMs show floodplain and floodway information for the streams in Knox County which have regulatory floodplains. The most recent FIS can be obtained for a nominal fee from FEMA (http://www.msc.fema.gov). The FIS can also be viewed and photocopied at the MPC Library in the City/County Building.

The FIS contains historical and background data on local streams, as well as a summary of technical data that was used to develop flood elevations, floodplain boundaries, and floodways. Two main features of the FIS that may be helpful to developers and site design engineers are Floodway Data Tables and Flood Profiles. For each stream in the FIS, the Floodway Data Tables present the distance along the stream at selected cross-sections, and the corresponding flood elevation, floodway width, mean velocity and section area. The Flood Profiles are graphs that present the water surface elevation along the studied portion of the stream for each flood event that was included in the FIS (the 100-year elevation at a minimum). The stream bed profile, cross-sections, road and other stream crossings, and stream confluences are also shown.

FIRMs depict the boundaries of SFHAs (i.e., 100-year floodplains), 500-year floodplains and approximate zones. FIRM maps typically have a zone designation associated with three areas, as designated by FEMA: 1) SFHAs; 2) other flood areas; and, 3) other areas. Zones within each area designation that are commonly shown on Knox County FIRMs are presented in Table 8-3, along with a general description of the information that is available for each zone. The developer or site designer should consult with the appropriate FIRM map for the exact definition of each zone.

The FBFM mapping presents the boundaries of the floodway, 100-year floodplain and 500-year floodplain. FBFMs also show the locations of selected hydraulic stream-cross sections that were used to develop the flood elevations and floodway widths.

8.3.1.1 FEMA Map Amendment and Revision Process

FEMA has established procedures for map amendments and map revisions for studied areas in which a property owner wishes to alter or encroach into a regulatory floodplain. Map amendments typically update the mapping by letter only; and therefore, a new FIS, FIRMs and FBFMs are not typically published. A list of the map amendments and revisions that are available through FEMA are presented in Table 8-4.

Table 8-3. Common Knox County Flood Insurance Rate Map Zones

FIRM Zone	General Description
Special Flood Hazard Areas (SFHA)	
AE, AH, AO, AR	Areas of the 100-year flood; BFEs and flood hazard factors determined.
A, A99	Areas of the 100-year flood; BFEs and flood hazard factors not determined.
Other Flood Areas	
X (shaded)	Areas of the 500-year flood. Areas of the 100-year flood with shallow flooding or other mitigating circumstances. BFEs and flood hazard factors determined.
Other Areas	
X (unshaded), D	Areas outside the 500-year floodplain. Flood hazards are undetermined.

Table 8-4. FEMA Map Amendments and Revisions

Letter Name	Letter Summary
Conditional Letter of Map Amendment (**CLOMA**)	A letter from FEMA stating that a proposed structure that is not to be elevated by fill (natural grade) would not be inundated by the base flood if built as proposed.
Letter of Map Amendment (**LOMA**)	A letter from FEMA stating that an existing structure or parcel of land that has not been elevated by fill (natural grade) would not be inundated by the base flood.
Conditional Letter of Map Revision Based on Fill (**CLOMR-F**)	A letter from FEMA stating that a parcel of land or proposed structure that will be elevated by fill would not be inundated by the base flood if fill is placed on the parcel as proposed or the structure is built as proposed.
Letter of Map Revision Based on Fill (**LOMR-F**)	A letter from FEMA stating that an existing structure or parcel of land that has been elevated by fill would not be inundated by the base flood.
Conditional Letter of Map Revision (**CLOMR**)	A letter from FEMA commenting on whether a proposed project, if built as proposed, would meet minimum NFIP standards.
Letter of Map Revision (**LOMR**)	A letter from FEMA officially revising the current NFIP map to show changes to floodplains, floodways, or flood elevations outlined in 44 CFR Part 65.

Map amendments and revisions are administered by FEMA. More information on this process and application forms for map amendments and revisions can be found at FEMA's website: www.fema.gov. The property owner or an agent of the owner is responsible for obtaining any required engineering or other services for a proposed map amendment or map revision.

Map amendments and map revisions require approvals from both FEMA and Knox County. Any encroachment or alteration of regulatory floodplains must be performed in accordance with the Knox County Stormwater Management Ordinance and Flood Damage Prevention Ordinance.

8.3.2 Knox County Engineering Floodplain Mapping

Knox County may have additional SFHA, floodplain and floodway information for streams in the FIS which were in need of improved flood management information, and for a number of streams that are not included in the FIS. This information may be used to regulate the development and redevelopment in floodplains in the unincorporated areas of Knox County in the same way that the FEMA-published data is regulated. Further, it should be noted the FIS, FBFMs and FIRMs **do not** include a depiction of the "no fill" boundaries. The location of no-fill boundaries is established by the Knox County Stormwater Management Ordinance, and has been mapped by Knox County Engineering for all studied streams. Floodplain, floodway, and no-fill boundary lines for all streams studied by Knox County are available from Knox County Engineering. Floodplain and floodway lines are available from the Knoxville, Knox County, Knoxville Utilities Board Geographic Information System (KGIS) (www.kgis.org).

It is the responsibility of the property owner or developer to obtain the appropriate regulatory floodplain management data for his/her property. When in doubt, Knox County Engineering should be consulted to determine if regulatory floodplain information is available. Data not included in the most recent FIS can be obtained from Knox County Engineering.

8.3.3 Stormwater Master Plans

A stormwater master plan is an engineering and planning study of a watershed and its stream network. Master plans can be used to regulate new development and redevelopment, address flooding (and sometimes water quality) problems, plan and prioritize stormwater capital improvements, and examine the potential stormwater flooding and quality impacts of future land use patterns and/or regulatory requirements. In some situations, the hydrologic and hydraulic models used to develop a stormwater master plan can be used to determine the stormwater impact of a single development.

Knox County has developed stormwater master plans for several County watersheds. The Director can utilize the information provided by a stormwater master plan for regulatory purposes to limit adverse impacts of a proposed development. Knox County Engineering should be consulted prior to development of a stormwater management plan to determine if a master plan will affect floodplain requirements.

8.4 Floodplain Management Regulations

This section summarizes the local regulations that are used to administer the local Floodplain Management Program, and provides additional policies and procedural guidance where necessary. Knox County ordinance language is presented in this section to the degree needed to allow an understanding of the supportive policy and technical guidance. The reader should consult the Knox County Stormwater Management Ordinance and the Flood Damage Prevention Ordinance for the complete floodplain management regulations. Both ordinances are provided in Volume 1, Appendix A and are discussed in more detail in Volume 1, Chapter 3 of this manual.

8.4.1 Summary of Ordinance Requirements for Development in Floodplains

Table 8-5 presents a summary of the floodplain development requirements stated in the Knox County Stormwater Management Ordinance. The requirements are separated into three categories, depending upon the location of the new development (or redevelopment).

8.4.2 Requirements for Land Use and Land Disturbance

The land use of areas located in floodway, no-fill, and floodplain areas are regulated by the Knox County Zoning Ordinance, as follows: uses permitted within the flood fringe shall be in accordance with Article 3.70, Flood Fringe Requirements; and uses permitted within the floodway shall be in accordance with Article 5.70, Floodway Zone. The reader should consult the Zoning Ordinance for specific requirements regarding zoning, structure placement, and land use planning for developments proposed in floodplain areas.

Table 8-5. Summary of Floodplain Development Requirements

Location of Development	Available Data	Summary of Requirements
AE, AH, AO, or AR, shaded X Zones	100-year floodplain elevations 500-year floodplain elevations (i.e., base flood elevations) Floodway widths No-fill boundaries	1. Floodplain Development Permit 2. FEMA Elevation Certificate for insurable structures 3. Floodway encroachments are prohibited. 4. Construction fill inside the no-fill boundary is prohibited. 5. An engineering study is required for exceptions to items 3 and 4 above. 6. A CLOMR and LOMR are required for modification of a floodway boundary.
A and A99 Zones	Approximate 100-year floodplain boundaries; no base flood elevations or floodway widths determined.	1. Floodplain Development Permit 2. FEMA Elevation Certificate for insurable structures 3. Developments greater than 40 lots or 10 acres require the determination of accurate floodplain, floodway, BFE and no-fill boundary data and conformance with requirements for AE zones. 4. For all other developments, encroachments are prohibited within a specified distance of the streambank. 5. An engineering study is required for an exception to item 4 above.
Unshaded X or D Zones adjacent to unstudied streams	No data available	1. Developments greater than 40 lots or 10 acres and adjacent to a stream that has an upstream drainage basin of 1mi^2 or greater require the determination of accurate floodplain, floodway, BFE and no-fill boundary data, and conformance with requirements for AE zones. 2. For all other developments, encroachments are prohibited within a specified distance of the streambank.

Note: the Zoning Ordinance's definition for "floodway zone" encompasses the entire 500-year floodplain. It is important that the use by MPC of the term "floodway" for this zone not be confused with the regulatory definition for floodway that has been established by FEMA, is utilized by Knox County Engineering, and is defined in section 8.2 above. For purposes of floodplain regulation and management by Knox County Engineering, "floodway" specifically refers to a regulatory width across a stream that is calculated for the 100-year storm event. In Knox County, there is no regulatory 500-year floodway.

Knox County Engineering allows open space land uses and land disturbances within the no-fill boundary, so long as the land use does not alter, fill, or cause an encroachment. Further, land uses and disturbances must be in keeping with Knox County's requirements for water quality buffers where the regulated floodplain areas and water quality buffer areas overlap. Where water quality buffer and floodplain management requirements conflict or overlap, that provision which is more restrictive or imposes higher standards or requirements shall prevail. More information on water quality buffer requirements can be found in Volume 2, Chapter 6 of this manual.

8.4.3 Requirements for Engineering Studies

Engineering studies performed to develop floodplain, floodway, BFE and no-fill boundaries where required shall be performed by a registered professional engineer in the State of Tennessee, and shall be prepared and reported in accordance with FEMA standards. Such standards are documented in the FEMA publication *Guidelines and Specifications for Flood Hazard Mapping Partners*, which can be found at the following website: http://www.fema.gov.

Both hydrologic (i.e., peak discharge) and hydraulic (i.e., floodplain elevation and floodway width) computations shall be performed to develop the required floodplain data for a given stream. Requirements for hydrologic and hydraulic models are presented in the next two sections.

8.4.3.1 Hydrologic Modeling – Determining Peak Discharges

Peak discharges shall be determined for the 2-year, 10-year, 25-year, 100-year and 500-year storm events. The peak discharges shall be determined for pre-development and post-development land use conditions, and shall be utilized as input to the hydraulic model that is used to determine floodplain elevations and other floodplain data.

Four methods are considered acceptable for determining peak discharges for the purpose of floodplain data development, as follows:

1. gage analysis using statistical hydrologic methods;
2. Clark Unit Hydrograph;
3. the use of Tennessee Valley Authority (TVA) regression equations; and,
4. the use of a hydrologic model, such as USACE's HEC-1 or HEC-HMS.

The Director shall select the method that will be used on a case-by-case basis. The selection is based on the availability of existing hydrologic models and/or gage data in the same area as the proposed development, a history of flooding in the same area as, or downstream of, the proposed development, FEMA standards and requirements, and engineering judgment. Each method is discussed briefly below.

Where the Director requires gage analysis using statistical hydrologic methods, peak discharges shall be determined using methods that are widely known and accepted, such as Gumbel extreme values or Log-Pearson Type III distributions. Such methods are appropriate for flood flow frequency analysis when suitable gage data is available with many years of record. Longer gage data records improve the peak flow estimates.

Where the Director requires the use of regression equations, the Director shall specify the set of equations required for analysis. Regression equations appropriate for rural and urbanized areas in East Tennessee have been developed by TVA. Detailed information on the use of regression equations to determine peak discharges is presented in Volume 2, Chapter 3 of this manual.

In the event that the Director requires the use of a hydrologic model, the United States Army Corps of Engineers (USACE) HEC-1 or HEC-HMS software shall be used. Both models and manuals are available for download at the following website: http://www.hec.usace.army.mil. The use of these models requires the division of the upstream drainage basin(s) based on topographic drainage divides, and/or an understanding of the basin organization and connectivity of a hydrologic model that may already exist for the area of proposed development. Knox County Engineering can provide input data and guidance for existing HEC-1 or HEC-HMS models.

For all HEC-1 and HEC-HMS models, Soil Conservation Service (SCS) curve numbers, the SCS time-of-concentration/lag time and Clark unit hydrograph methods shall be utilized to predict watershed response and generate design storm hydrographs and peak discharges. Channel routing shall be performed using Muskingum-Cunge routing techniques, and modified Puls routing shall be utilized to route flood storage areas, such as in-stream and off-line ponds and upstream of culverts and bridges.

8.4.3.2 Hydraulic Modeling – Determining Floodplain Elevations and Boundaries

The USACE HEC-RAS hydraulic modeling software shall be used for all hydraulic computations on streams where a HEC-RAS model, or no model, previously exists. On streams where the most current model is the older USACE HEC-2 hydraulic modeling software, the Director may allow the continued use of the HEC-2 model as long as FEMA modeling requirements are satisfied. Persons responsible for hydraulic analysis shall consult with Knox County Engineering to determine if an existing model will be utilized, or a new one must be created. Both programs are available from the Hydrologic Engineering Center's website: http://www.hec.usace.army.mil.

Flood Elevation

Flood elevations shall be determined for the 100-year and 500-year storm events. The Director may also require the determination of flood elevations for the 2-year, 10-year, 25-year, and/or 50-year storm events. FEMA requires the use of valid and accurate flood elevation, land survey, topographic and other pertinent information for the study area. It is important to understand that survey of stream cross-sections may be required upstream and downstream of the proposed development in order to adequately model the study area. Guidelines for stream cross-section surveys are presented in the next section.

Floodway

Regulatory floodway widths shall be developed for the 100-year storm event. For floodway development, the surcharges between the 100-year flood elevation profile and the floodway profile shall be no less than zero, and no greater than the maximum allowable surcharge of one (1) foot. Floodways shall initially be determined using equal conveyance reduction for the floodplain on opposite sides of the stream. The engineer can then optimize the floodway width to the narrowest width allowed within the surcharge limits, but without abrupt changes in width. Floodway boundaries that appear as a zig-zag or accordion-like will not be accepted unless the engineer can provide sufficient evidence to show that a more gradual and smooth boundary could not be obtained.

Floodway boundaries shall not be located outside of the 100-year floodplain boundary. For tributary streams, the regulatory floodway on the tributary shall be calculated without consideration of backwater flood elevations from the larger stream.

Base Flood Elevation

Base flood elevations shall be determined from the 500-year floodplain elevation, in accordance with the Knox County Stormwater Management Ordinance.

Flood Protection Elevation

Flood protection elevations shall be determined from the 500-year floodplain elevation. Where a higher backwater flood elevation (for the 500-year event) exists and at stream and tributary confluences, the backwater elevation shall prevail. If the future 100-yr floodplain elevation is higher than the current 500-yr elevation, then the future 100-yr elevation shall be used.

No-Fill Boundary

The "no fill' boundary shall be located at one-half the linear distance between the floodway boundary and the 100-year floodplain boundary. This measurement is relatively easy along most of the stream's length, where the floodway and floodplain are approximately parallel. In backwater areas (e.g., confluences with tributaries, ditches and channels) where these boundaries are not parallel, the method for delineating the no-fill boundary is as follows.

1. Delineate the incoming ditch or channel in the backwater area to the 100-year floodplain boundary.
2. Measure the distance along the ditch or channel from the floodway boundary to the 100-year floodplain boundary.
3. Mark the point that is halfway along the distance measured, and determine the topographic elevation at that point.
4. The no-fill boundary in the backwater area shall be drawn at (i.e., shall follow) the topographic elevation of the halfway point until the floodway and the 100-year floodplain boundary become approximately parallel to each other. At this point, the no-fill line shall be located at one-half the linear distance between the floodway boundary and the 100-year floodplain boundary.

8.4.3.3 Stream Cross-section Survey Guidelines

Determining floodplain and floodway data in accordance with FEMA standards often requires the collection of field survey data for stream cross-sections located along the stream reach, including cross-sections at bridge and culvert channel crossings. Field surveys shall be accomplished by differential leveling or differential GPS methods, with vertical error tolerances of 0.5 feet across the 100-year floodplain. Cross-section elevations and stations shall be determined at those points along the study reach that represent significant breaks in the ground slope, and at changes in the hydraulic characteristics of the floodplain. Stream cross-section survey data may be supplemented by KGIS topography in areas at least eight (8) feet vertically above the top of bank elevation.

Engineers responsible for the hydraulic analysis shall locate cross sections to accurately represent the stream reach. The cross-section through the stream channel shall be perpendicular to the channel from top-of-bank to top-of-bank. The cross-section shall be extended perpendicular to channel/floodplain slope to a point at least eight (8) feet vertically above the top-of-bank elevation. Where the channel is not aligned with the valley slope, these sections may be crooked (i.e., dog-legged) so that the flow direction is essentially perpendicular to the cross-section alignment. Elevation and location information shall be collected at each of the following locations on the cross-section: top of bank; toe of slope; stream invert; all edges of water; and all major changes of grade. Distance between survey points shall not exceed 100 feet.

For roadway and driveway crossings, the HEC-RAS and HEC-2 models require surveyed cross-sections for the roadway or driveway profile, as well as at the upstream face and downstream face of the structure. For level roadways, the roadway profile survey shall be collected down the middle of the road. If the roadway is super-elevated, the roadway profile survey shall be collected on the highest side (i.e., the side above which the road would be completely overtopped by a flood). If the crossing has a solid rail, also survey along the top-of-rail. For each face cross-section, the horizontal and vertical position of the following locations shall be surveyed:

- Tops of curb;
- Top of wingwalls at the junction with the upstream face and all changes in vertical or horizontal alignment (only required at upstream face);
- Low chord at the face of the abutments and at each substructure (e.g., culvert) where the depth to low chord varies;
- Ground at the face of the abutments (if footing is exposed, define shape of footing);

- Top of banks;
- Toe of slopes;
- All changes of grade;
- Edges of water;
- Stream invert; and
- Face of each substructure (if footing is exposed, define shape of footing).

For culverts, the survey shall obtain at a minimum: low chord (at top of each culvert), culvert invert, stream invert, culvert size and type (e.g., pipe arch, CMP, concrete box, etc.).

8.4.3.4 Reporting Requirements and Guidelines

Because the data provided in the engineering study may be submitted to FEMA, floodplain and floodway data shall be reported in accordance with FEMA standards. The Director may establish additional reporting requirements when deemed necessary to adequately document the flood study preparation and results. Typical items included with flood study reports include the following:

- Floodplain Study Report (see below);
- Digital copies of all models used;
- Scaled topographic maps that shows model cross-section locations, proposed structure locations and finished floor elevations;
- Stream profiles showing water surface elevations, stream bed, bridge and culvert crossings and selected cross-section locations;
- Support information for peak flow determination such as a map with watershed boundary and subareas or subbasins delineated;
- All input data used for computing water surface profiles in printed and electronic format (if applicable). Examples would be the HEC-2 and HEC-RAS input files;
- Summary output tables and charts showing computed water surface elevations, energy grade lines, velocities, cross-sectional flow areas, etc; and,
- Scaled flood elevation/boundary maps prepared in accordance with FEMA map standards.

At a minimum, flood elevation/boundary maps submitted to Knox County Engineering must include the following data for the study reach:

- pre-development topographic contours (2 to 4 ft intervals as available from KGIS);
- post-development topographic contours (1 to 2 ft intervals);
- the most current planimetric (e.g., roads, houses, streams etc.) information (field topography or KGIS planimetric information, if current);
- cross-section locations (labeled as named in hydraulic model) and river mile or distance;
- the 100-year floodplain boundary;
- base flood elevations;
- the 500-year floodplain boundary; and,
- the floodway boundary.

A second map (or set of maps) shall be prepared with all of the data described above and the no-fill boundary. FEMA does not regulate the no-fill boundary, therefore, two sets of maps are required (one with and one without the no-fill boundary).

Flood elevation/boundary maps can be submitted on the concept or site plan if desired, but must be "to scale" at least at 1"=200' or 1"=400', in Microstation (*.dgn) format, or other format suitable for use by KGIS. It is the responsibility of the person submitting the engineering study to consult with KGIS on a suitable format prior to submittal of the information to Knox County Engineering and/or KGIS.

Each engineering study shall include a Floodplain Study Report. At a minimum, the Floodplain Study Report shall be formatted and include the sections and information listed below. The Director may require information as necessary. Floodplain Study Reports shall be submitted with the Stormwater Management Plan for the proposed development.

I. General Description
   A. Site location
   B. Study limits
   C. Existing conditions
   D. Proposed conditions
   E. Boundary conditions
   F. Special considerations and assumptions
II. Hydrology (Peak Discharge Calculation)
   A. Description of methodology
   B. Peak discharge calculation locations
   C. Drainage basin delineation
   D. Assumptions
III. Hydraulics (Flood Data Calculation)
   A. Description of methodology
   B. Assumptions
IV. Results
   A. Summary output tables (as described above)
   B. Maps and profiles (as described below)
V. Supporting Data

8.4.4 Requirements for Construction in Floodplains

Construction requirements for structures located in floodplains are presented in the Knox County Stormwater Management Ordinance, Zoning Ordinance, and the Flood Damage Prevention Ordinance. Some construction requirements differ depending upon the type of construction (i.e., new residential, improved existing residential, manufactured homes, and non-residential); therefore, the person(s) responsible for construction should be sure to: 1) consult both ordinances; and 2) ensure conformance with the regulations that are appropriate with the type of construction being proposed. The Flood Damage Prevention Ordinance also contains provisions that address the construction of water supply and wastewater disposal systems (i.e., sanitary sewers and septic tanks) in floodplain areas.

It is important to note that conformance with Knox County's construction requirements for structures located in floodplains is required for all construction in floodplains, not just those that require submittal of a Stormwater Management Plan and/or approval of a building permit.

8.5 Regulations for Development in Sinkhole Areas

8.5.1 Background

In karst areas, such as Knox County, the surface drainage is significantly influenced by the hydrogeologic flow regime. Stormwater runoff is directed into the sinkholes, transmitted through the ground-water system through a network of interconnected conduits and finally discharged at resurgent locations (springs). If the stormwater runoff exceeds the capacity or rate of discharge of the sinkhole, flooding is probable.

Flooding in sinkhole areas can be caused by a number of factors. First, a lack of other surface drainage features in the area (e.g., streams) result in sinkholes and depressions serving as the primary drainage outlets for stormwater runoff. Developments that drain to these areas increase the volume of runoff that the sinkholes must handle, and often decrease the storage capacity of the sinkhole. Second, construction practices that may alter the karst terrain, such as sinkhole filling, blasting, and inadequate erosion and sediment control, can increase the potential for flooding. Partial or total filling of a sinkhole reduces the volume of surface storage available to contain stormwater runoff. Blasting can change the geometry of sinkhole throats and underground caves, blocking outflow pathways. Finally, a lack of adequate erosion and sediment control measures can increase the potential for silt and other debris to accumulate at sinkhole throats, effectively reducing the outflow efficiency. These factors, combined with seasonal hydrologic conditions such as saturated soils and periods of heavy rainfall, result in increased flooding potential in sinkhole areas and the propensity for geologic settlement (i.e., the appearance of previously unknown sinkholes) in sinkhole areas.

Knox County has significant areas that are characterized by karst geological formations, and therefore, have a high potential for the existence of sinkholes. In some areas, the presence of sinkholes is very evident. This is true in the Ten Mile Creek watershed near Cedar Bluff and Dutchtown Roads, where features such as rock outcrops, large enclosed depressions and rock-lined throats are numerous and easily seen. However, other sinkhole areas are not always so easily identified. Natural, gradual depressions can indicate the presence of sinkholes and the developer should be aware of regulations and policies that may apply to such depressions.

8.5.2 Regulations and Policies

The Knox County Stormwater Management Ordinance contains a number of stormwater and floodplain management requirements for developments that are located near and/or drain to sinkholes, which are summarized below. Persons responsible for development in or near sinkholes should consult the ordinance for a full detail of sinkhole development requirements.

8.5.2.1 General

- Developments in and around sinkholes require submittal of a stormwater management plan. A Floodplain Development Permit is required if structures are developed in or near a sinkhole floodplain.
- Minimum standards for flood management around sinkholes are based on the limits determined by the water surface elevation of the 100-year frequency, 4-day duration storm event (7.8 inches), assuming plugged conditions (0 cfs outflow).
- Where flood management provisions stated in the Knox County Stormwater Management Ordinance cannot be satisfied, or where development or redevelopment is proposed in critical sinkhole watersheds, stormwater master planning, or sound engineering judgment, calculations must be provided that show total retention of the difference between the pre-developed and

post-developed 100-year, 24-hour design storm. These calculations must include the entire contributing watershed area for all designed retention basins and sinkholes. Calculations must also be provided that show that the post-developed 100-year, 24-hour peak flow rate does not exceed the pre-developed 100-year, 24-hour peak flow rate. In basins or subbasins with a documented historical drawdown time, it may be acceptable to assume drawdown if the documented value is at least 1.5 times larger than the drawdown time for the region. In general, advanced subsurface testing must be performed and certified by a professional engineer registered in the State of Tennessee with a demonstrated expertise in hydrogeology. Subsurface testing shall reasonably determine the range of outflows under a variety of design conditions.

- The Director has authority to condition the approval of a permit upon the compliance with additional requirements, including but not limited to measures to avoid and/or protect the sinkhole throat, detention, conveyance facilities, or other stormwater management solutions required to reduce the adverse impact of the proposed development on other properties or on the subject development.

Additional policies pertaining to stormwater and floodplain management in and around sinkholes are as follows:

1. The minimum extent of a sinkhole area is determined by the sinkhole floodplain elevation using at a minimum, the limits determined by the water surface elevation of the 100-year frequency, 4-day duration storm event (7.8 inches), assuming plugged conditions (0 cfs outflow). For developments that drain to sinkholes, the pre- and post-condition runoff volumes shall be calculated using the CN procedure for wet or Antecedent Runoff Condition III (ARC III). The 100-year flood elevations for some sinkholes in Knox County (largely in the Dutchtown Road area in west Knox County) have been mapped by FEMA, and therefore are considered regulatory floodplains and shall be managed as such. The determination of the 100-year floodplain is presented in Example 8-1.
2. No person shall place or cause to be placed any substances or objects, other than stormwater runoff, in any sinkhole or sinkhole drainage area in such a way so as to allow such substances or objects to be washed into a sinkhole throat during storm events.
3. All persons draining stormwater runoff into sinkholes should coordinate with the State of Tennessee to insure appropriate compliance with the applicable provisions of the State of Tennessee Rules of the Tennessee Department of Environment and Conservation Chapter 1200-4-6 (Underground Injection Control) for Class V injection wells. Copies of approved permits and any pertinent site-specific provisions shall be submitted to the County prior to the issuance of a Grading Permit.
4. Disturbance of the immediate area around a sinkhole during construction activities shall be minimized to as little as possible. The use of mechanized equipment near the sinkhole throat should be avoided. All use of explosives shall be in compliance with the State Fire Marshall's Office. The underground system of caves and streams is dynamic and explosions in the vicinity can alter or block underground drainage passages. Sinkhole areas are known to be unstable for construction and structures placed on soil foundations in sink areas may be subject to both settling and collapse of the sink.
5. Knox County Engineering requires geotechnical studies for structures located within fifty (50) feet of the highest hachured contour (also called the "rim") of a sinkhole.
6. Uncontrolled fill placement may present additional settlement hazards when fill is placed in or near sinkholes. Knox County *requires* that appropriate geotechnical studies be done and measures taken to insure structure foundations are designed to take into account potential sinkhole locations and instability. Such studies should account for potential foundation problems for both undisturbed sink areas and those previously filled by others.
7. Drainage and flood calculations for sinkhole areas shall be performed by a professional engineer registered in the State of Tennessee.

8. The property owner shall contact TDEC to determine if any wells and/or springs are located on the proposed development site. Known wells shall be shown on the stormwater management plan for the site. In addition, the stormwater management plan shall include the name and phone number for the TDEC official that was contacted with regards to known wells and/or springs located on the site, and the date of the inquiry to TDEC. A copy of any written correspondence with TDEC may be included with the stormwater management plan.
9. The Director has the authority to require preparation of a sinkhole drainage study for developments that require a stormwater management plan that drain to sinkholes, are located adjacent to sinkholes, or are within a sinkhole drainage area. The purposes of the study and the minimum items that shall be included in the sinkhole drainage study are detailed in Section 8.5.3.2 and Section 8.5.4.3
10. Filling, altering or obstructing a throat or outlet to a sinkhole or system of sinkholes, or springs or system of springs is prohibited, without prior approval of a grading permit. Any excavation in a sinkhole must be approved by the Director prior to approval of a grading permit.
11. Developers and site contractors shall employ measures that will protect sinkhole throats and outlets. These may include, but are not limited to, including installation of erosion prevention and sediment control measures, limiting blasting activities or avoiding such activities altogether, avoid grading that causes an increase in area draining to the sinkhole after development, and locating structures and other proposed development away from the throat. No person shall fill or obstruct the outlet to a sinkhole or system of sinkholes, or fill over a spring.
12. When encroachment into a sinkhole has been approved, the Director may impose a no-fill zone requirement in sinkholes that have an obvious throat, a history of flooding, and/or where engineering judgment suggests that such a requirement is necessary for proper management of the local stormwater system. In such cases, a no-fill line shall be established by the contour line or interpolated contour line for the elevation that defines sixty (60) percent of the floodplain storage volume. The area encompassed by this line shall be defined as the no-fill zone for all development activities. No construction fill will be allowed in this zone. Any fill added in the floodplain but outside the no-fill lines must be compensated for by an equal volume cut below the sinkhole floodplain elevation. An example of the calculation of the no-fill zone is presented below in Example 8-2.
13. The Director may approve a reduction in the sinkhole storage volume if engineering calculations prepared by a professional engineer registered in the State of Tennessee document that the floodplain elevation resulting from the 100-year frequency, 24-hour duration storm, assuming post-development site conditions and a plugged sinkhole outlet (0 cfs) will not flood any structures. The calculation of the 100-year flood elevation for a sinkhole is presented in Example 8-1.
14. The finished floor elevation (FFE) of any habitable structure adjacent to a sinkhole must be at least one foot above the established floodplain elevation. No structure shall be located within the hachured contours of a sinkhole.
15. The County encourages site designers to limit the impervious area of the site as much as possible. Efficient use of parking areas can reduce the impervious area and reduce the post-development runoff volumes. Pervious surfaces can be used for overflow parking areas that will be used infrequently. Impervious area reduction techniques and other Better Site Design measures are discussed in detail in Volume 2, Chapter 5 of this manual.

8.5.2.2 Determining Sinkhole Floodplain and No-Fill Lines

The floodplain line for a sinkhole is defined by the sinkhole lip elevation or the flood elevation expected under extreme flood conditions. The storage volume beneath this elevation is the sinkhole floodplain storage volume. *The pre-development floodplain storage volume must be preserved under post development conditions.* If any fill is added in the floodplain outside the no-fill line this stipulation will require compensating excavation in the floodplain.

The no-fill line shall be established by the contour line or interpolated contour line for the elevation that defines *sixty (60) percent* of the floodplain storage volume. The area encompassed by this line shall be defined as a no-fill zone for all construction activities. No construction fill will be allowed in this zone.

The floodplain, no-fill line and dedicated easement shall be indicated on all preliminary and final plans.

The floodplain and no fill lines may be adjusted under following circumstances:

1. The floodplain line can be adjusted to equal the 100-year flood elevation (future conditions) determined from flood studies or Master Plans that consider future development in the watershed. The Director must approve the adjustment. The use of a flood study or Master Plan by the Director does not imply approval of the adjustment of the floodplain for the development.
2. If the contributing drainage area is less than 50 acres and there is documented evidence that resurgence is not a contributing factor to flood elevations, the floodplain elevation can be determined by the following procedure:
    A. Determine the volume of runoff for the 4-day, 100-year for the contributing drainage to the sinkhole for fully developed conditions. The volume of runoff will equal the drainage area multiplied by 7.8 inches of runoff.
    B. Based on the 4-day, 100-year volume of runoff, determine the predicted floodplain elevation in the sinkhole assuming no outflow from the sinkhole.
    C. If the predicted 4-day, 100-year floodplain elevation is less than the lip of the sinkhole this elevation can be used as the floodplain elevation.
3. The floodplain elevation can be reduced if the drainage directed to the sinkhole under current conditions can be safely redirected to a reliable surface drainage system. Calculations that quantify the pre- and post-development stormwater discharges to the surface drainage system must be provided in the Stormwater Management Plan.

Under all of the above scenarios, the no-fill line must be recalculated based on the revised floodplain elevation.

Example 8-1. Calculation of 100-year Flood Elevation for a Sinkhole

Assume: Sinkhole outflow is 0 cfs, all contributing watershed runoff outlets to the sinkhole and that all rainfall becomes runoff (no infiltration, evaporation, etc.).

Watershed Variable	Amount
Total area (ac)	10
100-year, 24-hour storm rainfall (in)	6.5
Total Watershed Runoff (ac-ft)	5.5

The incremental storage volume for the sinkhole is assumed to be calculated using a volume formula for the frustum of a cone. The frustum is the part of a right circular cone between the base and a plane parallel to the base whose distance from the base is less than the height of the cone. Let h be the height of the cone frustum, or for this case the difference between the elevations (2 feet here). The areas at the elevation increments are the areas at the top and bottom of the cone frustum, and are represented as A_1 and A_2, respectively. The incremental storage volume formula, including the conversion from cubic feet to ac-feet is:

$$\Delta V = \frac{(\frac{h}{3} \times (A_1 + A_2 + \sqrt{A_1 \times A_2}))}{43{,}560}$$

The calculated storage volume for receiving sinkhole is summarized in the table below:

Elevation (feet)	Area Inside Closed Contour (ft^2)	Incremental Storage Volume (ac-ft)	Cumulative Storage Volume (ac-ft)
938	0	NA	NA
940	225	0.00	0.00
942	2215	0.05	0.05
944	13250	0.32	0.37
946	69160	1.72	2.09
948	178000	5.48	7.57
950	260000	10.00	17.57
952	354000	14.04	31.61

To determine the 100-year floodplain elevation, the cumulative storage volume must be equal to, or exceed, the total watershed runoff of 5.5 ac-ft. This occurs between the 948 and 946 contours. Note that the sinkhole lip is located at elevation 952, so in this example, the 100-year, 24-hour flood is contained within the sinkhole without overflow.

Example 8-2. Calculation of Sinkhole No-Fill Line

The following elevation-storage table was developed for a sinkhole adjacent to a proposed development site:

Elevation (feet)	Cumulative Storage Volume (ac-ft)
900	0.0
904	3.5
906	5.0
908	8.0
910	11.0
912	15.0
914	19.0
916	25.0

The lip of the sinkhole is located at elevation 916, therefore the sinkhole floodplain line is defined by the 916 contour line. The sinkhole no-fill line is defined by the 912 contour line, as follows:

> 60% of 25 ac-ft = 15 ac-ft
> The 912 elevation contour has the required 60% of total storage volume of 15 ac-ft.

Therefore, the development can encroach between the 916 and 912 contour lines but, the total storage volume below elevation 916 must be kept equal to 25 ac-ft. Note that the fill volume within the sinkhole but beyond the "no fill" line must be compensated for by excavating below the sinkhole floodplain elevation.

Example 8-3. Determining Runoff Volumes

Note: This example is for volume calculations only and does not include discharge calculations that would be required.

A five (5)-acre site that is currently 50% woods and 50% meadow is to be developed into a commercial shopping center. The proposed development will include 2.5 acres of impervious area, 1 acre of maintained lawn and 1.5 acres will remain wooded. The site drains to a sinkhole, which is located off the property, but inside the Dutchtown Road Drainage Basin.

All soils were determined to be in Hydrologic Soil Group C.

Determine the pre-development runoff volume (Q_v):

P = 6.6 inches
CN = 86 (woods and meadow good condition, AMC=III, soil group C, source TR-55)
S = 1000/CN-10 = 1.63 inches
$Q_v = (P-0.2S)^2/(P+0.8S) = 4.98$ inches * 5 acres = 90,400 ft^3

Determine the post-development runoff volume (Q_v):

P = 6.6 inches
CN = (1.5 * 86 (woods) + 2.5 * 99 (imp. Area) + 1.0 * 88 (open space))/5 = 93
S = 1000/CN-10 = 0.75 inches
$Q_v = (P-0.2S)^2/(P+0.8S) = 5.78$ inches * 5 acres = 104,900 ft^3

To meet Knox County requirements, the designer must reduce the post-development runoff volume by 14,500 ft^3 (104,900 ft^3 – 90,400 ft^3) to meet pre-development levels.

References

City of Knoxville. *Land Development Manual.* City of Knoxville Engineering Department, Stormwater Division, June 2006.

Table of Contents

CHAPTER 9 — CONSTRUCTION SITE STORMWATER MANAGEMENT

This page left intentionally blank

CHAPTER

9

CONSTRUCTION SITE STORMWATER MANAGEMENT

This chapter provides clarification on Knox County's requirements for erosion prevention and sediment control (EPSC) for large construction sites and small residential lots, construction site waste management practices and stormwater pollution prevention practices. The objectives of Knox County's construction site stormwater management regulations are:

- To protect streams within Knox County from sedimentation and other pollutants that may result from construction activities;
- To rely first on erosion controls and phasing to reduce the potential for off-site sedimentation;
- To prevent off-site sedimentation from land-disturbances of any size;
- To mirror, to the extent practical, the requirements of the State of Tennessee Construction General Permit; and
- To comply with the requirements of the State of Tennessee Municipal Separate Storm Sewer System (MS4) permit.

9.1 TN Construction General Permit

In general, Knox County's requirements for construction site EPSC mirror the State's regulations, as defined in the General NPDES Permit for Discharges of Stormwater Associated with Construction Activities, also called the NPDES Construction General Permit (CGP). The CGP is administered by the Tennessee Department of Environment and Conservation (TDEC). The State of Tennessee requires that the owner or operator for each construction site that disturbs one or more acres, or sites that disturb less than one acre but are part of a larger plan of development or sale, obtain coverage under the Tennessee CGP. A Notice of Intent (NOI) to obtain coverage under the CGP and a copy of the Stormwater Pollution Prevention Plan (SWPPP) must be submitted to, and reviewed by, TDEC. Once TDEC has approved the NOI, a Notice of Coverage (NOC) is issued to the permittee.

