Design
of
Stormwater Filtering Systems

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Abstract:

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Abstract: The project is oriented to create a unified design manual for stormwater filtering systems to remove pollutants from urban runoff generated at smaller sites within the Chesapeake Bay watershed. The primary audience for the manual are engineers, planners and landscape architects at the local or state level that need to comply with stormwater regulations in urban or suburban areas.

The manual presents detailed engineering guidance on eleven different filtering systems. The term stormwater filter refers to a diverse spectrum of stormwater treatment methods utilizing various media, such as sand, peat, grass, soil or compost to filter out pollutants entrained in urban stormwater. These filters are typically designed solely for pollutant removal, and serve small development sites. The three broad groups include: sand filters (surface, underground, perimeter, organic, and pocket designs), bioretention, and vegetated channels (grass channels, dry swales and wet swales, filter strips, and gravel wetlands).

The seven chapter design manual promotes a volume-based sizing criteria for all filtering systems utilizing principles of *small storm hydrology*, provides detailed guidance on the selection of appropriate filter types for various applications, reviews pollutant performance data and pathways for stormwater filtering systems, and provides detailed engineering design principles and guidance. The manual provides several design examples and contains over one-hundred tables and figures.

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PREFACE

This project is oriented to create a unified design manual for stormwater filtering systems to remove pollutants from urban runoff generated at smaller sites within the Chesapeake Bay watershed. The primary audience for the manual are engineers, planners and landscape architects at the local or state level that need to comply with stormwater regulations in urban or suburban areas.

This manual continues the Center's efforts to produce urban stormwater practice design manuals targeted at specific categories of systems. Stormwater filtering is just one of these targeted areas. Existing and future manuals will cover areas such as wetland systems and pond systems.

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Special gratitude to Don Koch of Engineering Technologies Associate for his contributions on the bioretention water balance analysis, Tim Schueler and Rick Scaffidi of Environmental Quality Resources for their insightful comments and suggestions, Ricardo Gonzalez and Jeffery Everhart for their diligent work on the grass species selection guide, Keith Bowers for his contributions to the bioretention species selection guide, Dean Geiser and Donna deMars for their superb preparation of graphic figures and drawings, and Arlene Allegretto for her tireless efforts in making this manual a reality.

INTRODUCTION

The manual presents detailed engineering guidance on ten different filtering systems. The term stormwater filter refers to a diverse spectrum of stormwater treatment methods which utilize an artificial media , such as sand, peat, grass, soil or compost to filter out pollutants entrained in urban stormwater. These filters are typically designed solely for pollutant removal (quantity bypassed), and serve small development sites (usually less than five acres). The three broad groups include: sand filters (surface, underground, perimeter, organic, and pocket designs), bioretention and vegetated channels (grass channels, dry swales wet swales, and filter strips).

The underlying concept of the manual is that a common and unified approach was needed to design each type of stormwater filter, so that this useful technology can gain wider engineering acceptance at the local level. Therefore, each stormwater filter incorporates four standard engineering features: a flow regulator, a pretreatment mechanism , filter media and bed specification, and overflow channels. In addition, the manual presents a single volumetric sizing requirement for each filter which is to capture and treat 90% of the runoff producing events that occur each year.

Many prior design approaches had been rate-based, and resulted in limited and unreliable pollutant removal rates. A third feature of the manual is that it utilizes new techniques for calculating runoff rates and volumes that reflect small storm hydrology from small, heterogeneous urban sites. Field research has indicated these methods are superior to traditional applications of the National Resource Conservation Service (NRCS) runoff forecasting models (such as TR-55 and TR-20). The manual also includes numerous step-by-step design examples that help an engineer apply the new design techniques. Lastly, the manual synthesizes recent research and field experience on the pollutant removal performance, longevity, cost, and maintenance burden of each type of stormwater filter, drawn from a national literature and phone survey. This information has been condensed in a series of tables that help designers and municipal officials select the most effective stormwater filter for their situation, and compare the performance of stormwater filters to that of other stormwater BMP options (e.g., ponds, wetlands, and infiltration systems).

Although stormwater filters can be applied to a diverse range of development conditions as a group, individual designs are limited to a more narrow range of site conditions . The most economical and feasible options are identified for five broad categories of development: ultra-urban, parking lots, roads, residential subdivisions, and backyard/rooftop drainage. Key feasibility factors that influence the selection of stormwater filters include space consumption, minimum head, maintenance burden, cost/acre and soil conditions.

During the study, over thirty published and unpublished studies on the pollutant removal performance of stormwater filtering systems were consulted (and are abstracted in Appendix A and cited in the References). Estimated removal rates for each of the stormwater filters are derived in Chapter 4, based on monitoring studies, infiltration rates, modeling and inference from similar technologies. Despite their many differences in design, stormwater filters have many similarities in performance.

The performance, feasibility, and environmental restrictions of stormwater filters are compared to three other groups of stormwater BMPs that are currently in widespread use by engineers in the Chesapeake Bay region-ponds, wetlands and infiltration systems.

In general, stormwater filters are the most feasible option for smaller development sites (less than 5 acres) but are not typically cost effective beyond that drainage area. Other BMPs, most notably ponds and wetlands, also have higher or more reliable removal rates for nutrients, bacteria and hydrocarbons. Ponds and wetlands, however, cannot usually be applied on small development sites and ultra urban conditions. Another key advantage of stormwater filters as a group is their lack of environmental drawbacks, such as stream warming, groundwater contamination, wetland impairment, and public safety. On the other hand, with one notable exception (bioretention), most stormwater filters confer few if any amenity values to the community (such as habitat, flood control, landscaping or increase in property value.

In summary, stormwater filters have their greatest applicability for small development sites, and can generally provide reliable rates of pollutant removal if design improvement are made and regular maintenance is performed. Stormwater filters appear to have particular utility in treating runoff from urban "hotspot" source areas such as commercial parking lots, vehicle service centers, and industrial sites, as well as problematic street and highway sites when other BMPs are not feasible.

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CHAPTER 1
INTRODUCTION TO STORMWATER FILTERING SYSTEMS

1.1 WHAT ARE STORMWATER FILTERING SYSTEMS?

Stormwater filtering systems refer to a diverse group of techniques for treating the quality of stormwater runoff. The common thread is that each utilizes some kind of filtering media, such as sand, soil, gravel, peat or compost to filter out pollutants entrained in stormwater runoff. In addition, most filtering systems are typically applied to small drainage areas (five acres or less). Third, filtering systems are designed solely for pollutant removal. Flows greater than the water quality treatment volume are bypassed around the filter to a downstream stormwater management facility. Lastly, filtering systems incorporate four basic design components in every application.

1.2 COMMON DESIGN COMPONENTS

While stormwater filters are a diverse group of stormwater practices, they have several common design components. The four basic design components of a filtering system are: (a) inflow regulation that diverts a defined flow volume into the system; (b) a pretreatment technique to capture coarse sediments; (c) the filter bed surface and unique filter media, and (d) an outflow mechanism to return treated flows back to the conveyance system and/or safely handle storm events that exceed the capacity of the filter. Each of the design components are described in greater detail below:

1.2A INFLOW REGULATION

The inflow regulator is used to divert runoff from a pipe, open channel or impervious surface into the filtering system. The inflow regulator is designed to divert the desired water quality volume into the filter, and also allow large flow volumes to continue through the conveyance channel. With a few exceptions, most filtering systems are constructed *off-line* (i.e., runoff is diverted from the main conveyance system, treated, and then returned back to the conveyance system (Figure 1.1a). A few filtering systems are constructed *on-line,* such as the swale system depicted in Figure 1.1b. On-line filters are located within the conveyance system, and are exposed to the full range of flow events from the smallest storm up to and including the 100 year event.

FIGURE 1.1: SCHEMATIC: ON-LINE VS. OFF-LINE DESIGN

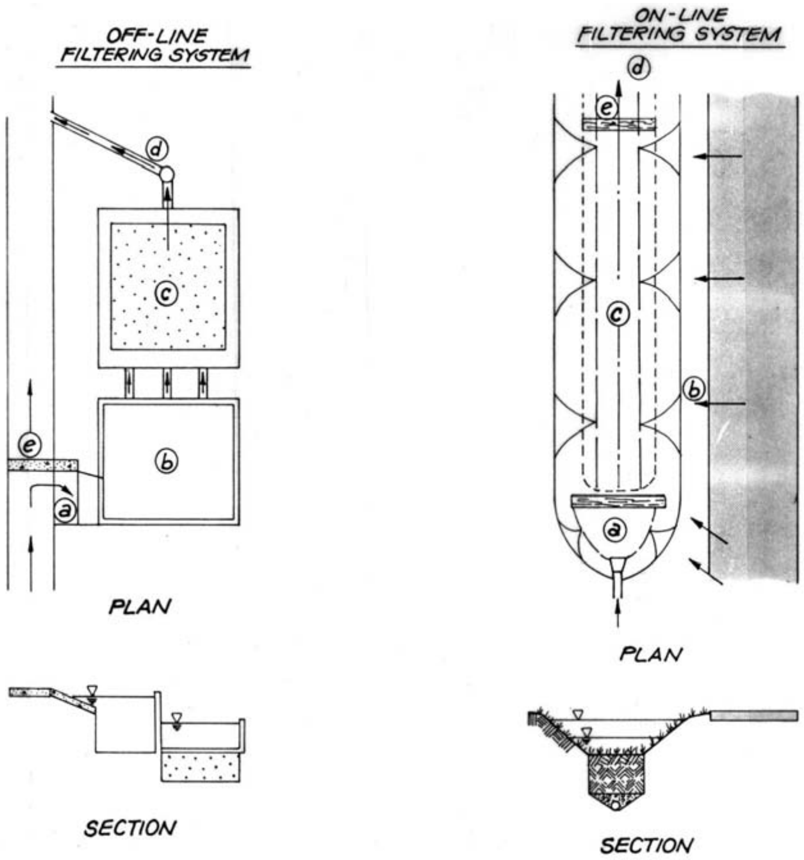


FIGURE 1.1A
OFF-LINE FILTERING SYSTEM

FIGURE 1.1B
ON-LINE FILTERING SYSTEM

1.2B PRETREATMENT

The second key component of any filtering system is pretreatment. Pretreatment is needed in every design to trap coarse sediments before they reach the filter bed.

Without pretreatment, the filter will quickly clog, and lose its pollutant removal capability. Each filter design differs with respect to the type and volume of pretreatment afforded. The most common technique of pretreatment is a wet or dry settling chamber. Geotextile screens, pea gravel diaphragms and grass filter strips may also be used as a secondary form of protection. Sediments deposited in the pretreatment chamber must be periodically removed to maintain the system.

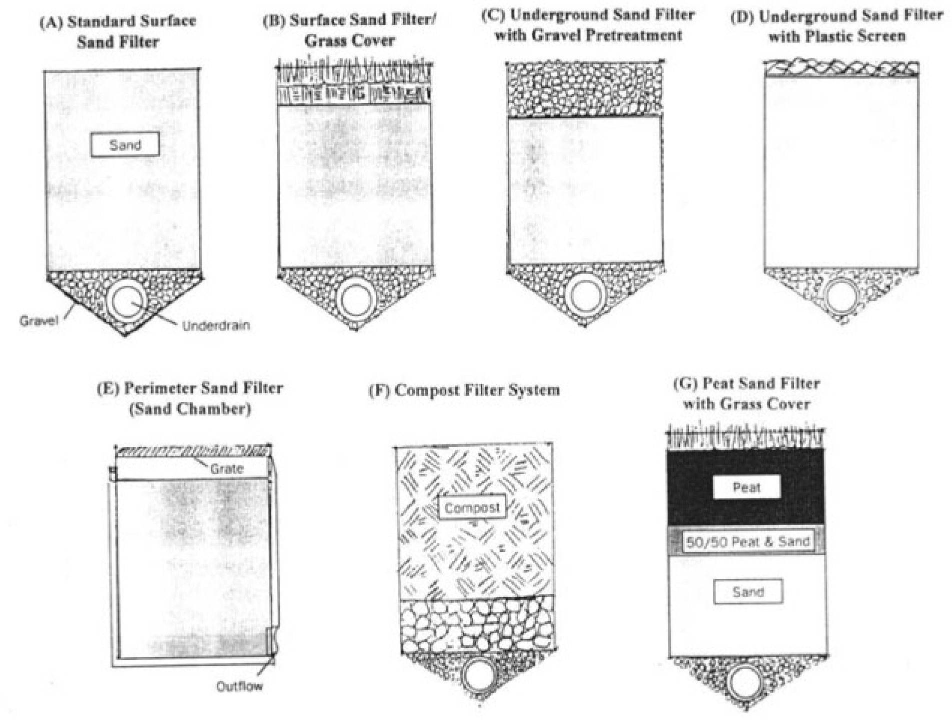
1.2C FILTER BED AND FILTER MEDIA

Each filtering system utilizes some kind of media such as sand, gravel, peat, grass, soil or compost to filter out pollutants entrained in urban stormwater, and some designs utilize more than one. The selection of the right media is important, as each has different hydraulic, pollutant removal and clogging characteristics.

The filter media is incorporated into the filter bed. The three key properties of the bed are its surface area, depth, and profile. The required *surface area* for a filter is usually based as a percentage of impervious area treated and the media itself, and may vary due to regional rainfall patterns and local criteria for water quality treatment volumes. The *depth* of most filtering systems ranges from 18 inches to four feet. A relatively shallow filter bed is used for hydraulic and cost reasons, and because most pollutants are trapped in the top few inches of the bed. Each design also utilizes a slightly different *profile* through the bed. An example of the variation in sand filter profiles is shown in Figure 1.2. As can be seen, each design has slightly different surface protection and layering through the bed.

1.2D OUTFLOW MECHANISM

FIGURE 1.2: CROSS-SECTION OF SAND FILTER DESIGN VARIATIONS



The final component of any stormwater filter design is the method(s) used to *collect or exfiltrate* the filtered runoff that leaves the filter bed and bypass the larger storm flows. The two primary methods for handling filtered runoff are to *collect* it in perforated pipes and return it back to the conveyance system, or to allow it to *exfiltrate* into the underlying soils where it may ultimately reach groundwater. Each method has its pros and cons. In the *collection* method, the bottom of the filter bed may be sealed with an impermeable liner which allows the filtered runoff to be captured in pipes and returned to the conveyance system. This is desirable if the contributing land use is considered a pollutant hotspot or if groundwater contamination is a concern. In the *exfiltration* method, the bottom of the filter bed is fully or partly permeable, and the filtered runoff continues downward through the soil and into groundwater. The uncollected runoff volume and pollutant mass drain into underlying soils and the water table. The advantage of exfiltration is that it provides groundwater recharge and takes advantage of the natural filtering capacity of soil to remove additional pollutants.