The Knox County Stormwater Management Ordinance requires that the design, installation, maintenance and inspection of EPSC design standards and best management practices (BMPs) be in accordance with the State of Tennessee CGP, the *Tennessee Erosion and Sediment Control Handbook*, and this manual. Since the CGP is renewed and potentially modified every five years, this requirement refers to the Tennessee CGP and TDEC handbook that are valid and in-use at the time that the grading permit application is submitted. The Director of Engineering and Public Works (Director) has the authority to invoke more stringent requirements for construction site stormwater management where necessary. When the provisions of the ordinance, this chapter and another local or State regulation conflict or overlap, that provision which is more restrictive or imposes higher standards or requirements must be followed.

Since construction site stormwater management requirements of the State and Knox County are so similar, much of the information that is submitted to TDEC as part of the Tennessee CGP NOI can also be used to apply for a grading permit in Knox County as part of the EPSC plan. Construction sites that require coverage under the Tennessee CGP must submit a copy of the NOC and SWPPP with the EPSC plan that is submitted to Knox County Engineering to obtain a grading permit. The EPSC plan will not be considered complete if a copy of the NOC and SWPPP are not included. More information on the grading permit application and EPSC plan contents can be found in Volume 1, Chapter 4 and Appendix D, respectively.

9.2 General Principles

Knox County's erosion prevention and sediment control program guidance is based upon the *Tennessee Erosion and Sediment Control Handbook*. The following paragraphs provide clarification and additional guidance for EPSC measures for land disturbing activities of any size that are located in Knox County.

- **Erosion prevention is the first line of defense to prevent off-site sedimentation.** In the past, erosion within a construction site has been considered acceptable and part of the overall construction process, and emphasis was placed on the control of eroded sediments. However, erosion increases the potential for off-site sedimentation and increases site grading costs. Relying first on erosion prevention measures reduces the potential for enforcement actions resulting from off-site discharges of sediment, the maintenance requirements for sediment control measures, and overall grading costs.
- **Minimize the area that is disturbed.** Developers and contractors should disturb only building envelopes, leaving the surrounding areas undisturbed thereby maintaining pre-development infiltration rates and runoff coefficients. If water quality buffers and natural areas will be used as post-construction water quality controls, limiting the disturbed area will be required in those areas at a minimum.
- **Sequence land disturbing activities to minimize the amount of time that such areas are exposed to storm events.** For example, if a development will ultimately disturb 75 acres, the land disturbing activities should be phased or sequenced into smaller, more manageable sections with EPSC measures prescribed for each section.
- **Sediment must be retained on-site.** Construction and land-disturbing activities inherently cause sediment migration. However, the Knox County Stormwater Management Ordinance prohibits off-site sediment discharges. Sediment controls must be designed to retain sediment on the development site and prevent sediment from discharging onto adjacent property, into the storm drain system, or into the street.
- **All disturbed areas must be permanently stabilized after construction has ceased.** To prevent the potential for ongoing erosion and sedimentation, permanent ground cover must be provided on all areas that were disturbed during construction. The permanent ground cover can consist of any of the following: permanent grasses or other permanent vegetative cover; asphalt or concrete pavement; rip rap or other hard armor for channels and slopes; or buildings.

9.3 Regulations and Policies

9.3.1 General

General regulations and policies for construction site stormwater management are as follows:

1. All EPSC measures must be properly selected, installed, and maintained in accordance with the manufacturer's specifications (where applicable) and good engineering practices. All EPSC measures selected must be able to slow runoff so that rill and gully formation is prevented. When steep slopes and/or fine particle soils are present at the site, additional physical or chemical treatment of stormwater runoff may be required, and must be fully described. If periodic inspections or other information indicate that a control measure has been used inappropriately, or incorrectly, the owner or operator must replace or modify the control for relevant site situations.

2. Discharges from construction sites must leave the project area in a non-erosive manner. Discharges from sediment basins and traps must be through a pipe or a conveyance lined with rip-rap or other stabilized spillway so that the discharge does not cause erosion.
3. Construction materials buried onsite shall meet the provisions of Section 4.8 of the Knox County Zoning Ordinance.
4. Construction related wastes shall be managed in accordance with the requirements of the Tennessee CGP.
5. For installation of any waste disposal systems on site, or sanitary sewer or septic system, the plan should provide for the necessary sediment controls. Owners or operators must also comply with applicable State and/or local waste disposal, sanitary sewer or septic system regulations for such systems, to the extent that these are located within the permitted area.
6. If permanent or temporary vegetation is to be used as a control measure, then the timing of the planting of the vegetation cover must be discussed in the EPSC plan or SWPPP. Delay in planting cover vegetation until winter months or dry months should be avoided, if possible.
7. If sediment escapes the construction site, off-site accumulations of sediment that have not reached a stream must be removed at a frequency sufficient to minimize offsite impacts (e.g., fugitive sediment that has escaped the construction site and has collected in a street must be removed so that it is not subsequently washed into storm sewers and streams by the next rain and/or so that it does not pose a safety hazard to users of public streets). Owners or operators shall not initiate remediation/restoration of a stream without consulting Knox County and TDEC first. Approval from Knox County or TDEC does not authorize access to private property. Arrangements concerning removal of sediment on adjoining property must be settled by the owner or operator with the adjoining landowner.
8. Sediment should be removed from sediment traps, silt fences, sedimentation ponds, and other sediment controls as necessary, and must be removed when design capacity has been reduced by 50%.
9. Sediment laden water to be pumped from excavation and work areas must be held in settling basins or filtered prior to its discharge into surface waters. Water must be discharged onto a stabilized outlet point so that the discharge does not cause erosion and sedimentation.
10. Litter, construction debris, and construction chemicals exposed to stormwater shall be picked up prior to anticipated storm events or before being carried off of the site by wind (e.g., forecasted by local weather reports), or otherwise prevented from becoming a pollutant source for stormwater discharges (e.g., screening outfalls, daily pick-up, etc.). After use, materials used for erosion prevention and sediment control should be removed or otherwise prevented from becoming a pollutant source for stormwater discharges.
11. Offsite erodible material storage areas (also including overburden and stockpiles of dirt, etc.) used primarily by the permitted project are considered a part of the project and shall be addressed in the EPSC and included in the calculation of the EPSC portion of the performance bond.
12. Construction must be phased for projects in which over **25** acres of soil will be disturbed. Areas of the completed phase must be stabilized within 15 days. No more than **25** acres of active soil disturbance is allowed at any time during the construction project.
13. The following records shall be maintained on or near site: the dates when major grading activities occur; the dates when construction activities temporarily or permanently cease on a portion of the site; the dates when stabilization measures are initiated; inspection records and rainfall records.
14. Owners or operators shall maintain a rain gauge and daily rainfall records at the site, or use a nearby reference site for a record of daily amount of precipitation. The location of the rain gauge shall be shown on the EPSC plan.

15. Owners or operators shall implement practices to protect the stormwater conveyance system from sedimentation.
16. Disturbed areas will be considered permanently stabilized and ready to be released from the Performance and Indemnity Agreement when a permanent vegetative cover has been established over at least 70% of the site. Note that the Director can require greater coverage when the potential for off-site sedimentation or accelerated erosion exists.

9.3.2 Protection of Water Quality Buffers

During construction, water quality buffers around streams, wetlands, ponds, and other water bodies must be protected from disturbance and from sediment-laden runoff from the site. Prior to beginning land-disturbing activities at a site, water quality buffers must be identified and flagged in the field for protection. Water quality buffers cannot be used as vegetated filters for sediment control.

9.3.3 Stabilization of Disturbed Areas

Disturbed areas within the project site must be permanently or temporarily stabilized within 15 days after construction activity in that portion of the site has temporarily or permanently ceased unless the soil is frozen, soggy, or otherwise unworkable.

Pre-construction vegetative ground cover must not be destroyed, removed or disturbed more than 10 days prior to grading or earth moving unless the area is seeded and/or mulched or other temporary cover is installed.

Stabilization of disturbed areas refers to measures that protect soil from accelerated erosion. Applicable practices include, but are not limited to, vegetative groundcovers, mulch, or the application of gravel base on areas to be paved. Unpacked gravel containing fines (silt and clay sized particles) or crusher run will not be considered a non-erosive surface. Soil stabilization specifications should be provided for year round seeding and the particular site conditions. Specifications for establishing a groundcover should include seeding mixtures for permanent and temporary seed establishment, soil amendments, mulch application, and application of a tackifier to hold the mulch in place.

Permanent vegetative cover shall be established on disturbed areas not otherwise permanently stabilized. Areas that were previously disturbed and provided with a permanent vegetative groundcover will not be considered established until a groundcover is achieved which, in the opinion of the Department of Engineering and Public Works, is mature enough to control soil erosion and to survive severe weather conditions. Generally, final stabilization of a site with a vegetative groundcover has been achieved when the following has been provided:

- All cut and fill slopes have a permanent groundcover with at least 70% coverage, and the slopes do not contain evidence of rill or gully erosion.
- All concentrated flow paths have been stabilized against erosion with a permanent measure (groundcover, rip rap, concrete, etc.) at least 70% coverage, and the concentrated flow paths do not contain evidence of erosion.
- All other areas of the site have a permanent groundcover with at least 70% coverage, and the site does not contain evidence of accelerated erosion.

The Engineering Director can have more stringent requirements in accordance with the Knox County Stormwater Management Ordinance.

When conditions are not conducive to establishing a permanent vegetative cover, a temporary cover can be established, provided that a permanent cover is provided when conditions have improved. Temporary covers can include annual grains or mulch anchored with netting, crimping or other appropriate anchor.

9.3.4 Off-Site Sedimentation

Properties adjacent to or downstream from a disturbed site must be protected from sediment. This may be accomplished by preserving a well-vegetated filter strip around the lower perimeter of the land disturbance or by installing perimeter controls such as sediment barriers, filters, diversion berms, or sediment basins.

9.3.5 Construction Sequencing

Sediment basins and traps, perimeter diversion berms, sediment barriers and other measures intended to trap sediment onsite must be constructed as a first step in grading and be made functional before upslope land disturbance takes place. Water quality buffers must also be identified and protected with flagging or fencing prior to beginning land-disturbing activities on the site. All sediment control practices at hydraulic outlets from the site must be installed before additional construction may take place within the area draining to the outlet. Earthen structures such as dams, dikes, cut and fill slopes and diversions must be seeded and mulched within 15 days of installation.

9.3.6 Structural Controls

The EPSC plan must include a description of structural practices to divert flows from exposed soils, store flows or otherwise limit runoff and discharge of pollutants from exposed areas of the site. Such practices may include silt fences, earth dikes, drainage swales, sediment traps, check dams, subsurface drains, pipe slope drains, level spreaders, storm drain inlet protection, rock outlet protection, reinforced soil retaining systems, gabions, and temporary or permanent sediment basins. Structural controls shall not be placed in streams or wetlands except as authorized by a Section 404 Permit and/or Aquatic Resource Alteration Permit (ARAP).

Erosion prevention and sediment control measures must be designed according to the size and slope of disturbed drainage areas with the goal of detaining runoff and trapping sediment. In addition, erosion prevention and sediment controls must be designed to control the rainfall and runoff from a 2-year return frequency, 24-hour duration storm event, as a minimum. When clay and other fine particle soils are present at the construction site, chemical treatment may be used to minimize the amount of sediment being discharged.

For an outfall in a drainage area of a total of 10 or more acres, a temporary (or permanent) sediment basin that provides storage for a calculated volume of runoff from a 2-year return frequency, 24-hour duration storm event and runoff from each acre drained, or equivalent control measures, must be provided until final stabilization of the site. The basin must have both wet storage (i.e., permanent pool) and dry storage components, in accordance with the *Tennessee Erosion and Sediment Control Handbook*. Where an equivalent control measure is substituted for a sediment retention basin, the equivalency must be justified to Knox County Engineering. Runoff from any undisturbed acreage should be diverted around the disturbed area and the sediment basin. Diverted runoff can be omitted from the volume calculation. Sediment storage expected from the disturbed areas must be included and a marker installed signifying the need for cleanout of the basin.

All calculations of drainage areas, runoff coefficients and basin volumes must be provided in the EPSC plan. The discharge structure from a sediment basin must be designed to retain sediment during the lower flows. Muddy water to be pumped from excavation and work areas must be held in settling basins or filtered or chemically treated prior to its discharge into surface waters. Water must be discharged through a pipe, well-grassed or lined channel or other equivalent means so that the discharge does not cause erosion and sedimentation.

9.3.7 Special Requirements for Priority Construction Activities

Priority construction activities have additional local and State requirements for BMP design, site inspections and buffer zones. Priority construction activities are defined in the Knox County Stormwater Management Ordinance as land disturbing activities that are located in a watershed that discharges into waters recognized by the State of Tennessee as impaired for siltation, or high quality waters. Knox County Engineering can assist owners or operators in determining if the land disturbing activity is considered a priority construction activity. The policies below are applicable to all priority construction activities.

- It must be clearly noted on the EPSC or small lot EPSC plan that the new development or redevelopment is a priority construction activity.
- Additional design, inspection, and buffer zone requirements for discharges into impaired or high quality waters that are stated in the Tennessee CGP must be implemented for all priority construction activities.
- Owners and operators of priority construction activities must attend a pre-construction meeting with Knox County Engineering prior to approval of a grading permit.
- The priority construction activity will be inspected for conformance with the EPSC plan and all local EPSC requirements by Knox County Engineering (or a duly authorized agent of Knox County Engineering) at least once per month.

9.3.8 Cut and Fill Slopes

Cut and fill slopes must be designed and constructed in a manner which will minimize erosion and potential slope failure. Consideration must be given to the length and steepness of the slope, the soil type, upslope tributary area, groundwater conditions, and other applicable factors. To promote proper maintenance, all cut and fill slopes should be graded no steeper than 3 feet horizontal to 1 foot vertical (3:1), unless otherwise specified in this manual. Cut and fill slopes must be stabilized once the final slope has been established.

9.3.9 Construction Exits

A stabilized stone construction exit shall be placed at any point where traffic will be leaving a construction site to a public right-of-way, street, alley, sidewalk, or parking lot. Stone pads shall contain ASTM-1 stone, 6 inches thick with a geotextile fabric liner underneath and be a minimum of one hundred (100) feet long and 20 feet wide. For individual lot construction, stone pads must be installed to the extent practicable, and contractors must use the stone pad during construction to avoid tracking soil, mud and debris off the project site. The intent of construction exits is to prevent tracking of mud or debris off the construction site.

9.3.10 Good Housekeeping Practices

On all construction sites, site operators are required to control construction site wastes to prevent water quality impacts to the maximum extent practicable. Non-stormwater discharges into the storm drain system constitute a violation of the Stormwater Management Ordinance (see Section 12). The EPSC plan must include good housekeeping practices as they relate to stormwater quality, as outlined in this section. Specific measures must be included in and shown on the plan to address:

- Spill prevention for chemicals used on site;
- Materials storage for chemicals stored on site;
- Waste management;
- Concrete truck and masonry washout areas;

- Vehicle maintenance conducted on site; and,
- Trash and construction debris management.

All contractors and sub-contractors working on the site must be made aware of the good housekeeping practices included in the plan.

9.3.10.1 Spill Prevention

Spills and leaks are often sources of stormwater pollutants and can be avoided. Avoiding spills is easier than cleaning up spills after they occur. Heavy equipment and other vehicles should be inspected daily for leaks and repaired as necessary. If equipment is to be fueled onsite, the fueling areas must be designated on the EPSC and other site development plans and provided with containment and spill cleanup measures.

When spills do occur, clean up should not include hosing the area off. The SWPPP must describe measures to be taken to prevent and clean up after a spill, such as excavating contaminated soils and placing them on heavy plastic sheeting or using absorbent material to absorb the material and then disposing of the material in approved containers to prevent contact with stormwater. Spills in excess of two (2) gallons must be reported to Knox County Department of Engineering and Public Works.

Portable sanitary facilities at the site must be maintained to prevent discharge of sewage into the storm drain system or into streams.

Concrete truck wash down areas must be designated on the EPSC and other site development plans. The designated area must fully contain the washings to prevent residual concrete tailings from entering a stormwater system or stream. Other equipment washing must be performed in a designated area, and these areas noted on the EPSC and other site development plans. Wash water must be adequately treated prior to discharging to the storm drain system or routed to the sanitary sewer.

Construction vehicle maintenance must be performed at least 50 feet away from any storm drain system. Drip pans and other containment systems must be employed to prevent spills.

9.3.10.2 Materials Storage

All chemicals must be stored in covered areas, with containment systems constructed in or around the storage areas. Areas must be designated for materials delivery and storage. These areas should be shown on EPSC and other site development plans. Shipping and receiving practices must minimize materials spills and exposure to stormwater.

9.3.10.3 Trash and Debris Management

All construction trash must be properly managed and disposed. Trash and construction debris should be controlled in a centralized area that is protected from wind and rain. All construction debris and trash must be removed and properly disposed of from the site after construction has been completed.

9.4 Erosion Prevention and Sediment Control Plans

The owner or operator of land development activities not exempted by Section 4.1 of the Stormwater Management Ordinance must obtain a grading permit prior to commencing land-disturbing activities. An EPSC plan must be submitted to obtain a grading permit.

EPSC plans identify the erosion prevention practices, both temporary and permanent, and sediment control measures that are appropriate for the individual site conditions. In addition, the plan must include a description of measures to be installed to control other construction-related wastes such as litter, construction debris, concrete washout and solvents. Other items to include are: a construction schedule; design calculations supporting structural controls on the project; a location map; vegetation specifications for stabilization purposes; and any other information necessary to support the EPSC plan for a specific development. Specific requirements for the EPSC plan are presented in the EPSC plan checklist, in Volume 1, Appendix D of this manual.

When developing the EPSC plan, the designer should consider the following items:

- Limit the disturbance to the minimum necessary for the development site. Keeping the disturbed area minimized in turn minimizes the sizes of sediment controls. Protect areas that are not to be disturbed to prevent inadvertent disturbances. In addition to minimizing sediment and erosion control required on the site, minimizing disturbed areas and protecting undisturbed areas aids in the treatment of post-construction stormwater management (see Volume 2, Chapter 5 for more information on post-construction stormwater management).
- Undisturbed off-site drainage should be routed around disturbed areas instead of through the project site. Large amounts of off-site drainage routing through a project site can cause measures to be grossly oversized and overwhelmed during storm events.
- Particular attention should be given to concentrated flow paths inside the development site. Disturbing concentrated flow paths should be avoided. Instead, the conveyance should be isolated from the surrounding disturbed area. In the event that the concentrated flow path must be disturbed, the conveyance should be stabilized as soon as possible and erosion control measures applied to prevent erosion.
- Sediment controls should be designed for all stormwater outfalls that have disturbed areas within their drainage boundaries. Stormwater outfalls are those points where runoff flows onto adjacent properties, into roadways, or into streams.
- Construction sequences provided in the EPSC plan should address the installation of perimeter measures for the initial grading phase prior to beginning any land disturbing. It should also address the application of temporary and permanent groundcovers, the ongoing maintenance of measures, and the final removal of temporary measures once the site has been stabilized. Consider providing a phased grading plan for large sites or those sites that have sensitive features (streams, wetlands, sinkholes, steep slopes, etc.).

9.4.1 Plan Modifications

Once EPSC plans have been approved and are under construction, field modifications are often necessary and beneficial. Minor modifications of grading plans are permissible without resubmitting plans to Knox County Engineering for approval. Minor modifications could consist of shifting a control to better fit the site topography, increasing the sediment control capacity, installing a more robust measure (for example, installing a rock berm instead of a silt fence), or decreasing the size of the disturbance. However, major modifications, such as omitting controls, using a different control, or substantially modifying the drainage patterns of a site, must first obtain approval from the Director before implementing the change. Knox County inspectors may require a plan modification when, upon inspection, measures are found to be ineffective.

9.5 Deficient Performance

If at any time it is determined by the Department of Engineering and Public Works, the property owner, or designated construction site inspector/manager that the erosion prevention and sediment control practices as originally designed are not capable of controlling erosion and/or preventing sediment from leaving the site under storm conditions, then additional controls must be implemented and a plan modification will be required. Additional controls must be implemented to a level and until a time in which the Director is satisfied that the controls are adequate. If Knox

County inspectors determine that adequate inspections and maintenance procedures are not being implemented or the controls as designed are not meeting performance objectives presented in the Stormwater Management Ordinance or these regulations, the inspector may issue a Notice of Violation citing actions to be taken and a time frame for compliance. If appropriate actions are not taken as specified on the notice-of-violation, a stop work order may be issued.

9.6 Inspections and Maintenance

All permitted land-disturbances must be inspected during the life of the project until the site has been permanently and finally stabilized. Documentation of inspections should be maintained at the project trailer or with the person/individual responsible for performing the inspections. Inspections must be conducted twice every week, at least 72 hours apart. Often, it is advantageous to inspect erosion prevention and sediment control measures during rain events to see how well the system of measures is actually working.

Inspections should be performed by qualified personnel that have taken the TDEC Level I Fundamentals of Erosion Prevention and Sediment Control for Construction Sites and have passed the exam. Inspections should be performed on all areas of active construction, including areas that have not been finally stabilized, areas used for storage of materials or stockpiles, exits and entrances for construction traffic, outfall points from the project, and structural controls. These areas should be inspected for signs of erosion, sedimentation, or the discharges of other pollutants from the site. Any controls found to be inadequate must be repaired within 7 days following the inspection, and the EPSC plan modified (if necessary) within 14 days of the inspection.

The following information should be provided on an inspection form:

- Name of the project;
- Name of inspector;
- Date, time, site and soil conditions;
- Major observations related to the implementation of the EPSC plan, including measures that need maintenance or upgrade; signs of pollutant discharge; evidence of erosion and areas that have been or need to be stabilized;
- Any actions taken since the last inspection to repair measures or mitigate any previous problems.

An inspection checklist can be found in Appendix F of Volume 1 of this manual. When requested, copies of completed inspection reports must be provided to Knox County within a timely manner.

9.7 Small Lot EPSC Plans

Prior to issuing a building permit, a project that has not previously been required to obtain a grading permit must provide a small lot EPSC plan. Preparation of a small lot EPSC plan is much simpler than the EPSC plan required for a grading permit. A template that can be utilized to prepare a small lot EPSC Plan can be found in Appendix E of Volume 1 of this manual.

The permittee is required to adhere to the small lot EPSC plan and prevent sediment from leaving the building site. The permittee is also responsible for stabilizing the site prior to the issuance of a Certificate of Occupancy.

In some circumstances, a small lot EPSC plan may not provide adequate protection to streams, sinkholes and other stormwater systems from sedimentation. In such situations, the Director can require a more detailed EPSC plan.

References

Tennessee Department of Environment and Conservation (TDEC). *General NPDES Permit for Discharges of Stormwater Associated with Construction Activities.* Permit No. TNR100000. Issued June 16, 2005.

Tennessee Department of Environment and Conservation (TDEC). *Tennessee Erosion and Sediment Control Handbook.* Second Edition. March, 2002.

Table of Contents

CHAPTER 10 — POLLUTION PREVENTION AFTER CONSTRUCTION

Chapter 10 – List of Tables

This page left intentionally blank

Table of Contents

CHAPTER 10 — POLLUTION PREVENTION AFTER CONSTRUCTION

Chapter 10 – List of Tables

This page left intentionally blank

CHAPTER

10

POLLUTION PREVENTION AFTER CONSTRUCTION

This chapter describes the pollution prevention measures to be taken once the site has been permanently and finally stabilized and no additional construction activities are anticipated on the site.

10.1 Overview

As noted in Chapter 1, stormwater pollution, or non-point source pollution, comes from many different sources: active construction sites, agricultural practices, forestry practices, and urbanized areas. Some types of urban land uses contribute higher than normal pollutant loadings. Inherent with these types of land uses are the storage, use and/or production of higher amounts of solvents, oils, lubricants, fertilizers, grease, and/or bacteria. If traditional stormwater quality controls are installed in runoff discharges that carry higher pollutant levels, the controls are quickly overwhelmed and stormwater quality downstream suffers. Therefore, for such discharges, additional measures must be taken to protect Knox County's stormwater management system from these higher pollutant loadings.

Pollution prevention begins during active construction, as outlined in Volume 2, Chapter 9 of this manual. However, pollution prevention must be practiced throughout the life of the site and is the responsibility of the property owner and business/activity operator. Pollution prevention activities should be tailored to capture typical pollutants from the land use activity occurring on the site. General guidance for land uses that are often identified as having a higher than normal pollutant potential is presented in the following paragraphs.

- **Vehicle maintenance, washing or storage facilities.** Pollution prevention activities for vehicle maintenance, washing, or storage land uses must focus on spill prevention and cleanup, oil and other fluid and material recycling, pre-treatment of wash water or runoff from maintenance areas, staff education on proper pollution prevention techniques, and customer education about the activities that are or are not acceptable on the premises. For businesses where vehicles will be stored, pollution prevention activities must also include routine inspection of the vehicles for leaks or discharges. Drip pans must be used to capture leaks and discharges until the vehicle can be maintained or fluids should be drained completely from vehicles that will remain unused. Discharges of wash water resulting from the hosing or cleaning of vehicles, equipment and/or facilities is considered an illegal non-stormwater discharge. Therefore, wash water must be prevented from entering the stormwater system. These activities could include blocking the stormwater system or diverting the wash water into a pre-treatment measure and then into the sanitary sewer system.
- **Recycling and salvage yard facilities.** Where the land use is a business that recycles or salvages vehicles or other equipment, the pollution prevention practices for that site should address draining the equipment of all fluids before storage. If the storage area is uncovered, pre-treatment controls are required to treat additional pollutants that could result from the storage or deterioration of the equipment or vehicles before the runoff discharges to traditional best management practices (BMPs), such as those discussed in Chapter 4.
- **Restaurants, grocery stores, and other food service facilities.** Grease and organic pollutants are pollutants that are typically encountered around restaurants, grocery stores, and other food service facilities. Pre-treatment to remove such pollutants prior to discharging to

traditional BMPs is required, in order to prevent clogging of downstream BMPs and the stormwater system. As well, wash water from equipment and/or facility cleaning activities must either be discharged to the sanitary sewer or be pre-treated prior to discharging to a stormwater quality BMP.

- **Facilities that temporarily or permanently house animals outside.** Animal housing facilities, such as veterinary clinics, boarding facilities, livestock stables, hatcheries and animal shelters, have the potential to deliver higher than normal bacterial loadings to the stormwater system. High counts of bacteria in streams and rivers can cause water quality impairments, but can also cause illnesses in people. Pollution prevention practices for these types of facilities must include pet waste management practices, such as collecting and properly disposing of pet waste at landfills or wastewater treatment facilities. Animal bedding should be removed when soiled and properly disposed. Wood shavings or chips must not be allowed to migrate into the stormwater system.

10.2 Special Pollution Abatement Permit

A Special Pollution Abatement Permit (SPAP) may be required for new developments and redevelopments on the basis of: 1) land use or type of business; 2) a history of air or water pollution at a site; 3) a history of air or water pollution by an owner/operator at other sites; 4) the potential to impact environmentally sensitive areas, such as wetlands; or 5) at the discretion of the Director upon sound engineering judgment. A SPAP form is provided in Volume 1, Appendix D of this manual.

As required by Section 4.6 of the Knox County Stormwater Management Ordinance, SPAPs are required for the following hotpot land uses:

- Vehicle, truck or equipment maintenance, fueling, washing or storage areas including but not limited to: automotive dealerships, automotive repair shops, and car wash facilities;
- Recycling and/or salvage yard facilities;
- Restaurants, grocery stores, and other food service facilities;
- Commercial facilities with outside animal housing areas including animal shelters, fish hatcheries, kennels, livestock stables, veterinary clinics, or zoos;
- Other producers of pollutants identified by the Director by information provided to or collected by him/her or his/her representatives, or reasonably deduced or estimated by him/her or his/her representatives from engineering or scientific study.

A SPAP is not required for outfalls that have been previously permitted through the state's NPDES program. A copy of the NPDES permit must be submitted to Knox County Engineering. Typically, the need for a SPAP is identified during grading permit application review.

To obtain coverage under a SPAP, the property or business owner must submit a SPAP application form and the appropriate application fee. In the event that a SPAP is required for a new development or redevelopment site, grading and/or stormwater management plans will not be approved until the SPAP application form and any appropriate application fee has been received and approved by Knox County. The SPAP application requires supporting documentation for the proposed BMP(s), including BMP specifications and maintenance information. An As-Built Certification must be included for any structural BMPs installed at the site.

Once issued, the SPAP will be valid for five (5) years and must be renewed prior to the expiration date. SPAP renewal requires completion and submittal of an updated application form, including supporting documentation for the stormwater BMP(s) installed at the site, and payment of any appropriate application fee.

Coverage under a SPAP must be renewed if, at any time during the five-year permit period, pollution pre-treatment devices or stormwater BMPs that are reflected in the current SPAP are removed or otherwise significantly altered. A SPAP application that reflects the proposed modifications, along with a SPAP application fee, must be submitted to and approved by Knox County Engineering prior to instituting the changes. Renewal of a SPAP is not required for routine BMP maintenance and repair activities or for replacement of poorly functioning or failed BMPs as long as the replacement is similar to, in form and function, and serves the same purpose as the original BMP.

The following minimum standards shall be addressed in the SPAP application form:

- **Employees and/or staff of the business or land use type shall be trained annually on the requirements of the SPAP,** specifically addressing pollution source controls such as spill control and cleanup, proper waste management, chemical storage, and fluids management with vehicle servicing. The type of training shall be tailored to and appropriate for the land use or business. Documentation of the training shall be maintained with the SPAP and made available to Knox County upon request.
- **Parking lots shall be swept monthly to remove gross solids.** Waste gathered during sweeping activities shall be disposed of properly.
- **Animal waste shall be prevented from entering streams, sinkholes, wetlands, ponds or any other component of the storm drain system.** Controls shall be instituted to collect the animal waste and properly treat or dispose of it.
- **Structural BMPs that have been designed to specifically address the target pollutants associated with the land use shall be utilized where appropriate to reduce pollutant loadings.** This requirement does not alleviate new developments and redevelopments from water quality treatment design criteria for total suspended solids (TSS), as discussed in Volume 2, Chapter 3. BMPs that are implemented to comply with SPAP minimum standards can factor into the % TSS calculation, provided that they have TSS removal capabilities. Percent TSS removal values and policies for stormwater treatment BMPs are presented in Volume 2, Chapter 3 of this manual. Table 10-1 presents target pollutants for the land uses required to obtain coverage under a SPAP.
- **Structural BMPs shall be inspected and maintained by the owner/permittee.** Inspections must be conducted at least annually. Maintenance shall be conducted as needed and as required by the manufacturer of the structural BMP or by Knox County. Documentation of such inspections shall be maintained by the owner and made available to Knox County upon request.

Table 10-1. Target Pollutants for SPAP Permitted Land Uses

Land use	Target Pollutant
Vehicle, truck or equipment maintenance, fueling, washing or storage areas including but not limited to: automotive dealerships, automotive repair shops, and car wash facilities	Oil, grease, detergents, solids, metals
Recycling and/or salvage yard facilities	Oil, grease, metals
Restaurants, grocery stores, and other food service facilities	Oil, grease, trash
Commercial facilities with outside animal housing areas including animal shelters, fish hatcheries, kennels, livestock stables, veterinary clinics, or zoos	Bacteria, nutrients
Other producers of pollutants identified by the Director by information provided to or collected by him/her or his/her representatives, or reasonably deduced or estimated by him/her or his/her representatives from engineering or scientific study	As identified by the Director

10.3 Sediment Disposal for Structural BMP Maintenance

Many of the structural BMPs (presented in detail in Volume 2, Chapter 4 of this manual) that are utilized to prevent stormwater pollutants from entering the waters of the state will accumulate sediment deposits over time and will require maintenance and cleaning to ensure that they continue to work at optimum efficiency. Depending on the characteristics of the drainage area to each structural BMP, there could be a wide nature of substances contained within the sediments. The appropriate sediment disposal method will depend on the type of contamination, if any, in the soil. Proper assessment and disposal of accumulated sediment is necessary to ensure that the sediment removed from structural BMPs does not cause discharge of pollutants to the environment. The text in this section shall be regarded as Knox County policy for proper assessment and disposal of accumulated sediments that are removed from structural BMPs. (Note: the text below was adapted for Knox County from the City of Knoxville Land Development Manual – Policy 11, June 2003.)

When properly designed, structural BMPs will accumulate significant quantities of sediment over time. Sediment gradually reduces the available stormwater storage capacity. A rule of thumb for BMPs such as detention ponds, extended detention ponds and stormwater ponds is that approximately 1% of the storage volume capacity associated with the 2-year design storm can be lost annually due to accumulated sediment. Therefore, approximately 20% of a pond's total storage capacity can be lost within 20 years.

The actual sediment accumulation rate is dependent upon a number of factors including watershed size, facility sizing, upstream construction, nearby industrial activities and land uses, numbers of leaking vehicles, use of sand and salt during winter, etc. Thick grass and vegetation will retain sediment and silt at a faster rate.

In addition to long-term maintenance, sediment disposal is usually necessary during the construction process. Erosion prevention and sediment control practices and devices are not 100% effective at reducing and eliminating all sediment. Therefore, the developer must ensure that the designed detention volume has been restored and that all graded surfaces have been completely stabilized at the end of construction.

Structural BMPs shall be inspected on a regular basis to determine the impact of existing sedimentation on the capacity. The frequency of inspection is dependant upon the upstream land use(s), type of BMP, and other factors. Inspections should occur during dry weather and wet weather conditions. In general, remove sediment prior to significant accumulations using a combination of equipment methods and hand shoveling. Typical intervals for sediment removal will be every 5 to 7 years for some BMP types, 10 to 20 years for others. Typical intervals for sediment removal for sediment forebay or other pretreatment settling basin will be once a year. Detailed guidance on the frequency of inspection and maintenance activities relating to sediment accumulation specific to each structural BMP that is presented in this manual is provided in Chapter 4. This guidance must be included in the Operations and Maintenance plan for each development and must be followed by the owner of the structural BMP.

<u>Guidance for Assessment and Disposal:</u>

1. If the structural BMP meets any of the following criteria, then the structural BMP owner must contact the Tennessee Department of Environment and Conservation (TDEC) for further regulations and recommended disposal guidelines.
    a. known contaminants are contained in the stormwater runoff that discharges to the structural BMP or in the sediment that has accumulated in the structural BMP.
    b. the structural BMP receives stormwater runoff from an industrial site.

c. the structural BMP receives stormwater runoff from a fueling center.

d. The structural BMP receives stormwater runoff from one or more commercial businesses with a total parking area of at least 120,000 square feet or 400 parking spaces.

e. the Director has reason to believe that contaminants are present based upon scientific or engineering information.

In all cases, treat sediment from structural BMPs as potentially hazardous soil until proven otherwise. Sediments should be sampled and identified before removal and disposal operations proceed. Contact the local office of TDEC – Division of Water Pollution Control (865-594-6035) to discuss special disposal procedures.

2. If the structural BMP does not meet any of the above criteria, or if the sediment has been tested and is determined to be free of contamination, then the following disposal practices are allowed:

   a. disposal at a Class III or Class IV landfill.

   b. use for fill material, cover material or land spreading on the project site.

   c. other disposal materials as approved by the Engineering Director.

All sediment which is disposed onsite must be prevented from re-entering the structural BMP, or entering any other BMP, drainage channel or culvert, natural creeks or streams, or any other component of the stormwater drainage system.

Table 10-2 is a list of local landfills that may accept sediment. Contact each landfill for costs and regulations associated with sediment disposal. This list is not intended to be complete or inclusive.

Table 10-2. Local Landfills for Sediment Disposal

Landfill	Location	Phone	Type
Burnett-Armstrong Demolition Landfill	3330 Delrose Avenue Knoxville, Tennessee	865-525-6645	Demolition
Poplar View Class III/IV Landfill	7826 Rutledge Pike Knoxville, Tennessee	865-525-7720	Demolition
Ridgeview Demolition Landfill	8723 Oak Ridge Highway Knoxville, Tennessee	865-690-9436	Demolition
Yarnell Demolition Landfill, LLC	1550 Lamon Quarry Road Knoxville, Tennessee	865-470-0023	Demolition
Chestnut Ridge Landfill & Recycling Center	240 Fleenor Mill Road Heiskell, Tennessee (Anderson County)	865-457-7810	Sanitary

References

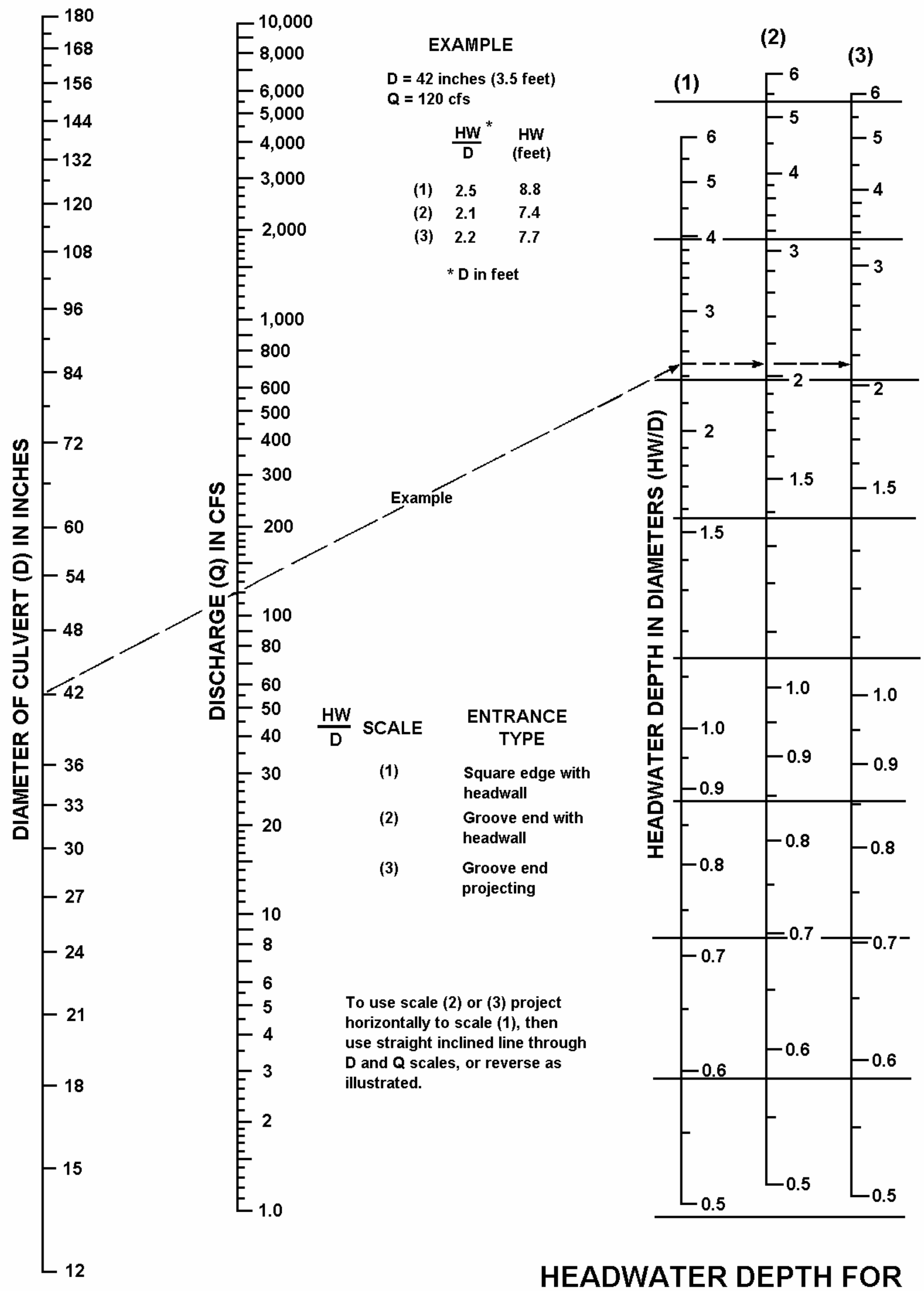
City of Knoxville. *Land Development Manual.* City of Knoxville Engineering Department, Stormwater Division, June 2006.

This page left intentionally blank

APPENDIX A

This Appendix contains Culvert Design Charts and Nomographs taken from the AASHTO Model Drainage Manual, 1991.

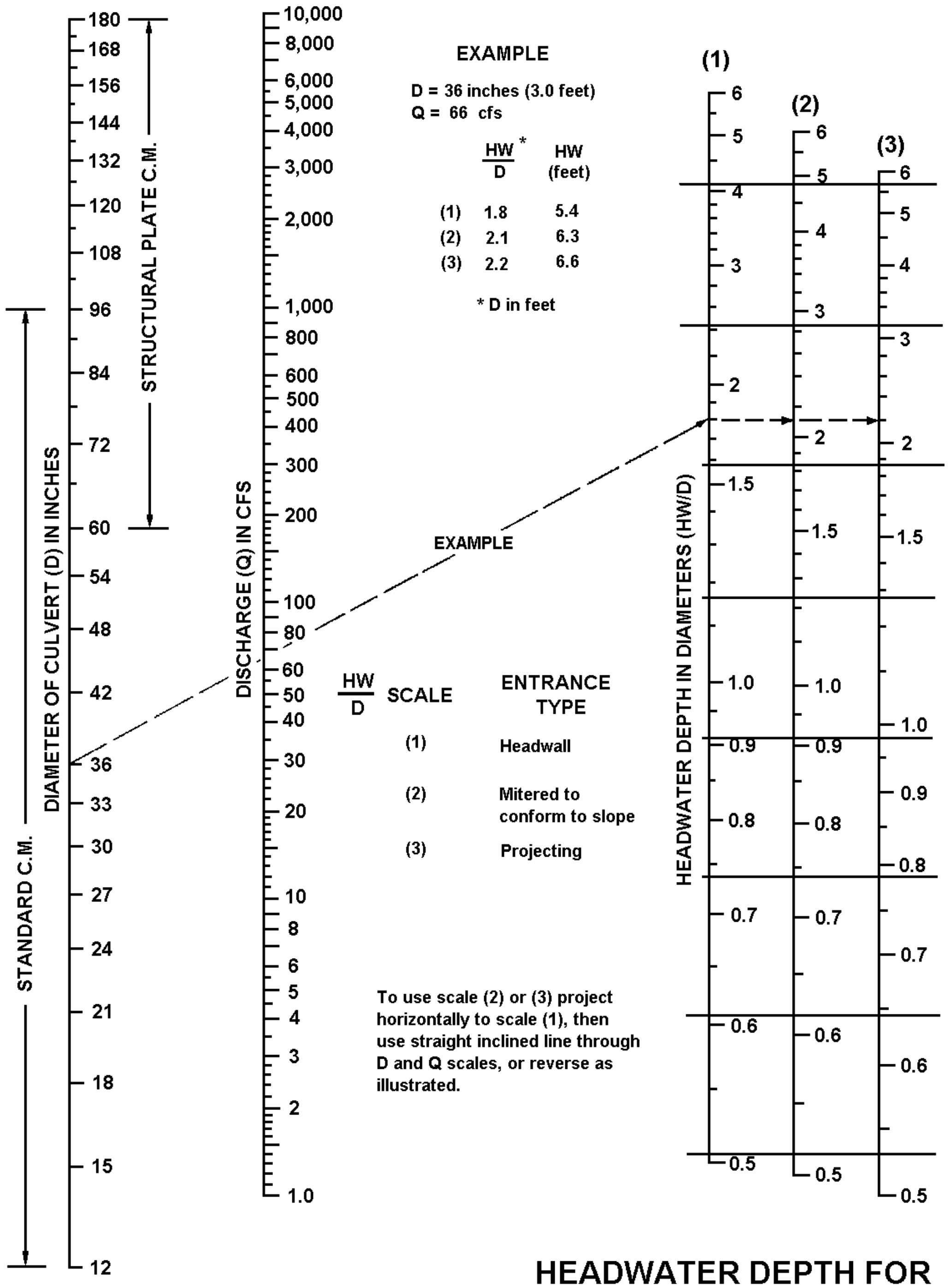
CHART 1



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963
HEADWATER SCALES 2&3 REVISED MAY 1964

CHART 2

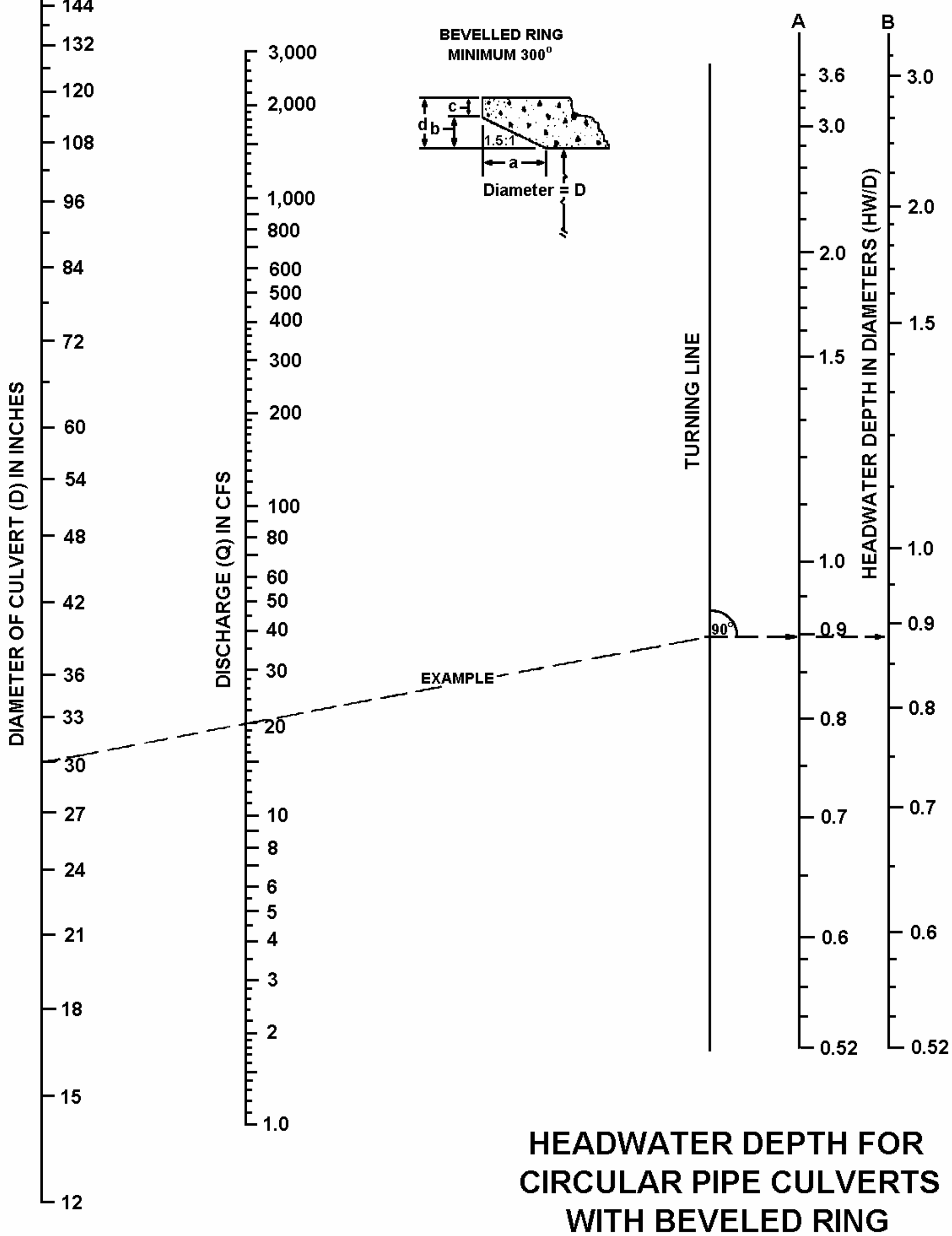


HEADWATER DEPTH FOR C.M. PIPE CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 3

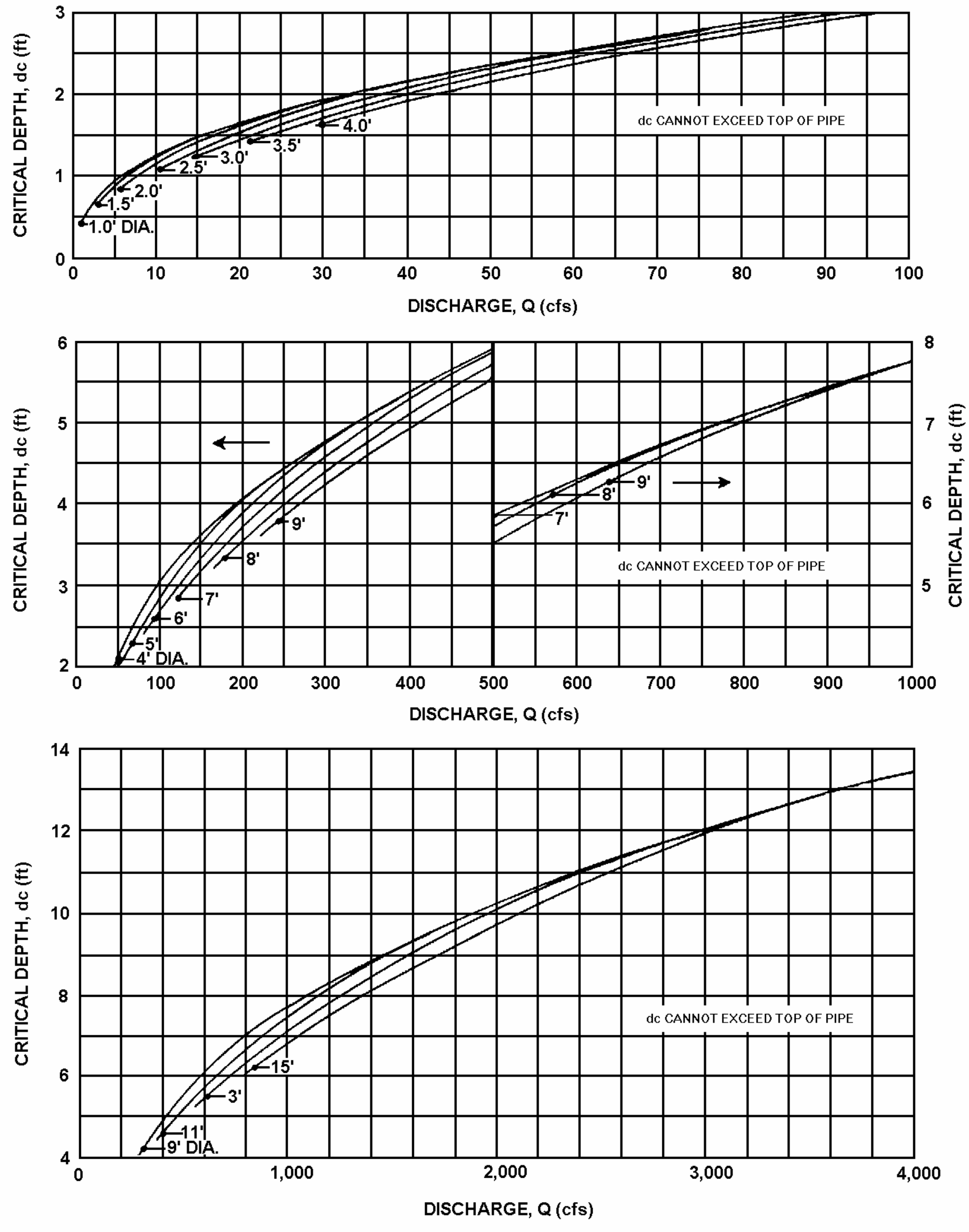
b/D	a/D	c/D	d/D	Entrance Type
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL

FEDERAL HIGHWAY ADMINISTRATION MAY 1973

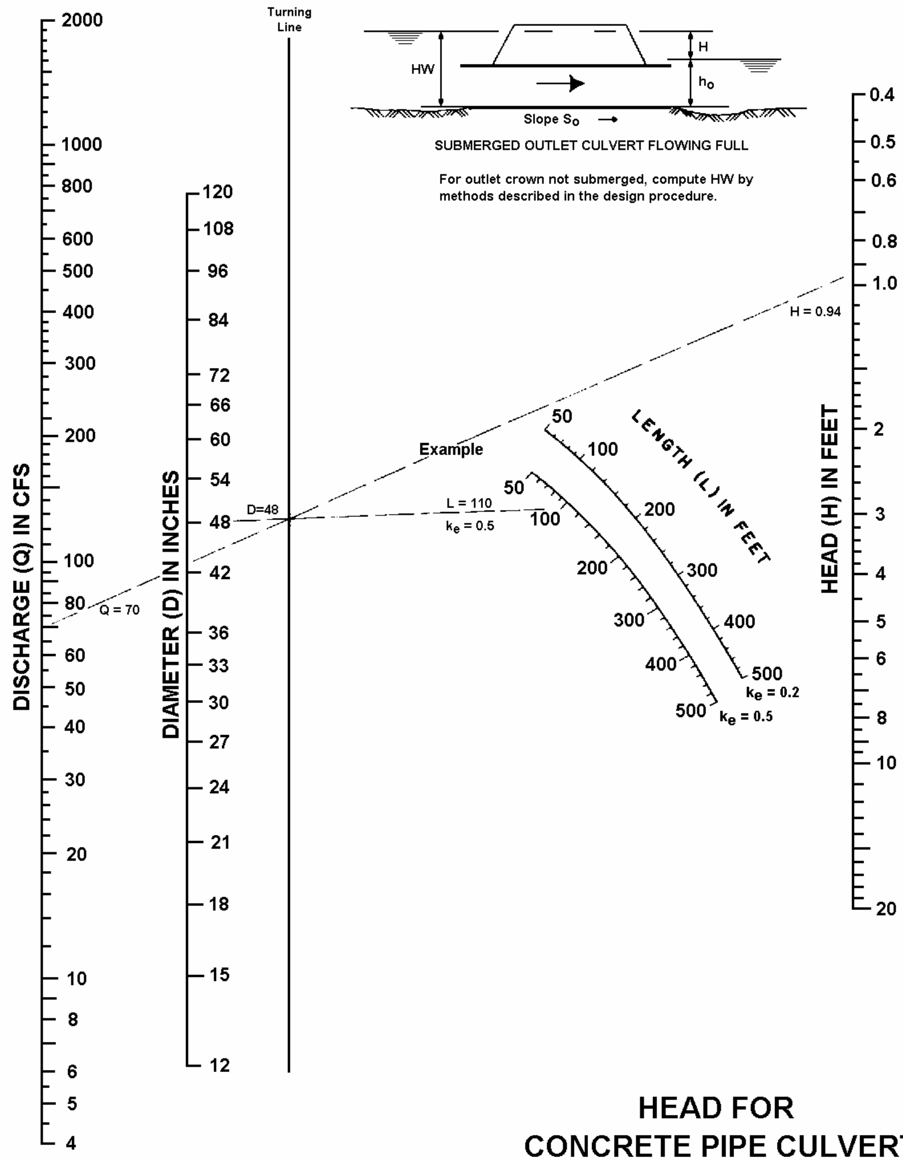
CHART 4



BUREAU OF PUBLIC ROADS JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

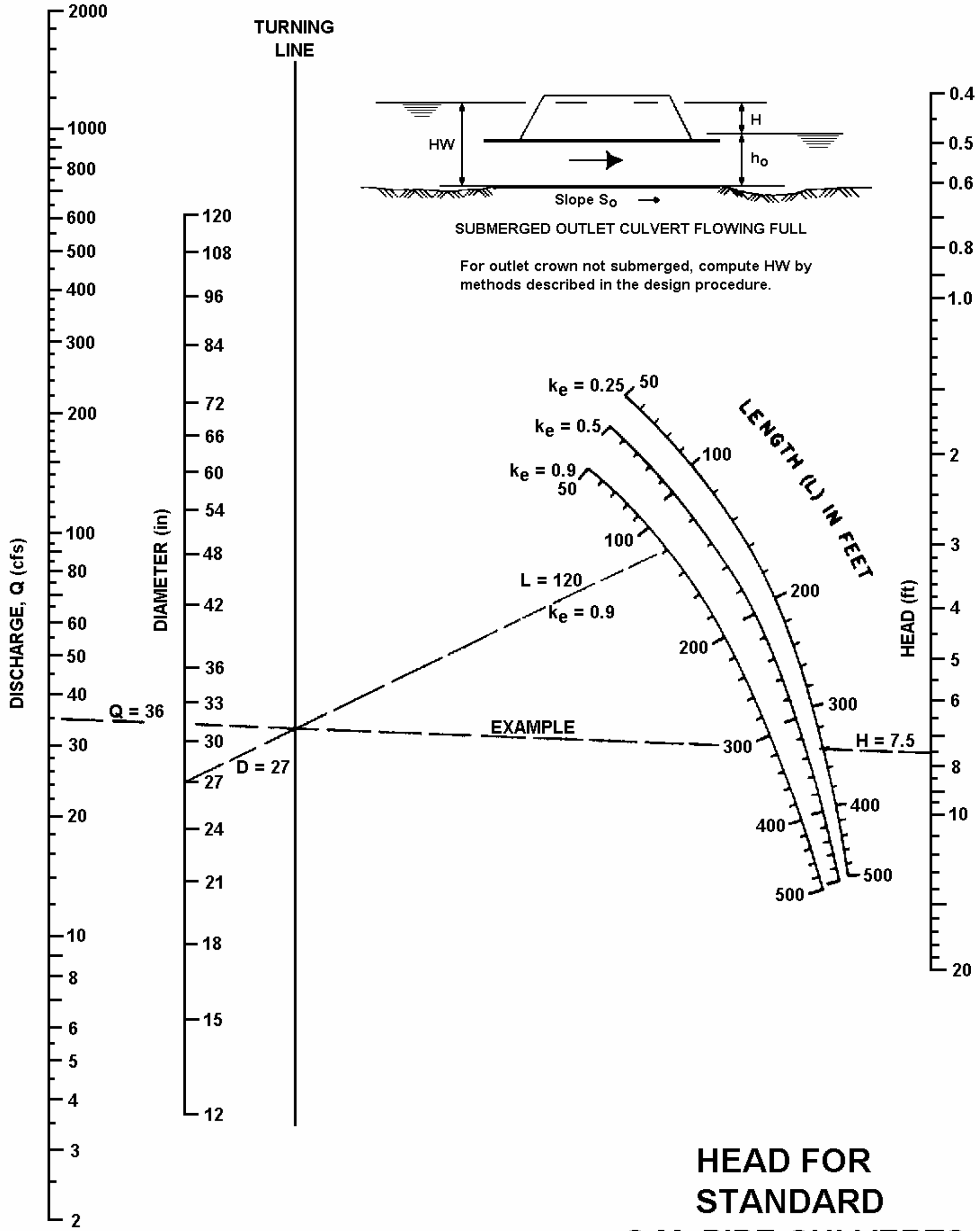
CHART 5



HEAD FOR CONCRETE PIPE CULVERTS FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

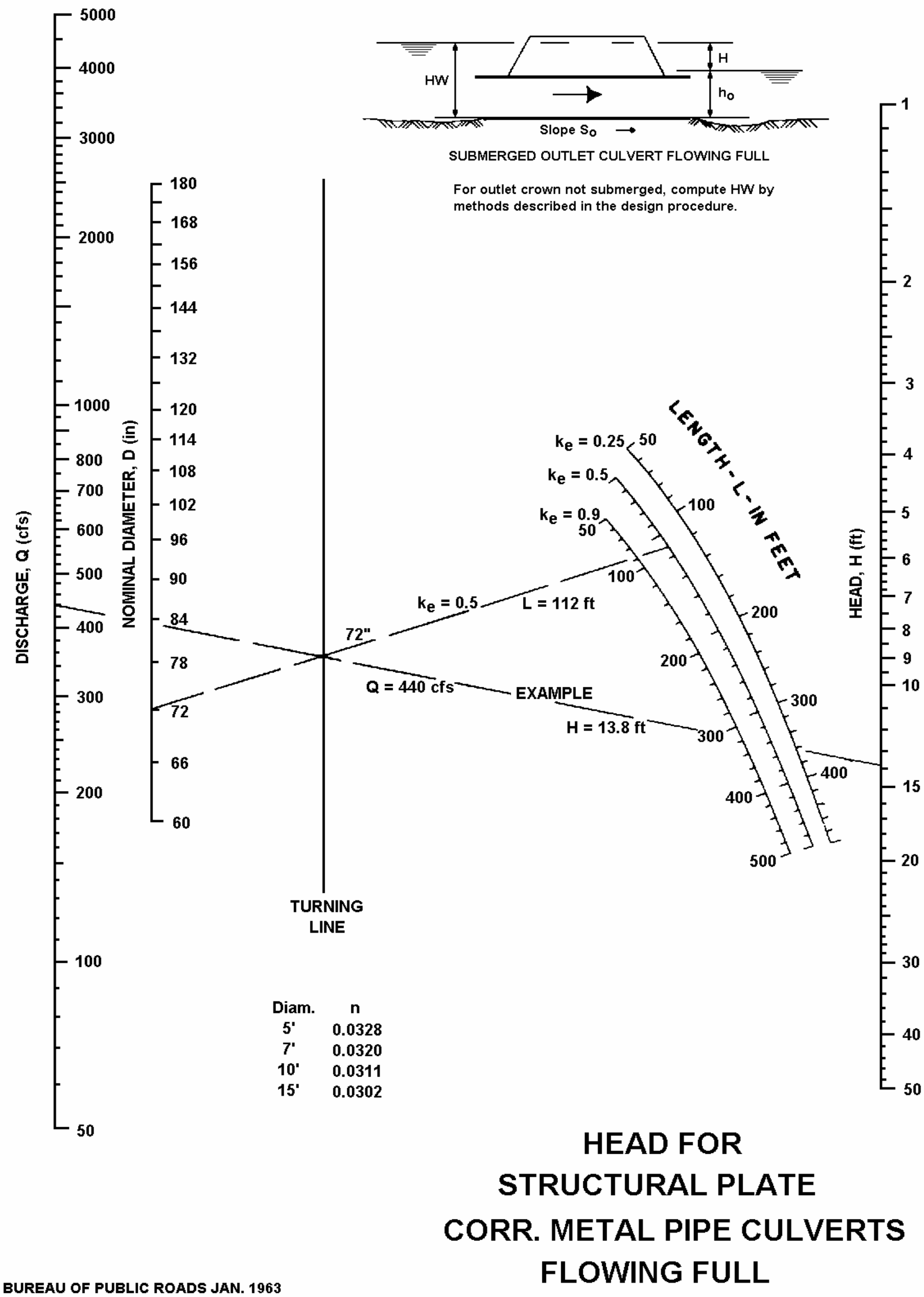
CHART 6



HEAD FOR STANDARD C.M. PIPE CULVERTS FLOWING FULL n = 0.024

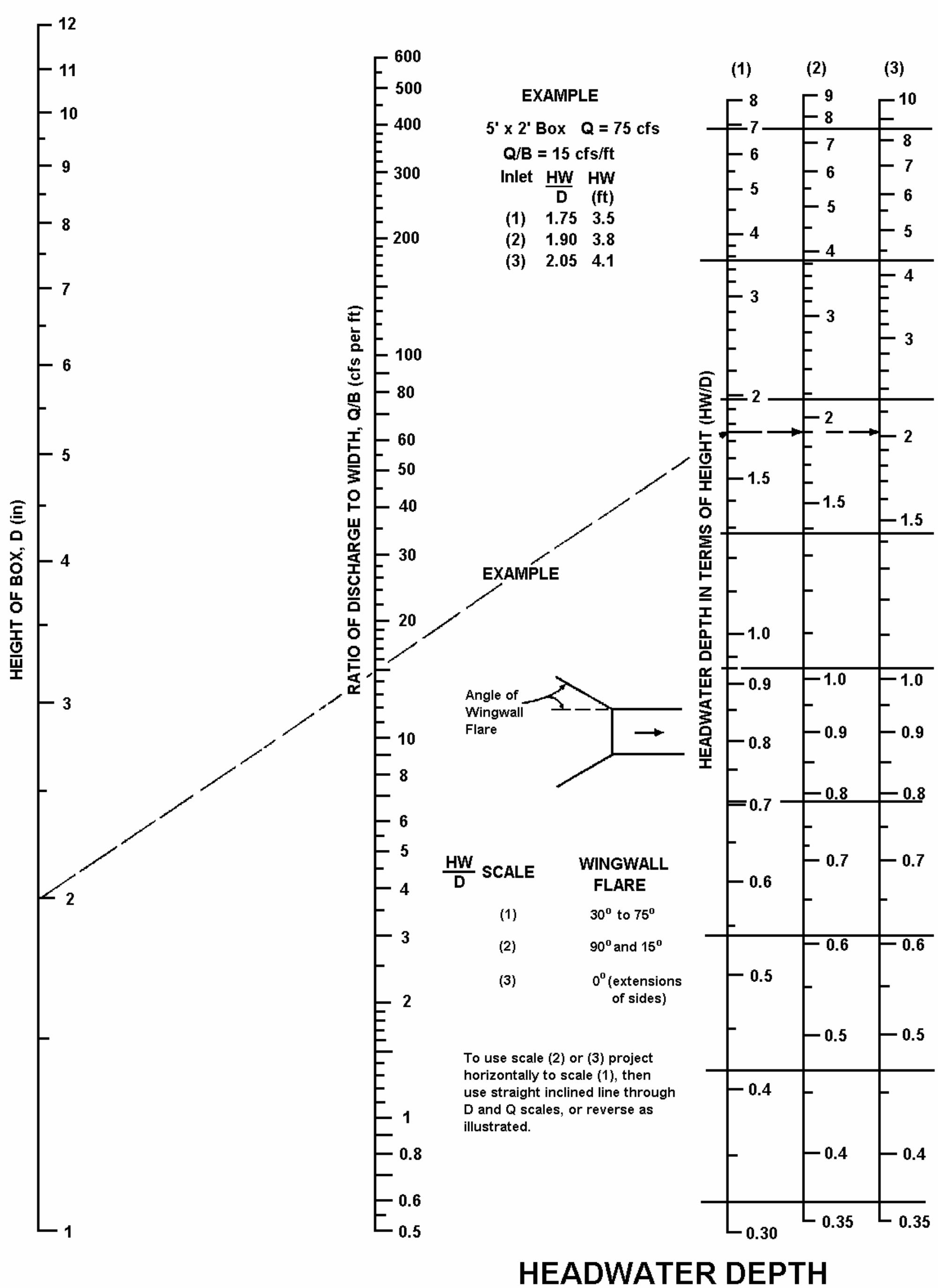
BUREAU OF PUBLIC ROADS JAN. 1963

CHART 7



HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL n = 0.0328 TO 0.0302

CHART 8



BUREAU OF PUBLIC ROADS JAN. 1963

HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

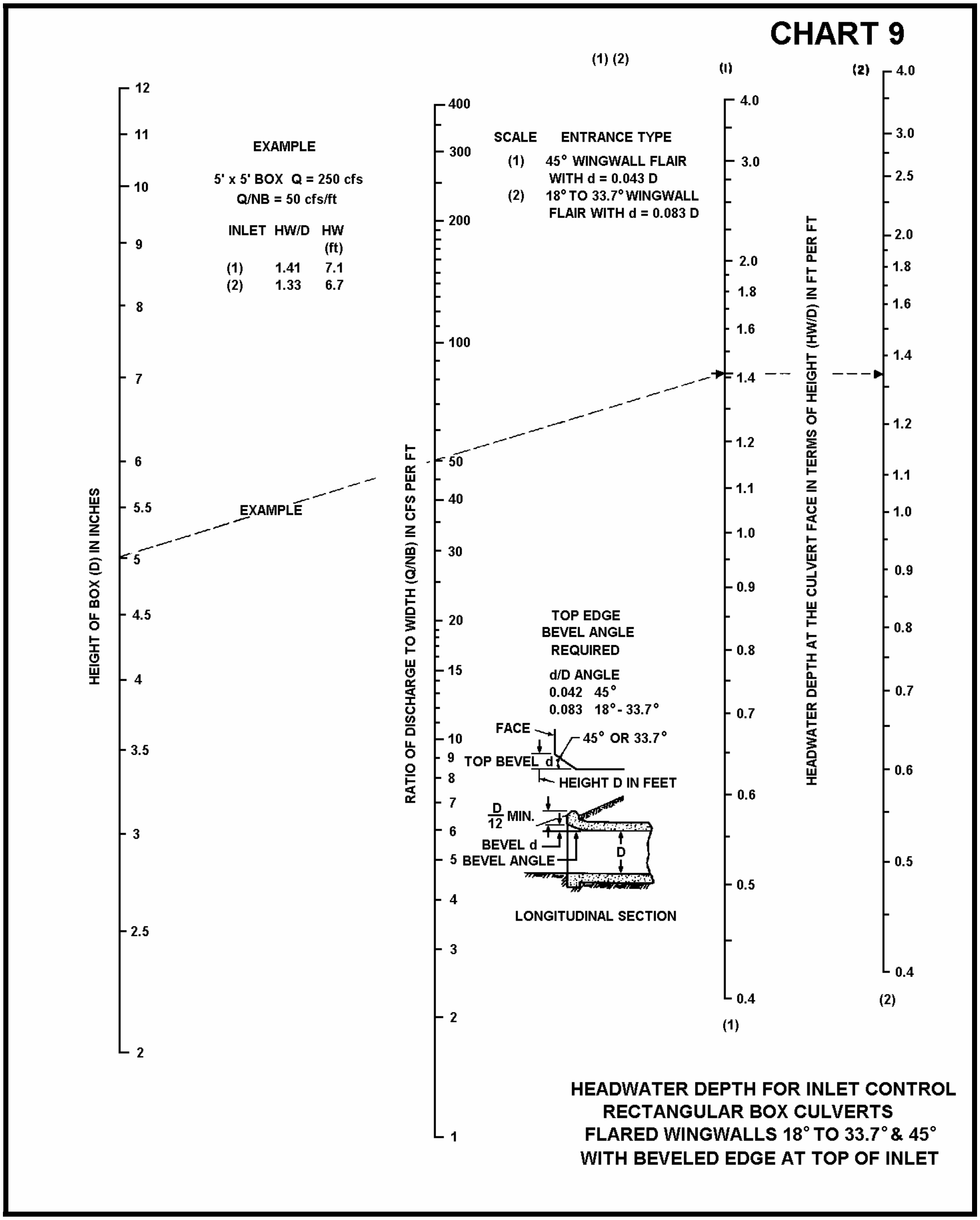
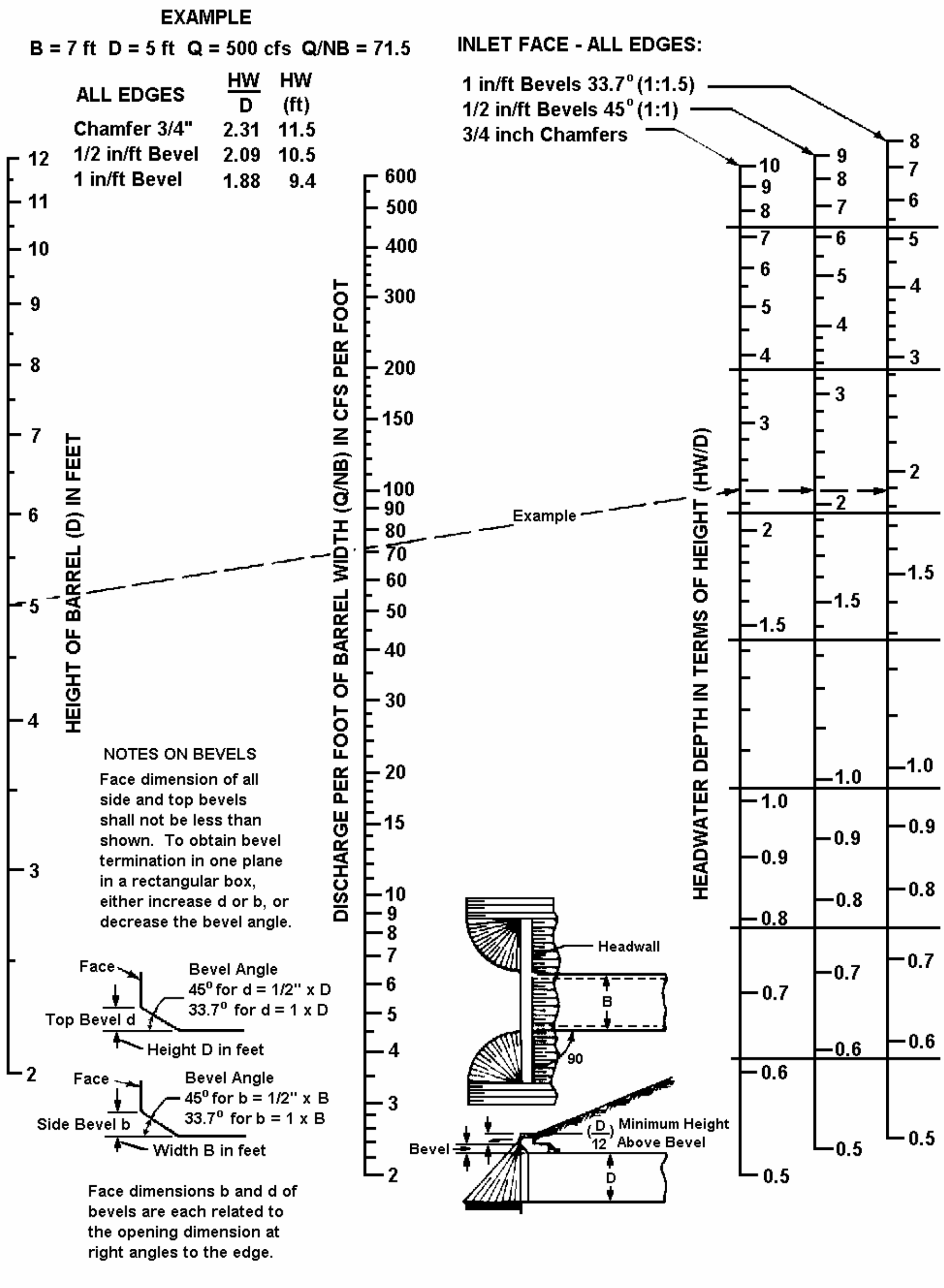