1.3 TYPES OF STORMWATER FILTERING SYSTEMS

This section describes the five broad groups of filtering systems that can be used for stormwater treatment. They include sand filters, open vegetated channels, bioretention areas, filter strips and submerged gravel filters. Within each group of filters are a number of important design variants that need to be considered.

1.3A SAND FILTERS

The City of Austin, Texas first pioneered the use of sand filters to treat urban stormwater runoff in the early 1980's. Since then the practice has rapidly evolved, with nearly a dozen variants of the basic sand filter design developed in response to different climatic, development and site conditions. For purposes of this manual, sand filter designs are grouped into five broad categories:

- Surface Sand Filter
- Underground Sand Filter
- Perimeter Sand Filter
- Organic Filter
- Pocket Sand Filter

SURFACE SAND FILTER

The earliest design was the surface sand filter, shown in Figure 1.3. A flow splitter is used to divert the first flush of runoff into an off-line sedimentation chamber. The chamber may be either wet or dry, and is used for pretreatment. Coarse sediments drop out as the runoff velocities are reduced. Runoff is then distributed into the second chamber, which consists of an 18 inch deep sand filter bed and temporary runoff storage above the bed. Pollutants are trapped or strained out at the surface of the filter bed. The filter bed surface may have a sand or grass cover. A series of perforated pipes located in a gravel bed collect the runoff passing through the filter bed, and return it into the stream or channel at a downstream point. If underlying soils are permeable, and groundwater contamination unlikely, the bottom of the filter bed may have no lining, and the filtered runoff may be allowed to exfiltrate.

UNDERGROUND SAND FILTER

The underground sand filter was adapted for sites where space is at a premium. In this design, the sand filter is placed in a three chamber underground vault accessible by manholes or grate openings. (Figure 1.4). Pioneered in the District of Columbia, the vault can be either on-line or off-line in the storm drain system. The first chamber is used for pretreatment and relies on a wet pool as well as temporary runoff storage. It is connected to the second sand filter chamber by an inverted elbow, which keeps the filter surface free from trash and oil. The filter bed is 18 inches in depth and may have a protective screen of gravel or permeable geotextile to limit clogging. During a storm, the water quality volume is temporarily stored in both the first and second chambers. Flows in excess of the filter's capacity are diverted through an overflow weir. Filtered runoff is always collected, using perforated underdrains that extend into the third “overflow” chamber.

PERIMETER SAND FILTER

The “Delaware” sand filter, developed by Shaver and Baldwin (1991), consists of two parallel trench-like chambers that are typically installed along the perimeter of a parking lot (Figure 1.5). Parking lot runoff enters the first chamber which has a shallow permanent pool of water. The first trench provides pretreatment before the runoff spills into the second trench, which consists of an 18 inch deep sand layer. During a storm event, runoff is temporarily ponded above the normal pool and sand layer, respectively. When both chambers fill up to capacity, excess parking lot runoff is routed to a bypass drop inlet. The remaining runoff is filtered through the sand, and collected by underdrains and delivered to a protected outflow point.

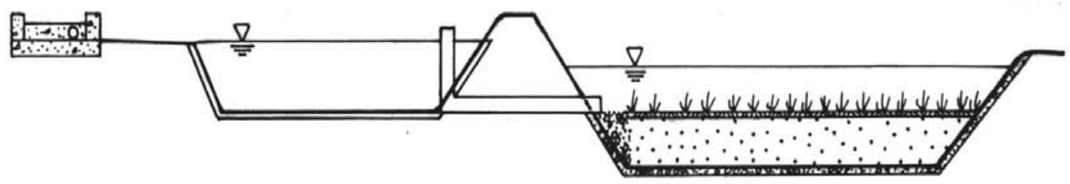
ORGANIC FILTER

The organic filter functions the same as a surface sand filter design, with the exception that it uses compost or peat/sand as the filter media. The basic design of an organic filter is shown in Figure 1.6. A flow splitter diverts runoff into a pretreatment chamber, and then passes into a series of filter cells. Each filter bed contains an 18 inch layer of compost or peat, followed by a filter fabric, and six inches of perforated pipe and gravel. Runoff filters through the organic media and is then collected by a perforated pipe and directed toward the outlet. In most organic filters, the filter bed and subsoils are separated by impermeable polyliner to prevent movement into groundwater.

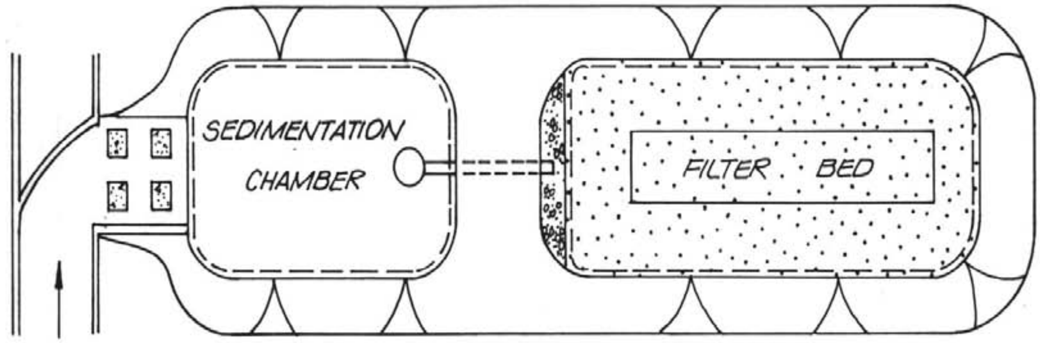
POCKET SAND FILTER

The pocket sand filter is a simplified and low cost design that may be used on smaller sites. Runoff is diverted within a manhole (Figure 1.7). A bypass pipe sends excess runoff along the storm drain system, and a flow diversion pipe routes the water quality volume into the system. Pretreatment is provided by a concrete flow spreader, a grass filter strip and a plunge pool. The filter bed is also a relatively simple affair. A shallow basin is excavated, and contains the sand filter layer. Most of the water quality volume is temporarily stored above the filter bed. The surface of the filter bed contains a soil layer and grass cover crop. In the event of clogging, the pocket sand filter has a pea gravel “window” to direct runoff into the sand, as well as a cleanout and observation well. In most cases, the filtered runoff is allowed to exfiltrate into the underlying soils, although underdrains may be needed if the soils are not suitably permeable.

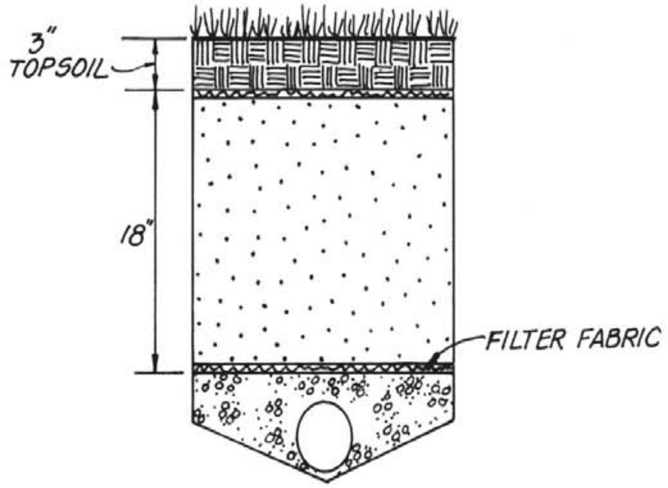
FIGURE 1.3: SURFACE SAND FILTER



PROFILE



PLAN



TYPICAL SECTION

FIGURE 1.4: UNDERGROUND SAND FILTER

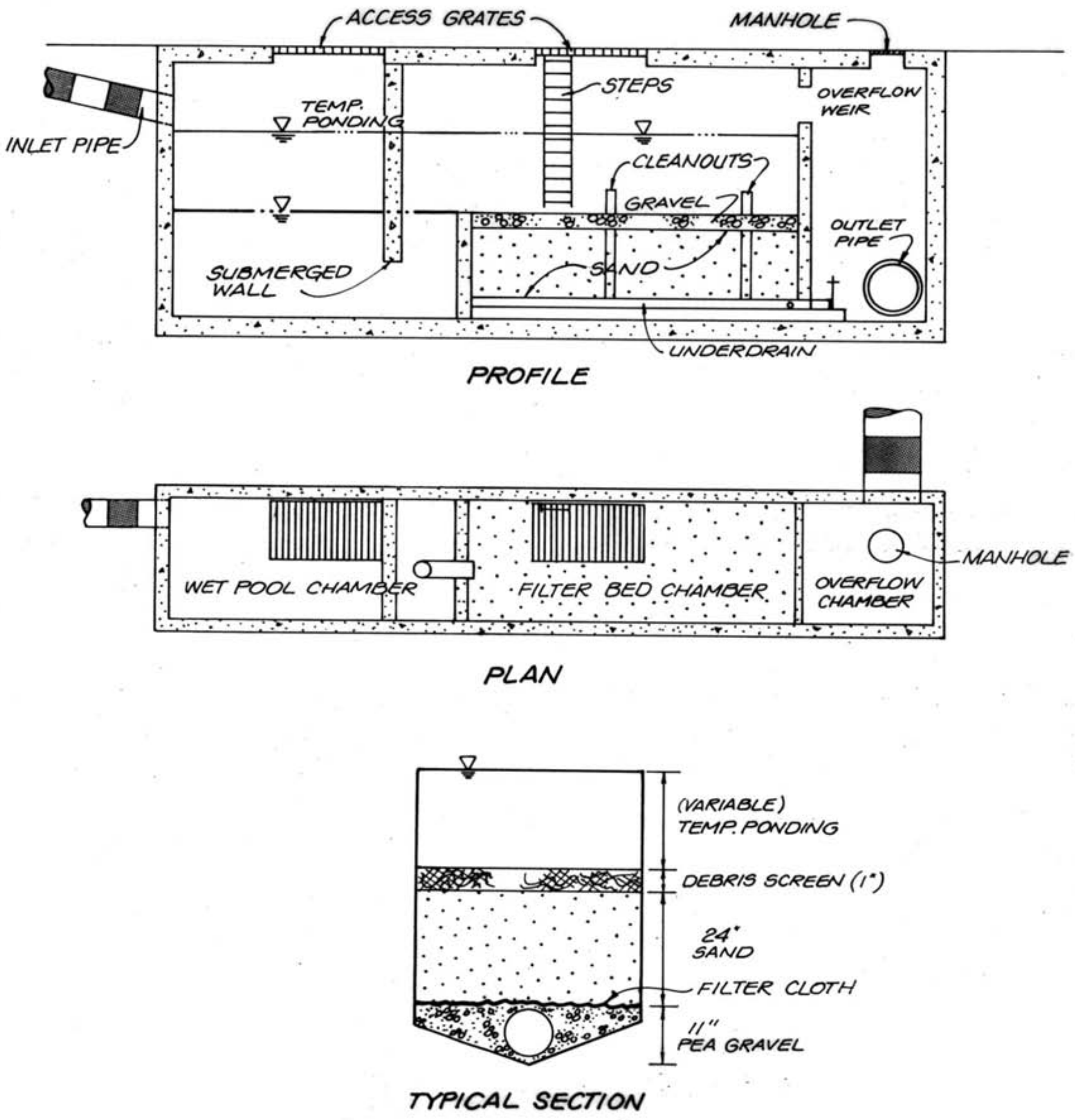
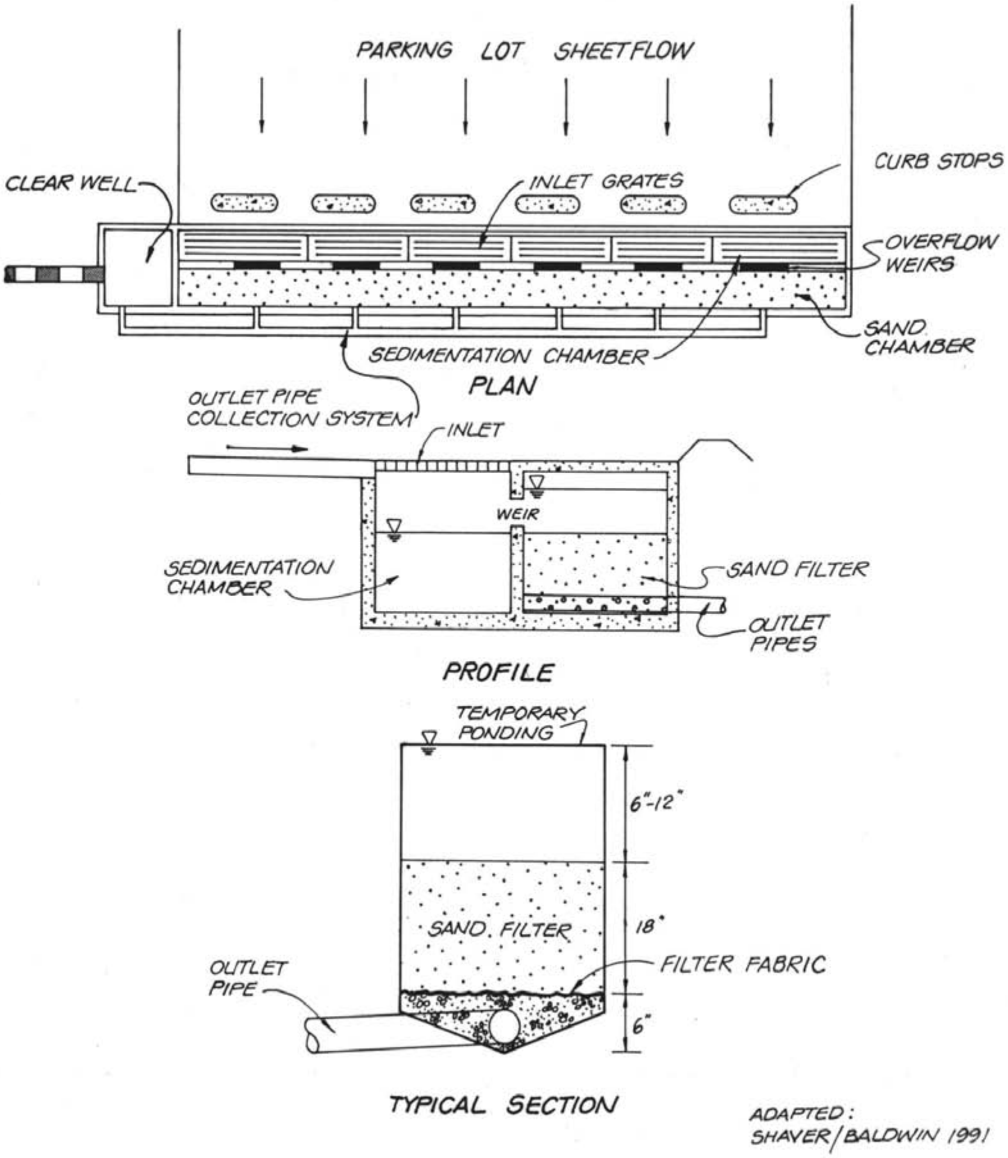


FIGURE 1.5: PERIMETER SAND FILTER



ADAPTED: SHAVER/BALDWIN 1991

FIGURE 1.6: ORGANIC FILTER

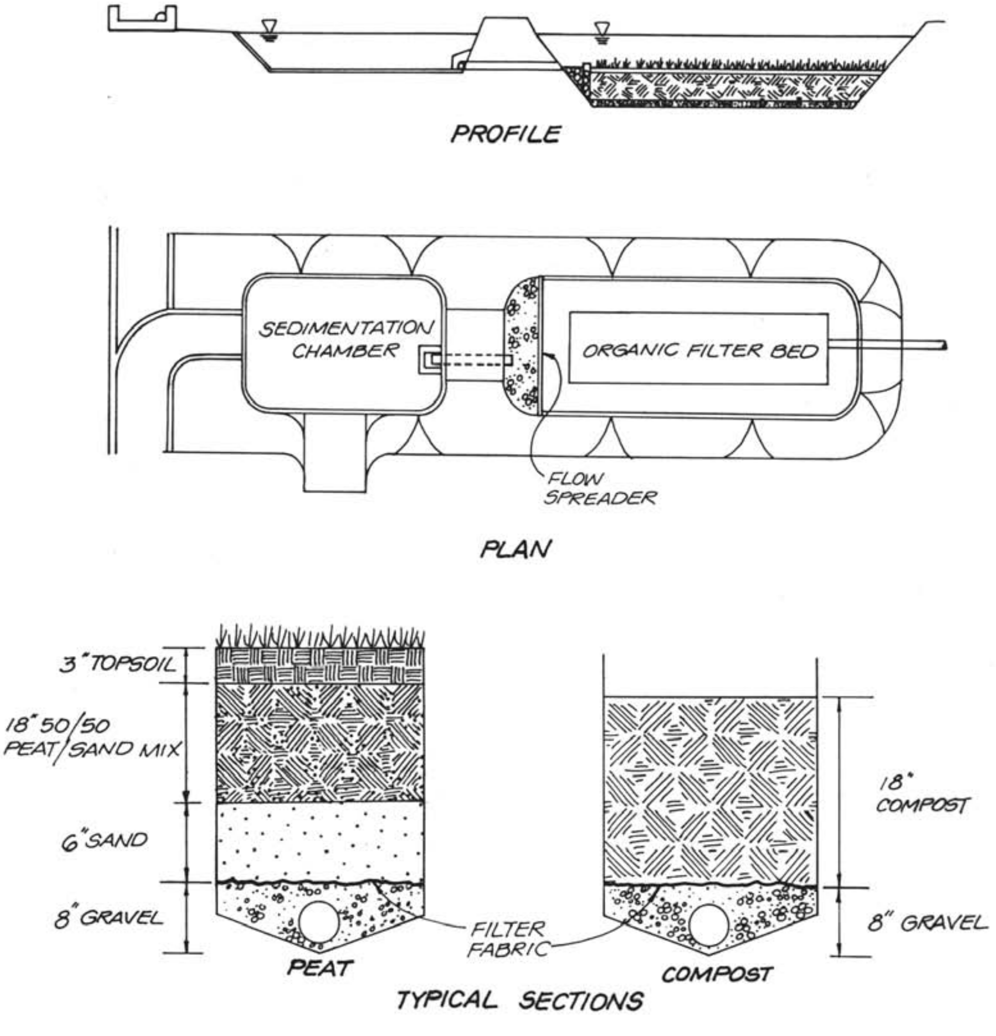
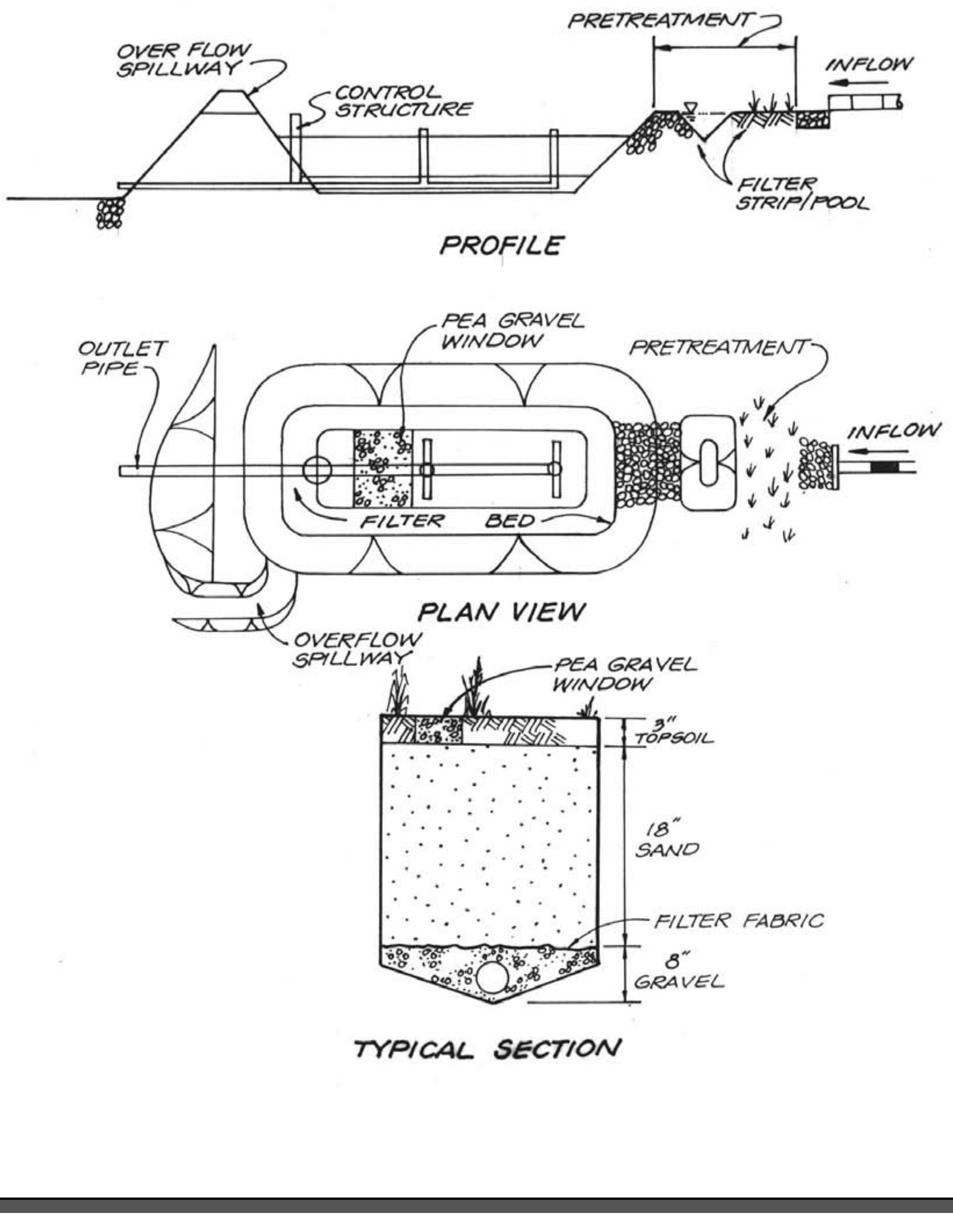


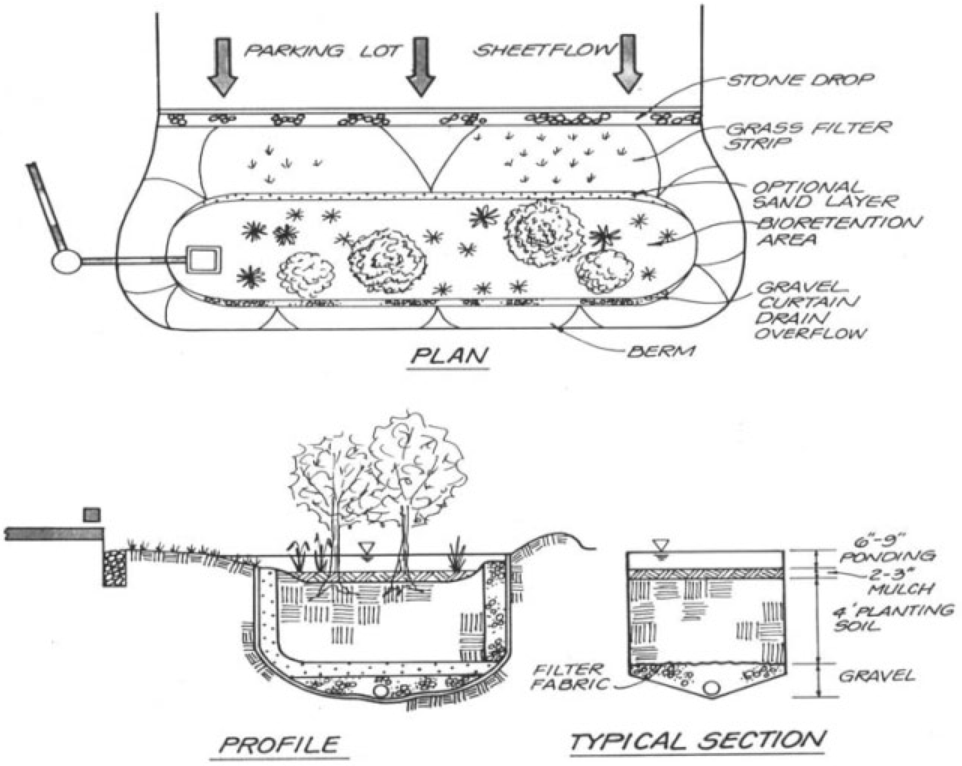
FIGURE 1.7: POCKET SAND FILTER



1.3B BIORETENTION

This filtering system utilizes parking lot islands and planting strips for on-site treatment of the water quality volume. Surface runoff is directed into shallow, landscaped depressions in the parking lot, known as bioretention areas. These depressions are modeled to incorporate many of the pollutant removal mechanisms that operate in forested ecosystems. Key elements include a grass filter, sand layer, loamy soils, mulch layer, shallow ponding of stormwater and plantings of native trees and shrubs (Figure 1.8). Pretreatment mechanisms include a stone drop at the edge of the parking lot that leads over a grass filter strip and a sand layer. During storms, the water quality volume is ponded up to nine inches above the mulch. Runoff in excess of the water quality volume rises to a higher elevation, but is then diverted into a standard drop inlet connected to the storm drain system. The remaining runoff filters through the mulch and prepared soil mix, which is about four feet in depth. Typically, the filtered runoff is collected in a perforated underdrain and returned to the storm drain system.

FIGURE 1.8: BIORETENTION FILTER

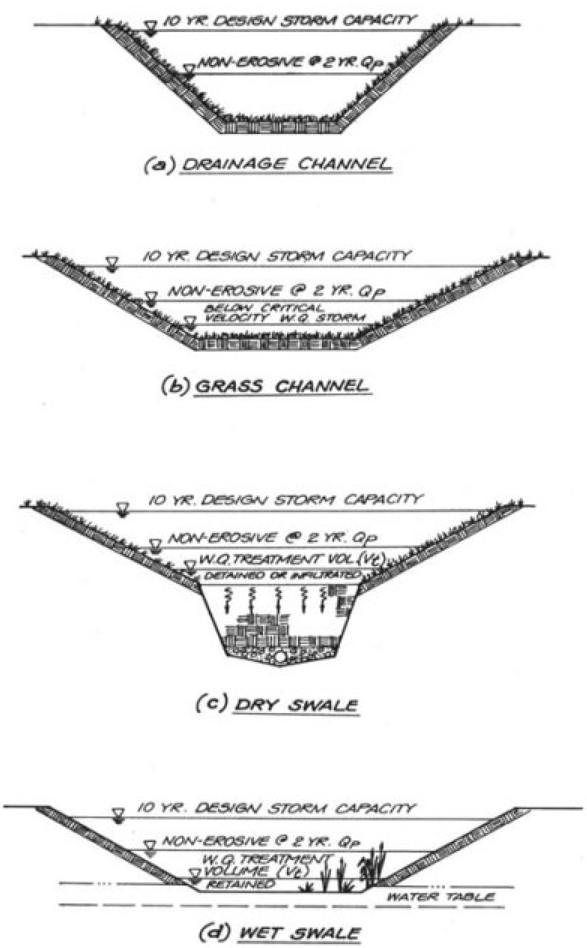


The benefits of bioretention include low land consumption, as the entire bioretention area can fit within the 5 to 10% of a parking lot that is typically devoted to landscaping. In addition, regular maintenance can be provided by commercial landscaping companies, and the “planting hole” provided by the bioretention area often increases the survival rates of landscaping.

1.3C OPEN VEGETATED CHANNELS

Stormwater engineers frequently use open channels or grass swales to convey stormwater runoff. In some cases, open channels can be redesigned to provide significant pollutant removal. It is therefore quite important to define what is meant by open channels, so as to better distinguish the potential differences in pollutant removal potential that various channel designs can have during small storms. In this sense, open channels can be classified into one of four possible categories, based on their hydrologic design. They are the drainage channel, grassed channel, dry swale and wet swale (Figure 1.9).