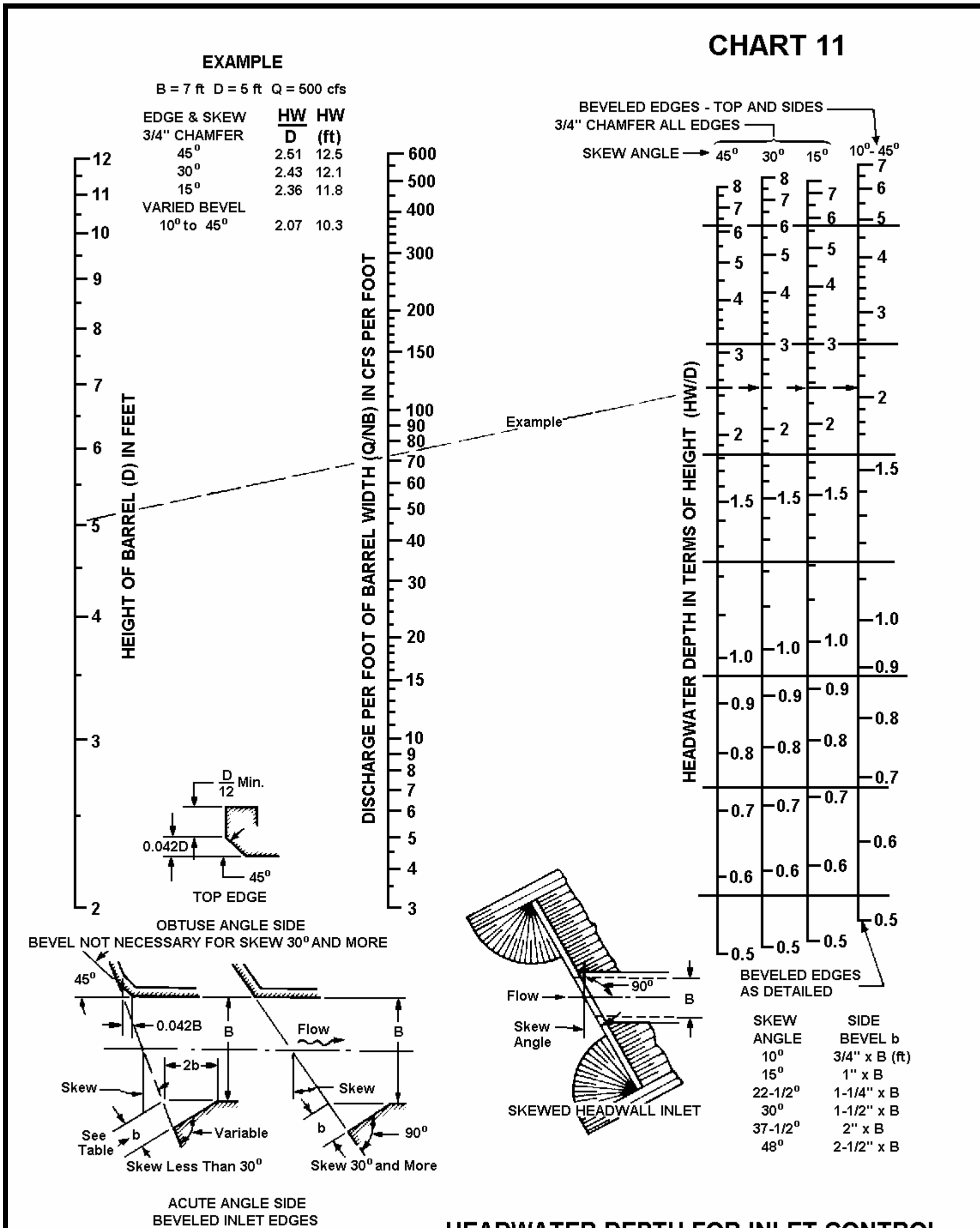
CHART 9
EXAMPLE
5' x 5' BOX Q = 250 cfs
Q/NB = 50 cfs/ft
INLET HW/D HW (ft)
(1) 1.41 7.1
(2) 1.33 6.7
SCALE ENTRANCE TYPE
(1) 45° WINGWALL FLAIR WITH d = 0.043 D
(2) 18° TO 33.7° WINGWALL FLAIR WITH d = 0.083 D
HEIGHT OF BOX (D) IN INCHES
RATIO OF DISCHARGE TO WIDTH (Q/NB) IN CFS PER FT
HEADWATER DEPTH AT THE CULVERT FACE IN TERMS OF HEIGHT (HW/D) IN FT PER FT
EXAMPLE
TOP EDGE BEVEL ANGLE REQUIRED
d/D ANGLE
0.042 45°
0.083 18° - 33.7°
FACE
45° OR 33.7°
TOP BEVEL d
HEIGHT D IN FEET
D/12 MIN.
BEVEL d
BEVEL ANGLE
D
LONGITUDINAL SECTION
HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° & 45°
WITH BEVELED EDGE AT TOP OF INLET

CHART 10



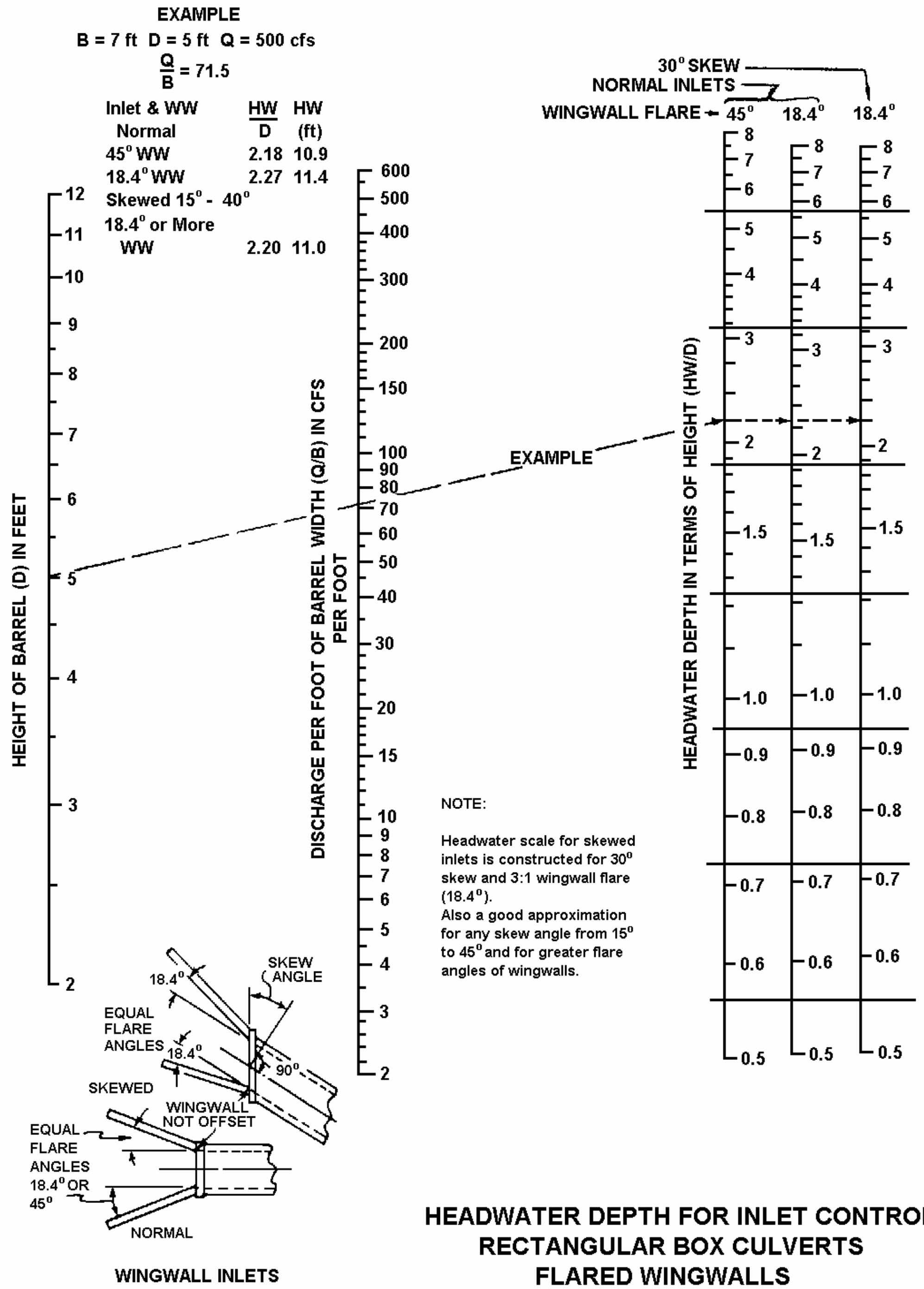
FEDERAL HIGHWAY ADMINISTRATION MAY 1973

HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
90° HEADWALL
CHAMFERED OR BEVELED INLET EDGES



HEADWATER DEPTH FOR INLET CONTROL
SINGLE BARREL BOX CULVERTS
SKEWED HEADWALLS
CHAMFERED OR BEVELED INLET EDGES

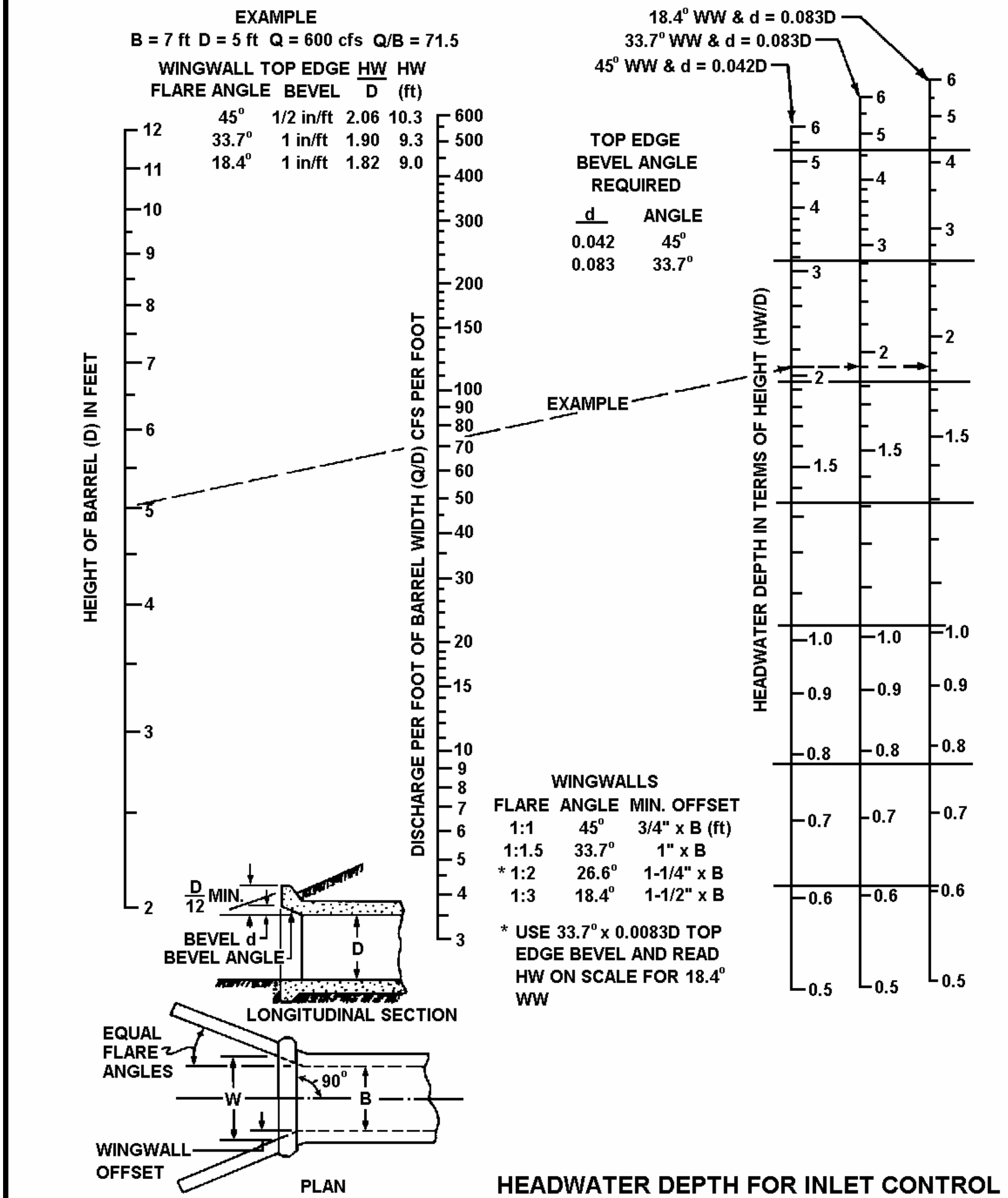
CHART 12



HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS
NORMAL AND SKEWED INLETS
3/4" CHAMFER AT TOP OF OPENING

BUREAU OF PUBLIC ROADS
OFFICE OF R&D AUGUST 1968

CHART 13



HEADWATER DEPTH FOR INLET CONTROL RECTANGULAR BOX CULVERTS OFFSET FLARED WINGWALLS AND BEVELED EDGE AT TOP OF INLET

BUREAU OF PUBLIC ROADS
OFFICE OF R&D AUGUST 1968

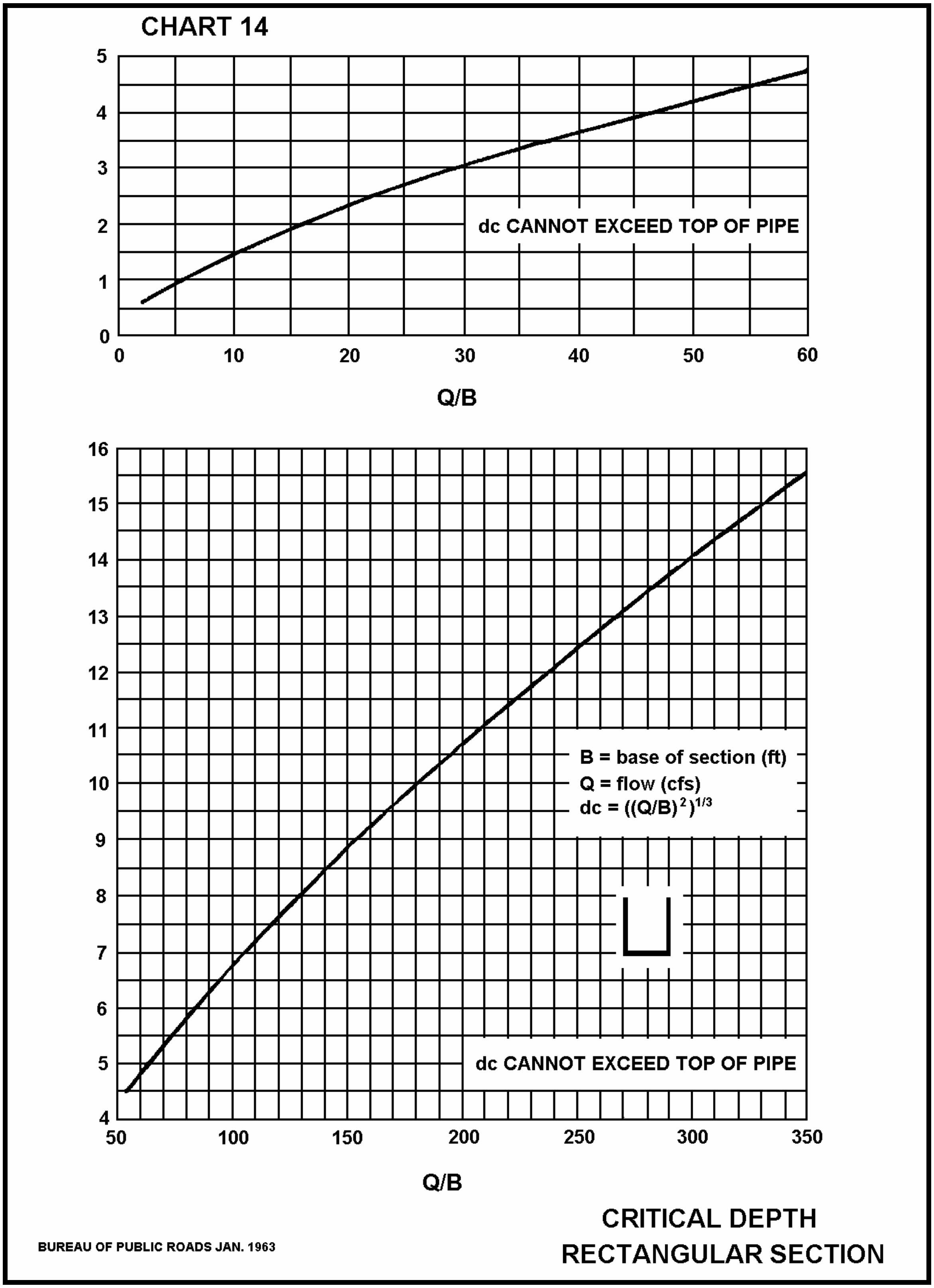
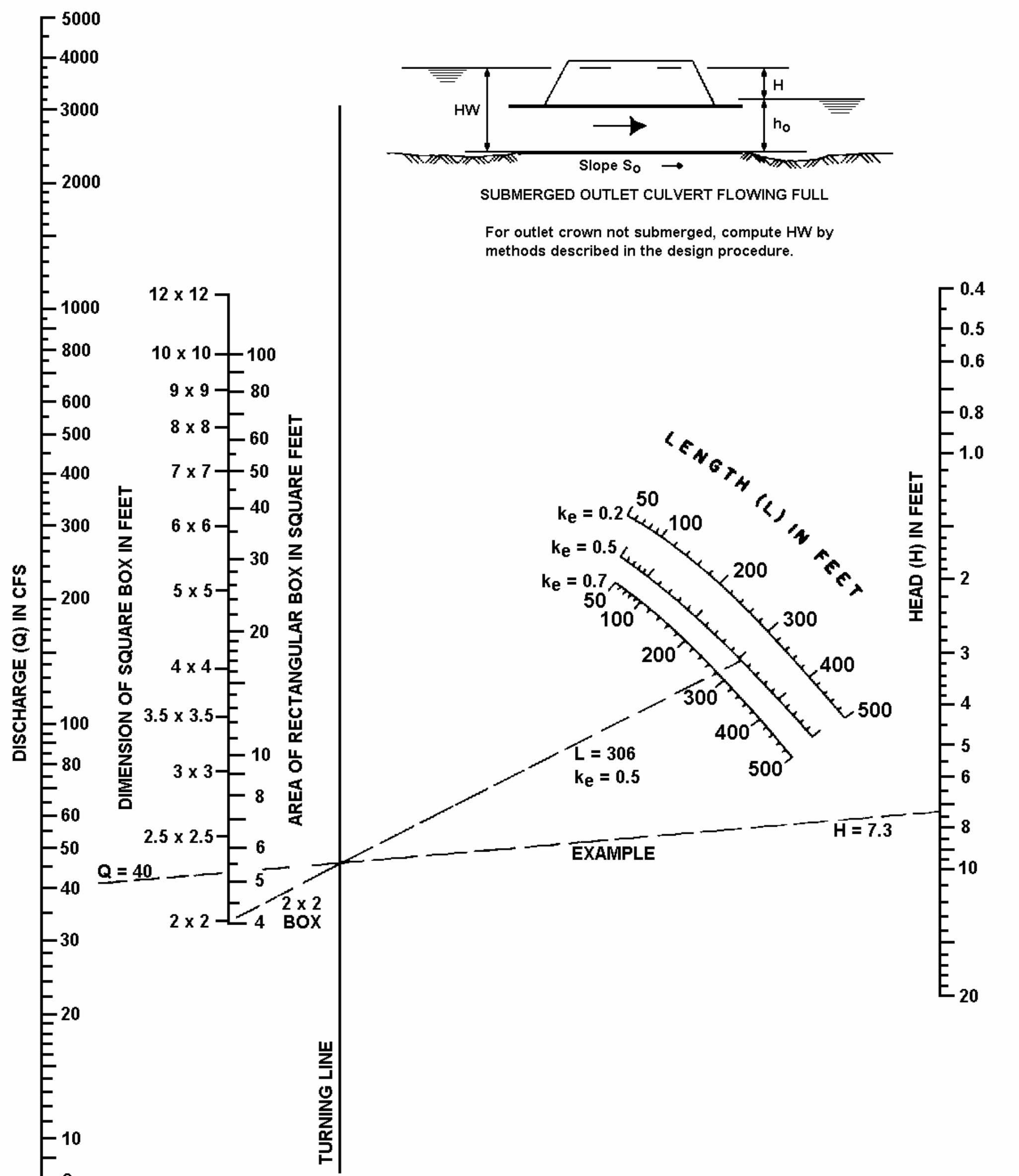


CHART 14
5
4
3
2
1
0
0
10
20
30
40
50
60
Q/B
dc CANNOT EXCEED TOP OF PIPE
16
15
14
13
12
11
10
9
8
7
6
5
4
50
100
150
200
250
300
350
Q/B
B = base of section (ft)
Q = flow (cfs)
dc = ((Q/B)²)^1/3
dc CANNOT EXCEED TOP OF PIPE
BUREAU OF PUBLIC ROADS JAN. 1963
CRITICAL DEPTH
RECTANGULAR SECTION

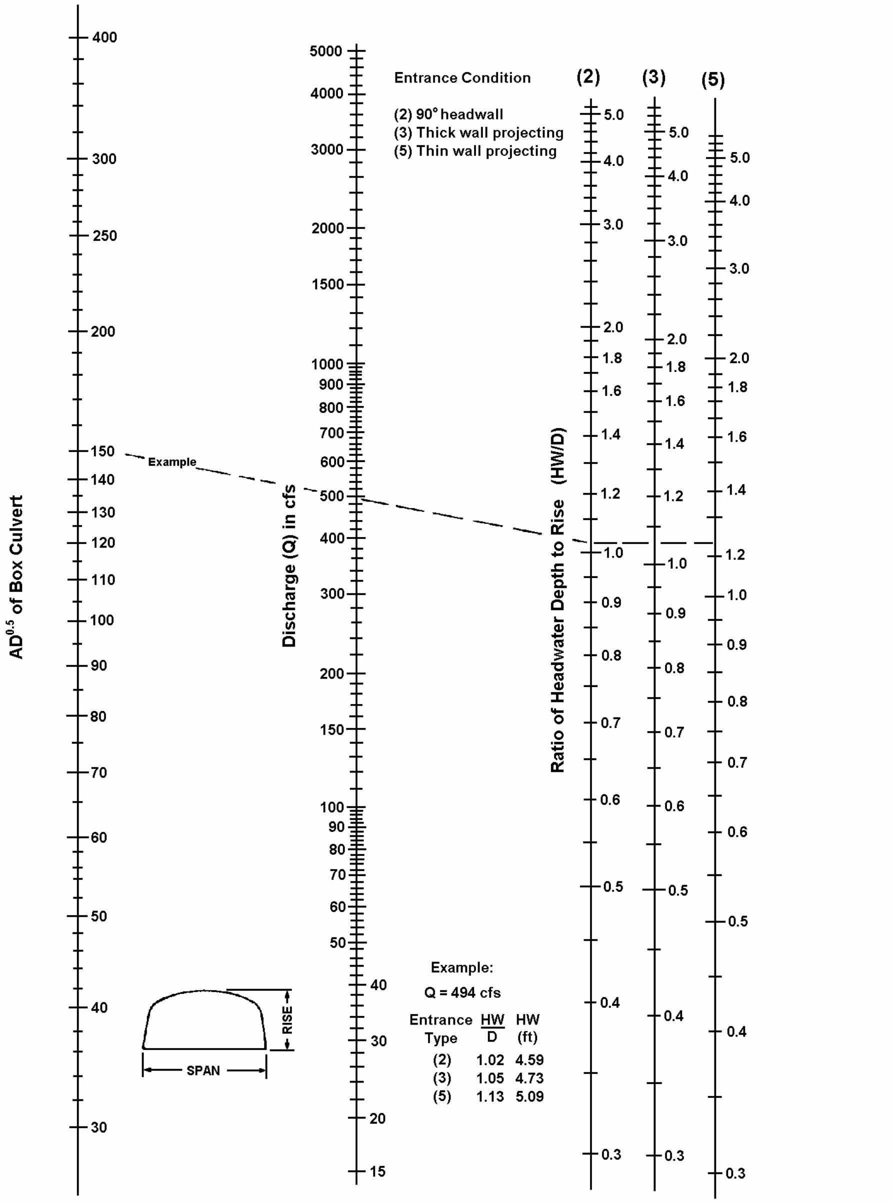
CHART 15



HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

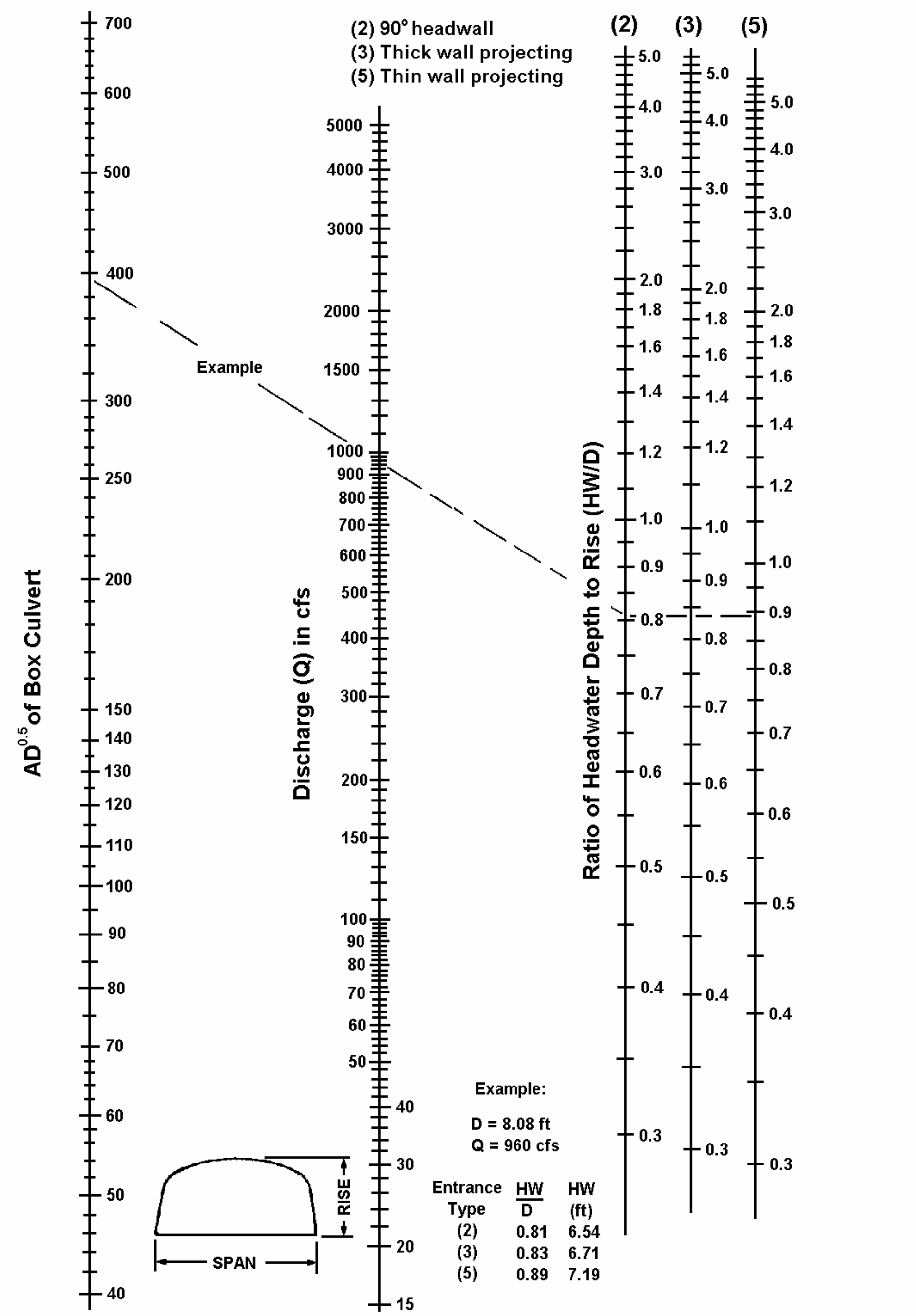
CHART 16



Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.

HEADWATER DEPTH FOR C.M. BOX CULVERTS RISE/SPAN < 0.3 WITH INLET CONTROL

CHART 17



HEADWATER DEPTH FOR C.M. BOX CULVERTS 0.3≤ RISE/SPAN < 0.4 WITH INLET CONTROL

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.
Duplication of this nomograph may distort scale.

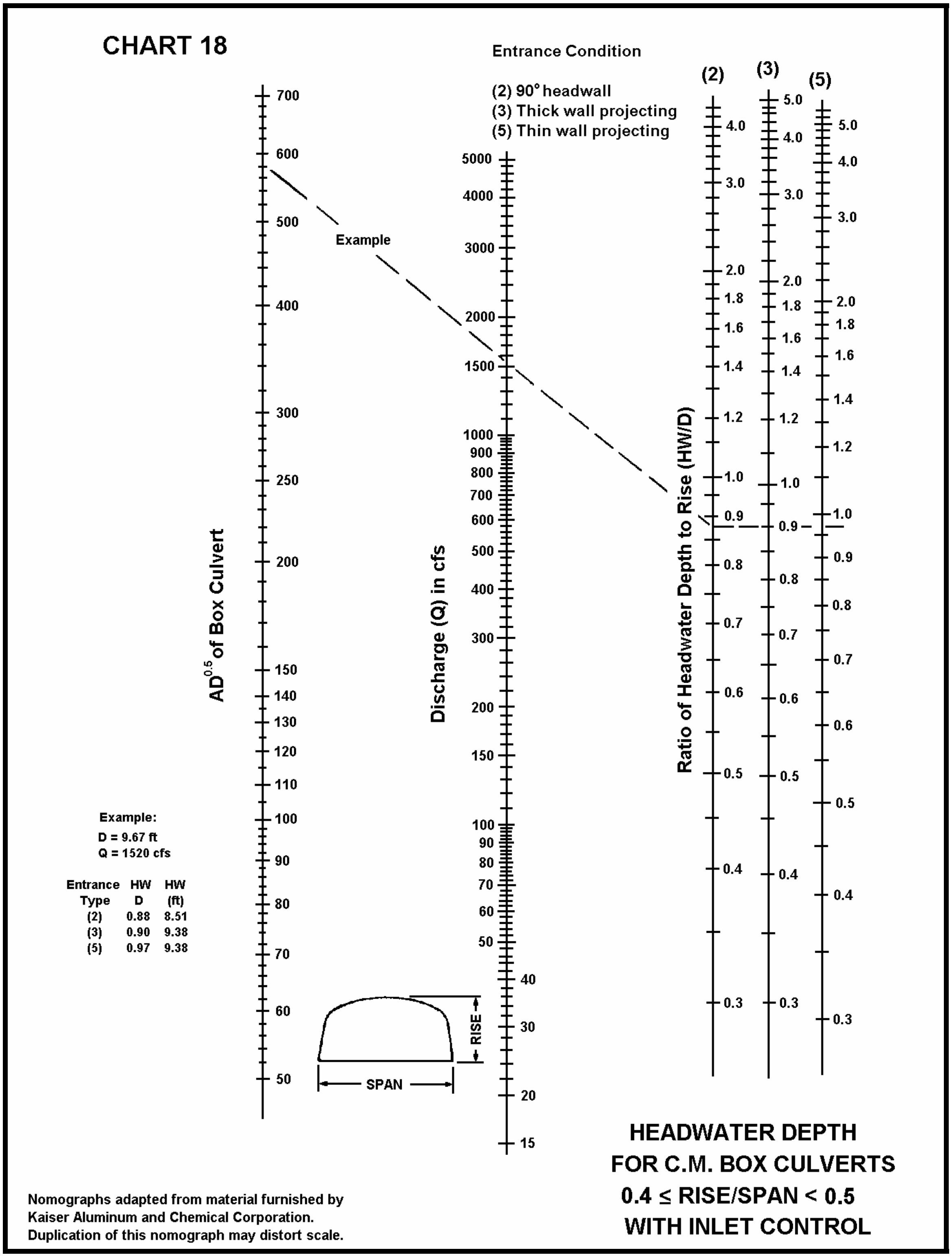
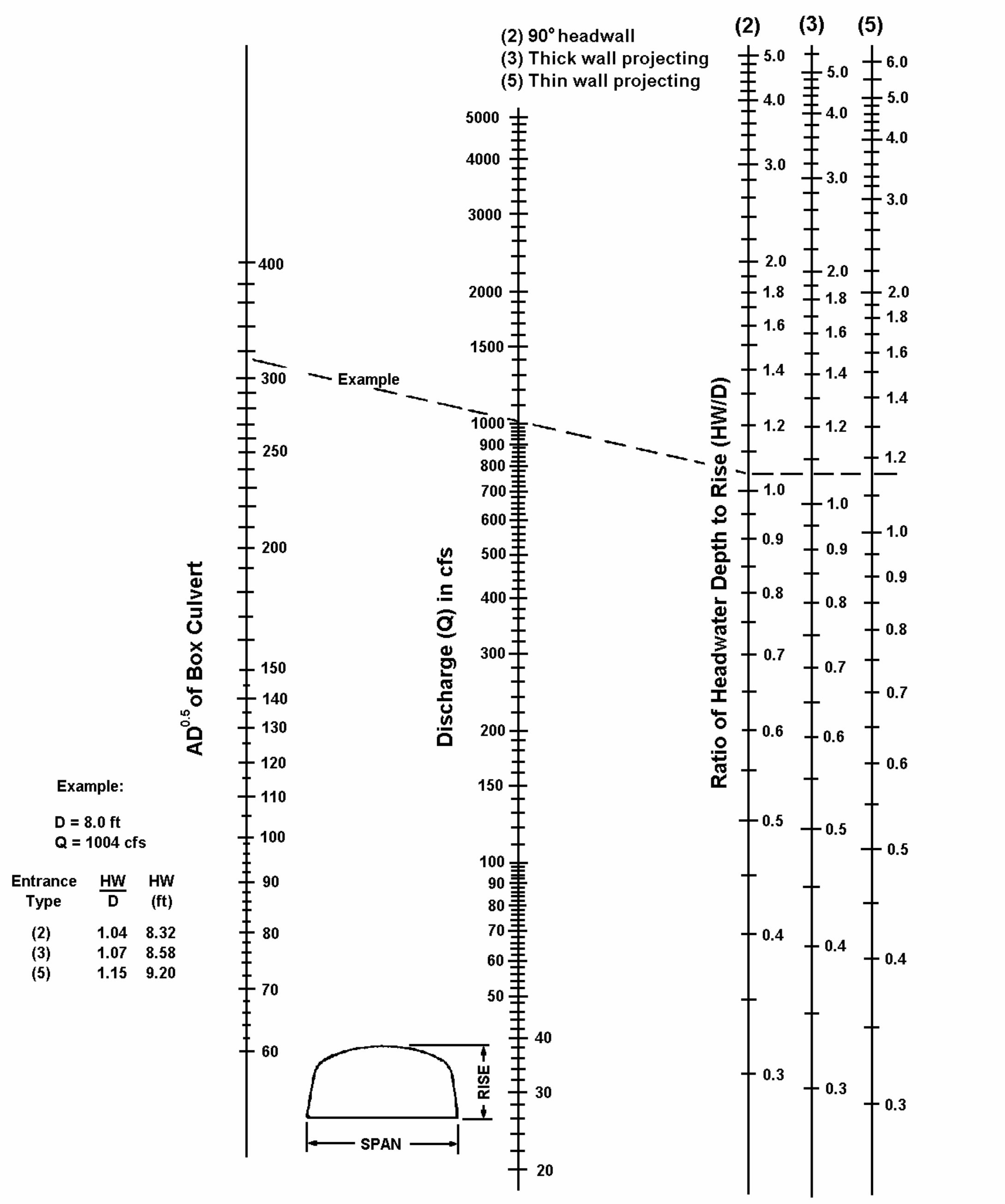


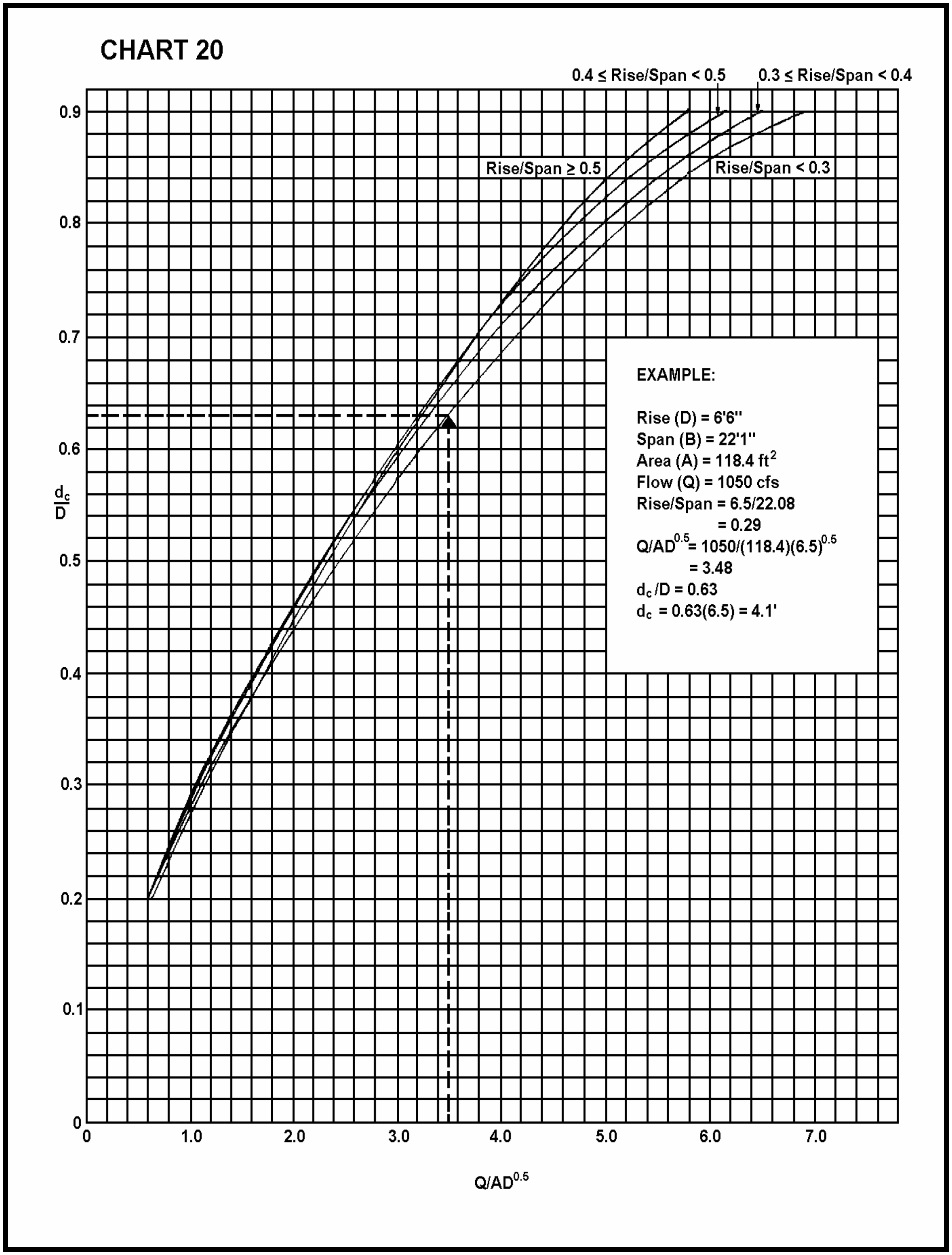
CHART 18
Entrance Condition
(2) 90° headwall
(3) Thick wall projecting
(5) Thin wall projecting
(2)
(3)
(5)
Example
AD^0.5 of Box Culvert
Discharge (Q) in cfs
Ratio of Headwater Depth to Rise (HW/D)
Example:
D = 9.67 ft
Q = 1520 cfs
Entrance Type | HW/D | HW (ft)
(2) | 0.88 | 8.51
(3) | 0.90 | 9.38
(5) | 0.97 | 9.38
RISE
SPAN
HEADWATER DEPTH
FOR C.M. BOX CULVERTS
0.4 ≤ RISE/SPAN < 0.5
WITH INLET CONTROL
Nomographs adapted from material furnished by
Kaiser Aluminum and Chemical Corporation.
Duplication of this nomograph may distort scale.