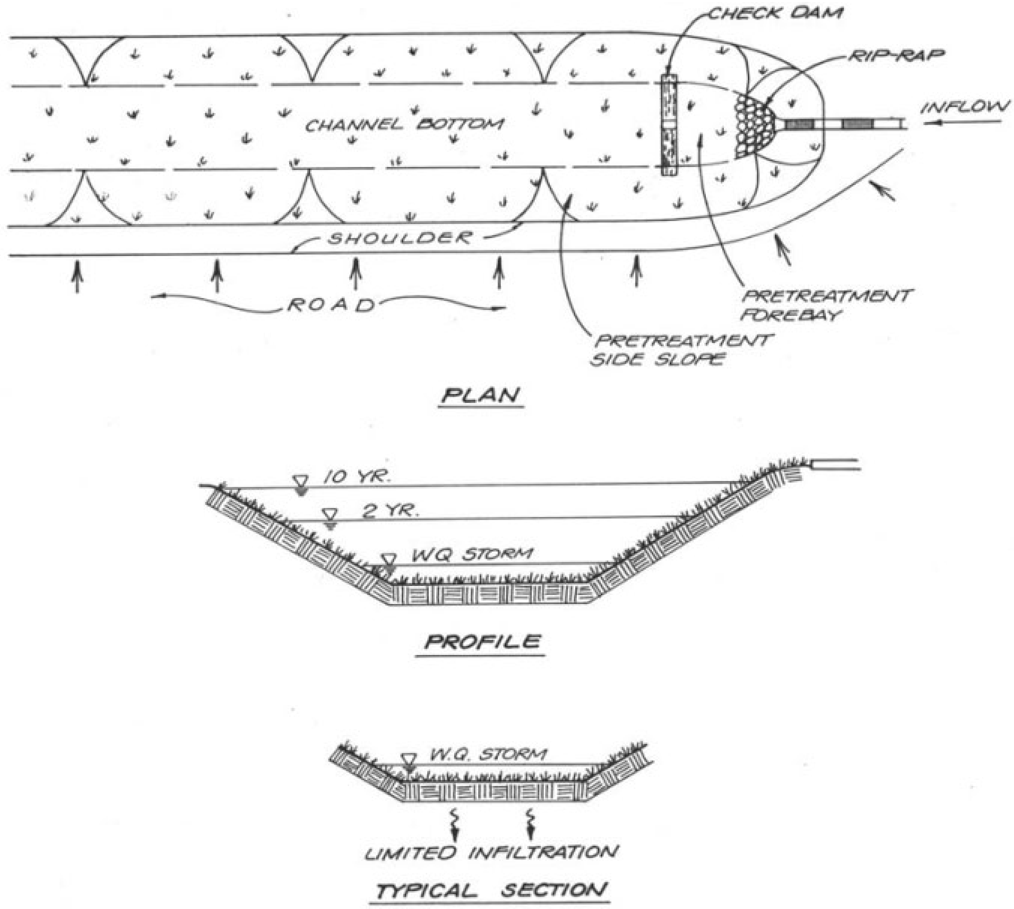
FIGURE 1.9: OPEN VEGETATED CHANNELS



The open channel design in most common use is termed a drainage channel, and is designed to have enough capacity to safely convey runoff during large storm events without erosion. Typically, a drainage channel has a cross-section with hydraulic capacity to handle the peak discharge rate for the ten year storm event, and channel dimensions (i.e., slope and bottom width) that will not exceed a critical erosive velocity during the peak discharge rate associated with the two year storm event (Figure 1.9a). Consequently, most drainage channels provide very limited pollutant removal, unless soils are extremely sandy and slopes are very gentle.

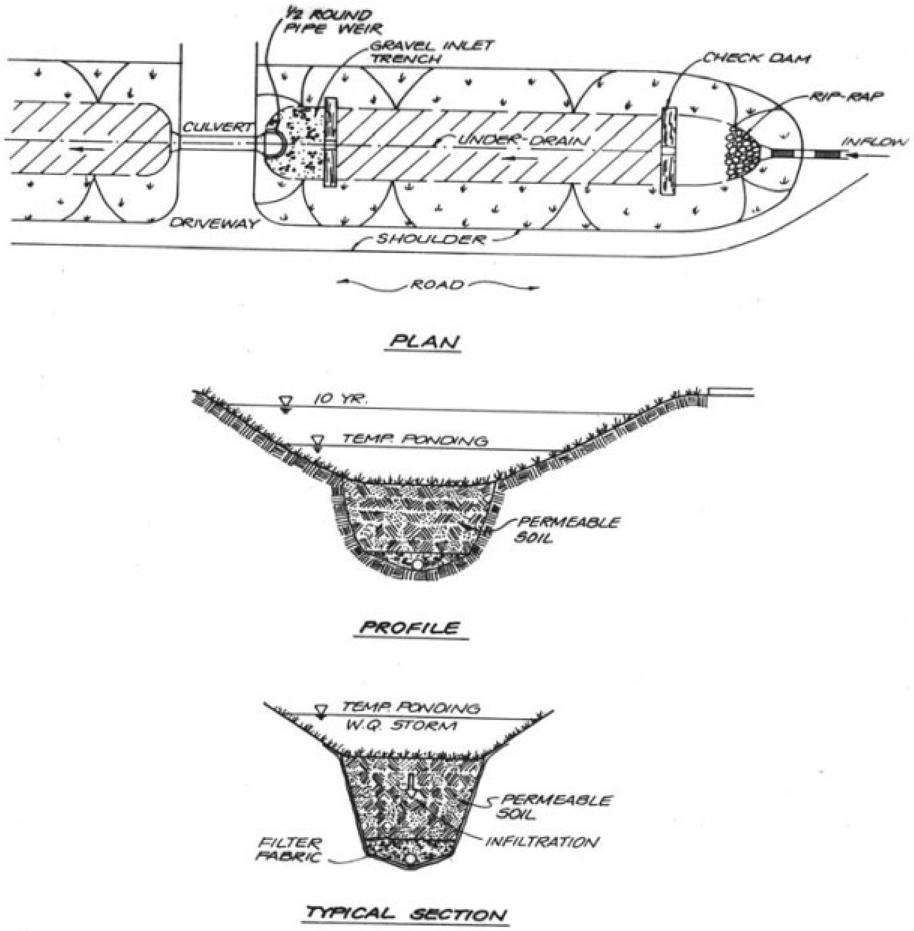
To achieve greater pollutant removal, stormwater engineers have recently employed *grass channels*. Grass channels are designed to meet runoff velocity targets for two very different storm conditions—a water quality design storm and the two year design storm (Figure 1.10). During the "water quality storm," runoff velocity typically cannot exceed 1.0 fps during the peak discharge associated with the water quality design rainfall event, and the total length of the channel must provide at least ten minutes residence time. In some regions of the country, grass channels are termed "biofilters" (Seattle METRO, 1992). To meet the water quality criteria, grass channels must have broader bottoms, lower slopes and denser vegetation than most drainage channels. Nominal pretreatment is created by placing checkdams across the channel below pipe inflows, and at various other points along the channel. The filter bed area in a grass channel is usually confined to the top inch of soil and thatch, since most runoff events will traverse the length of channel in about ten minutes.

FIGURE 1.10: GRASS CHANNEL



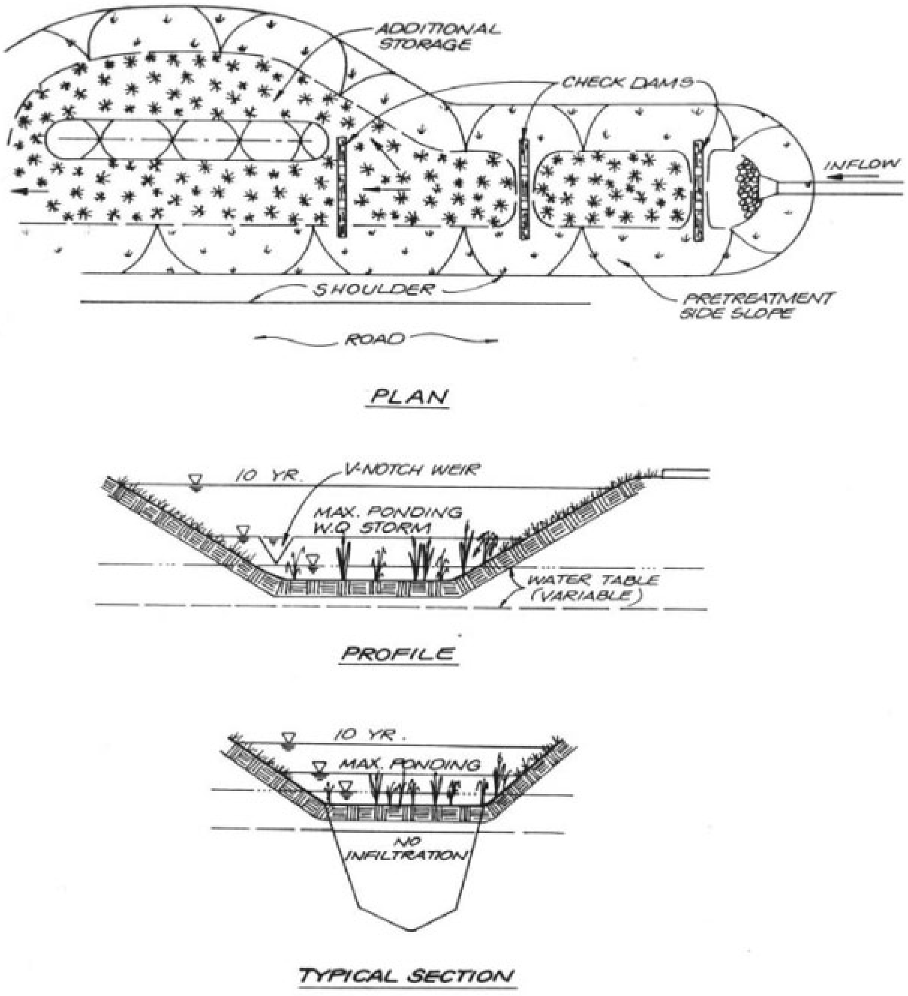
A third open channel is termed the dry swale. In a dry swale, the entire water quality volume is temporarily retained by checkdams during each storm. Unlike the grass channel, the filter bed in the swale consists of 30 inches of prepared soil (sandy loam) that is then collected by an underdrain pipe (see Figure 1.11). The swale is designed to rapidly dewater, thereby allowing front yards to be more easily mowed. Again, pretreatment is provided through check dams at pipe inflow points, and by keeping side slopes gentle if they are adjacent to impervious areas. In the event that surface soils clog, the dry swale has a pea gravel window on the downstream side of each checkdam to route water to the underdrain. A dry swale is often the preferred open channel option in residential settings since it is designed to prevent standing water that makes mowing difficult and generates complaints.

FIGURE 1.11: DRY SWALE



The last open channel design is termed a wet swale, and occurs when the water table is located very close to surface (Figure 1.12). As a result, swale soils often become fully saturated, or have standing water all or part of the year after the channel has been excavated. This "wet swale" essentially acts as a very long and linear shallow wetland treatment system. Like the dry swale, the entire water quality treatment volume is stored and retained within a series of cells in the channel, formed by berms or checkdams. The notched checkdams are set so that the invert creates the pool level when the water table is high. The dimensions of the notches are set to provide the desired detention time within each cell for the storm. In some cases, the cells may be planted with emergent wetland plant species to improve removal rates. If land is available, some wetland cells can be placed off-line, as shown in Figure 1.12.

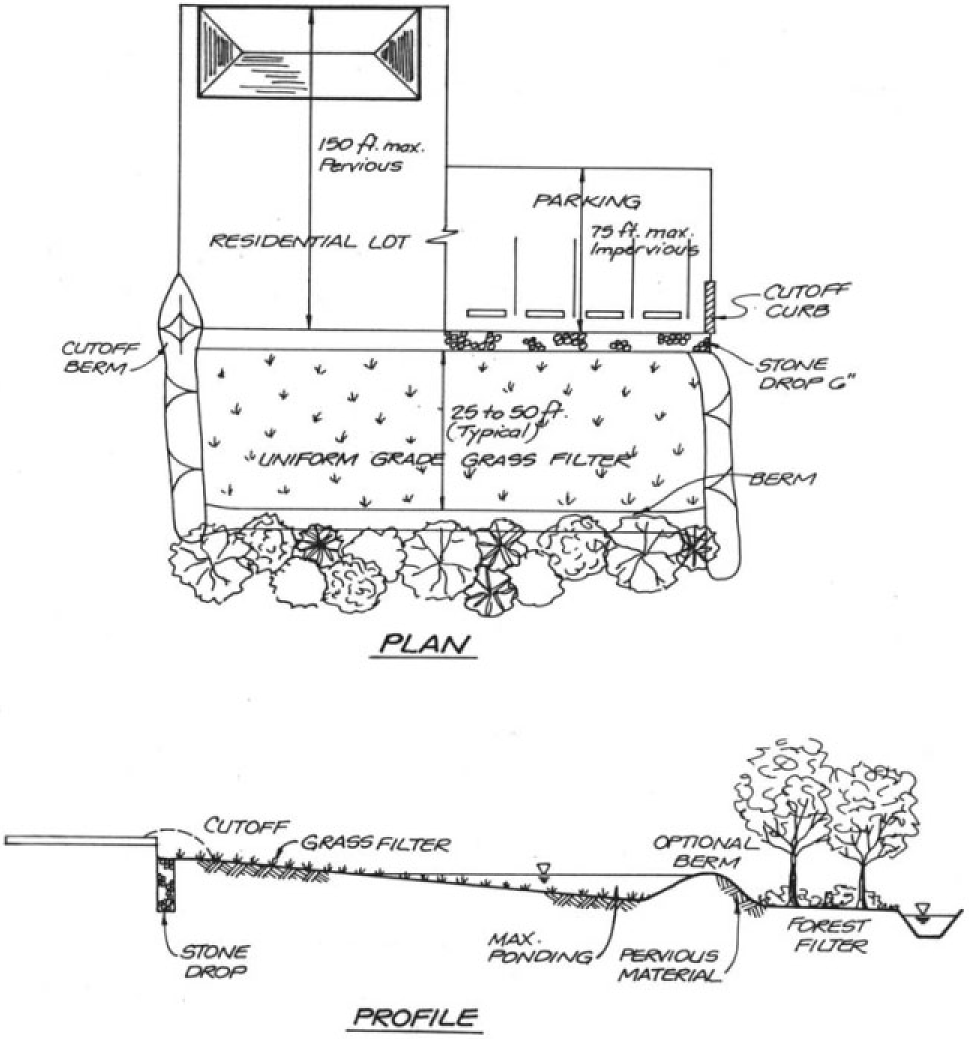
FIGURE 1.12: WET SWALE



1.3D FILTER STRIP

Filter strips rely on the use of vegetation to slow runoff velocities and filter out sediment and other pollutants from urban stormwater. To be effective, however, filter strips require the presence of sheet flow across the entire strip. Once flow concentrates to form a channel, it effectively short-circuits the filter strip. Unfortunately, this usually occurs within a short distance in urban areas. It is doubtful, for example, whether sheetflow can be maintained over a distance of 150 feet for pervious areas, and 75 feet for impervious areas (or about one parking bay). In the most common design, runoff is directed from a parking lot into a long filtering system composed of a stone trench, a grass strip and a longer wooded strip (see Figure 1.13).

FIGURE 1.13: FILTER STRIP

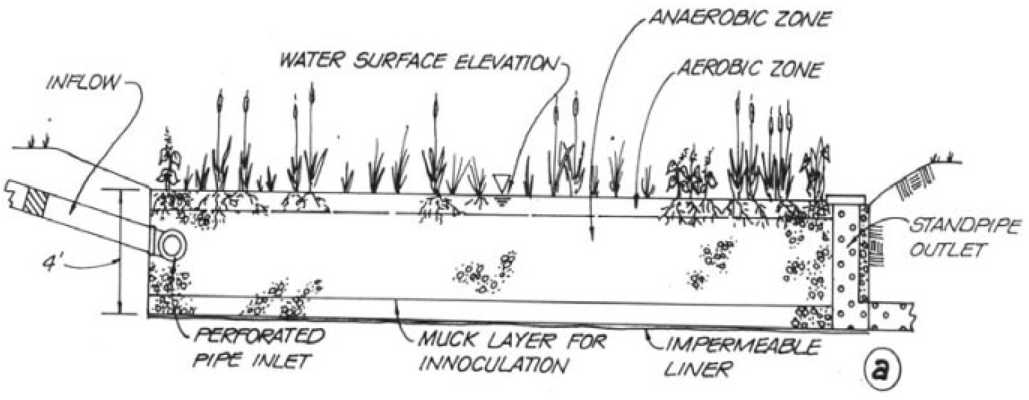


The grass portion of the filter strip provides pretreatment for the wooded portion. In addition, a six inch stone drop is located at the edge of the parking lot and the filter strip to prevent sediments from depositing at this critical entry point. The filter strip is typically an on-line practice, so it must be designed to withstand the full range of storm events without eroding (i.e., up to the peak discharge associated with the 100 year design storm). In snowier climates, the grass portion of the system provides a handy location to stockpile snow where the meltwater can gradually infiltrate into the soil. The maintenance requirements include scraping the sediment buildup at the edge of the parking lot to maintain inflows, and mowing the grass portion of the filter strip.

1.3E SUBMERGED GRAVEL FILTER

A recent design innovation is the submerged gravel filter. It consists of a series of cells that are filled with crushed rock or gravel (Figure 1.14). The standpipe from each cell is set at an elevation that keeps 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 enhanced surface area of the rocks. In particular, the anaerobic conditions on the bottom of the filter can foster the denitrification process (Kadle and Knight, 1996). Although widely used for wastewater treatment in recent years, only a handful of submerged gravel filters have been designed to treat stormwater. In general, the submerged gravel filter has similar design components to the pocket sand filter.

FIGURE 1.14: SUBMERGED GRAVEL FILTER



1.4 A UNIFIED DESIGN APPROACH

The remainder of the manual presents detailed engineering guidance on each of the first four groups of filtering systems. Some unique features of the manual include:

1.4A A UNIFIED DESIGN APPROACH

The underlying concept of the manual is that a common and unified approach is needed to design each type of stormwater filter, so that this useful technology can gain wider engineering acceptance at the local level throughout the Chesapeake Bay.

1.4B SMALL STORM HYDROLOGY AND STORMWATER HOTSPOTS

A key feature of the manual is the presentation of methods to determine the hydrologic response and pollutant loading from small storms for smaller sites (Chapter 2). Small sites are not always the same, and can often be best modeled with new techniques for calculating runoff rates and volumes that reflect small storm hydrology from small, heterogeneous urban sites. Field research has indicated these methods are superior to the conventional NRCS runoff forecasting methods (such as TR-55 and TR-20) on small sites.

1.4C VOLUME-BASED SIZING

The manual presents a single volumetric sizing requirement for each filter which is to capture and treat 90% of the runoff producing rainfall events that occur each year. Many prior design approaches had been rate-based, and resulted in limited and unreliable pollutant removal rates.

1.4D FILTER SELECTION CRITERIA

What is the most appropriate stormwater filter for a particular development site? Are other BMP systems such as ponds, wetlands or infiltration more effective or appropriate? To answer these questions the manual synthesizes recent research and field experience on the pollutant removal performance, longevity, cost, and maintenance burden of each type of stormwater filter. This information has been condensed in a series of tables in Chapter 3 that help designers and municipal officials select the most effective stormwater filter for their development situation, and compare it against the performance and feasibility of other stormwater BMP options.

1.4E REVIEW OF POLLUTANT REMOVAL PATHWAYS

The latest performance monitoring data for stormwater filtering systems is reviewed in Chapter 4 to identify key pollutant removal pathways that can be enhanced in design. Both practical and innovative techniques for enhancing pollutant removal in each group of filter practices are recommended.

1.4F STANDARD DESIGN FEATURES AND DESIGN EXAMPLES

Chapter 5 presents detailed engineering design guidance for sand filters. The design of bioretention systems is presented in Chapter 6. Open channel systems and filter strip design are outlined in Chapter 7. Each design chapter outlines the basic filter sizing criteria, and incorporates standard engineering specifications for flow regulation, pretreatment, filter bed and media, and outflow mechanisms. This standardization should increase the effectiveness of each filtering practice and reduce maintenance problems. In addition, step-by-step design examples are presented for most practices that walk the engineer through the design methods.

CHAPTER 2

RUNOFF AND WATER QUALITY CHARACTERISTICS OF SMALL SITES

In this chapter, we explore the hydrology and pollutant loading dynamics of small sites, where stormwater filtering systems are typically applied. Such understanding about the quality and quantity of runoff generated from small sites is essential to design effective stormwater filtering systems. Small sites are not homogeneous, and their hydrologic and water quality characteristics can be very different. For example, a site may exhibit a sharply different runoff response depending whether impervious areas are directly or indirectly connected to the storm drain system. Other source areas demonstrate different abilities to accumulate pollutants, or are heavily influenced by a unique pollutant loading source (industry, vehicles, pets, fertilizer). In other cases, certain impervious surfaces themselves may actually produce greater pollutant loadings (e.g, leaching of zinc from rooftops). Even pervious areas, such as lawns, can exhibit different hydrologic and water quality responses, depending on the degree of soil compaction or lawn management.

To better understand the hydrology and pollutant loading dynamics of small sites, the chapter is divided into six sections.

The first section explores the "average" concentration of pollutants in stormwater runoff for a range of different urban source areas, based on a synthesis of small site monitoring research. This section also examines specific stormwater "hotspots" that have the potential to generate greater loads of hydrocarbons, metals and priority pollutants, and therefore may warrant greater treatment and/or a higher level of groundwater protection.

The second section presents a simple method to break down small sites into individual source area units that have distinct hydrologic or water quality properties.

The third section investigates the unique hydrology during small storms for urban source areas. Recent research suggests that the hydrologic response of a small site can differ greatly depending on the nature of a source area. A simple adaptation of the NRCS stormwater peak discharge model (TR-55) is proposed to get more accurate runoff predictions from small sites.

The concept of the rainfall frequency spectrum is presented in the fourth and fifth sections. This simple rainfall analysis technique is a powerful tool for determining how much rainfall can be treated in a stormwater filter, and how much must be bypassed.

A unified method for defining and deriving the water quality volume and flow rate is advanced in the last three sections. This standard method is then used as the basis for sizing all stormwater filters considered in this manual.

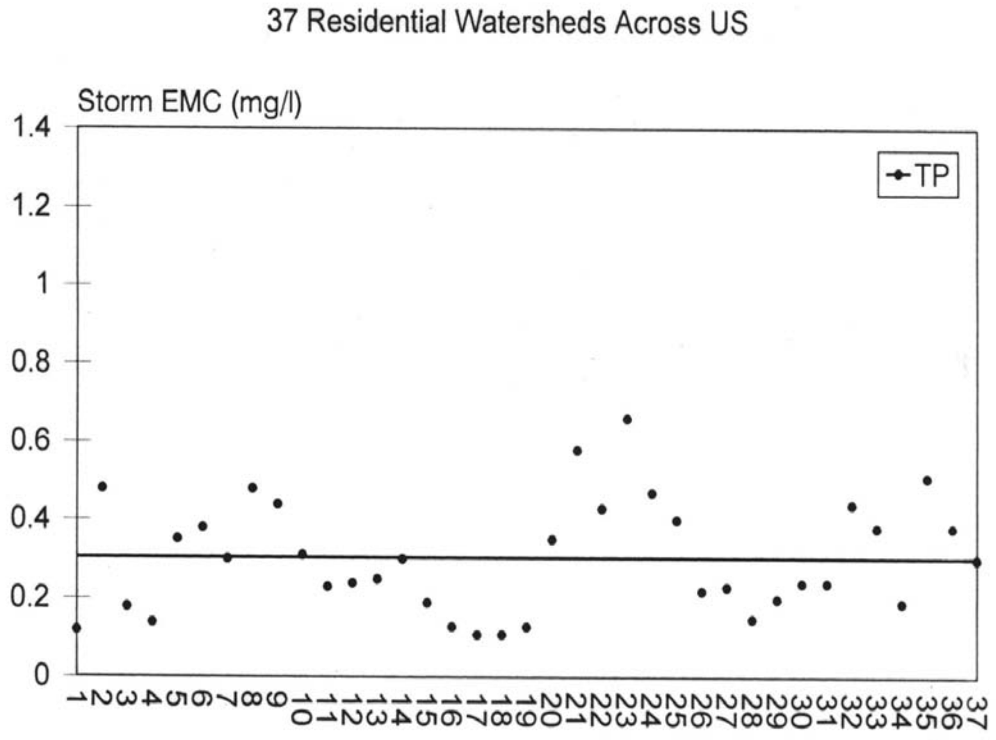
2.1 COMPARISON OF STORMWATER QUALITY FROM DIFFERENT SOURCE AREAS

One of the conclusions of the massive national EPA NURP monitoring study was that while pollutant concentrations were indeed variable at each site, there appeared to be no statistical difference among commercial, industrial and residential land uses at the catchment level (25 to 500 acres). In general, mean pollutant concentrations found in stormwater runoff were surprisingly consistent at the catchment or watershed level (see Table 2.1). One example of this consistency is the mean phosphorus concentration observed in stormwater runoff at 37 different catchments across the U.S. with widely different climate, soils, density and vegetative cover (Figure 2.1). Despite such differences, the average concentration of total phosphorus is about the same no matter where the runoff was sampled.

TABLE 2.1: MEAN POLLUTANT CONCENTRATIONS FOR SELECTIVE PARAMETERS FOR STORMWATER RUNOFF (SOURCE: EPA, 1983)

Pollutant	*New Suburban NURP Sites (Washington, DC)*	*National NURP Study Average*
Phosphorus		
Total	0.26	0.46
Ortho	0.12	-
Nitrogen		
Total	2.00	3.31
Nitrate	0.48	0.96
TKN	1.51	2.35
COD	35.6	90.8
BOD (5-Day)	05.1	11.9
Metals		
Zinc	0.037	0.176
Lead	0.018	0.180
Copper	-	0.047

FIGURE 2.1: TOTAL PHOSPHORUS CONCENTRATION IN STORMWATER RUNOFF



2.1A ATMOSPHERIC DEPOSITION AS THE PRIMARY POLLUTANT SOURCE

The primary explanation for the observed consistency is that the primary source of pollution was more or less the same regionally and nationally—atmospheric deposition. Thus, the basic model is that pollutants are deposited from the atmosphere as either dryfall or wetfall where they accumulate on impervious surfaces. The stored pollutants are subsequently washed off during storm events. Monitoring of the deposition rates for common pollutants found in stormwater runoff does indeed suggest that atmospheric deposition is the dominant source of many pollutants found in stormwater runoff. This is evident in the summary of Washington metropolitan area deposition rates provided in Table 2.2. As can be seen, atmospheric deposition alone can account for most, if not all, of the observed concentrations of total nitrogen, zinc and several other pollutants. While atmospheric deposition provides the base loading for many pollutants, other sources can significantly add to the overall pollutant loading for a site. These urban source areas are described in the next section.

TABLE 2.2: AVERAGE ANNUAL ATMOSPHERIC POLLUTANT DEPOSITION IN THE WASHINGTON METROPOLITAN AREA - LBS/AC/YR (ADAPTED FROM MWCOG, 1983)

Pollutant	*Suburban*	*Urban*	*Dryfall or Wetfall*
Solids	155	243	D>>W
Chemical Oxygen Demand	133	210	W>D
Total Nitrogen	1.8	17	W=D
Organic Nitrogen	7.2	10.2	W>D
Nitrate-Nitrogen	5.6	6.8	W>>D
Total-Phosphorus	0.5	0.8	D>W
Ortho-Phosphorus	0.26	0.35	D>W
Copper	0.21	0.61	W>>D
Cadmium	0.09	0.003	ND
Lead	0.44	0.53	D>>W
Nickel	0.56	0.08	ND
Zinc	1.35	0.65	W>>D

2.1B URBAN SOURCE AREAS

Monitoring of individual urban source areas is a relatively new line of research. Instead of monitoring an urban catchment that has many source areas that contribute to the observed pollutant concentration, researchers focus their monitoring on a single source area, such as a rooftop, parking lot, street, or lawn, to determine the range and concentration of pollutants that it produces. Are pollutant levels higher or lower than the national or regional average? Does the particular source area frequently produce hydrocarbons, metals or toxics that are not commonly found in urban runoff?

To answer these questions, mean pollutant concentrations were computed for 15 individual source areas, based on 20 published research studies conducted from a variety of geographic areas. As with any compilation derived from such diverse data sources, several caveats should be kept in mind in interpreting them. For example, the mean concentrations used to characterize a given source area represent a group mean averaged over one to nine monitoring sites. The sample size for individual studies ranged from two to 200 runoff samples. Monitoring was performed in all regions of the country, and different sampling methods were employed by different researchers. Characteristics of individual source areas (e.g., commercial parking lots) were not frequently reported; this prevents an exact comparison between groups. Literature sources and the group means are provided in Appendix A. Thus, mean stormwater concentrations for each of the 15 source areas should be considered very provisional, and subject to further change and refinement as more data is acquired.

2.1C STORMWATER HOTSPOTS

Stormwater hotspots are defined as a land use or activity that generates higher concentrations of hydrocarbons, trace metals or toxicants than are found in typical stormwater runoff, based on monitoring studies. The increased pollutant loadings from these hotspots can generate concerns about the risk of groundwater contamination and or the toxicity in sediments or the water column of surface waters. If a site is deemed a stormwater hotspot, it has two important implications. First, a higher or more effective form of stormwater treatment is required to remove the elevated concentration of pollutants. Second, treated runoff from a BMP must be prevented from infiltrating into groundwater.