CHART 19



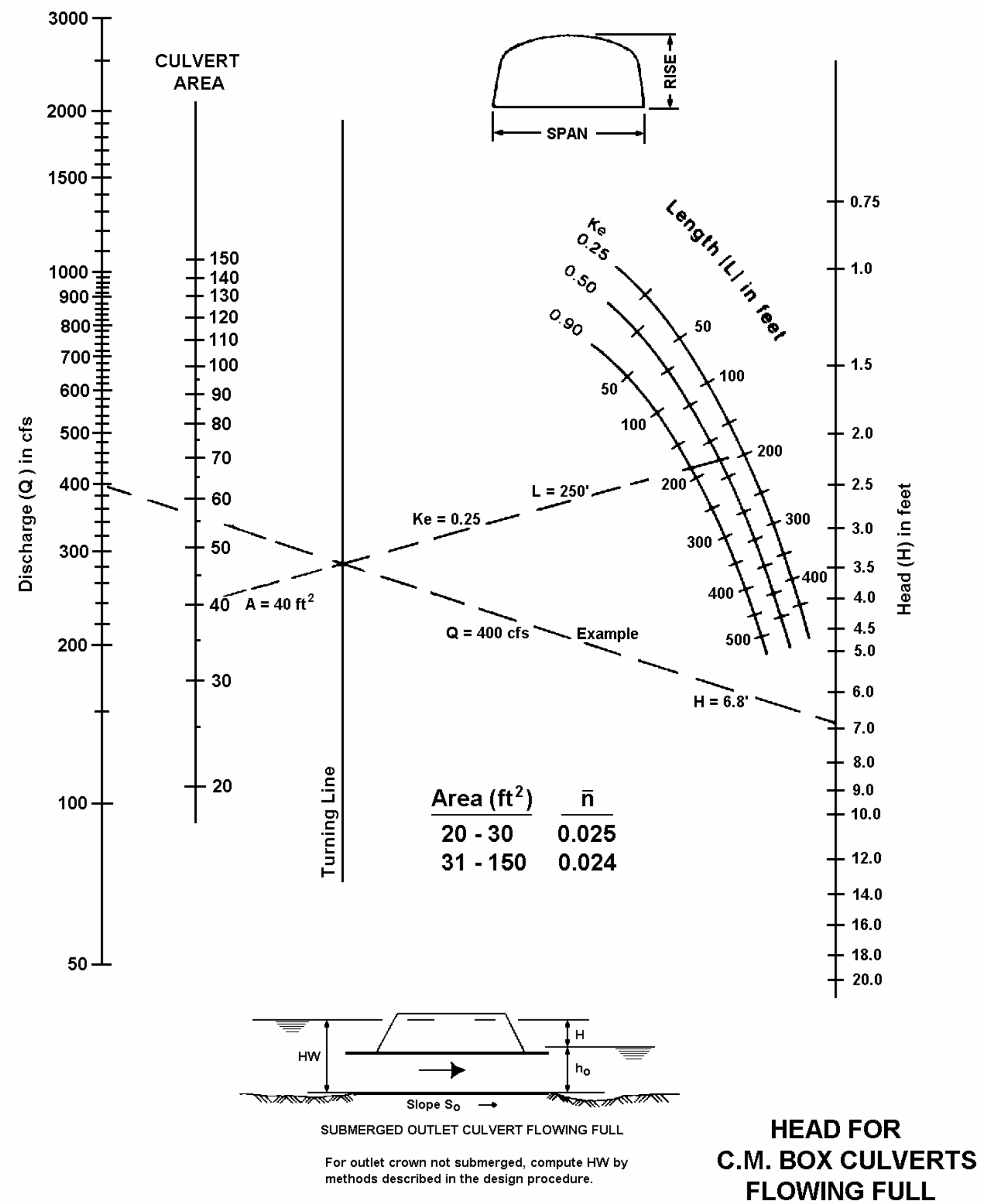
Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.

HEADWATER DEPTH FOR C.M. BOX CULVERTS $0.5 \leq$ RISE/SPAN WITH INLET CONTROL



**DIMENSIONLESS CRITICAL DEPTH
FOR C.M. BOX CULVERTS**

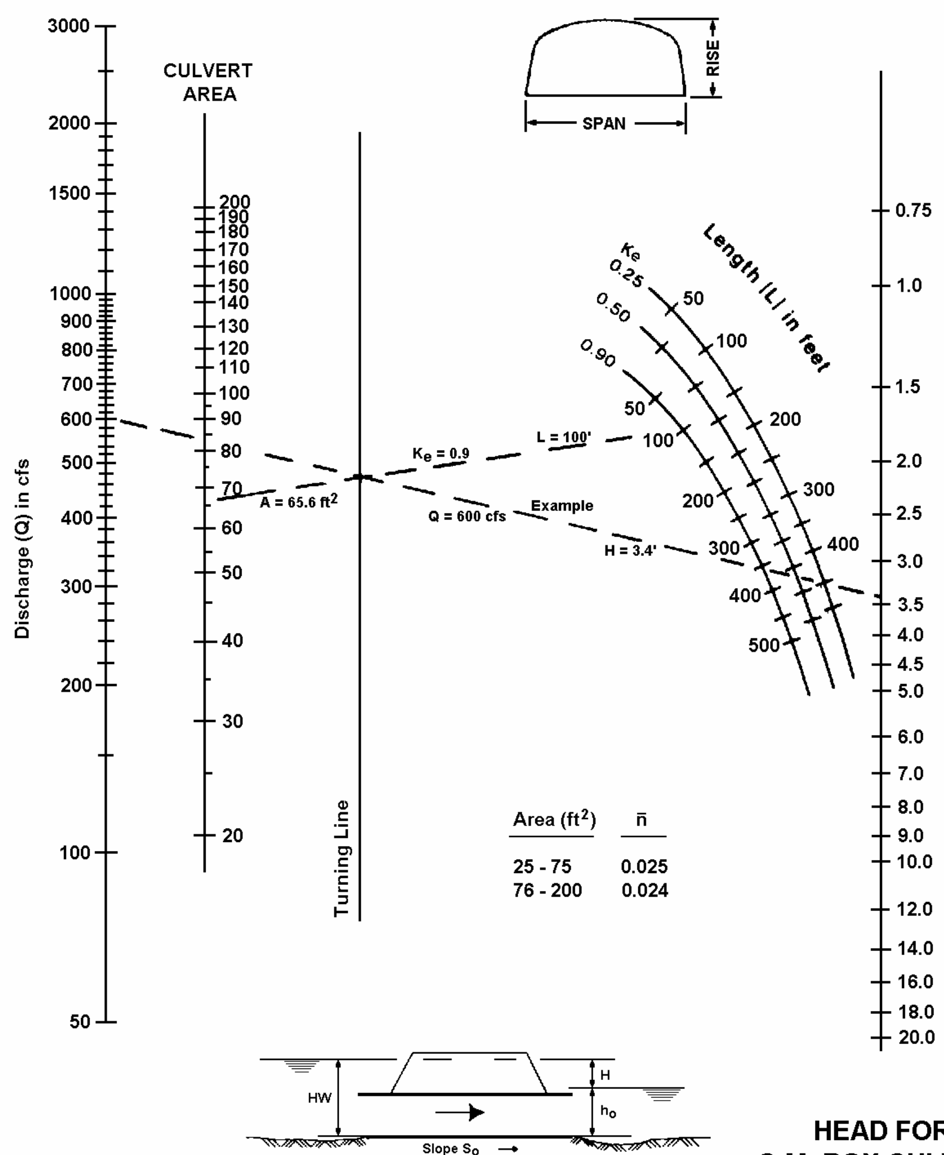
CHART 21



HEAD FOR
C.M. BOX CULVERTS
FLOWING FULL
CONCRETE BOTTOM
RISE/SPAN < 0.3

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.
Duplication of this nomograph may distort scale.

CHART 22



Area (ft²)	n̄
25 - 75	0.025
76 - 200	0.024

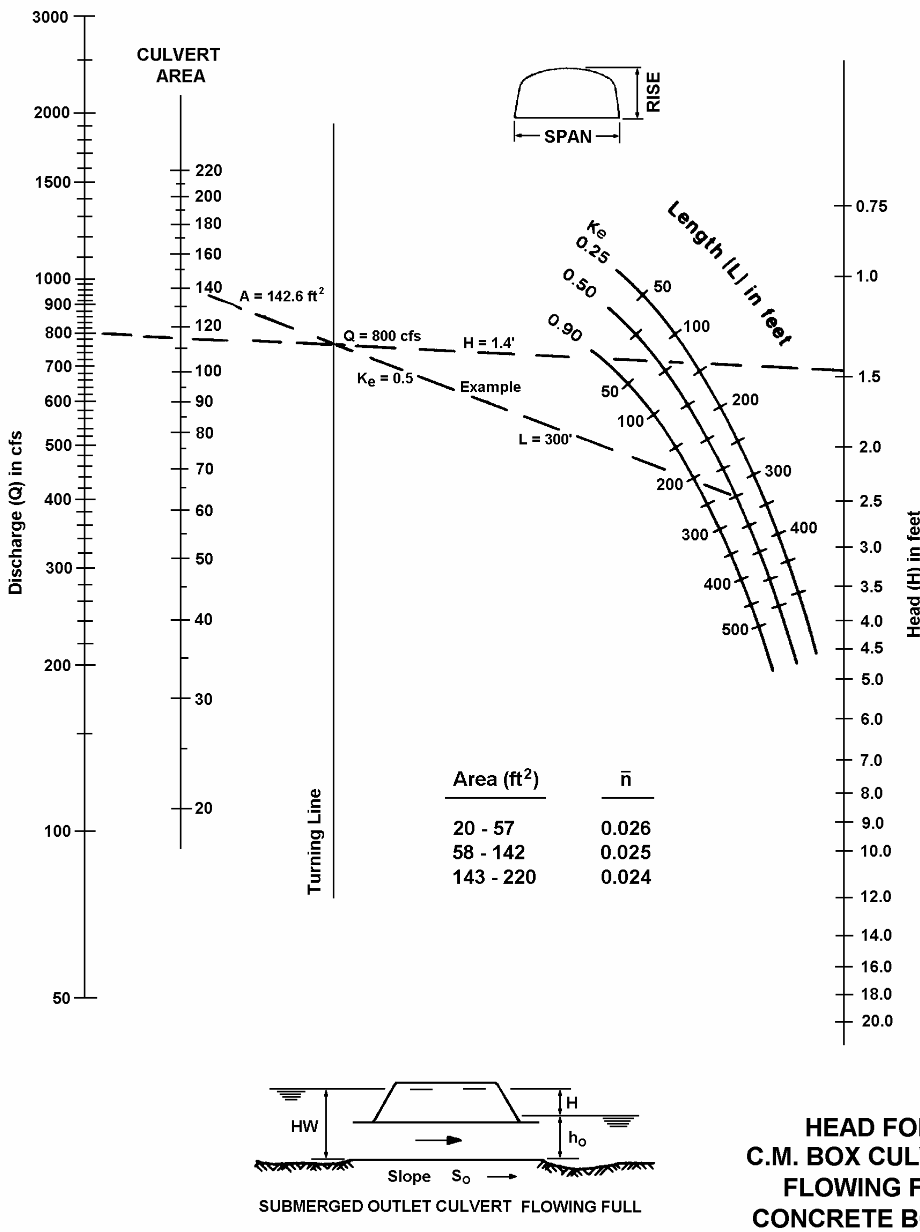
SUBMERGED OUTLET CULVERT FLOWING FULL

For outlet crown not submerged, compute HW by methods described in the design procedure.

HEAD FOR C.M. BOX CULVERTS FLOWING FULL CONCRETE BOTTOM $0.3 \leq$ RISE/SPAN < 0.4

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

CHART 23

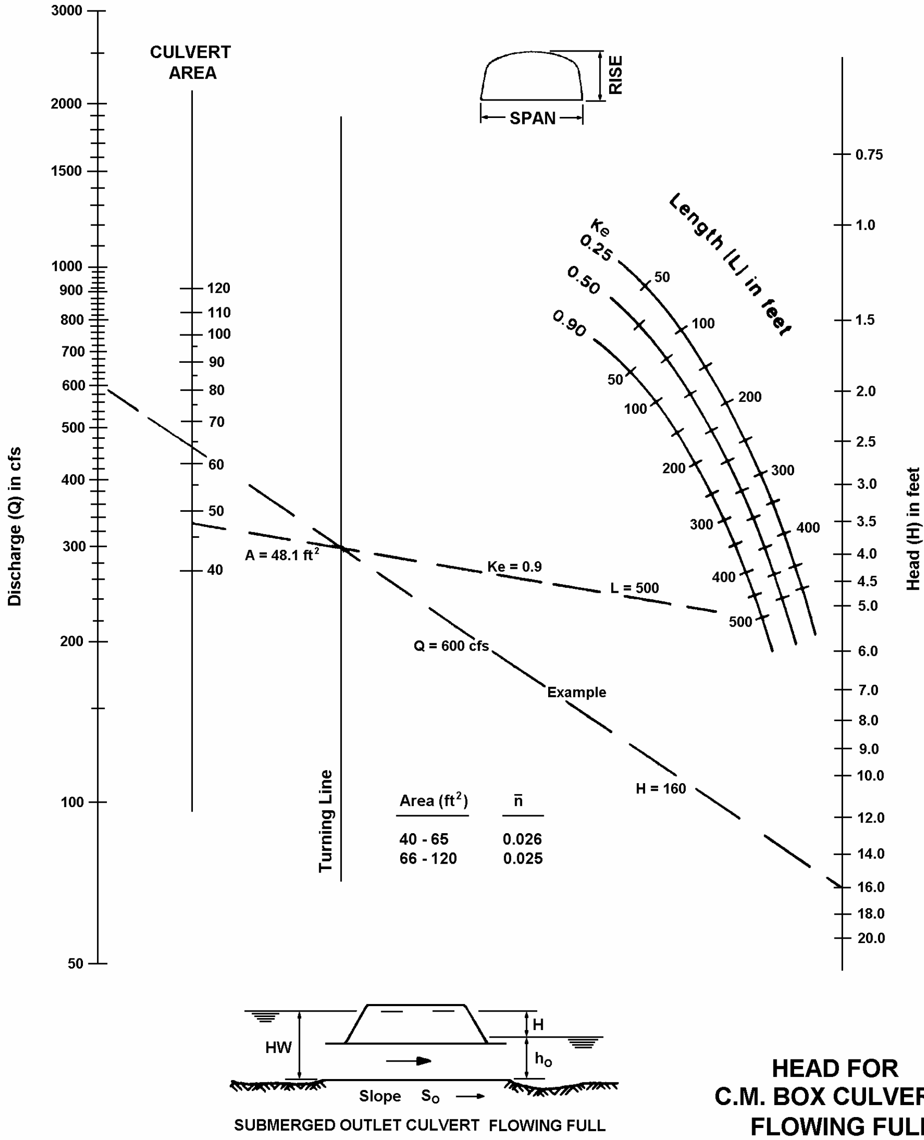


Area (ft^2)	$\bar{n}$
20 - 57	0.026
58 - 142	0.025
143 - 220	0.024

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

HEAD FOR C.M. BOX CULVERTS FLOWING FULL CONCRETE BOTTOM $0.4 \leq$ RISE/SPAN < 0.5

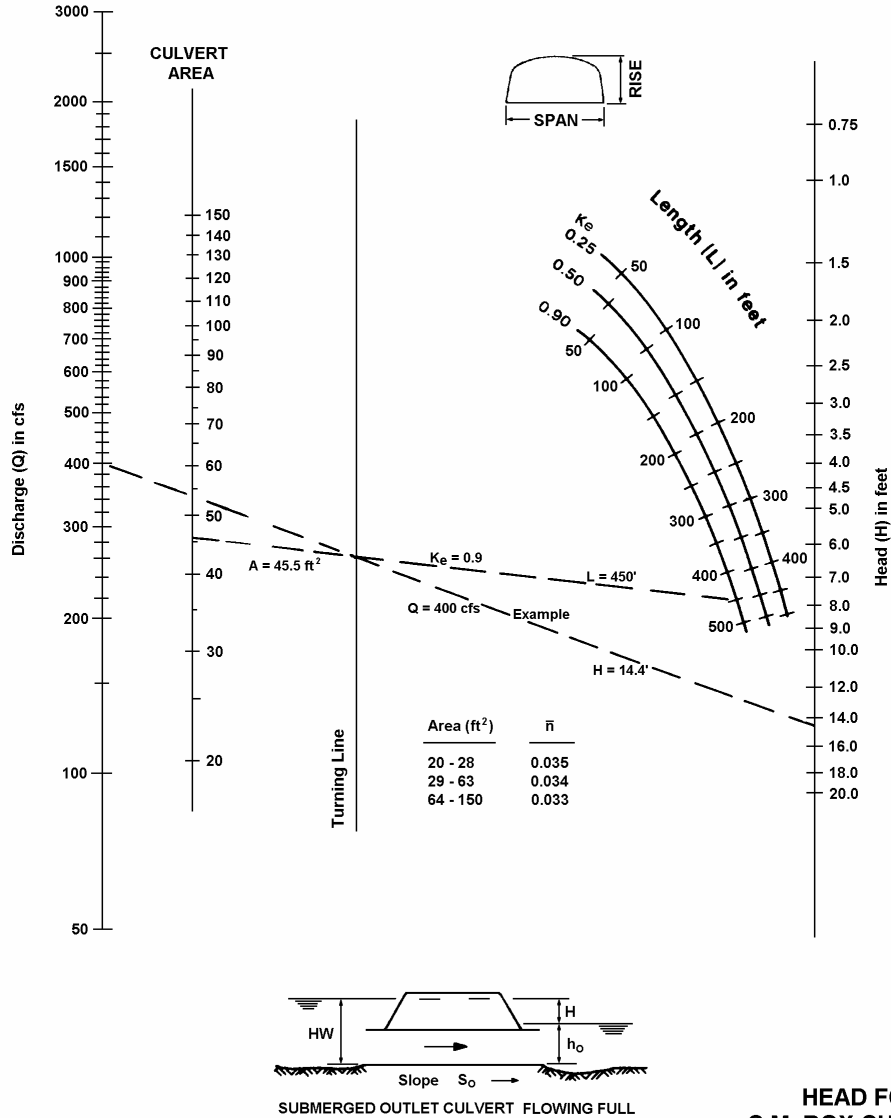
CHART 24



HEAD FOR
C.M. BOX CULVERTS
FLOWING FULL
CONCRETE BOTTOM
0.5 ≤ RISE/SPAN

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

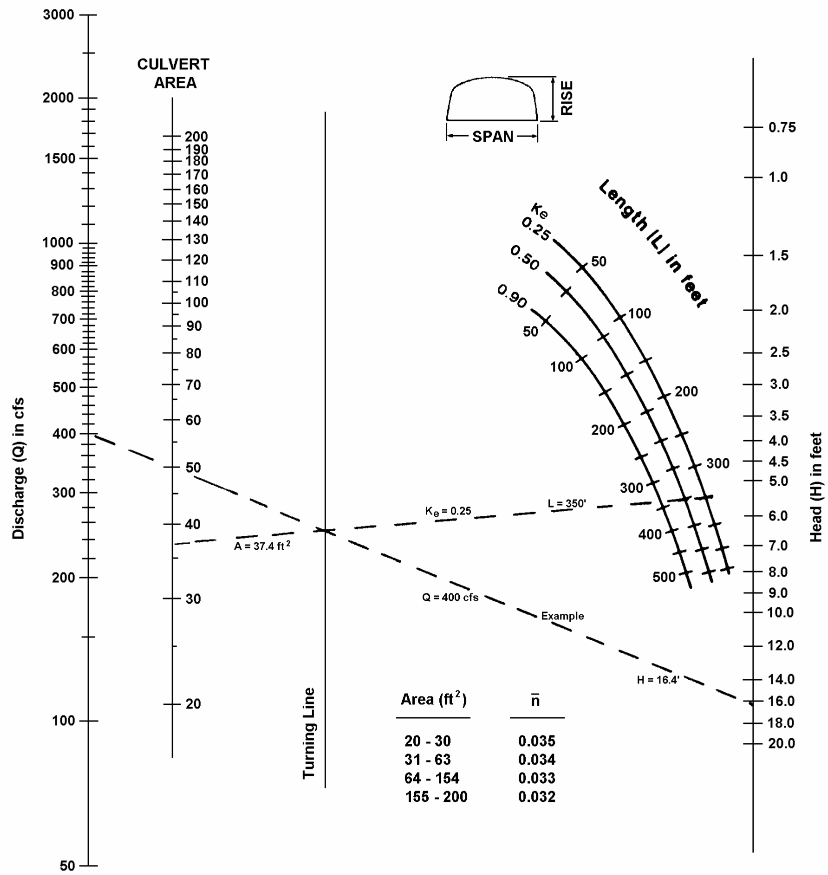
CHART 25



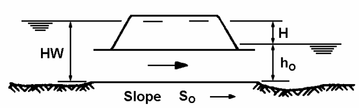
Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

HEAD FOR C.M. BOX CULVERTS FLOWING FULL CORRUGATED METAL BOTTOM 0.3< RISE/SPAN

CHART 26



Area (ft²)	n̄
20 - 30	0.035
31 - 63	0.034
64 - 154	0.033
155 - 200	0.032

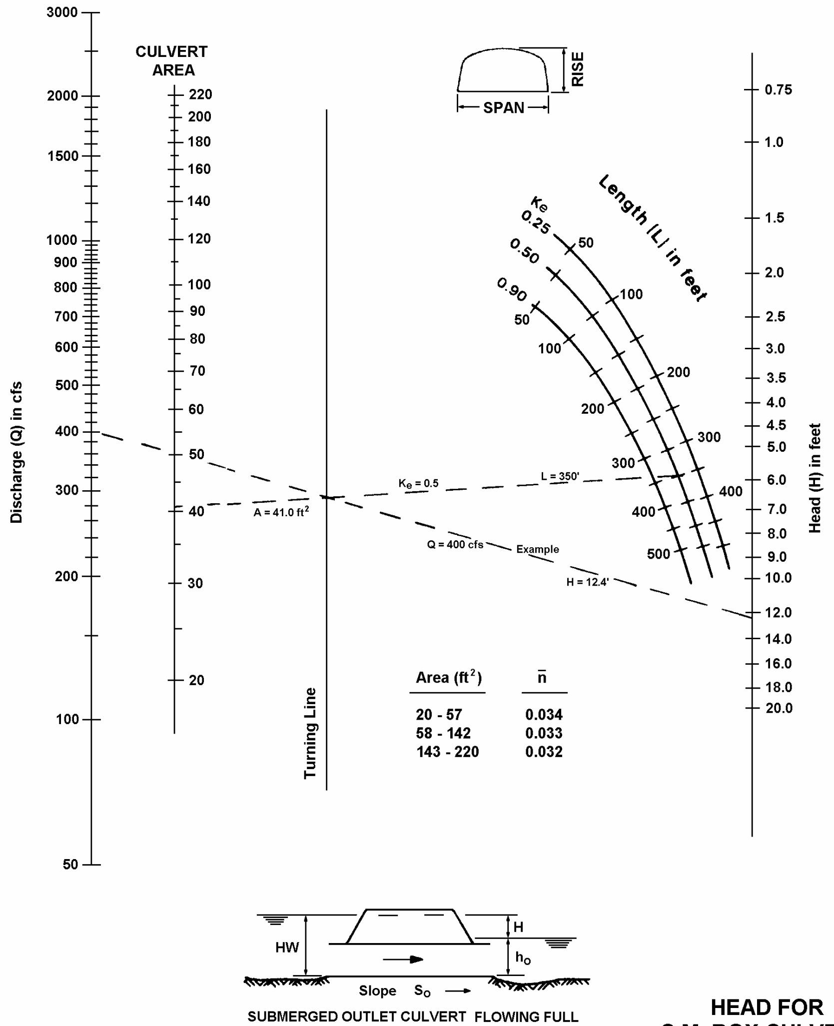


SUBMERGED OUTLET CULVERT FLOWING FULL

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

HEAD FOR C.M. BOX CULVERTS FLOWING FULL CORRUGATED METAL BOTTOM $0.4 \le$ RISE/SPAN < 0.5

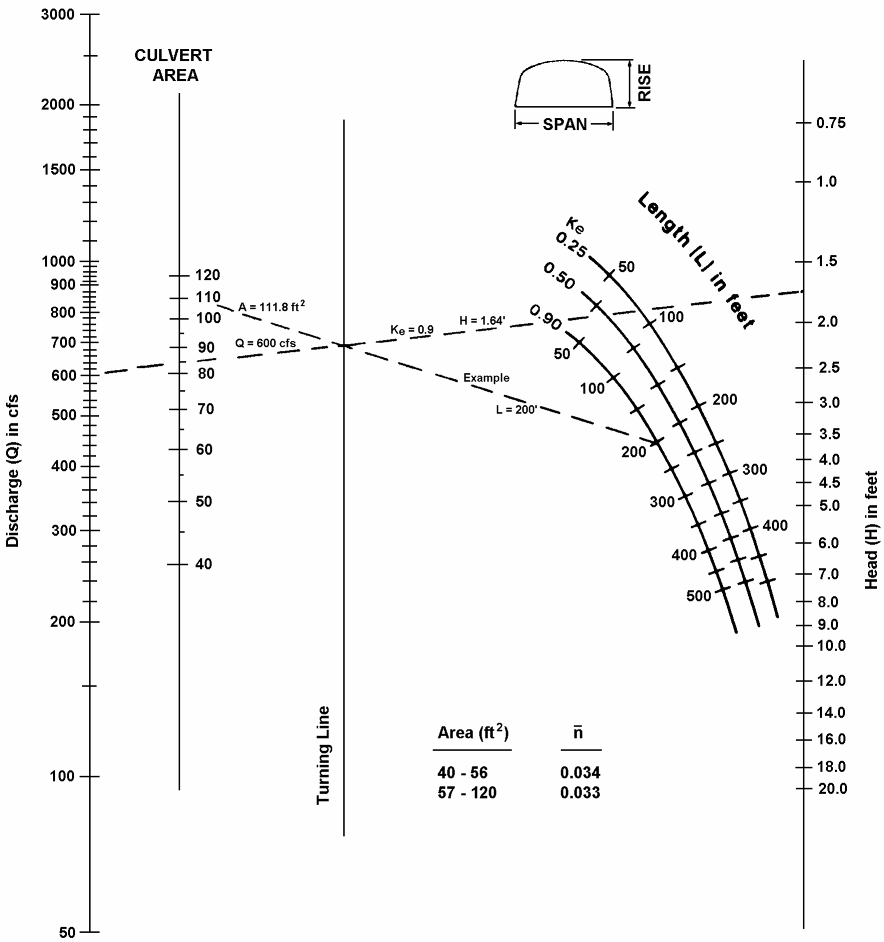
CHART 27



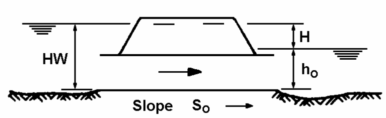
Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

HEAD FOR C.M. BOX CULVERTS FLOWING FULL CORRUGATED METAL BOTTOM $0.4 \leq$ RISE/SPAN < 0.5

CHART 28



Area (ft^2)	$\bar{n}$
40 - 56	0.034
57 - 120	0.033

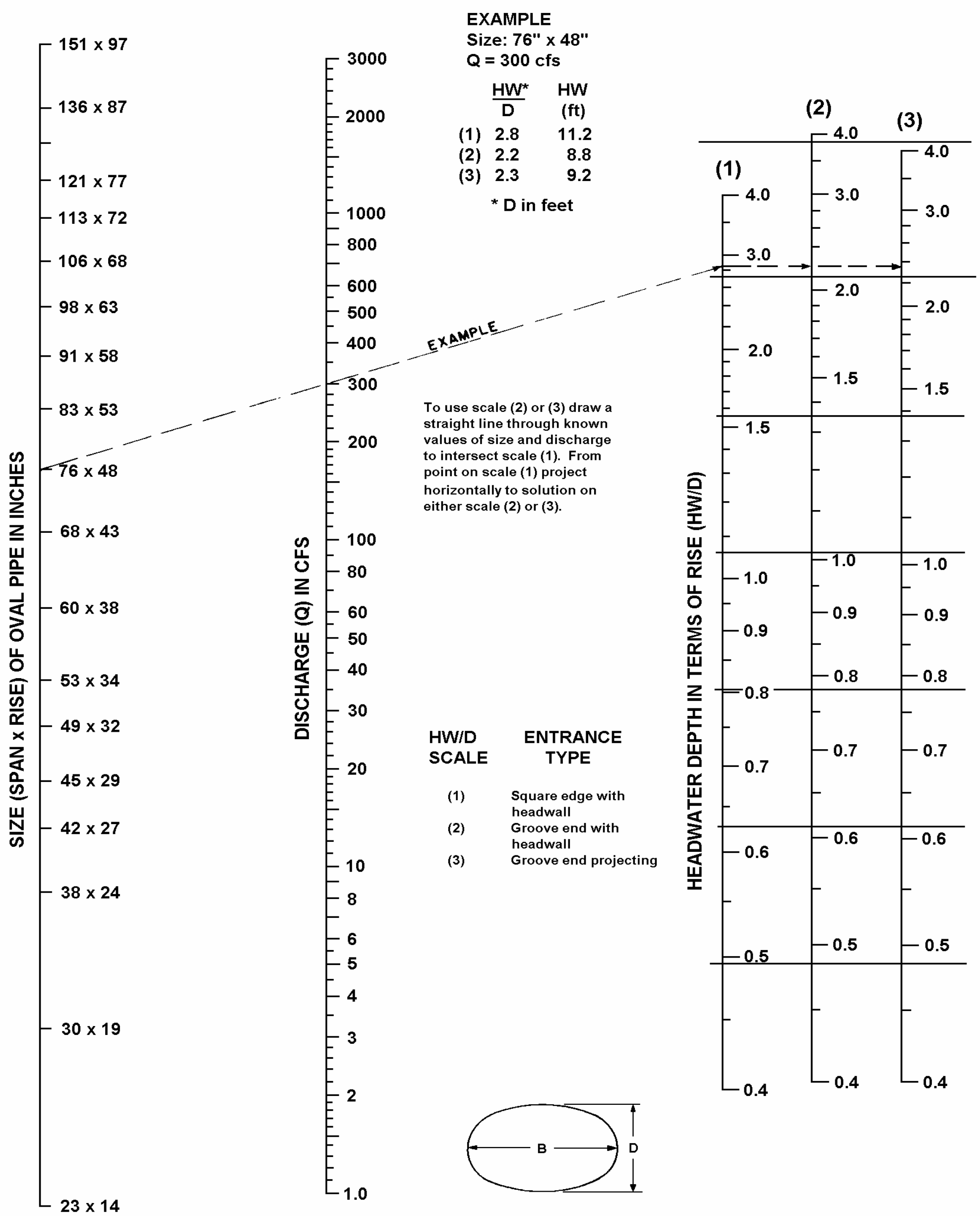


SUBMERGED OUTLET CULVERT FLOWING FULL

HEAD FOR
C.M. BOX CULVERTS
FLOWING FULL
CORRUGATED METAL BOTTOM
0.5 ≤ RISE/SPAN

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation.
Duplication of this nomograph may distort scale.

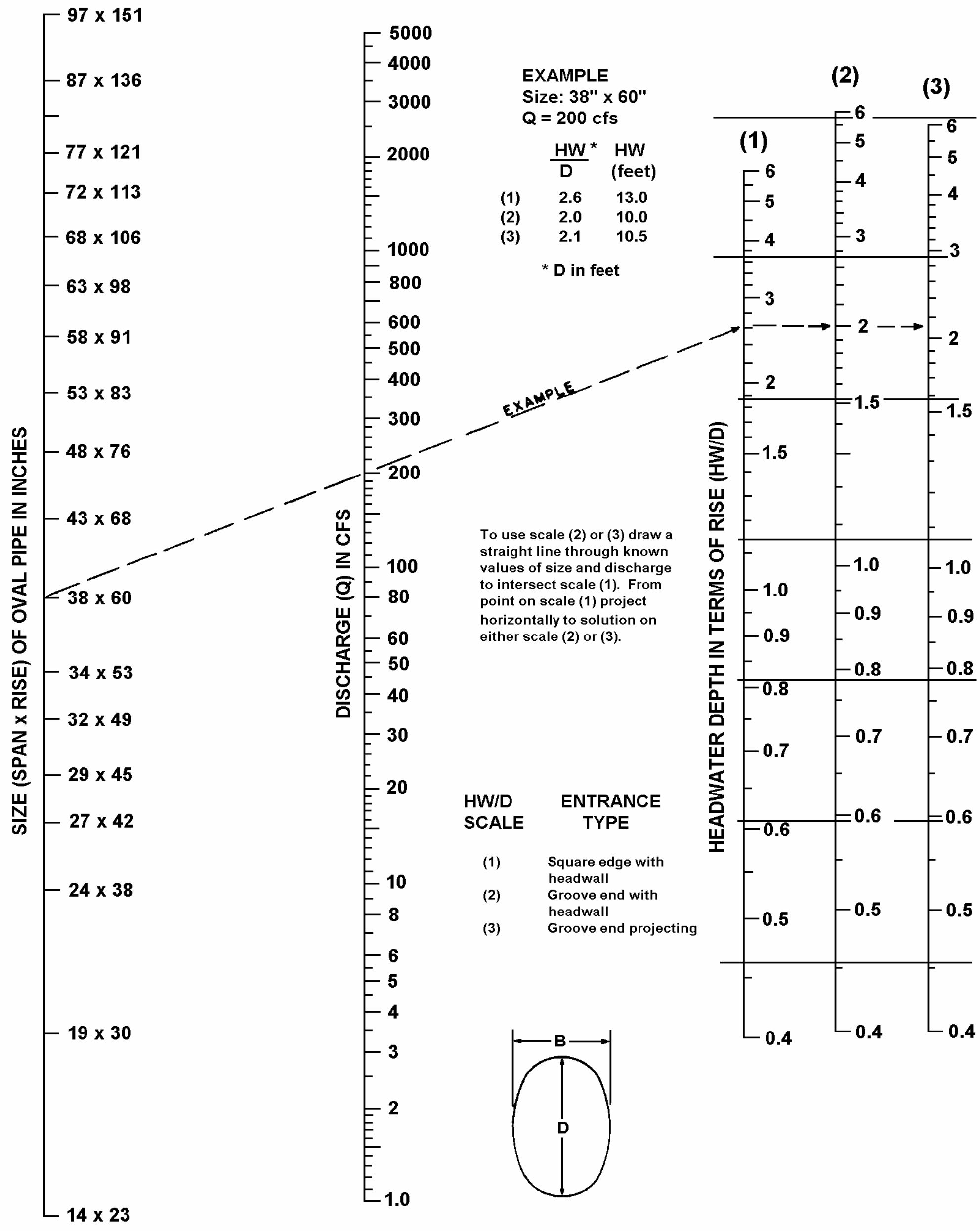
CHART 29



HEADWATER DEPTH FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

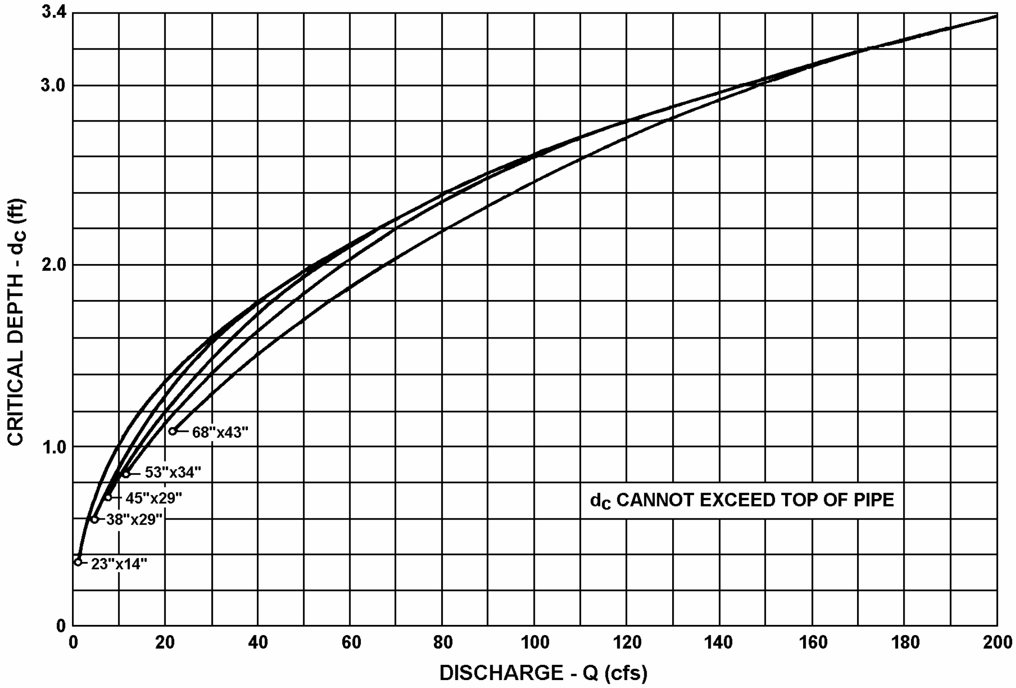
CHART 30

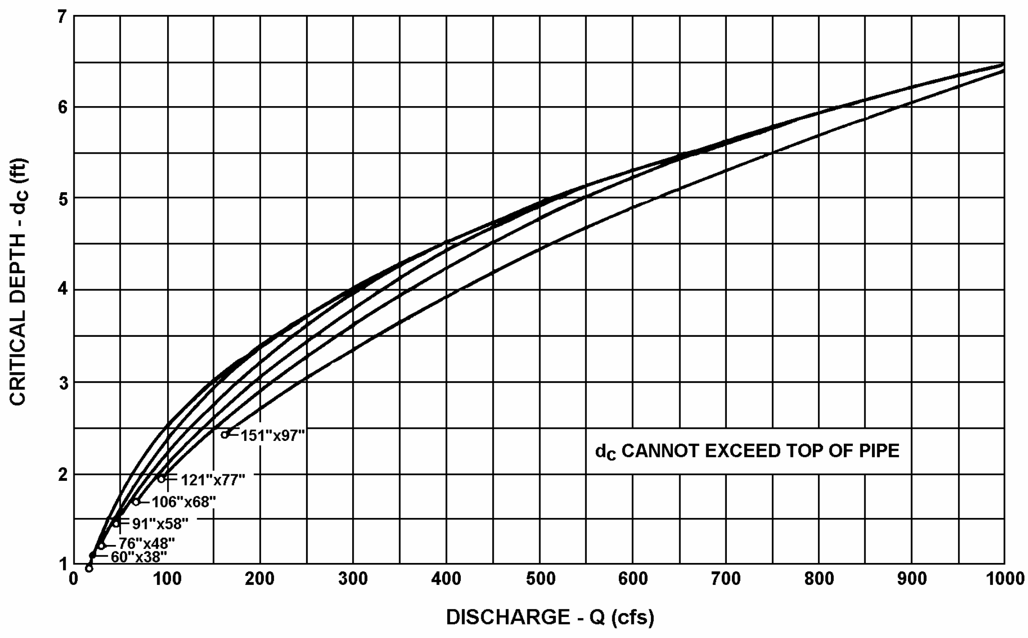


HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS VERTICAL
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 31