Designers should assess their sites to determine if any potential hotspots are present. It should be kept in mind that not all urban land uses or activities have been fully monitored to characterize the quality of their stormwater. Based on recent monitoring, however, a number of sites do appear to have hotspot characteristics. A preliminary list of potential stormwater hotspots includes the following land uses or activities:

- airport deicing facilities
- auto recycler facilities
- commercial nurseries
- commercial parking lots
- fueling stations
- fleet storage areas (bus, truck)
- industrial rooftops (depending on the nature of the roof surface)
- marinas
- outdoor container storage of liquids
- outdoor loading/unloading facilities
- public works storage areas
- SARA 312 generators (only if materials or containers are exposed to rainfall)
- vehicle service and maintenance areas
- vehicle and equipment washing/stream cleaning facilities

While it would be tempting to classify any industrial facility as a potential stormwater hotspot, in reality, these sites can range from very clean to very dirty, depending on the industrial process, site conditions, and the nature of chemical inputs. The key point is to analyze the site to see if rainfall comes into contact with materials or surfaces, and has the potential for subsequent washoff. Many industrial facilities are required to have a NPDES stormwater discharge permit and pollution prevention plan, under the Clean Water Act. As part of their EPA stormwater NPDES permit, many individual facilities have collected monitoring data to characterize the quality of their stormwater runoff. This data can be very helpful in determining whether an industrial site has the potential to become a stormwater hotspot.

At the same time, there are many land uses and activities that are not normally considered to have the potential to create a hotspot. Runoff quality from these areas is comparatively low with respect to hydrocarbons, metals and pollutants. These include:

- streets and highways
- residential developments
- institutional developments
- office developments
- non-industrial rooftops
- pervious areas

2.2 BREAKING DOWN A SITE INTO SOURCE AREAS

Small sites are not always homogenous in urban and suburban areas. In broad terms, every site can be broken down to those areas that are **pervious** to rainfall and those that are **impervious** to it (and therefore create more runoff). Within these two broad categories, however, there is quite a bit of variation in hydrological response and pollutant dynamics.

2.2A TYPES OF IMPERVIOUS COVER

Impervious areas can be further broken down into five subcategories.

ROOFTOP SOURCE AREAS

Rooftop source areas often tend to produce cleaner runoff given their elevation and pitch. Usually, the only major source of pollutants is atmospheric deposition. In some cases, however, acid rainfall can leach or desorb pollutants contained within the rooftop surface. This effect is best seen when the rooftop source area monitoring data are grouped into *residential*, *commercial* and *industrial* surfaces (Table 2.3). As can be seen, sediment and phosphorus levels are both well below mean national stormwater concentrations. Bacterial levels are also comparatively low, presumably caused only by birds. On the other hand, rooftop runoff often contains higher metal concentrations than many source areas, especially copper and zinc. The source of the metals appear to be from leaching of painted roof surfaces, galvanized gutters and downspouts, or copper flashing. In general, the effect is modest for residential rooftop runoff (about twice the national average), but is much more pronounced in commercial and industrial rooftops.

TABLE 2.3: SOURCE AREA MONITORING SUMMARY - ROOFTOP RUNOFF

Parameter	*Residential*	*Commercial*	*Industrial*
TSS (100 mg/l)	19	9	17
Total P (0.3 mg/l)	0.09	0.20	0.08
f. Coliforms (c/100ml)	2600	1100	5800
Copper (10 ug/l)	20	7	62
Zinc (160 ug/l)	312	256	1390

PARKING LOTS

Parking lots are a fairly diverse source area, depending on their size, vehicle turnover, age and land use. This source area is strongly influenced by emissions and leakage from vehicles, as well as atmospheric deposition. Consequently, parking lot runoff tends to have greater concentrations of trace metals than the national stormwater mean (e.g, cadmium, copper, lead and zinc) as well as oil and grease (Table 2.4). In addition, monitoring of parking lots results in a greater number of detections of priority pollutants. Some variation is seen between parking lots that serve *industrial* versus *commercial* land uses. In many cases, industrial parking lots may also contain stored materials, loading docks, parked equipment or fueling/ service areas that may be pollutant sources.

TABLE 2.4
SOURCE AREA MONITORING SUMMARY - PARKING LOTS

Parameter	Commercial	Industrial
T.S.S. (100 mg/l)	27	228
Cadmium (0.5 ug/l)	8	2
Copper (10 ug/l)	51	34
Lead (18 ug/l)	28	85
Zinc (160 ug/l)	139	224
Oil/Grease (1-2 mg/l)π	8.5	15.0

STREETS AND HIGHWAYS

As might be expected, pollutant concentrations in road runoff are also heavily influenced by vehicles, and tend to increase as traffic volume increases (Bannerman, 1994). *Urban highways* carry the greatest average daily traffic volume and tend to have sediment, organic carbon, nutrient and metal levels that are about twice the national stormwater mean (Table 2.5). Streets exhibit a similar trend, but also appear to be influenced by the land use that they serve. Table 2.6 shows sediment, metal and oil/grease levels for commercial and residential streets. In general, *commercial streets* appear to be a potent source area for many stormwater pollutants, which may reflect traffic volume, poor upkeep of shoulders, sanding or other factors. These include cadmium, copper, zinc and priority pollutant detection. Runoff from *residential streets*, on the other hand, is about two to five times lower for sediment, metals, and hydrocarbons, and is fairly close to the national stormwater mean. Notable exceptions in residential street runoff are very high bacteria and nutrient levels, which are presumably due to pets and blow-in from adjacent pervious areas.

TABLE 2.5: COMPARISON OF MEDIAN EVENT MEAN CONCENTRATION AT HIGHWAY AND NURP RUNOFF SITES (mg/l)
(SOURCE: FEDERAL HIGHWAY ADMINISTRATION, 1990)

	Urban Highway Sites	*NURP Mixed Sites (Come & Res)*	*Ratio of Highway/ NURP Mixed Sites*
TSS	142	67	2.1
COD	114	65	1.8
TKN	1.83	1.29	1.4
PO4-P	0.40	0.26	1.5
Copper	0.054	0.027	2.0
Lead	0.400	0.114	3.5
Zinc	0.329	10.154	2.1
		Average	2.0

TABLE 2.6: SOURCE AREA MONITORING SUMMARY - STREETS AND HIGHWAYS

Parameter	*Residential Street*	*Commercial Street*	*Urban Highway*
TSS (100 mg/l)	172	468	142
Cadmium (0.5 ug/l)	1.0	6.7	1.0
Copper (10 ug/l)	25	73	54
Lead (18 ug/l)	51	170	410
Zinc (160 ug/l)	173	450	329
Oil/Grease (1-2 mg/l)	2.0	3.7	ND

AUTOMOTIVE

Areas where vehicles are fueled, serviced or disposed can represent a significant source area of many stormwater pollutants. Table 2.7 shows reported stormwater concentrations for three automotive areas: *car service or fueling stations*, *auto recyclers* and *auto manufacturing*. In each case, the concentration of cadmium, copper, zinc, oil and grease and priority pollutant detections is among the highest reported for any urban source area. The primary reason is the higher risk that automotive fluids will spill, leak or drip onto impervious surfaces, or that other automotive parts (such as batteries, brake linings, paint, and metal dust) will come into contact with rainwater. Higher stormwater concentrations are often observed when wrecked cars are present, or where cars are scrapped.

TABLE 2.7: SOURCE AREA MONITORING SUMMARY - AUTOMOTIVE

Parameter	*Vehicle Service/Fuel*	*Recycler*	*Industrial*
TSS (100 mg/l)	31	355	124
Cadmium (0.5 ug/l)	9	8.5	ND
Copper (10 ug/l)	88	103	148
Lead (18 ug/l)	80	182	290
Zinc (160 ug/l)	290	520	1600
Oil/Grease (1-2 mg/l)	14	25	ND

RESIDENTIAL (NON-ROOFTOP)

The last major urban source area is residential areas, and includes *lawns* and *driveways*. In general, pollutant concentrations in residential runoff are relatively innocuous, and are usually well below the national stormwater mean for most metals, hydrocarbons and priority pollutants (see Appendix A Stormwater Pollutant Concentrations from Different Source Areas and Hotspots). Runoff from lawn and driveways, however, ranks among the highest concentration of several conventional stormwater pollutants, including sediment, total phosphorus (Table 2.8) and bacteria (Table 2.9). The importance of lawns as a source area is tempered, however by the fact that they generate relatively low runoff volumes, in comparison to impervious surfaces, and therefore produce a smaller annual mass loading of sediment and nutrients than other urban source areas (Bannerman, 1994).

TABLE 2.8: TOTAL PHOSPHORUS SOURCE AREAS IN THE URBAN LANDSCAPE

Source Area	*Concentration (mg/l)*
Rooftops	0.11
Commercial Parking Lot	0.45
Industrial Parking Lot	0.65
Residential Street	0.63
Commercial Street	0.47
Urban Highway	0.40
Lawns	1.67
Driveway	1.16
Snowbelt	0.70
Typical Stormwater	0.30

TABLE 2.9: FECAL COLIFORM BACTERIA SOURCE AREAS

Source Area	*Concentration (counts/100 ml)*
Rooftops	2,400
Parking Lots	2,500
Residential Streets	37,000
Commercial Streets	12,000
Lawns	24,000
Driveways	34,000

2.2B TYPES AND DISTRIBUTION OF PERVIOUS COVER

Pervious areas are a very diverse and complex mosaic of surfaces—forests, wetlands, meadows, lawns, turf, landscaping and the ubiquitous "vacant" lands. While the mix varies based on the history and intensity of past development, pervious cover can be grouped into one of five categories, depending on their vegetative cover and management (Figure 2.2). The estimated distribution of each type of pervious cover in a typical suburban landscape is shown in Figure 2.3.

FIGURE 2.2: TYPES OF PERVIOUS COVER

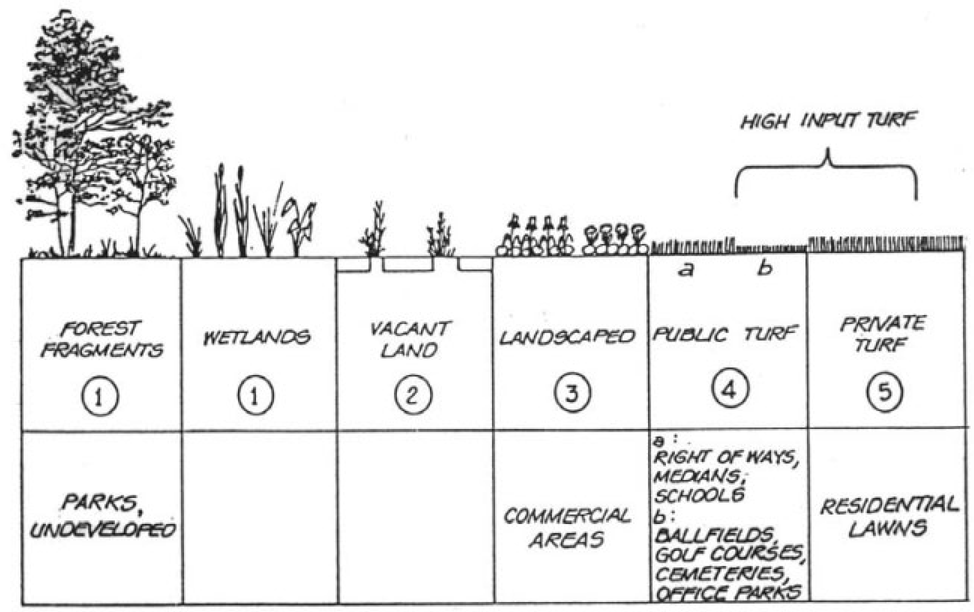
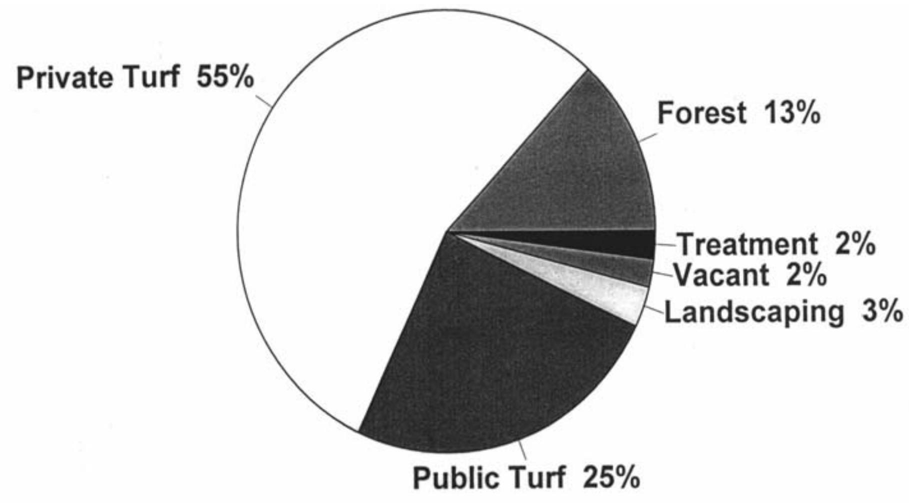


FIGURE 2.3: DISTRIBUTION OF PERVIOUS COVER



URBAN FORESTS AND WETLANDS

The extent of forests and wetlands in the urban landscape varies considerably from one region of the country to another, and even one city to another. The composition and diversity of the forest often changes remarkably due to urbanization, with a strong shift to non-native tree species and invasive shrubs and vines (Adams, 1994). As many as 30 to 60% of native forest species disappear from the highly urban forest community. Much of the forest cover in urban areas is often limited to isolated stands or individual street trees. While these small forest islands are important, they lack the structure, soils, and understory found in natural forests.

PRIVATE TURF (LAWNS)

The best estimate of the extent of home lawns is that they comprise about 70% of the total turf area of the urban landscape (Cockerham and Gibeault, 1985). The lawn category can be further subdivided into *high* and *low* input lawns. *High-input* lawns are defined as those that are regularly fertilized, irrigated and receive applications of herbicides or insecticides. Homeowners apply chemicals to roughly two-thirds of high-input lawns, while the remaining third is treated by lawn care companies. *Low-input* lawns are defined as those lawns that are regularly mowed, but seldom receive any chemical inputs. Surveys indicate that the percentage of high and low-input lawns are about equal in most urban areas.