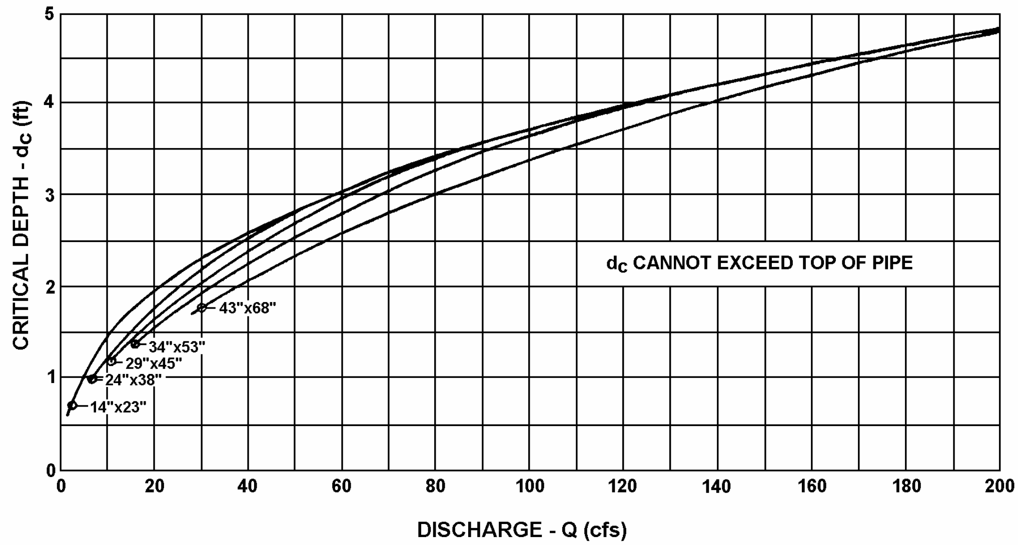


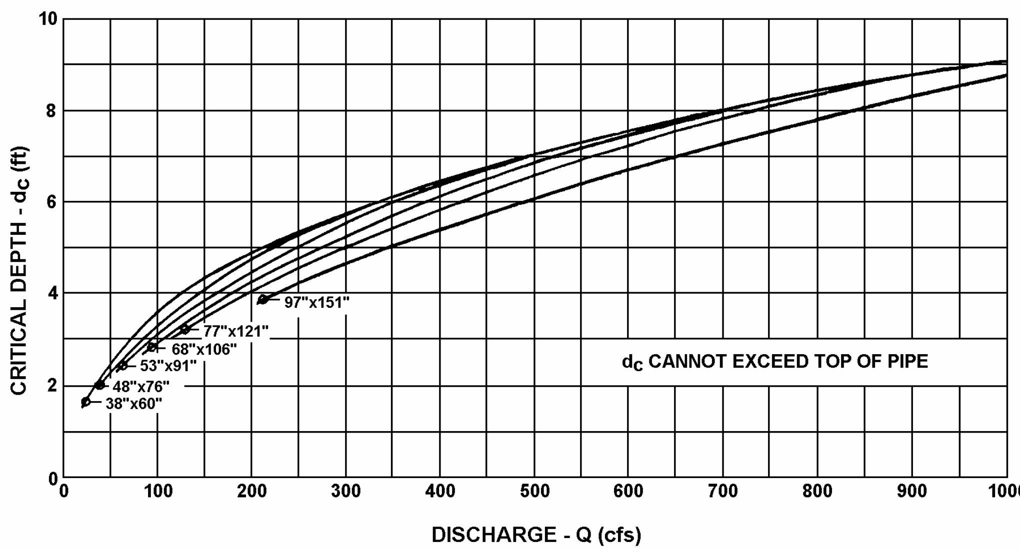


BUREAU OF PUBLIC ROADS JAN. 1964

CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL

CHART 32

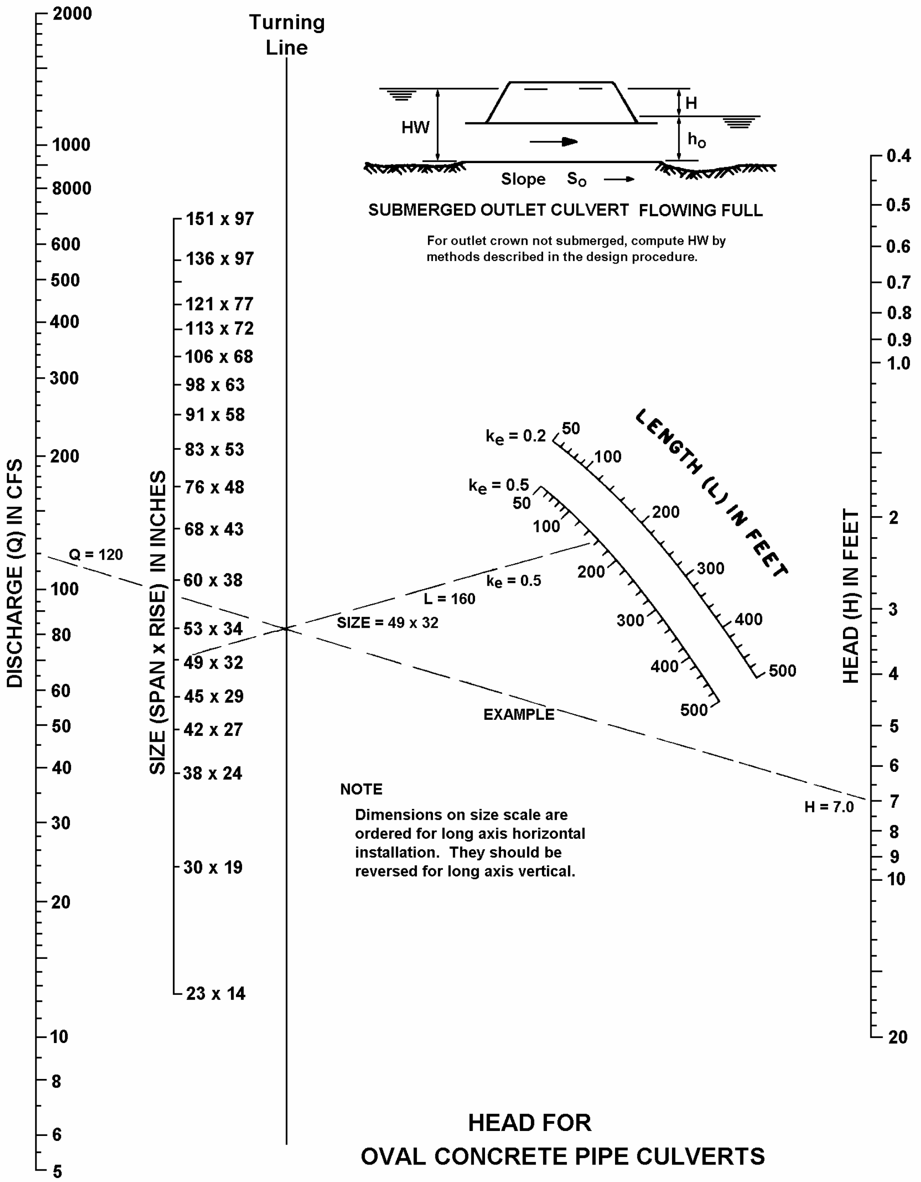




BUREAU OF PUBLIC ROADS JAN. 1964

**CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS VERTICAL**

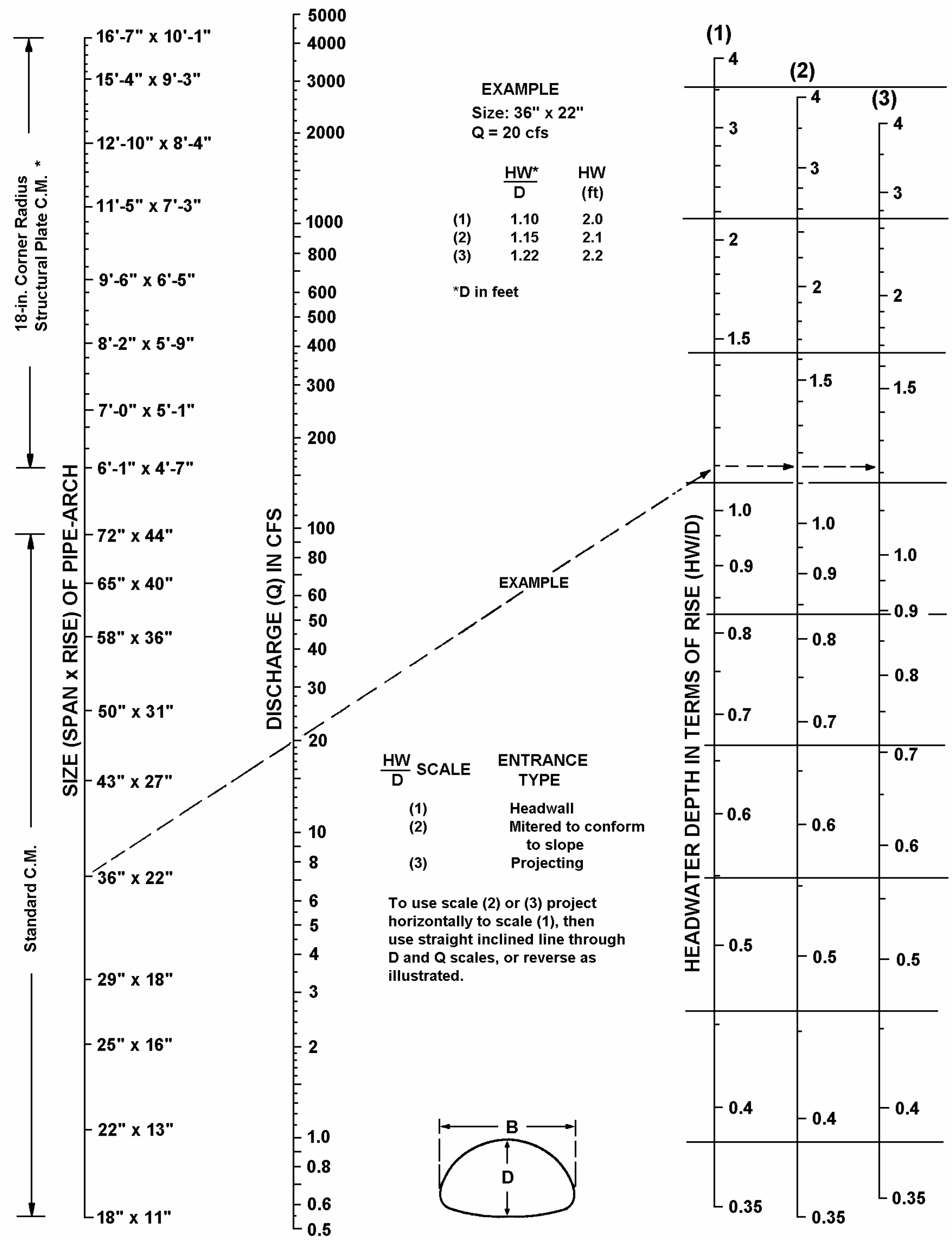
CHART 33



HEAD FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL OR VERTICAL
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 34



* Additional sizes not dimensioned are listed in fabricator's catalog.

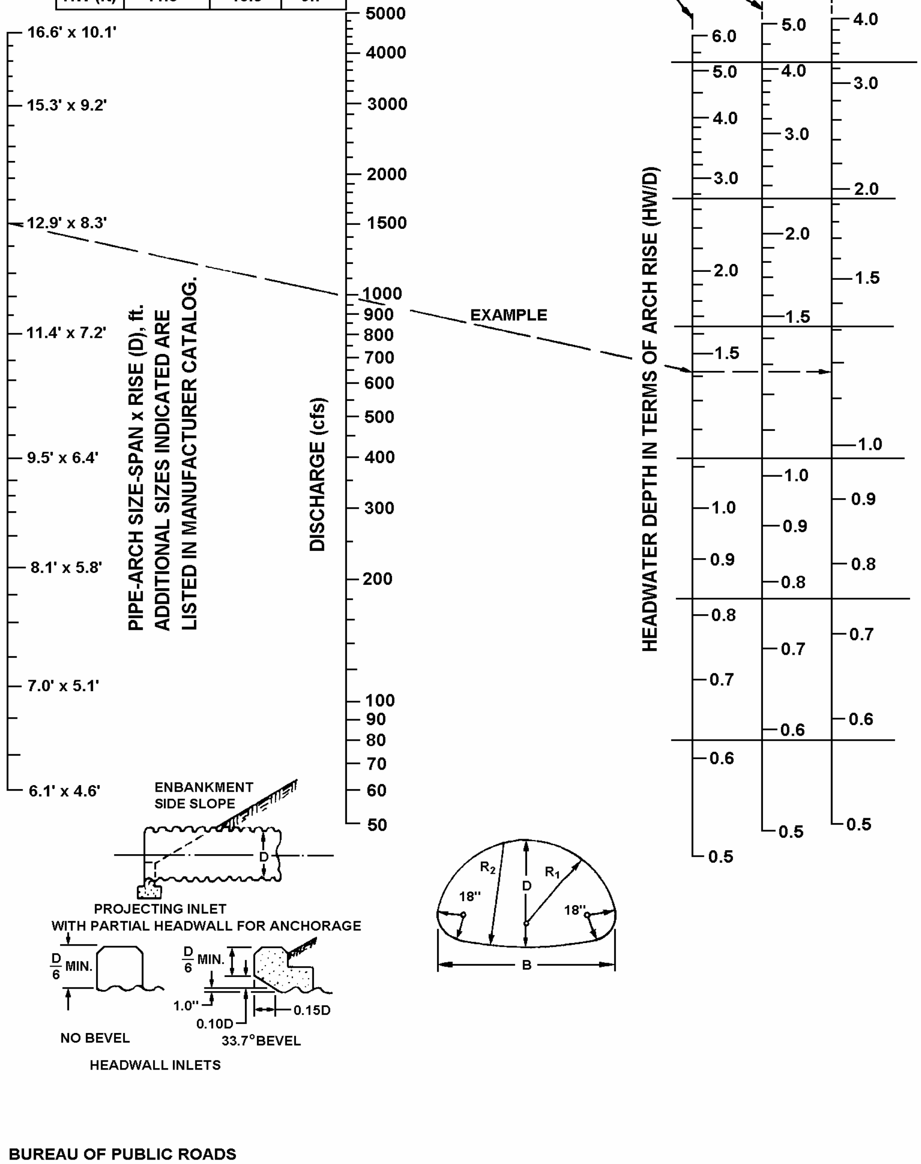
HEADWATER DEPTH FOR C.M. PIPE-ARCH CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 35

EXAMPLE
SIZE 12.9' x 8.3' Q = 1000 cfs

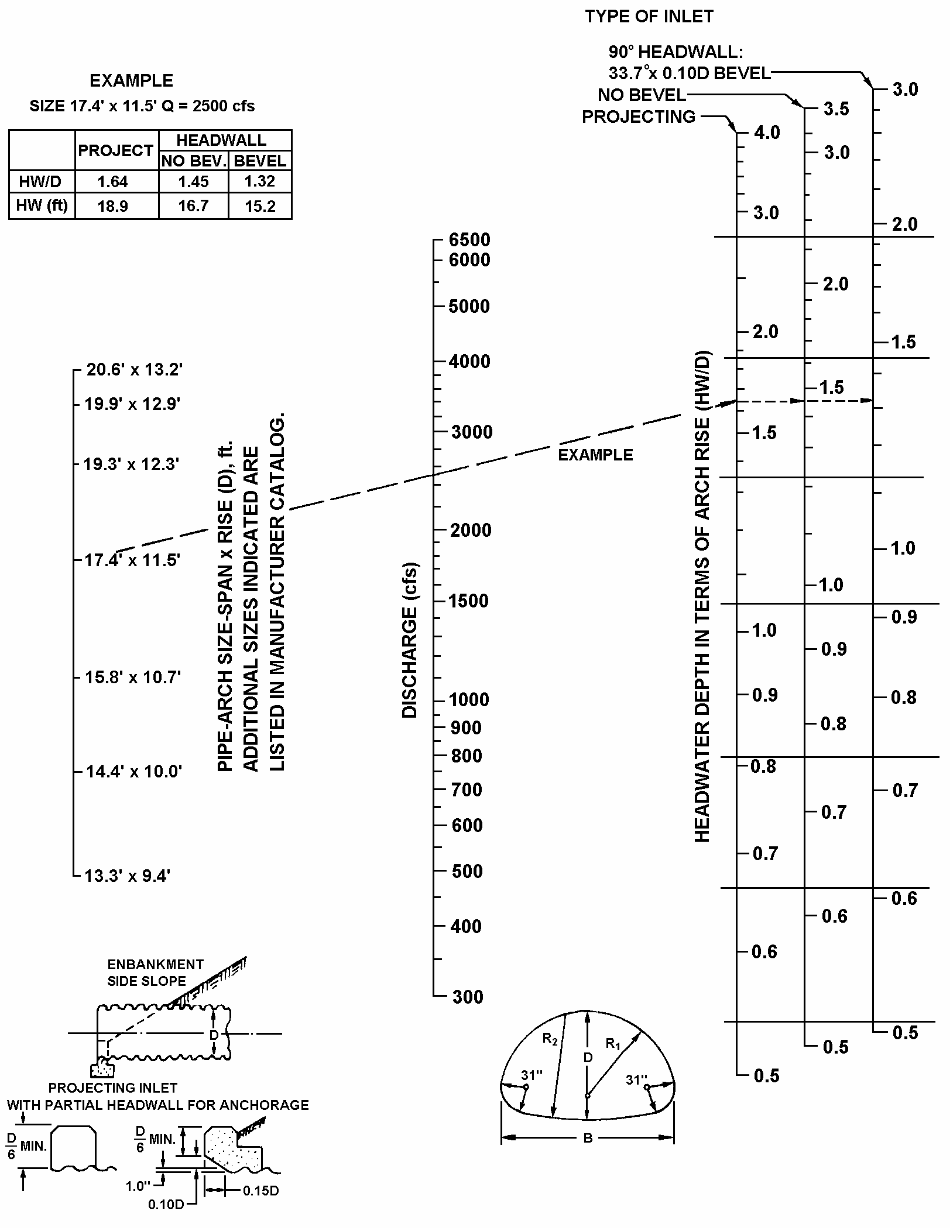
	PROJECT	HEADWALL	
		NO BEV.	BEVEL
HW/D	1.42	1.27	1.17
HW (ft)	11.8	10.5	9.7



BUREAU OF PUBLIC ROADS
OFFICE OF R&D JULY 1968

HEADWATER DEPTH FOR INLET CONTROLS
STRUCTURAL PLATE PIPE-ARCH CULVERTS
18 in. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

CHART 36

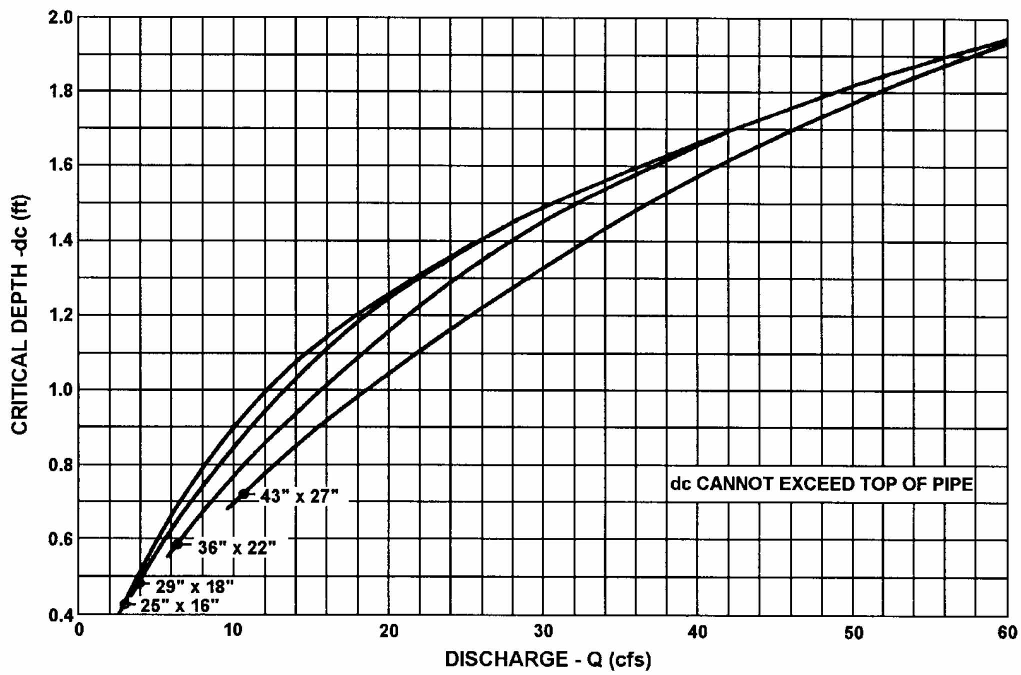


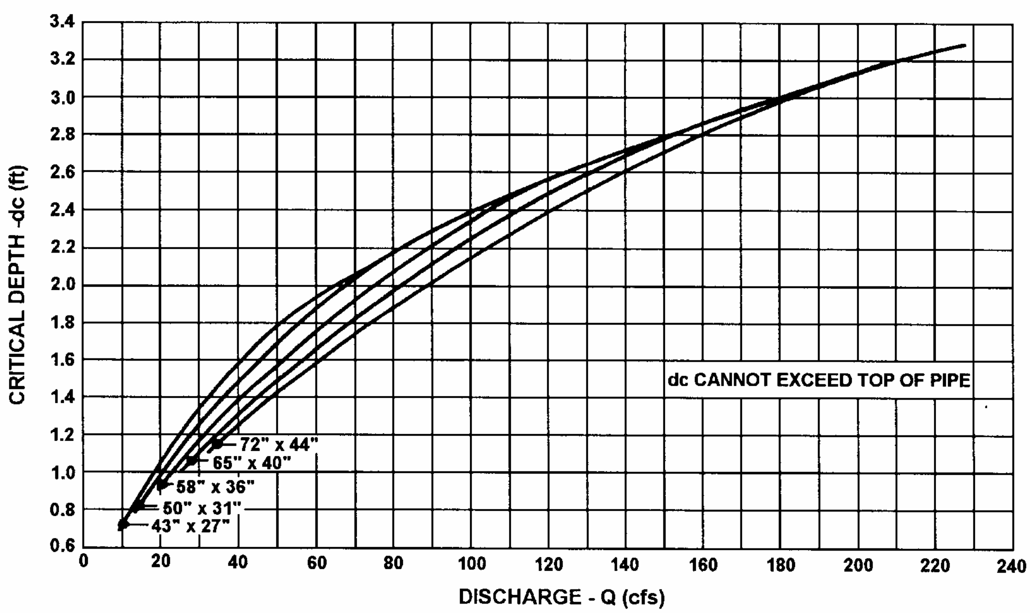
	PROJECT	HEADWALL	
		NO BEV.	BEVEL
HW/D	1.64	1.45	1.32
HW (ft)	18.9	16.7	15.2

HEADWATER DEPTH FOR INLET CONTROL
STRUCTURAL PLATE PIPE-ARCH CULVERTS
31 in. RADIUS CORNER PLATE
PROJECTING OR HEADWALL INLET
HEADWALL WITH OR WITHOUT EDGE BEVEL

BUREAU OF PUBLIC ROADS
OFFICE OF R&D JULY 1966

CHART 37

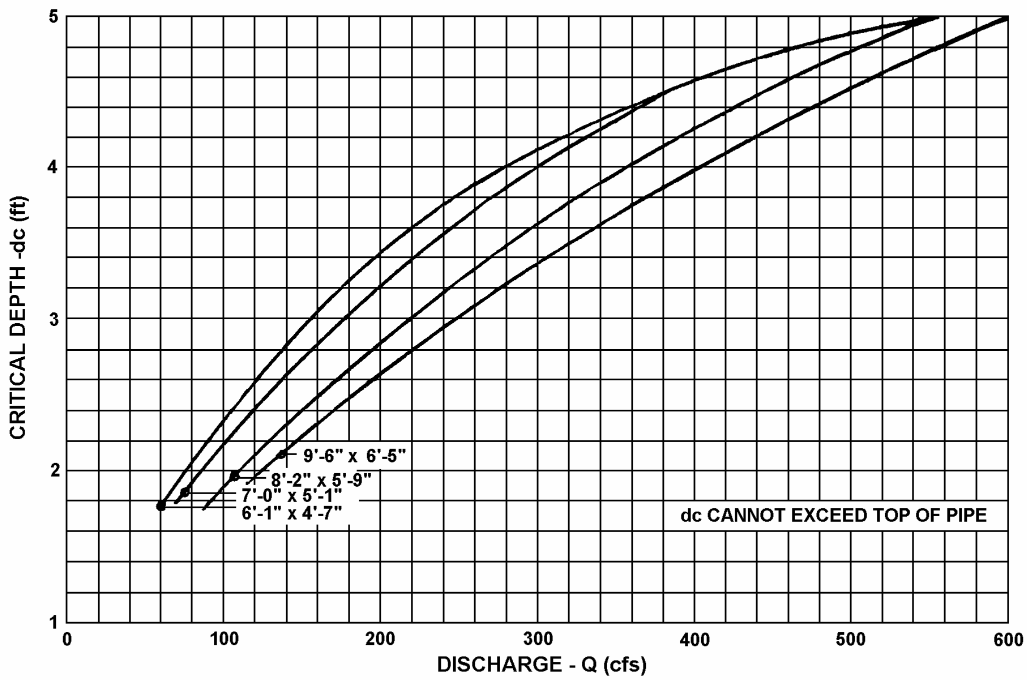


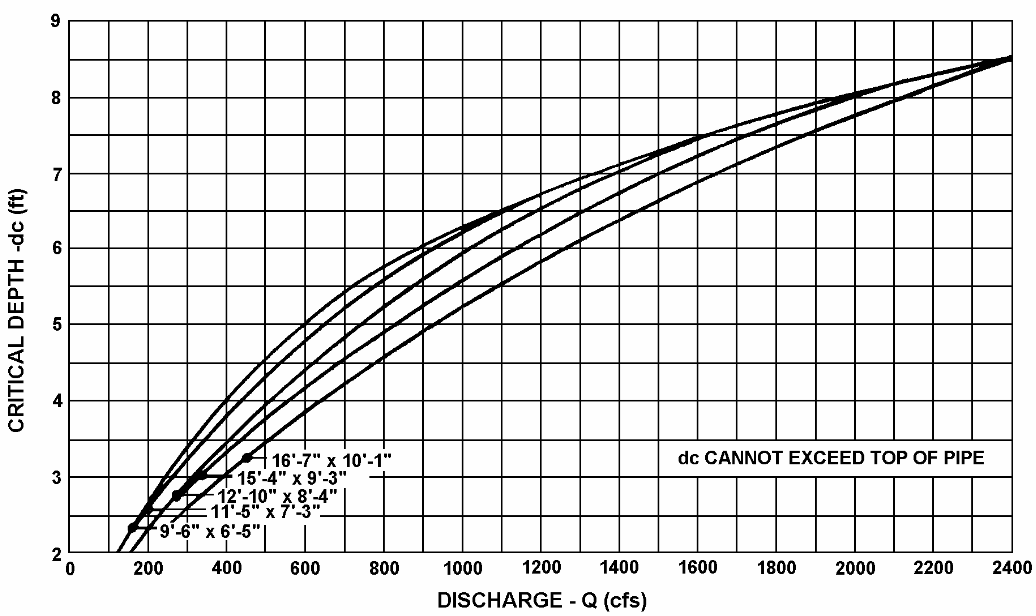


BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
STANDARD C.M. PIPE ARCH

CHART 38

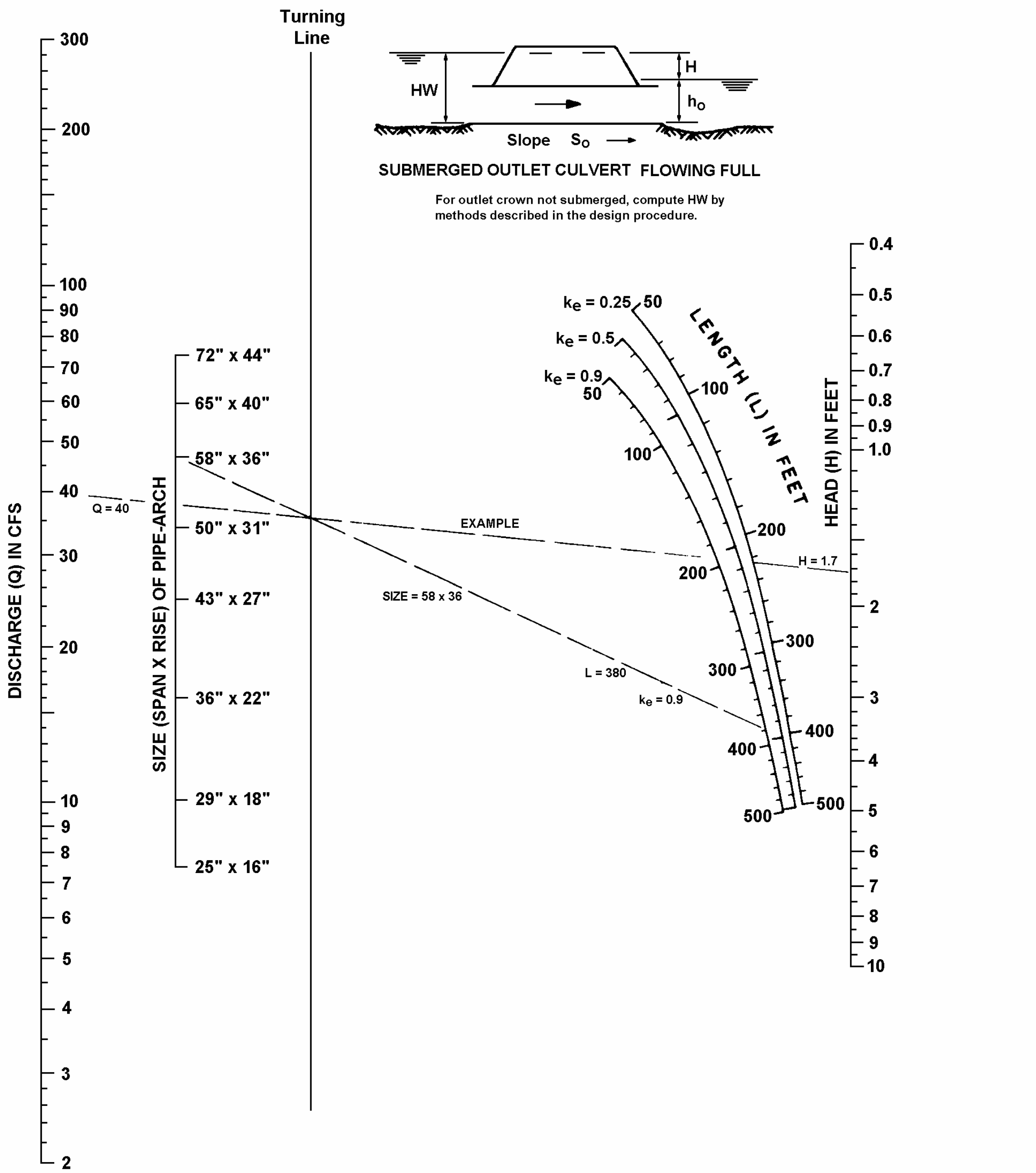




BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
STRUCTURAL PLATE
C.M. PIPE ARCH
18 in. CORNER RADIUS