PUBLIC TURF

About 30% of the remaining turf in urban areas is devoted to "public turf," which include parks, golf courses, schools, churches, cemeteries, median strips, utility corridors and office parks. The greatest share of public turf appears to be contained within parks, golf courses and school grounds. Management of public turf runs the gamut from regular mowing to very intensive turfgrass management (e.g., golf courses).

INTENSIVELY LANDSCAPED AREAS

Commercial areas can comprise up to 20% of the urban landscape. Although commercial areas are highly impervious, many localities require that 5 to 10% of the site be intensively landscaped to provide visual relief, shade and create a more attractive environment. Much of this landscaping is in small fragments that are graded to run onto adjacent impervious areas.

VACANT LANDS

Some portion of urban lands is always in transition from one use to another and remains vacant until that change occurs. In general, these vacant or open lands are temporary in nature and they receive little in the way of vegetative management (although they may be quickly invaded by invasive or pioneer species). Depending on how long the area has been vacant, the cover can range from bare earth, weeds, meadow or shrubs. Erosion can be severe if vegetative cover is poor.

Each of the five types of pervious cover have been highly disturbed by man and lack many of the qualities associated with similar cover types located in natural areas. Perhaps the greatest single change relates to the disturbance of native soils. Development usually involves wholesale grading of the site, removal of topsoil, severe erosion during construction, compaction by heavy equipment, and filling of depressions. In recognition of this disturbance, most soil surveys change the native soil type to the ubiquitous moniker "urban soils" after a site is developed. Urban soils tend to be highly compacted, poor in structure and low in permeability. As a result, these soils often produce more runoff than before they were disturbed. For example, Pitt (1994) noted that one third of the disturbed urban soils he tested in Milwaukee had an infiltration rate of zero or near zero, exhibiting the same runoff response as concrete or asphalt.

2.2C THE EDGE EFFECT: RELATIONSHIP BETWEEN PERVIOUS AND IMPERVIOUS COVER

When seen from the air, most impervious areas are small islands interspersed in a sea of pervious cover, ranging from a few hundred square feet to a few acres in size. The urban landscape is a complex mosaic of pervious and impervious cover that are linked and interlaced together. Since many impervious areas are linear in form (e.g., roads, sidewalks, and parking lots), extensive edges are created between the two types of cover. We tend to think of pervious and impervious areas as distinct and separate. Indeed, most hydrological models simulate the hydrological and water quality response of each area independently. Given the close proximity to each other, the assumption that the two areas do not interact is questionable.

From a hydrological perspective, pervious cover can only be understood in relation to its adjacent impervious cover. More precisely, if the direction of flow is from pervious cover to impervious cover, then the stormwater will occur as *runoff*. On the other hand, if water flows from impervious cover to pervious cover, then the stormwater will occur as *runon*, and is much more likely to infiltrate into the soil. The practical implication is that if a site is graded to produce runon, it may be possible to significantly reduce the volume of stormwater runoff (see section 2.7). Under some conditions, it may be possible to reduce stormwater pollutant loads, as well.

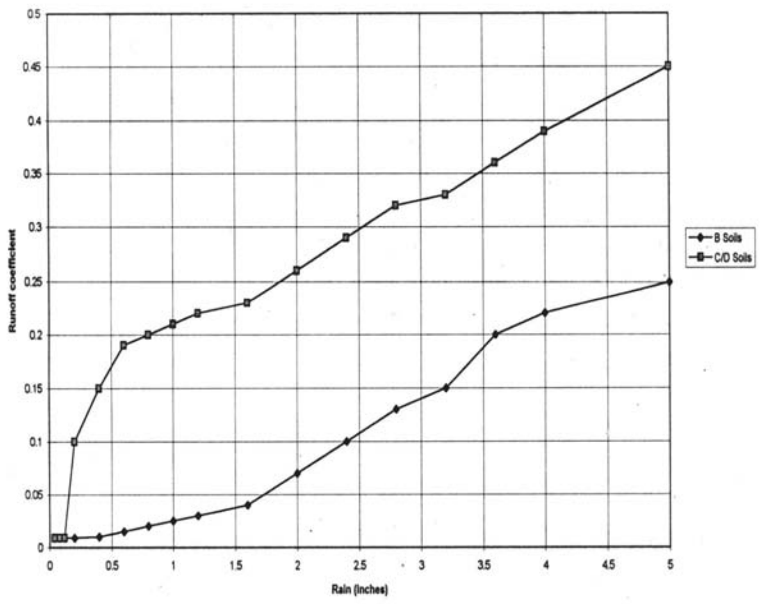
Some examples include directing rooftop runoff to travel through downspouts and over grassed yards, road runoff into swales rather than curb and gutters, and parking lot runoff to drain to forests or fields. The hydrologic effect of disconnecting impervious areas can be very significant, particularly in low-density residential watersheds. In some cases, disconnecting these impervious areas can create enough runon to reduce the "effective" impervious cover in a watershed by 20 to 50% (Sutherland, 1995).

RUNOFF FROM PERVIOUS AREAS

While every effort should be made to maximize runon to pervious areas, drainage considerations often dictate that most pervious areas will still be graded to drain to impervious areas or storm drain collection systems. Consequently, the hydrologic response of each of the five types of pervious cover is of great interest. Most hydrologic research, however, has lumped all the types of pervious cover into a single category, or has assumed that pervious cover has the same properties as well tended turfgrass. Thus, the majority of urban hydrology models utilize the SCS curve number approach, where the runoff rate is dependent primarily on the soil type and to a lesser extent the vegetative cover at a site.

While these models have proven effective for predicting runoff volumes from pervious areas during larger storm events (3 to 5 inches or more), the curve number approach tends to grossly over-predict the runoff volumes produced during the smaller but more common events (Pitt, 1994). The small storm hydrology data presented by Pitt for two test watersheds (Figure 2.4) illustrate the increased runoff properties of urban lawns, presumably due to soil compaction. The volumetric runoff coefficients at these sites tended to progressively increase with rainfall volume, and were in the 0.10 to 0.23 range for soils in the "D" hydrologic soil group for moderate storm events. Lawns that had more permeable soils (in the "B" soil group) produced less runoff volume (Rv's ranging from 0.01 to 0.04 for small to moderate sized storms). Clearly, lawns may produce greater runoff volume then has been traditionally assumed. Even runoff testing of well-tended turfgrass has revealed that turfgrass still produces about half the runoff of bare soil during larger storms (McLean, 1995).

FIGURE 2.4: RUNOFF COEFFICIENT FOR PERVIOUS URBAN AREAS



PARTICLE SIZE DISTRIBUTION

One additional important aspect of stormwater runoff from different source areas is the relationship of particle size to pollutant load. Work done by Sartor and Boyd (1974) and Pitt (1987) starting in the early 1970's suggests that most of the total particulate load from urban runoff is made up by the coarser fractions, consisting of sand/gravel particle sizes greater than approximately 40 microns. Shaver and Baldwin (1991) reported that while nearly 94% of the urban runoff particulate load is from these coarser grained fractions, more than half of the phosphorus load and significant percentages of other pollutants are associated with fine grained silts and clays.

Particle size distribution is an important consideration for sizing the sedimentation chamber of a filter system. Shaver and Baldwin (1991) and Bell et al. (1995) specify that sand filters should only be used to treat runoff from impervious, or nearly-impervious surfaces. They argue that the larger percentage of particulates from impervious surfaces are in the coarser fractions, and therefore, filtering systems will be less prone to clogging. The logic follows that the sedimentation chamber will capture the coarser grained material, and the filter chamber will capture and treat the relatively small amount of finer grained material. Therefore, filters designed to treat runoff from purely impervious surfaces require less sedimentation area and volume than those designed to treat runoff from more pervious surfaces.

The City of Austin (1988) allows the use of sand filters for a range of land uses and drainage areas. They use a smaller, silt size particle (20 microns) as the target for sizing the sedimentation chamber, probably recognizing that more pervious areas are likely to contribute more fine grained particles In order to quantify and resolve the apparent discrepancy between the above criteria, this manual recommends that for drainage areas less than 75% impervious, the target particle size for designing the sedimentation chamber be set at 20 microns. For drainage areas with imperviousness greater than 75%, the target particle size should be set at 40 microns. See Chapter 5 for discussion and application of these sizing principles.

2.3 SMALL STORM HYDROLOGY

Small storms are responsible for most annual urban runoff and likewise are responsible for most pollutant washoff from urban surfaces. Therefore, the small storms are of most concern for water quality resource protection.

Large storms occur infrequently, and although they may contain significant pollutant loads (Chang, G., et al., 1990), their contribution to the annual average pollutant load is really quite small (due to the infrequency of their occurrence). In addition, there are longer periods of recovery available to receiving waters between larger storm events allowing systems to flush themselves and the aquatic environment to recover.

The runoff **volume** is the most important hydrologic variable for water quality protection and design because water quality is a function of the capture and treatment of the mass load of pollutants. The runoff **peak rate** is the most important hydrologic variable for drainage system design and flooding analysis. Water quality facilities are designed to treat a specified quantity or volume of runoff for the full duration of a storm event as opposed to accommodating only an instantaneous peak at the most severe portion of a storm event.

To design effective BMPs and evaluate water quality impacts in urban watersheds, it is necessary to predict the amount of rainfall converted to runoff. The amount of rainfall which is converted to runoff is a function of storm characteristics such as rainfall amount, storm duration, rainfall intensity, and the urban land surface. These surfaces can be broken down into two main categories, pervious and impervious surfaces.

Impervious surfaces are traditionally thought to convert almost all rainfall into runoff, with pervious surfaces contributing much less runoff. In urban areas, particularly for small storms, this is not necessarily the case. Pervious surfaces can be heavily compacted and can have a surprisingly high runoff potential. Impervious surfaces, with minor cracks and expansion joints can have a remarkably high infiltration capability.

Impervious surfaces have five main components which contribute to rainfall losses:

- Interception of rainfall by over-hanging vegetation
- Flash evaporation
- Depression storage
- Sorption by dirt particles
- Infiltration through cracks and seams

The first four processes predominately occur immediately after the start of a rainfall event and dissipate within a relatively short time period and are therefore often referred to as initial abstractions. Infiltration through cracks and seams continues throughout the storm event and depending on the amount of rainfall, can account for significant losses. Many runoff models incorrectly estimate initial abstractions by holding them constant, and few consider infiltration through impervious surfaces for the duration of the storm event (Pitt, 1994).

The amount of runoff generated by pervious surfaces is related to the size of the pervious area, the relationship to impervious surfaces, the permeability of the underlying soils and the condition and type of vegetative cover.

The primary hydrologic methods to estimate storm runoff peak discharges in the Chesapeake Bay Watershed are the Rational Formula and SCS Methods, particularly, TR-55, “Urban Hydrology for Small Watersheds” (USDA, 1986). Several computer models, including SCS, TR-20, “Project Formulation, Hydrology” (USDA, 1982) and the U.S. Army Corps of Engineers’, HEC-1 (U.S. Army, COE 1982) also utilize SCS methods to compute discharge rates. These methods are valuable for estimating peak discharge rates for large storms (i.e., >2") and larger drainage areas (> 10 to 25 acres), but can significantly underestimate the runoff from small storm events.

The limiting factors for the Rational Formula are in the computation of the time of concentration (usually set at a minimum of 5 minutes, which is hard to achieve on many small sites), the selection of "C" values for urban developments which do not address soil infiltration capability, and the equal weight placed on drainage area. The rational method is ideally suited for drainage design where peak rates of runoff are required, but does not estimate storm volume and therefore should not be used for water quality design.

Urban Hydrology For Small Watersheds (TR-55), as the title suggests, is recommended for urban watersheds with small drainage basins. This methodology has been used extensively for stormwater management design for quantity control (i.e., 2, 10, and 100 year management). TR-55 relies on a Curve Number (CN) instead of the "C" to reflect the percentage of rainfall converted to runoff. The TR-55 methodology also has the same limitations associated with computing the time of concentration for extremely small drainage areas.

One of the principal short comings of TR-55 is that the methodology assumes a constant CN for a large range of rainfall events. While this assumption does not significantly affect the accuracy of the model for larger storm events (> 2"), smaller rainfall events produce more runoff than are predicted by the SCS procedure (Pitt, 1994). This chapter presents a method for estimating the volume of runoff and peak discharge from small storms. Standard SCS methods should be used by designers for computing volumes and peak discharges for larger storm events (i.e., 2, 10 and 100 year storms).

Dr. Robert Pitt and his colleagues, have conducted several years of research on small storm hydrology, in several diverse geographic regions, over a wide range of land uses with remarkable consistency between simulated and observed results. The results of Pitt’s research are described in Table 2.10.

TABLE 2.10: PRINCIPLES OF SMALL STORM HYDROLOGY (ADAPTED FROM PITT, 1994)

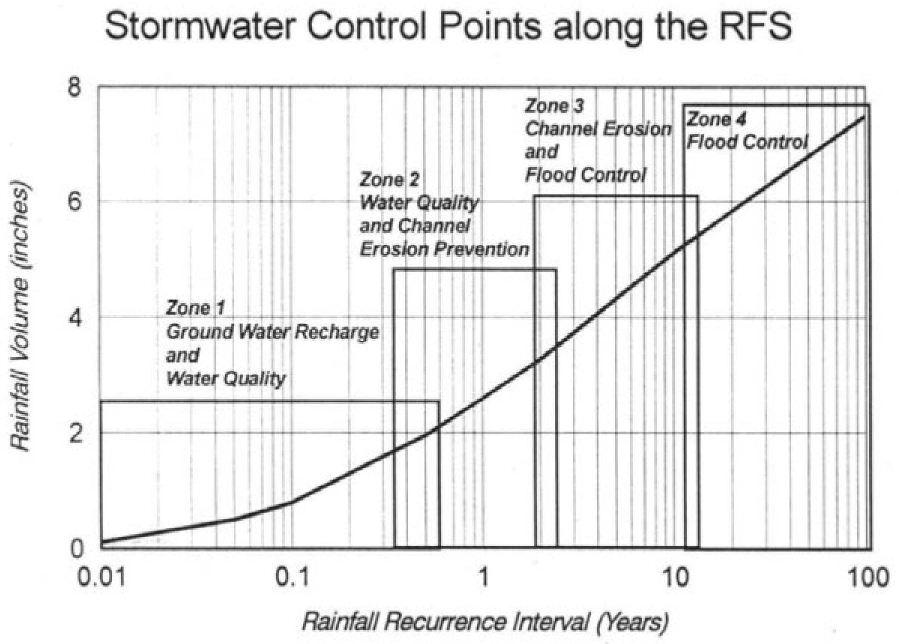
Larger rainfall events correspond reasonably well with SCS CN procedures.
Smaller rainfall events produce more runoff than is predicted by SCS CN procedures.
For strictly pervious surfaces, published CN's are much lower than observed CN's for small storm events. Therefore, less runoff is predicted from pervious areas during small storm events and SCS methodology incorrectly attributes more flow to impervious surfaces. This translates into inaccurate pollutant loading estimates from both pervious and impervious surfaces.
For impervious surfaces, the type of surface (i.e., rooftop, large paved surface, narrow street) has a significant impact on the amount of runoff for small storm events. The infiltration characteristics of these surfaces vary greatly. Remarkably, narrow streets can have a higher infiltration capability than some compacted urban pervious surfaces (such as ballfields).
Disconnecting impervious surfaces can significantly reduce the volume of runoff. The relative amount of reduction is a function of the pervious area flow path, the amount of impervious area draining to pervious areas, and the infiltration capacity of the pervious surfaces. Substantial reductions in runoff are observed for a wide range of land uses when impervious surfaces are disconnected and drained through permeable soils (SCS, Hydrologic Soil Groups (A and B). Reductions are only slight for relatively low density land uses when impervious surfaces are disconnected and drained through relatively impermeable soils (HSG's - C and D). Not surprisingly, disconnecting paved surfaces and rooftops for commercial areas does not result in significant reductions in runoff.