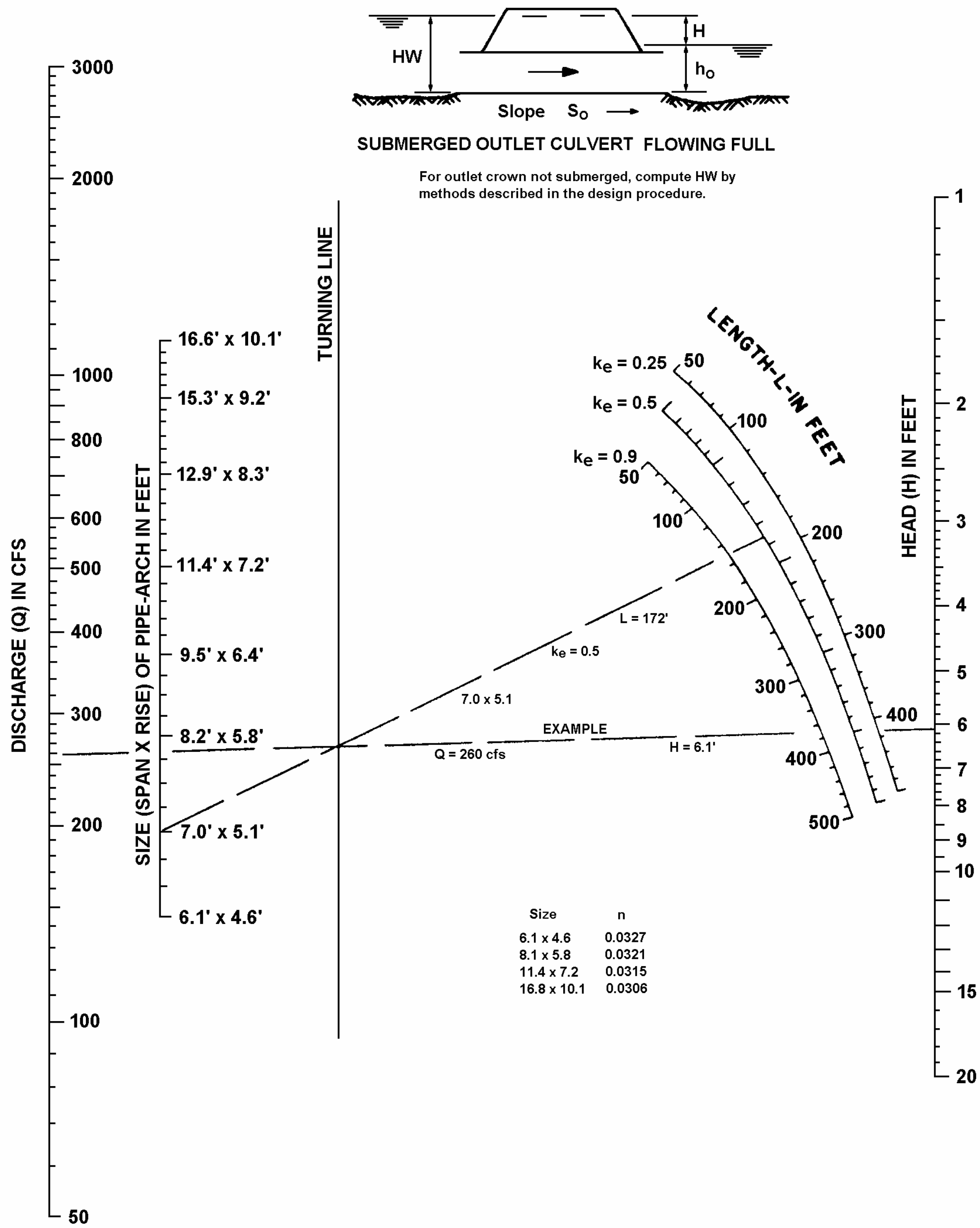
CHART 39



BUREAU OF PUBLIC ROADS JAN. 1963

HEAD FOR
STANDARD C.M. PIPE-ARCH CULVERTS
FLOWING FULL
n = 0.024

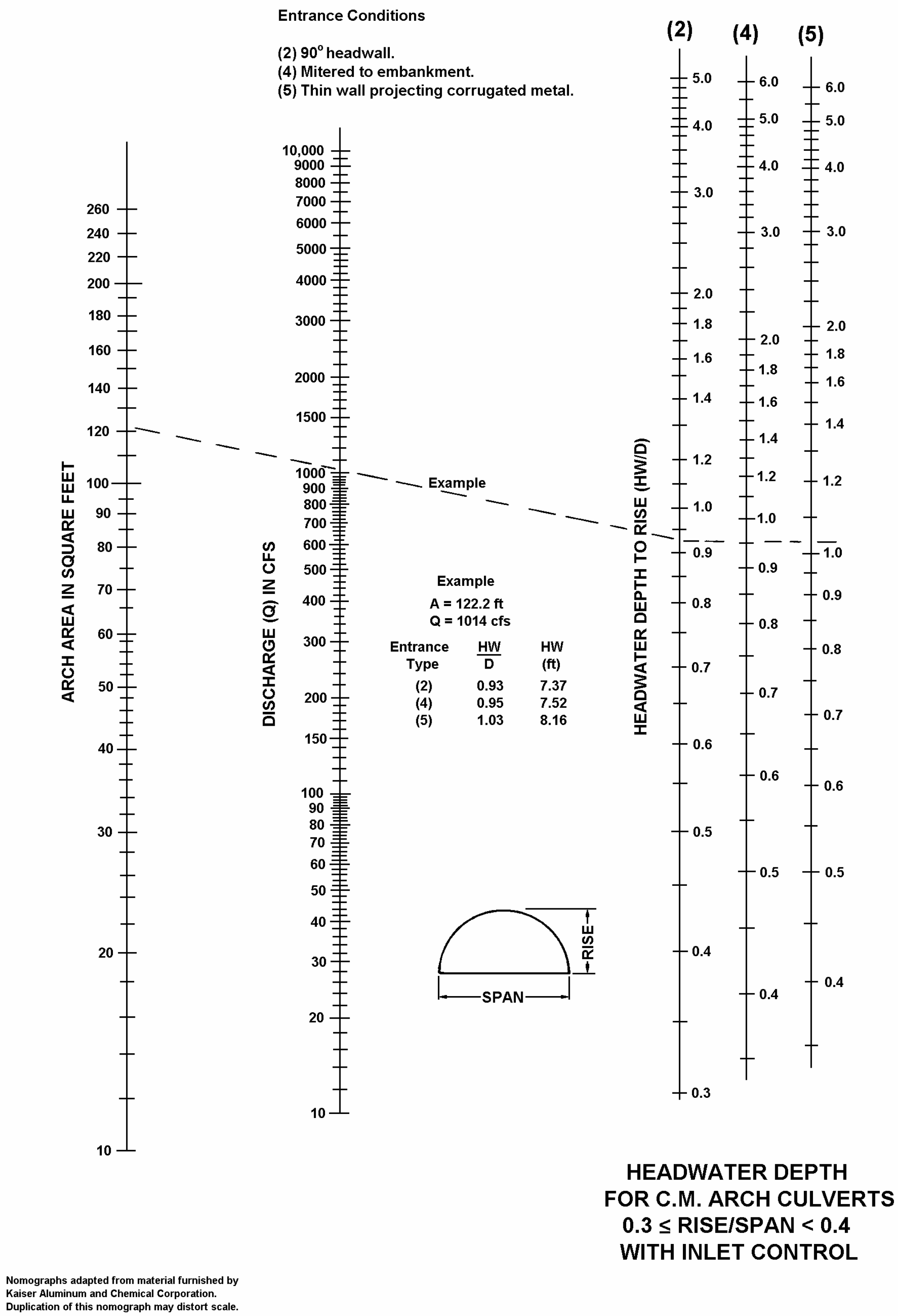
CHART 40



BUREAU OF PUBLIC ROADS JAN. 1963

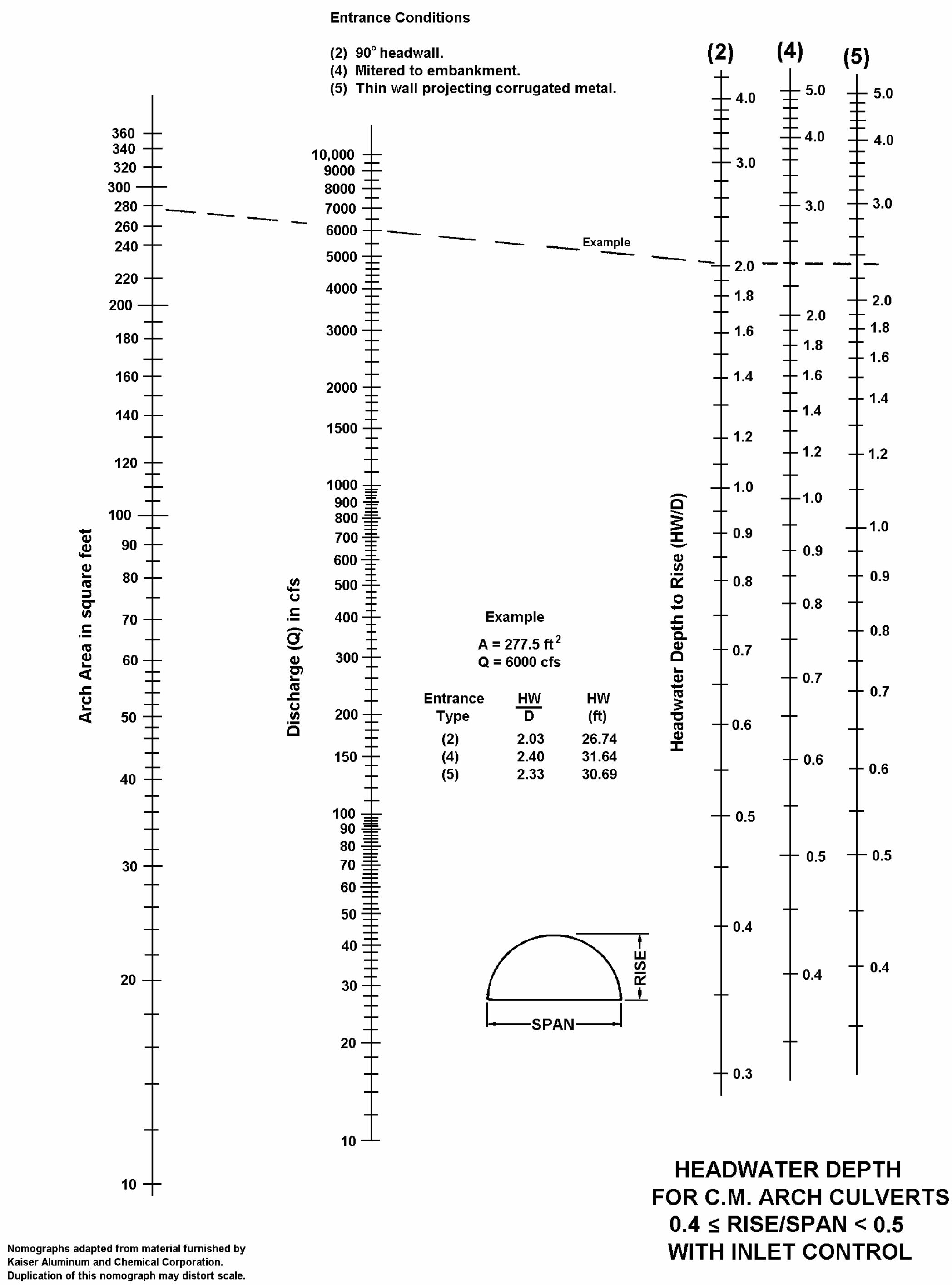
HEAD FOR
STRUCTURAL PLATE
C.M. PIPE ARCH CULVERTS
18 in. CORNER RADIUS
FLOWING FULL
n = 0.0327 TO 0.0306

CHART 41



HEADWATER DEPTH
FOR C.M. ARCH CULVERTS
$0.3 \leq$ RISE/SPAN < 0.4
WITH INLET CONTROL

CHART 42


Entrance Conditions
(2) 90° headwall.
(4) Mitered to embankment.
(5) Thin wall projecting corrugated metal.
Arch Area in square feet
Discharge (Q) in cfs
Headwater Depth to Rise (HW/D)
Example
Example
A = 277.5 ft²
Q = 6000 cfs
Entrance Type | HW/D | HW (ft)
(2) 2.03 26.74
(4) 2.40 31.64
(5) 2.33 30.69
RISE
SPAN
HEADWATER DEPTH FOR C.M. ARCH CULVERTS 0.4 ≤ RISE/SPAN < 0.5 WITH INLET CONTROL
Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

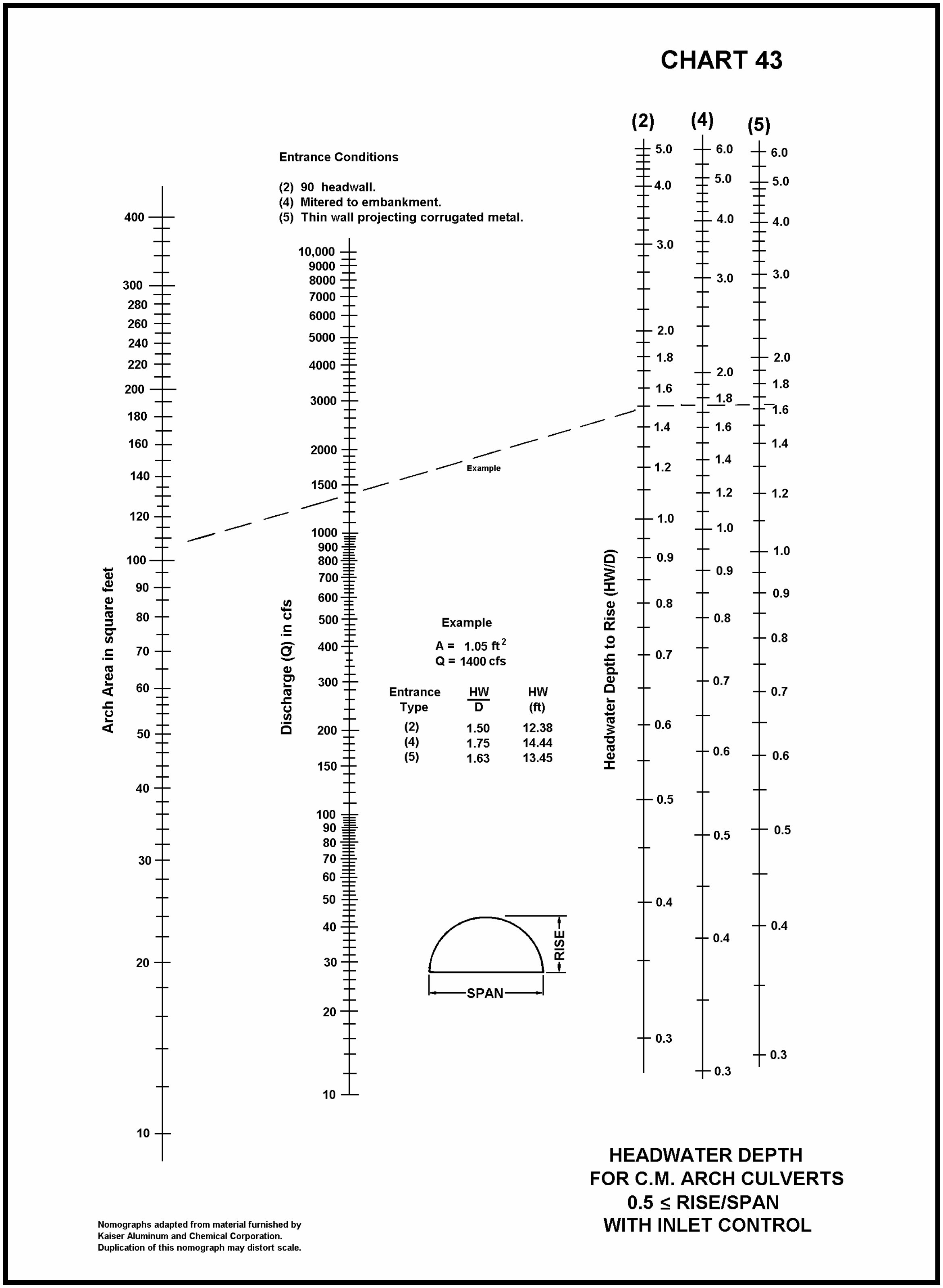
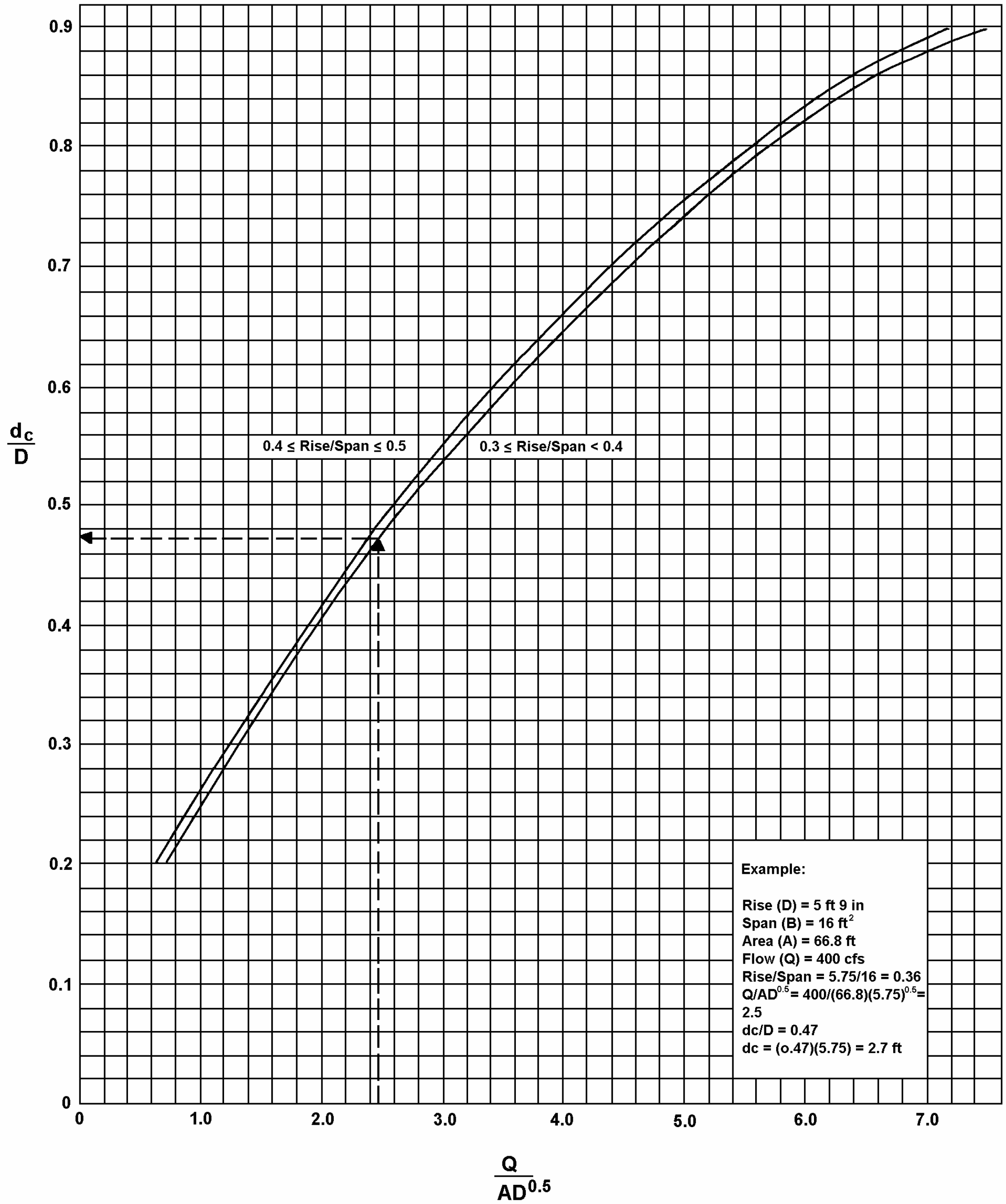


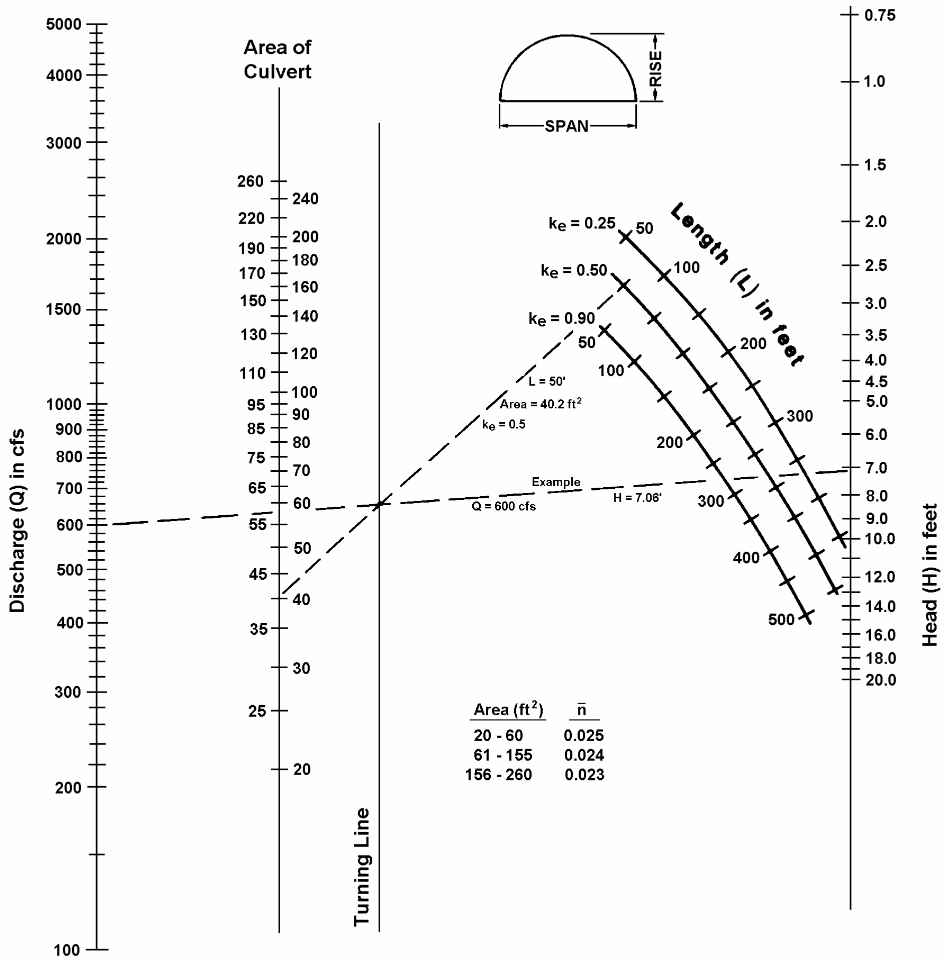
CHART 43
Entrance Conditions
(2) 90 headwall.
(4) Mitered to embankment.
(5) Thin wall projecting corrugated metal.
Arch Area in square feet
Discharge (Q) in cfs
Headwater Depth to Rise (HW/D)
(2)
(4)
(5)
Example
Example
A = 1.05 ft2
Q = 1400 cfs
Entrance Type
HW/D
HW (ft)
(2) 1.50 12.38
(4) 1.75 14.44
(5) 1.63 13.45
RISE
SPAN
HEADWATER DEPTH
FOR C.M. ARCH CULVERTS
0.5 ≤ RISE/SPAN
WITH INLET CONTROL
Nomographs adapted from material furnished by
Kaiser Aluminum and Chemical Corporation.
Duplication of this nomograph may distort scale.

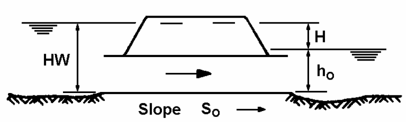
CHART 44



DIMENSIONLESS CRITICAL DEPTH CHART FOR C.M. ARCH CULVERTS

CHART 45



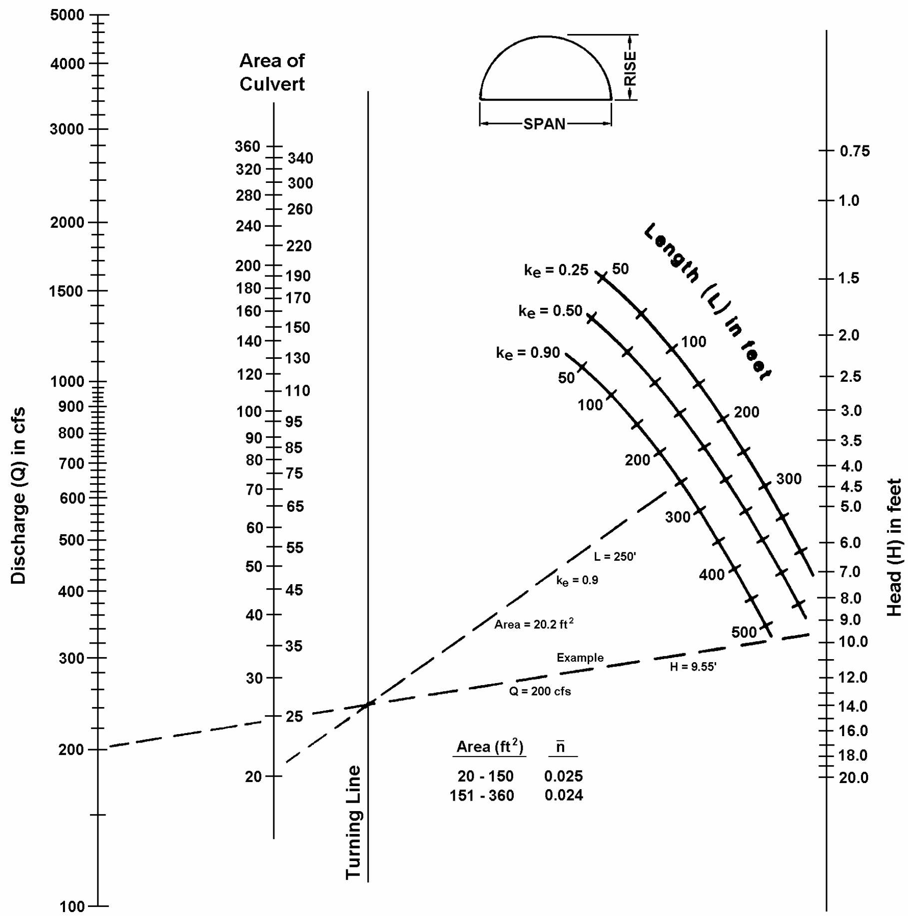


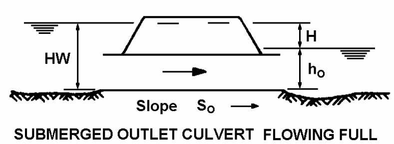
SUBMERGED OUTLET CULVERT FLOWING FULL

HEAD FOR
C.M. ARCH CULVERTS
FLOWING FULL
CONCRETE BOTTOM
0.3 ≤ RISE/SPAN < 0.4

Nomographs adapted from material furnished by
Kaiser Aluminum and Chemical Corporation.
Duplication of this nomograph may distort scale.

CHART 46

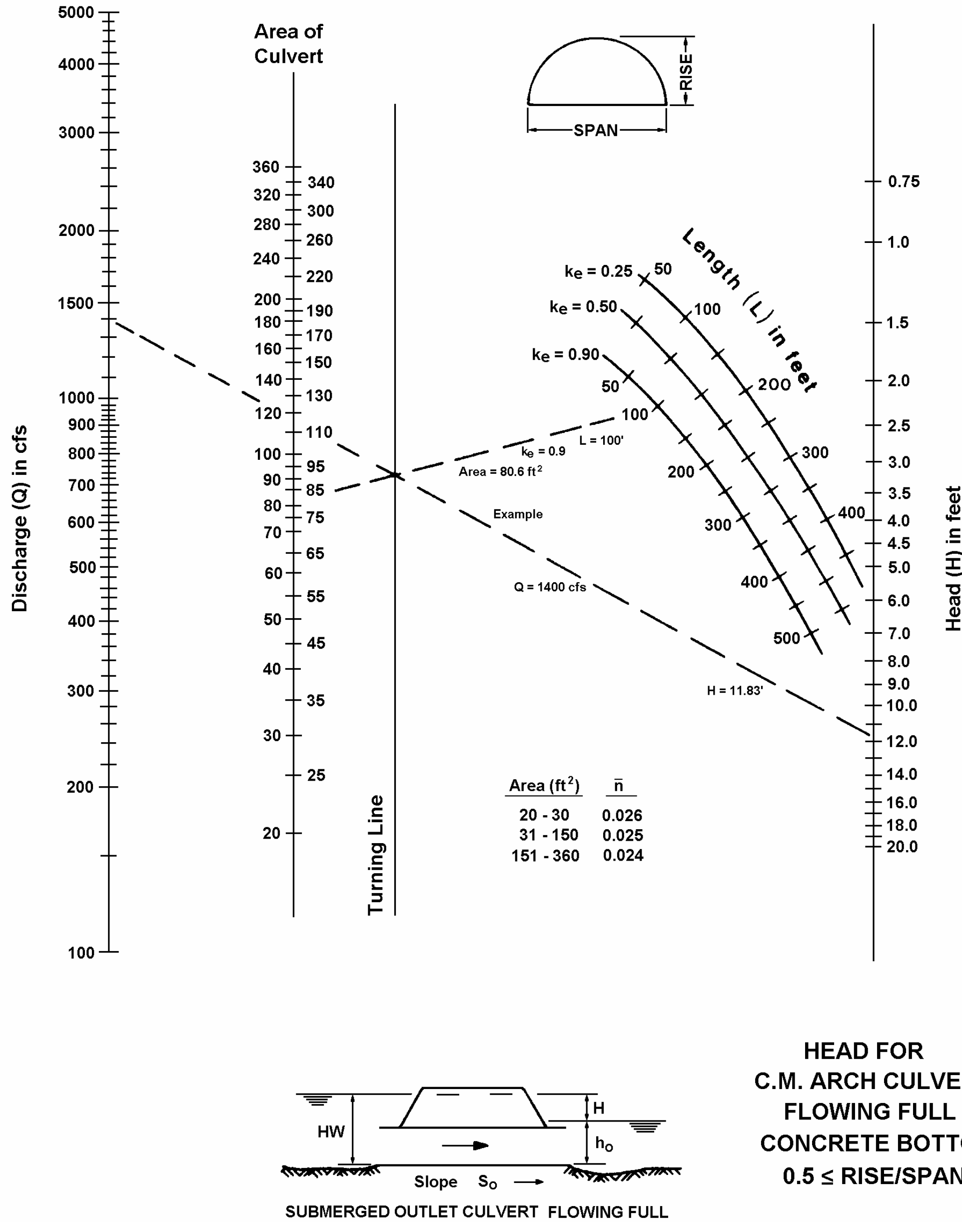




**HEAD FOR
C.M. ARCH CULVERTS
FLOWING FULL
CONCRETE BOTTOM
0.4 ≤ RISE/SPAN < 0.5**

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

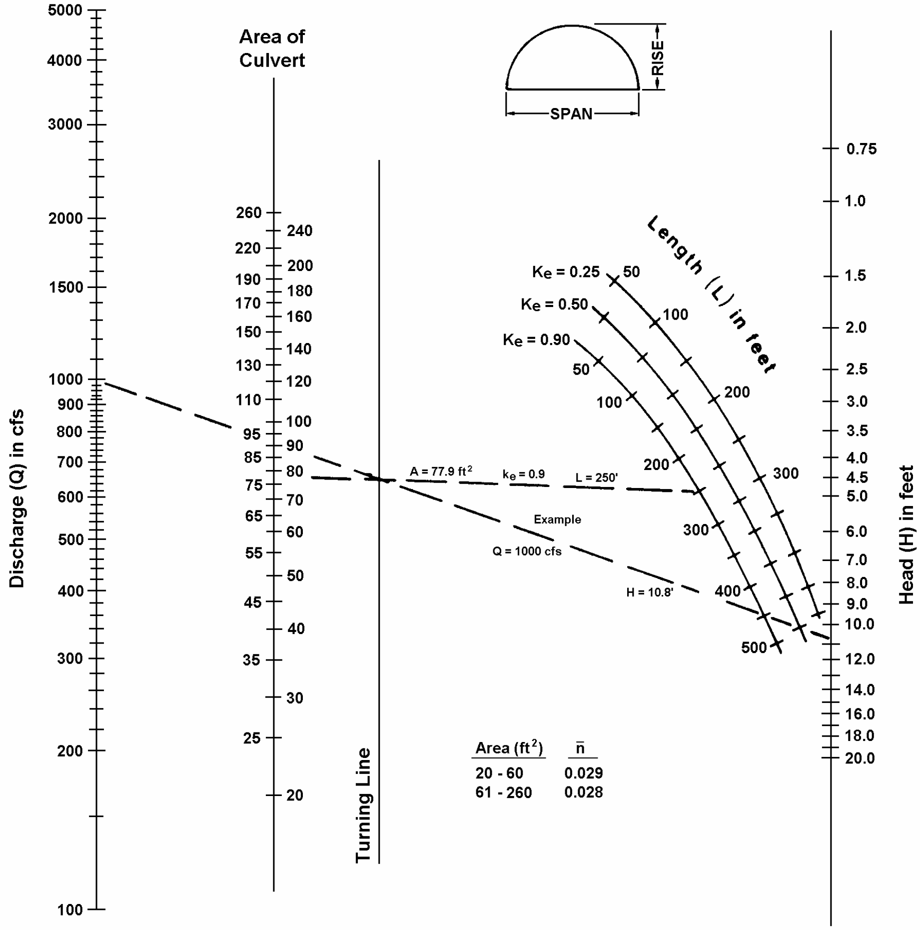
CHART 47

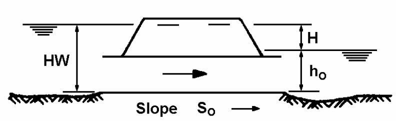


HEAD FOR
C.M. ARCH CULVERTS
FLOWING FULL
CONCRETE BOTTOM
0.5 ≤ RISE/SPAN

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

CHART 48



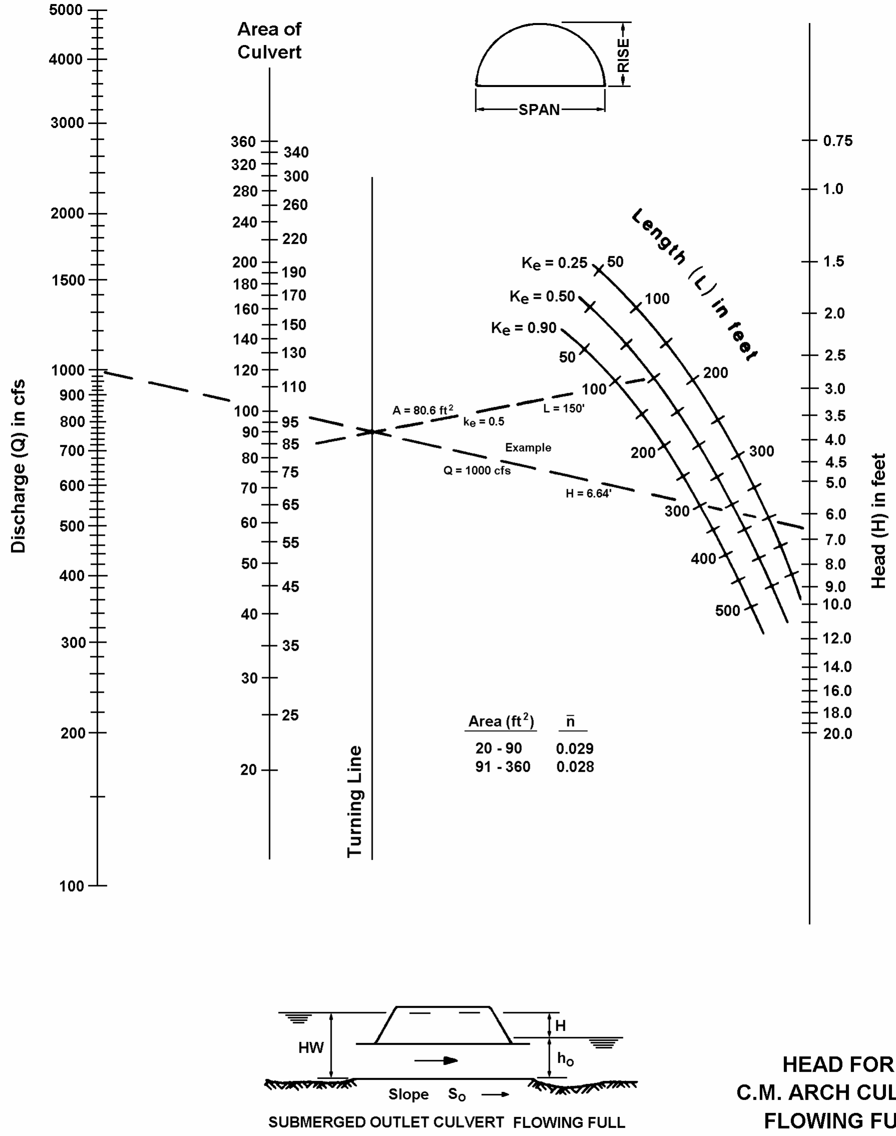


SUBMERGED OUTLET CULVERT FLOWING FULL

HEAD FOR
C.M. ARCH CULVERTS
FLOWING FULL
EARTH BOTTOM (n_b = 0.022)
0.3 ≤ RISE/SPAN < 0.4

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

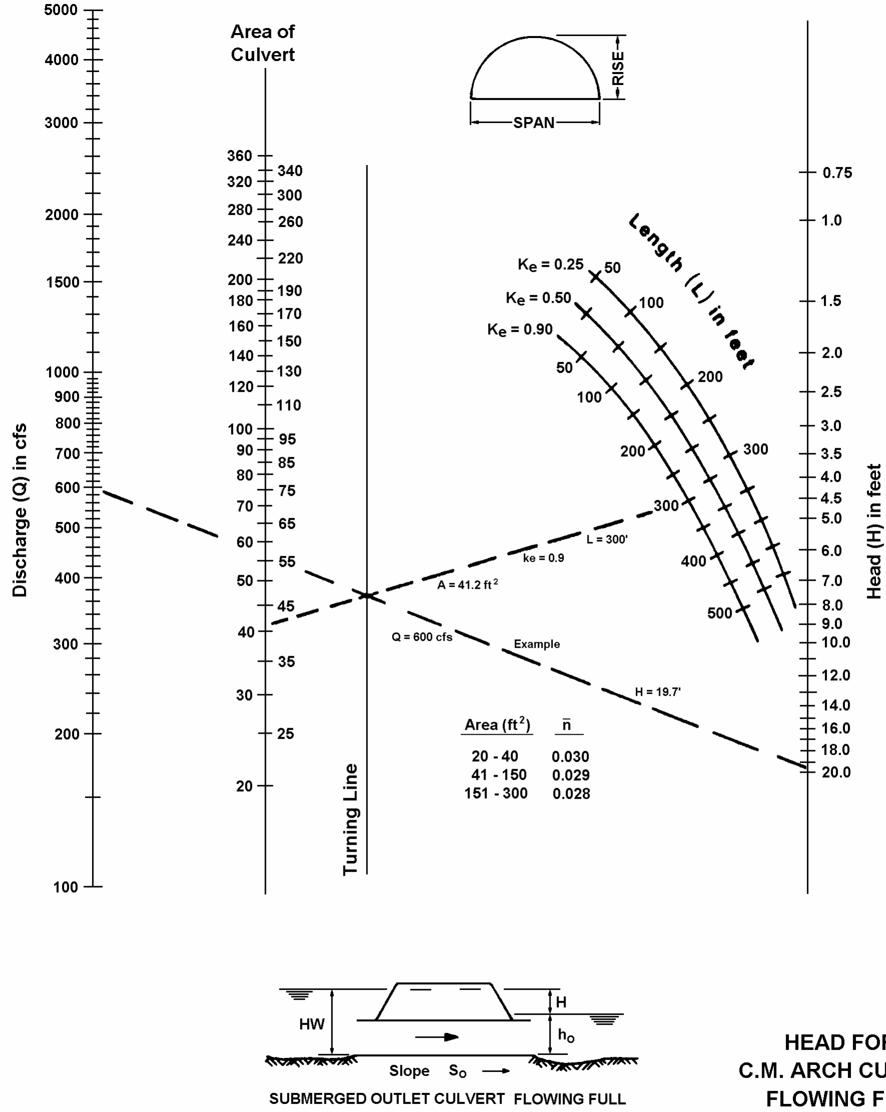
CHART 49



HEAD FOR C.M. ARCH CULVERTS FLOWING FULL EARTH BOTTOM (n_b = 0.022) 0.4 ≤ RISE/SPAN < 0.5

Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

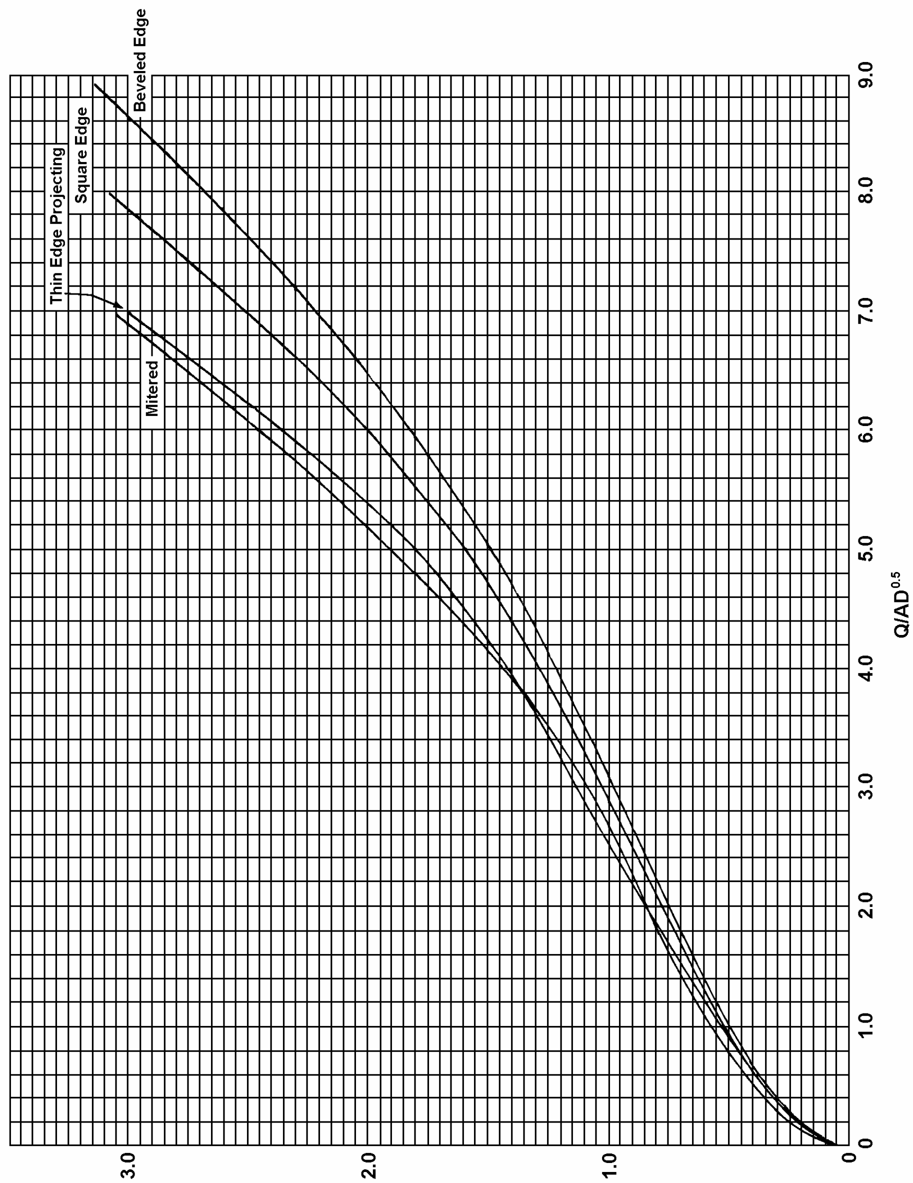
CHART 50



HEAD FOR
C.M. ARCH CULVERTS
FLOWING FULL
EARTH BOTTOM (n_b = 0.022)
0.5 ≤ RISE/SPAN

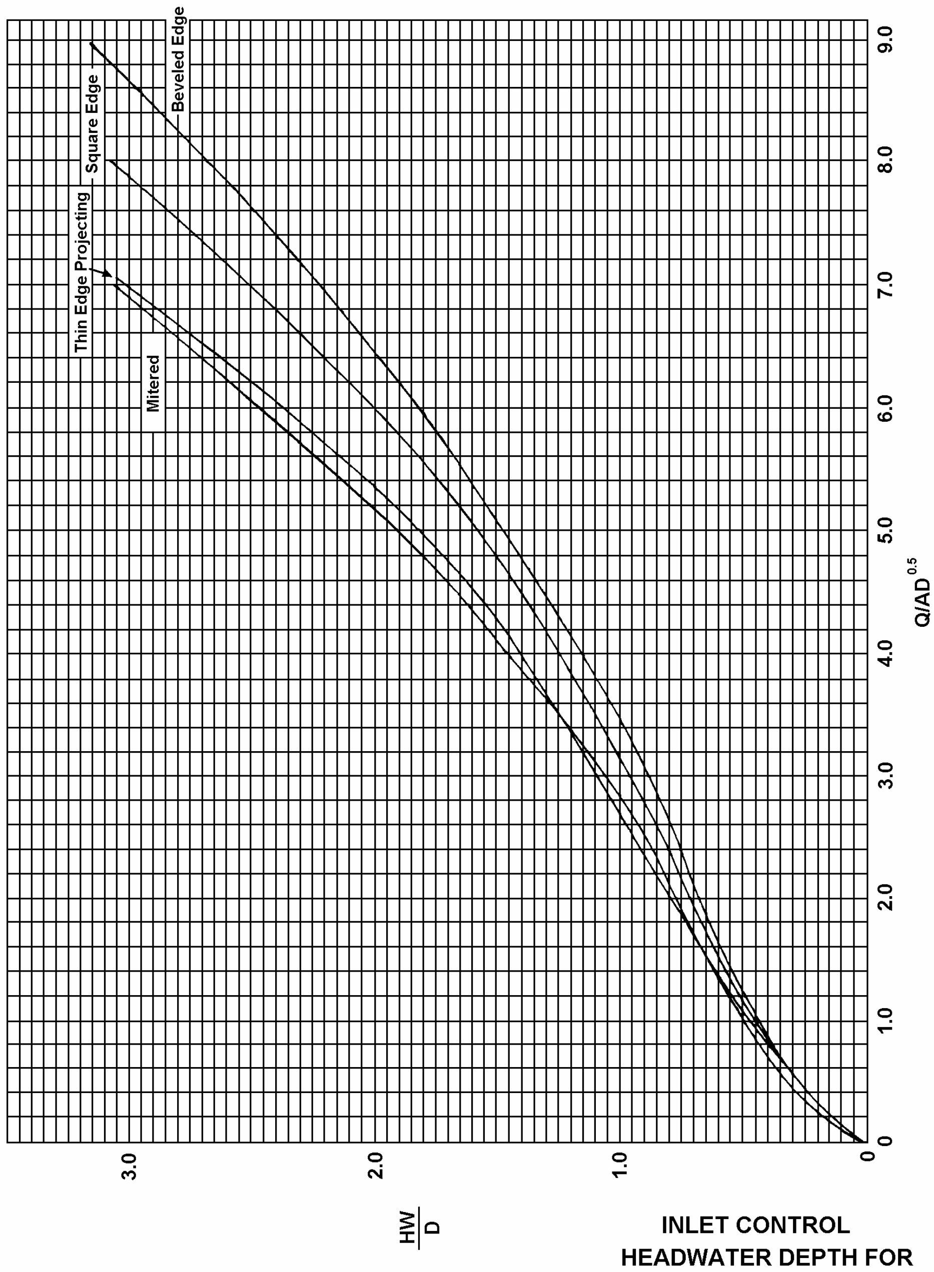
Nomographs adapted from material furnished by Kaiser Aluminum and Chemical Corporation. Duplication of this nomograph may distort scale.

CHART 51



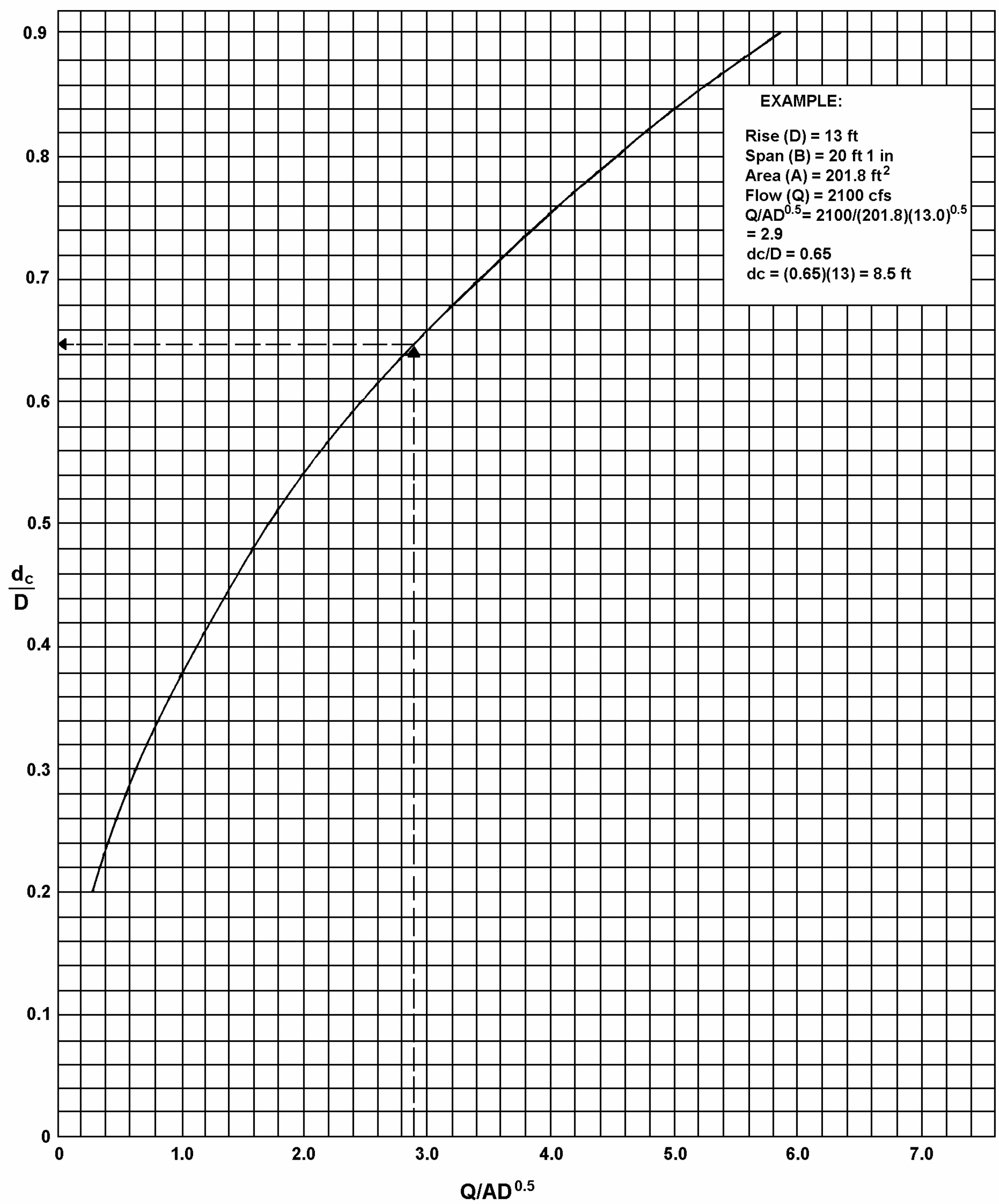
INLET CONTROL
HEADWATER DEPTH FOR
CIRCULAR OR ELLIPTICAL
STRUCTURAL PLATE
C.M. CONDUITS

CHART 52



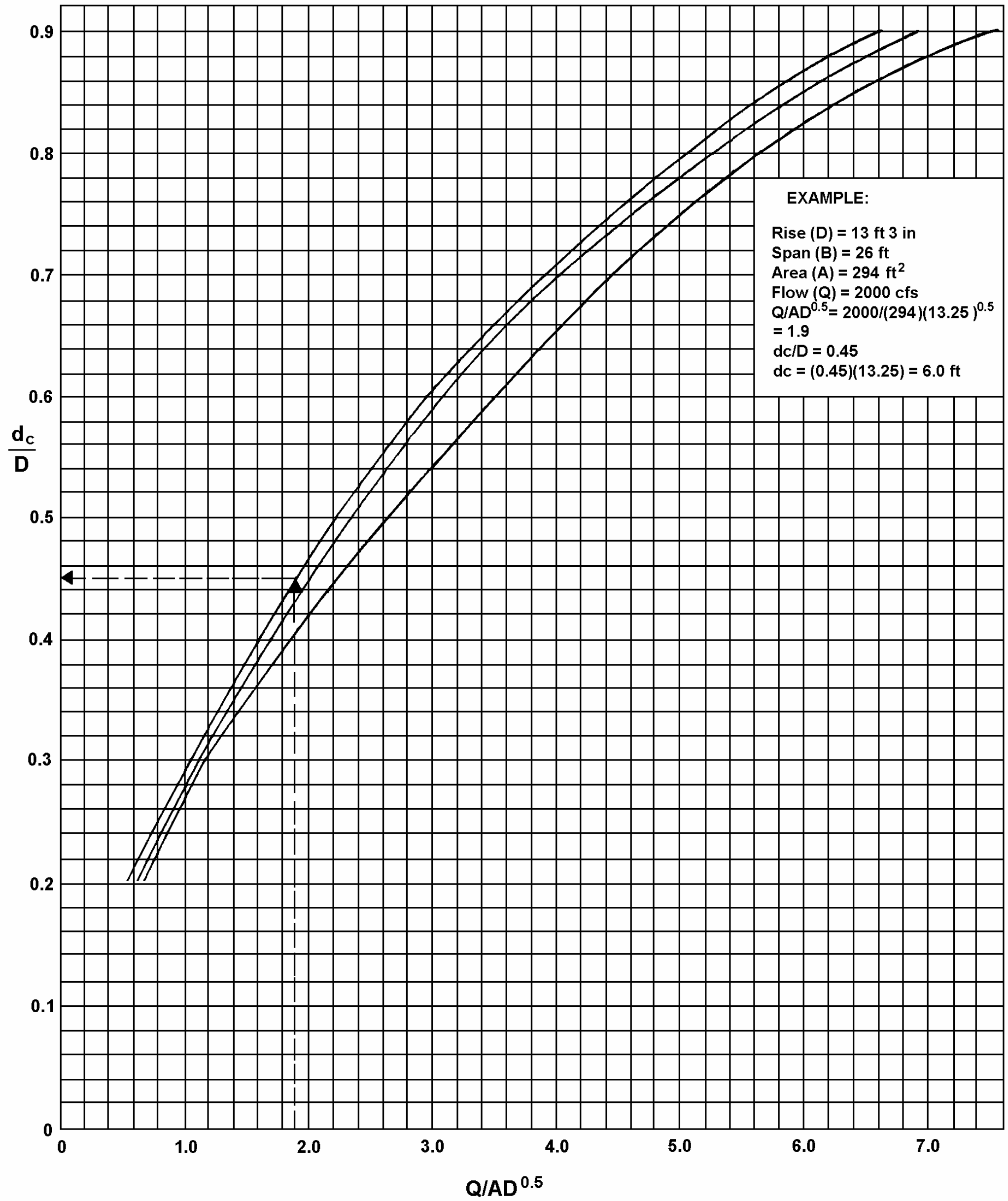
INLET CONTROL HEADWATER DEPTH FOR HIGH AND LOW PROFILE STRUCTURAL PLATE C.M. ARCH

CHART 53



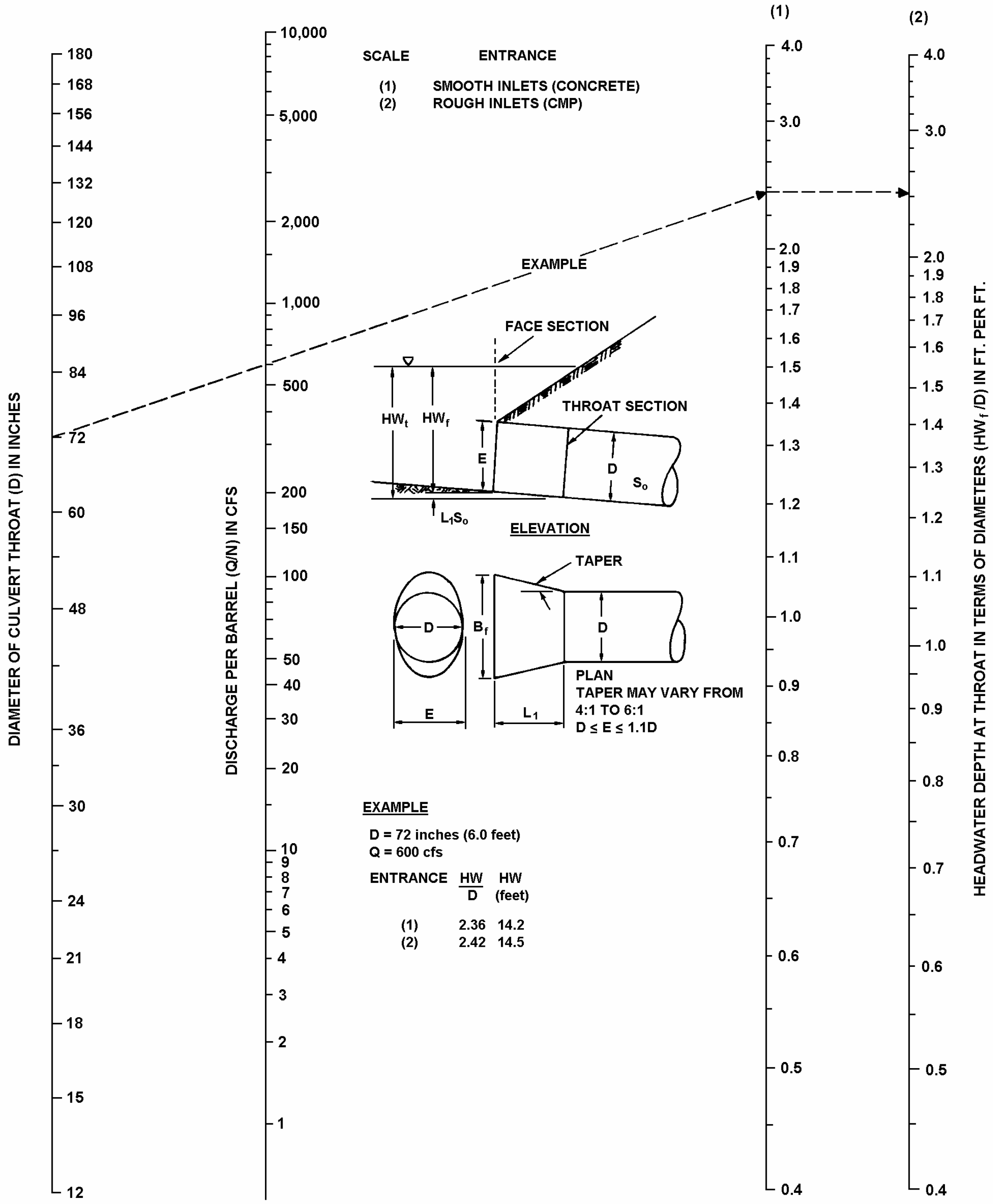
DIMENSIONLESS CRITICAL DEPTH CHART FOR STRUCTURAL PLATE ELLIPSE LONG AXIS HORIZONTAL

CHART 54



DIMENSIONLESS CRITICAL DEPTH CHART
FOR STRUCTURAL PLATE
LOW- AND HIGH-PROFILE ARCHES

CHART 55



**THROAT CONTROL
FOR SIDE-TAPERED INLETS
TO PIPE CULVERT
(CIRCULAR SECTION ONLY)**

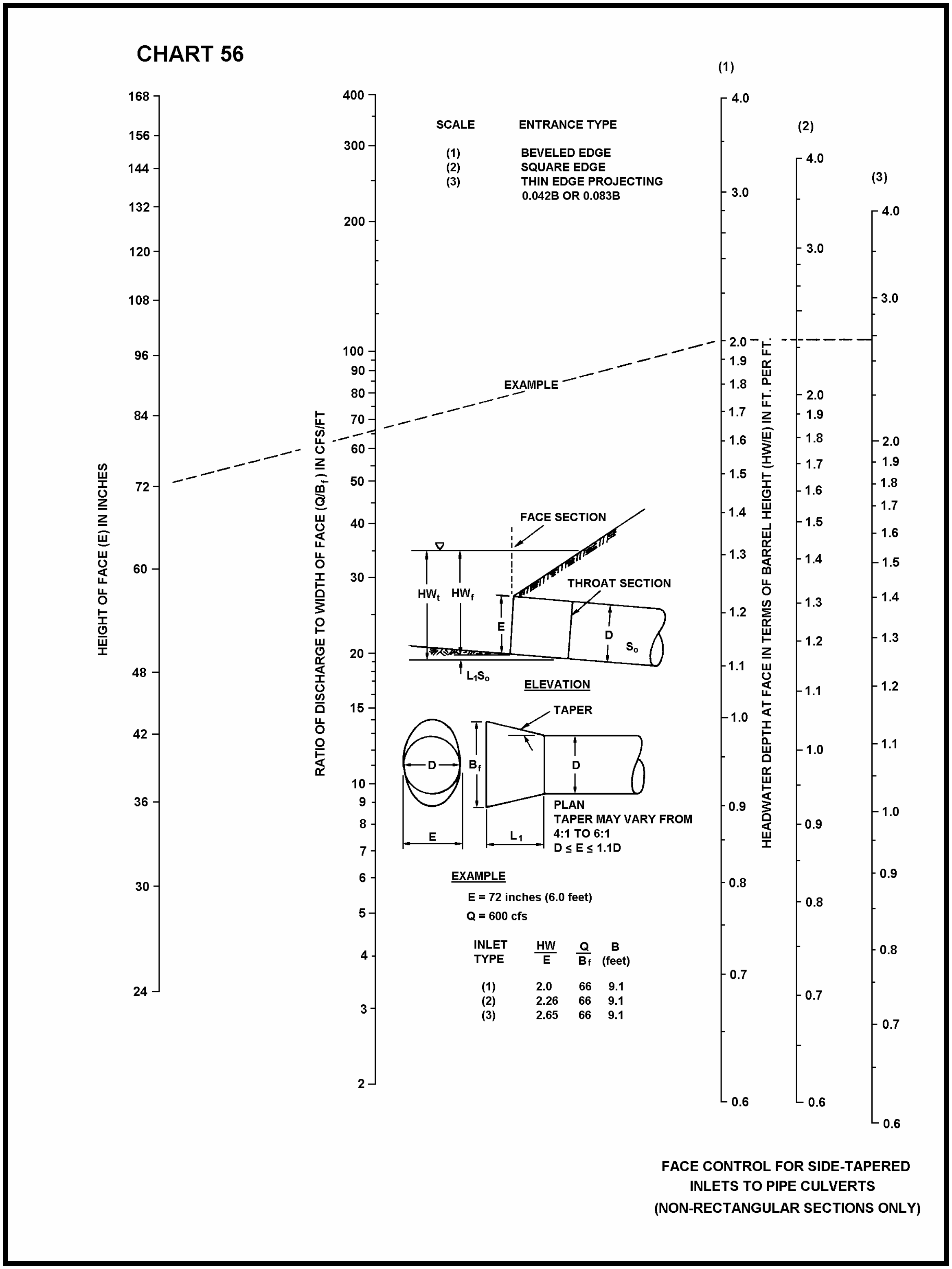
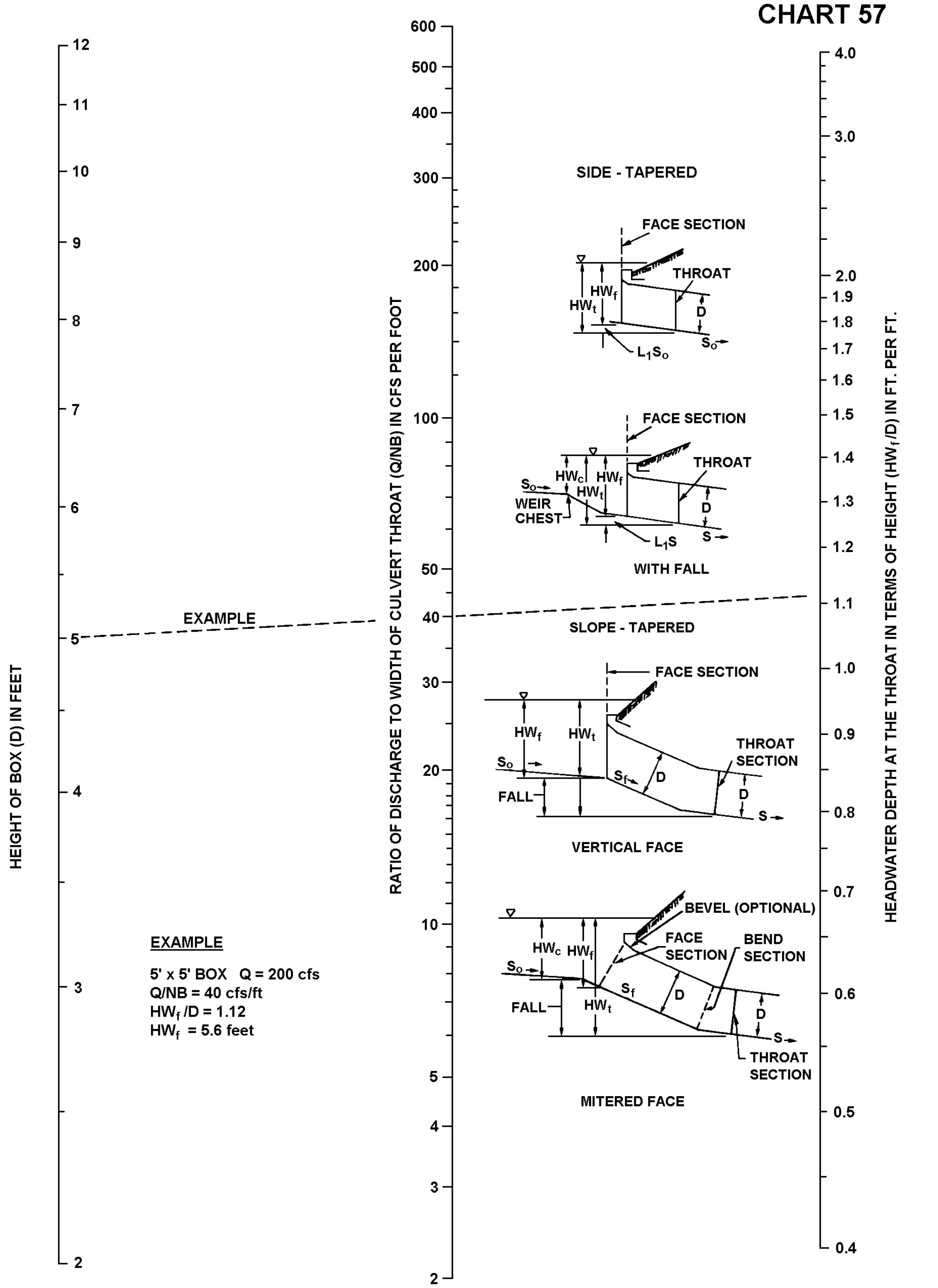


CHART 56
HEIGHT OF FACE (E) IN INCHES
RATIO OF DISCHARGE TO WIDTH OF FACE (Q/Bf) IN CFS/FT
HEADWATER DEPTH AT FACE IN TERMS OF BARREL HEIGHT (HW/E) IN FT. PER FT.
SCALE ENTRANCE TYPE
(1) BEVELED EDGE
(2) SQUARE EDGE
(3) THIN EDGE PROJECTING 0.042B OR 0.083B
EXAMPLE
FACE SECTION
THROAT SECTION
HWt HWf
E
D
So
L1So
ELEVATION
TAPER
D
Bf
D
E
L1
PLAN
TAPER MAY VARY FROM 4:1 TO 6:1
D ≤ E ≤ 1.1D
EXAMPLE
E = 72 inches (6.0 feet)
Q = 600 cfs
INLET TYPE HW/E Q/Bf B (feet)
(1) 2.0 66 9.1
(2) 2.26 66 9.1
(3) 2.65 66 9.1
FACE CONTROL FOR SIDE-TAPERED INLETS TO PIPE CULVERTS
(NON-RECTANGULAR SECTIONS ONLY)

CHART 57



THROAT CONTROL FOR BOX CULVERTS WITH TAPERED INLETS

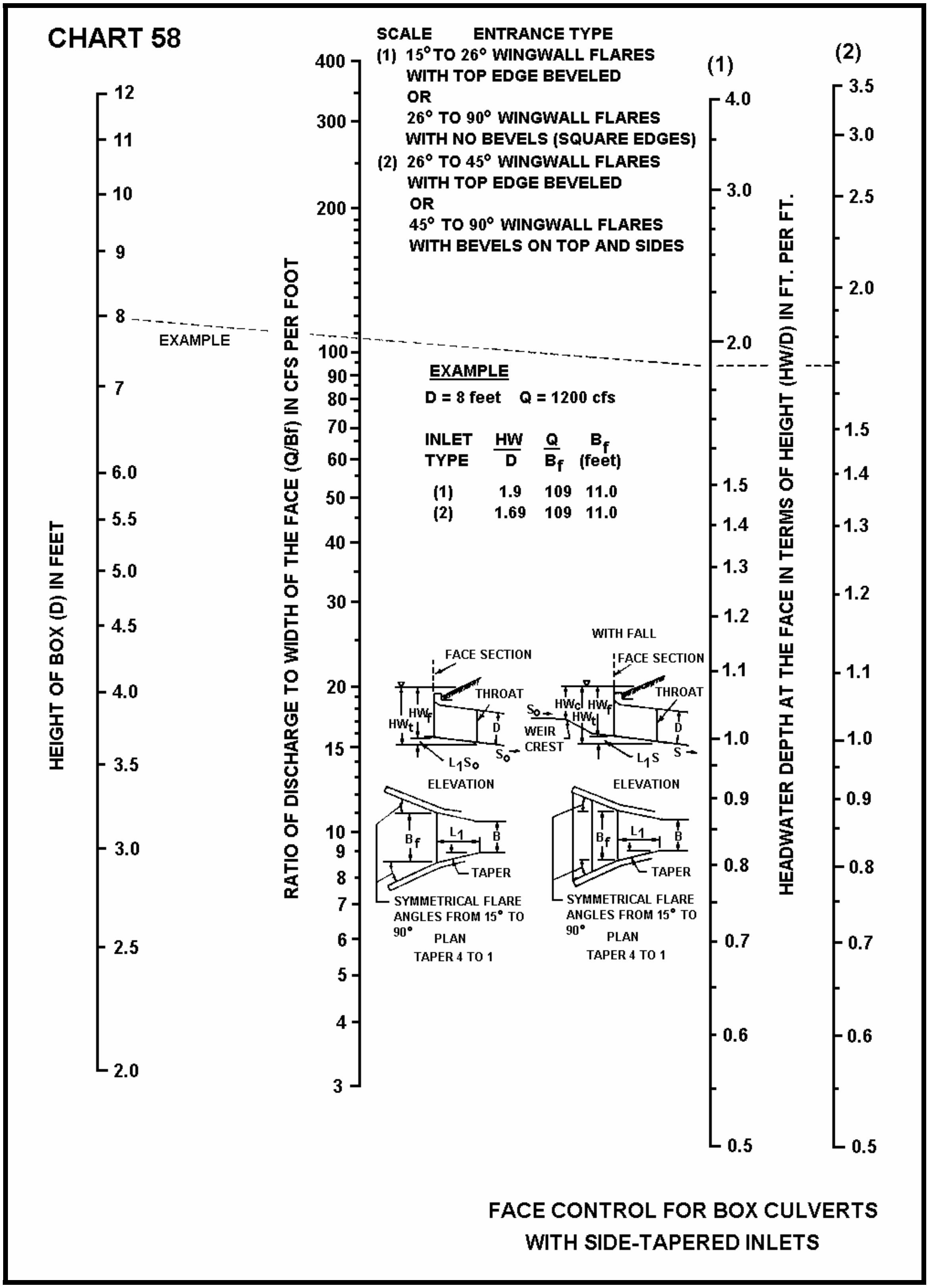


CHART 58
SCALE ENTRANCE TYPE
(1) 15° TO 26° WINGWALL FLARES WITH TOP EDGE BEVELED OR 26° TO 90° WINGWALL FLARES WITH NO BEVELS (SQUARE EDGES)
(2) 26° TO 45° WINGWALL FLARES WITH TOP EDGE BEVELED OR 45° TO 90° WINGWALL FLARES WITH BEVELS ON TOP AND SIDES
HEIGHT OF BOX (D) IN FEET
RATIO OF DISCHARGE TO WIDTH OF THE FACE (Q/Bf) IN CFS PER FOOT
HEADWATER DEPTH AT THE FACE IN TERMS OF HEIGHT (HW/D) IN FT. PER FT.
EXAMPLE
D = 8 feet Q = 1200 cfs
INLET TYPE | HW/D | Q/Bf | Bf (feet)
(1) 1.9 109 11.0
(2) 1.69 109 11.0
FACE SECTION
THROAT
ELEVATION
WITH FALL
WEIR CREST
TAPER
SYMMETRICAL FLARE ANGLES FROM 15° TO 90°
PLAN
TAPER 4 TO 1
FACE CONTROL FOR BOX CULVERTS WITH SIDE-TAPERED INLETS

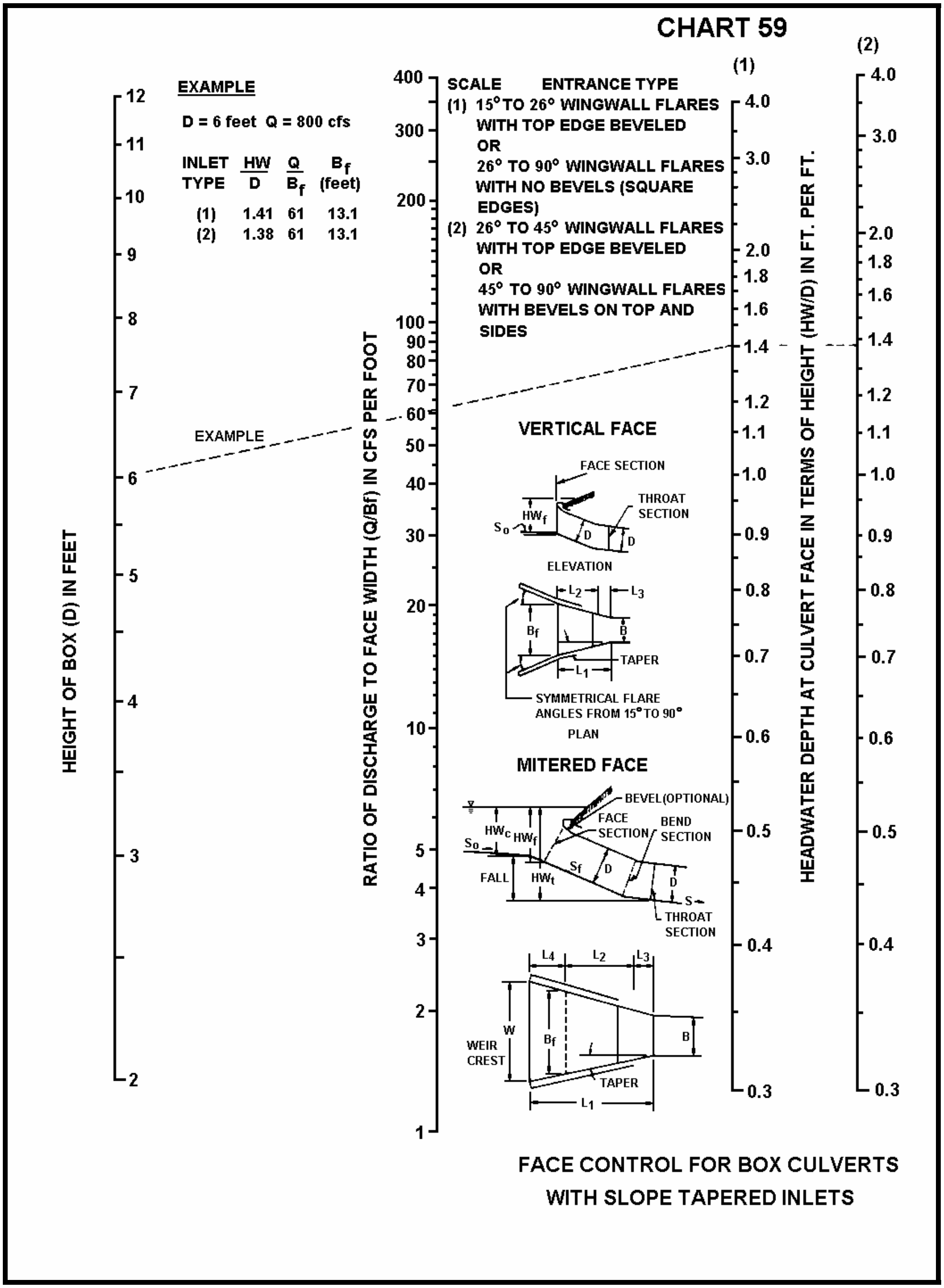
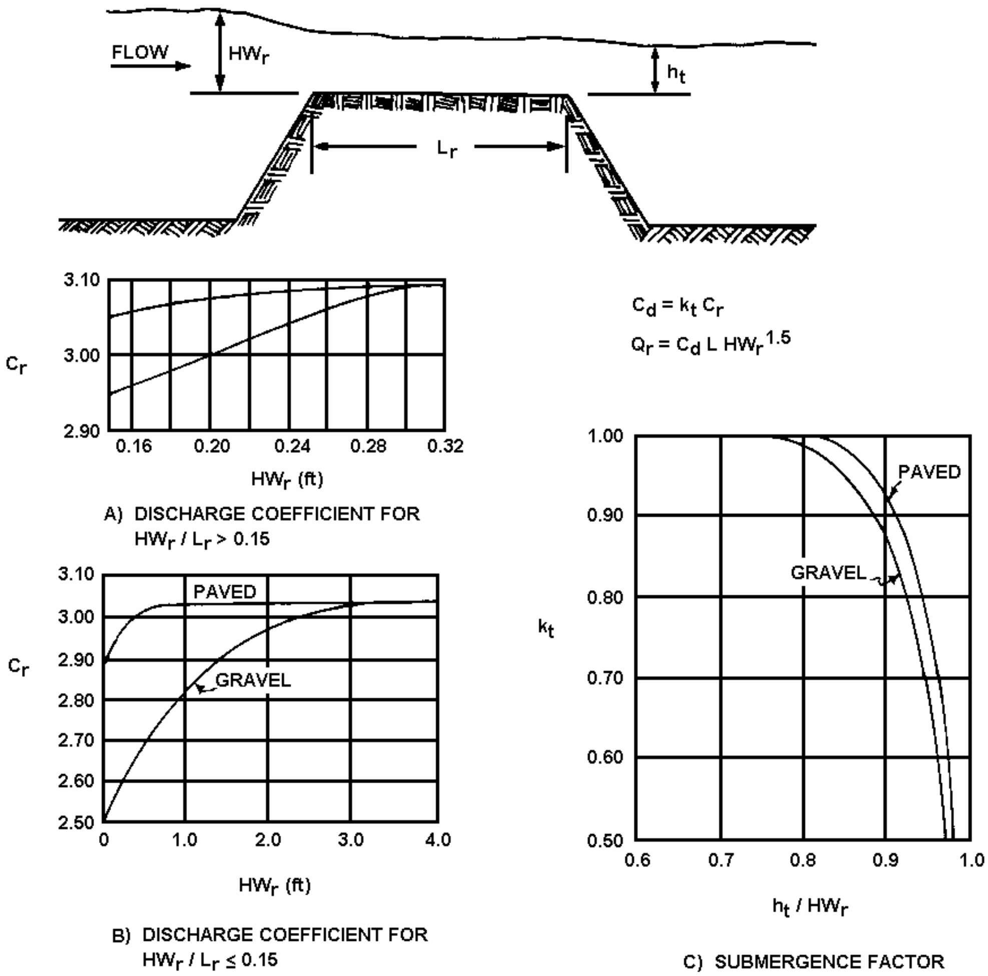


CHART 59
EXAMPLE
D = 6 feet Q = 800 cfs
INLET TYPE | HW/D | Q/Bf | Bf (feet)
(1) 1.41 61 13.1
(2) 1.38 61 13.1
SCALE ENTRANCE TYPE
(1) 15° TO 26° WINGWALL FLARES WITH TOP EDGE BEVELED OR 26° TO 90° WINGWALL FLARES WITH NO BEVELS (SQUARE EDGES)
(2) 26° TO 45° WINGWALL FLARES WITH TOP EDGE BEVELED OR 45° TO 90° WINGWALL FLARES WITH BEVELS ON TOP AND SIDES
HEIGHT OF BOX (D) IN FEET
RATIO OF DISCHARGE TO FACE WIDTH (Q/Bf) IN CFS PER FOOT
HEADWATER DEPTH AT CULVERT FACE IN TERMS OF HEIGHT (HW/D) IN FT. PER FT.
EXAMPLE
VERTICAL FACE
FACE SECTION
THROAT SECTION
ELEVATION
SYMMETRICAL FLARE ANGLES FROM 15° TO 90°
PLAN
TAPER
MITERED FACE
BEVEL(OPTIONAL)
FACE SECTION
BEND SECTION
FALL
THROAT SECTION
WEIR CREST
TAPER
FACE CONTROL FOR BOX CULVERTS WITH SLOPE TAPERED INLETS

CHART 60



DISCHARGE COEFFICIENTS FOR ROADWAY OVERTOPPING

This page left intentionally blank