2.4 RAINFALL FREQUENCY SPECTRUM (RFS)

The effectiveness of any stormwater water quality treatment practice is a function of how much stormwater runoff is treated by the system and how much bypasses the practice. Since storms vary dramatically in magnitude, stormwater best management practices must be sized to capture a reasonable percentage of all runoff but bypass excessively large events. The rainfall frequency spectrum or RFS, which is defined as the distribution of all rainfall events, is a useful tool for establishing water quality treatment volume sizing criteria. This distribution is the cumulative volume from all storm events ranging from the smallest most frequent events in any given year to the largest most extreme events over a long duration, say, the 100 year frequency event.

The RFS consists of classes of frequencies often broken down by return interval, such as the two year storm return interval. Four principle classes are typically targeted for control by stormwater management practices. The two smallest, most frequent, classes are often referred to as water quality storms, where the control objectives are groundwater recharge, pollutant load reduction, and to some extent, control of channel erosion producing events. The two larger classes are typically referred to as quantity storms, where the control objectives are channel erosion control, overbank control, and flood control. Figure 2.5 illustrates a theoretical representation of these four classes.

FIGURE 2.5: FOUR CLASSES OF RAINFALL DISTRIBUTION



The distribution and magnitude of the RFS varies from region to region and to some extent, from year to year. Therefore, in order to establish a reasonable water quality treatment design volume for stormwater filtering practices it is necessary to define the RFS for the region of application. Within the Chesapeake Bay Watershed the average precipitation characteristics vary somewhat. This manual presents a sizing criteria based on an in-depth analysis conducted for the Washington, DC metropolitan area, compared with three other locations within the Bay and makes recommendations for establishing the RFS for other locations within the Bay Watershed.

Schueler (1987 and 1992), conducted a detailed evaluation of 50 years of hourly rainfall data in the Washington D.C. area. The recorded precipitation data from Washington National Airport consisted of all storm events separated by at least 3 hours from the next event. The base data collected at National Airport included minor storm events which normally do not produce measurable runoff. These minor events make up approximately 10% of all annual rainfall, are usually less than 0.1 inches, and are therefore excluded from the RFS analysis.

Table 2.11 outlines the RFS for the Washington D.C. metropolitan area and illustrates that the vast majority of all annual runoff is produced from the small frequent storm events.

TABLE 2.11: RAINFALL FREQUENCY SPECTRUM WASHINGTON, DC AREA[a]
SOURCE: DESIGN OF STORWATER WETLAND SYSTEMS (SCHUELER, 1992)

Percent of All Storm Events [b]	*Return Interval*	*Rainfall* [c] *Volume*
30	7 days	0.25
50	14 days	0.40
70	Monthly	0.75
85	Bi-monthly	1.05
90	Quarterly	1.25
95	Semi-annually	1.65
98	Annually	2.40
99	Two-year	2.90

a. 50 year analysis of hourly rainfall record at Washington National Airport, excluding all storms less than 0.10 inches that were separated by three consecutive hours from the next storm. These small storms seldom produce measurable stormwater runoff, yet are numerically the most common rainfall event.

b. Equal to or less than given rainfall volume

c. Watershed inches

2.5 THE 90% RULE-CUMULATIVE RAINFALL VOLUME FOR WATER QUALITY TREATMENT

A careful examination of Table 2.11 suggests that a BMP which is sized to capture and treat the three month storm frequency storm (or 1.25" rainfall) will effectively treat 90% of the annual average rainfall. While this is true, such a practice will also capture and at least partially treat the first 1.25" of larger rainfall events. Therefore treating the 1.25" rainfall will result in a capture efficiency of greater than 90%.

Given the economic considerations of capturing and storing a reasonably large water quality volume, and the realization that stormwater filters tend to lose efficiency as pollutant load input concentrations decrease (Bell, et. al, 1995), a smaller storm event was investigated to evaluate the effectiveness of an alternative treatment criteria. Many jurisdictions require storage of the first one-half inch of runoff from impervious surfaces. While this volume appears to have gained widespread acceptance, there has been little research on the cumulative pollutant load bypassing facilities sized on this principle. One notable exception, is a study conducted in Texas by Chang and his colleagues (1990), where the annual total solids load captured using the half-inch rule showed significant drop-off when imperviousness approached 70%.

To balance the desire to capture and treat as much cumulative rainfall as possible while avoiding an overly burdensome sizing criteria, additional rainfall data was evaluated throughout Chesapeake Bay watershed. In addition to Washington, DC, Three other locations were selected to evaluate longer term rainfall characteristics.

Daily precipitation data was analyzed for an 11 year period (January 1980 through December 1990) at four locations within the Chesapeake Bay Watershed. Norfolk VA, Washington, DC, Frederick MD, and Harrisburg, PA were selected as representative of the bay-wide watershed where new development activity is occurring. In addition locations are separated by 100 to 150 miles and represent a distribution from coastal to inland, and south to north.

The one-inch rainfall was evaluated to assess whether this value could be used to effectively capture 90% of the annual runoff. The average capture percentage using the 1.0" rainfall ranges from approximately 85% to 91% for the four locations. The analysis included the first one-inch of larger rainfall events which will be captured, but probably not completely treated. It is recognized that during these large events treatment conditions may be less than ideal. But it is safe to say that approximately 90% of the annual average rainfall events will be captured and treated using a **one-inch rainfall criteria**.

The results presented in Table 2.12 provide justification for using the 1.0" rainfall event for sizing stormwater filtering practices throughout the Chesapeake Bay Watershed. It must be emphasized that regional rainfall characteristics will differ from specific location to location. Additional rainfall frequency analysis is required for more complete reliance on this value. If a particular jurisdiction has the resources and long term data, a complete RFS should be conducted and the 90% rule applied to establish a local water quality precipitation value. In addition a longer data-set (say 50 years) will make some of the extreme rainfall events or drought periods less statistically significant and may have a minor effect on the capture value derived herein.

TABLE 2.12: COMPARISON OF PRECIPITATION DATA FOR FOUR LOCATIONS WITHIN THE CHESAPEAKE BAY WATERSHED 1980 - 1991 (DAILY ANALYSIS)

	Norfolk, VA	***Washington, DC***	***Harrisburg, PA***	***Frederick, MD***
Annual average precipitation	43.4 inches	37.9 inches	39.6 inches	37.0 inches
Annual average snowfall	7.7 inches	17.2 inches	31.3 inches	Not Obtained
Annual average # of precipitation days *	76 days	67 days	71 days	68 days
Annual average # of precipitation days more than 1.0"	10.5 days	9.5 days	9.5 days	7.7 Days
Annual average # of precipitation days less than 0.1"	39.0 days	45.4 days	55.1 days	Not Obtained
Percent of annual average rainfall # 1.0" *	85.3%	91.4%	86.8%	89.9%
Percent of annual precipitation days # 1.0" *	86.2%	85.9%	86.7%	88.6%

* adjusted to exclude rainfall events # 0.1 (assumed to produce no runoff)

2.6 STORMWATER FILTERING SYSTEMS - SIZING CONSIDERATIONS

In general, stormwater filtering systems should be sized based on the **volume** of runoff to be filtered. All practices identified in this manual utilize the volume based sizing criteria, except for the grass channel practice, where a peak rate is utilized. It is necessary, however, to utilize a peak rate of discharge for sizing off-line flow diversion structures.

As presented earlier in this chapter, the target rainfall event for estimating the Water Quality Volume (WQV) for sizing all filtering devices is based on the **90% Rule** for capturing annual runoff volume. For the Mid-Atlantic region and much of the Chesapeake Bay Watershed, a rainfall value of **1.0 inches** is suggested.

Some jurisdictions may elect to use other sizing guidelines, such as the ½ inch rule (measured in watershed inches). This criteria may be acceptable for lower imperviousness but will have decreased pollutant capture efficiencies for a higher imperviousness and a lower capture percentage of the annual runoff volume. The individual practice sizing principles contained in this manual are applicable for alternative treatment volumes so a reliance on the 90% Rule is not mandatory. In addition, several filtering practices are ideally suited for retrofit applications where full storage is often constrained. Designers and regulators should recognize that the 90% Rule is targeted mainly at new construction and is based on maximizing pollutant load capture. Practices sized for smaller treatment volumes are certainly acceptable in many situations.

2.7 ESTIMATING WATER QUALITY VOLUME (WQV)

Two methods can be utilized to estimate the Water Quality Volume (WQV). Both rely on computing a volumetric runoff coefficient (R_v) and multiplying this by the rainfall volume to obtain a runoff volume in watershed inches.

The first method, or what we call the **Short Cut Method,** utilizes equation 2.1 to estimate the volumetric runoff coefficient R_v, (Schueler, 1987). It is recommended that the Short Cut Method be utilized where the site consists of predominately one type of land surface or for quick calculations to obtain a reasonably accurate estimate of treatment volume.

$$R_v = 0.05 + 0.009(I) \qquad \textbf{Equation 2.1}$$

where I = site percent impervious

Therefore, the required treatment volume for a site will be equal to:

$$WQV = P \; (\; R_v \qquad \textbf{Equation 2.2}$$

P = rainfall, in inches
and WQV = Water Quality Volume, in watershed inches

EXAMPLE CALCULATION

Assume a 3.0 acre shopping center which is 87% impervious, for a 1.0 inch rainfall event.

$R_v = 0.05 + 0.009(87\%)$
$R_v = 0.83$

for P = 1.0 inches
WQV = (1.0")(.83) = .83 watershed inches
WQV = .83"(1/12 "/ft)(3.0 ac)(43,560 ft^2/ac) = 9,039 ft^3

The second method, or **Small Storm Hydrology Method** utilizes the work done by Pitt and others, to compute a volumetric runoff coefficient (R_v) based on the specific characteristics of the pervious and impervious surfaces of the drainage catchment. This method presents a relatively simple relationship between rainfall amount, land surface, and runoff volume. The R_vs used to compute the volume of runoff are identified in Table 2.13. The small storm hydrology model involves the following:

< For a given rainfall depth, the runoff coefficients for land surfaces present on the subject site are selected.

< A weighted runoff coefficient for the entire site is computed.

< If a portion of the site has disconnected impervious surfaces, reduction factors are applied to R_v. The reduction factors (from Table 2.14) are multiplied by the computed R_v for connected impervious areas to obtain the corrected value.

< For the given rainfall, the runoff volume (in watershed inches) is computed. WQV is equal to the rainfall times the R_v (same as equation 2.2 above).

TABLE 2.13: VOLUMETRIC COEFFICIENTS FOR URBAN RUNOFF (DIRECTLY CONNECTED IMPERVIOUS AREAS, ADAPTED FROM PITT, 1994)

Rainfall (inches)	Flat roofs and large unpaved parking lots	Pitched roofs and large impervious areas (large parking lots)	Small impervious areas and narrow streets	Sandy soils HSG-A	Silty soils HSG-B	Clayey soils HSG-C & D
0.75	.82	.97	.66	.02	.11	.20
1.00	.84	.97	.70	.02	.11	.21
1.25	.86	.98	.74	.03	.13	.22
1.50	.88	.99	.77	.05	.15	.24

TABLE 2.14: REDUCTION FACTORS TO VOLUMETRIC RUNOFF COEFFICIENTS FOR DISCONNECTED IMPERVIOUS SURFACES (ADAPTED FROM PITT, 1994)

Rainfall (inches)	Strip commercial and shopping center	Medium to high density residential with paved alleys	Medium to high density residential without alleys	Low density residential
0.75	.99	.27	.21	.20
1.00	.99	.38	.22	.21
1.25	.99	.48	.22	.22
1.50	.99	.59	.24	.24

In order to use the reduction factors for disconnected impervious surfaces, as general guidance, the impervious area above the pervious surface area should be less than one-half of the pervious surface and the flowpath through the pervious area should be at least twice the impervious surface flowpath.

The Small Storm Hydrology method has the advantage of evaluating the precise elements of a particular site and should be utilized for most design applications to estimate accurate runoff volumes. The method requires somewhat more effort to identify the specific land surface area ratios and additional effort is needed to assess the disconnections of impervious areas. The method rewards site designs which utilize disconnections of impervious surfaces by lowering the computed R_v and the required WQV.

EXAMPLE CALCULATION

Assume a 3.0 acre small shopping center having a 1.0 acre flat roof, 1.6 acres of parking and a 0.4 acre open space (sandy soil), for a 1.0 inch rainfall event and no disconnection of impervious surfaces. The weighted volumetric runoff coefficient is:

flat roof: 1.0 acre x .84 = 0.84
parking: 1.6 acres x .97 = 1.55
open space: 0.4 acre x .02 = 0.01

total: 3.0 acres = 2.40

weighted volumetric runoff coefficient R_v = 2.40/3.0 = .80

for P = 1.0 inches
Water Quality Volume (WQV) = (1.0")(.80) = .80 watershed inches
= (.80") ((1 ft/12") ((3.0 ac) ((43,560 ft^2/ac)
= 8,712 ft^3

2.8 ESTIMATING PEAK DISCHARGE FOR THE WATER QUALITY STORM (Q_P)

The peak rate of discharge is needed for the sizing of off-line diversion structures and to design grass channels. As discussed earlier in this chapter, conventional SCS methods underestimate the volume and rate of runoff for rainfall events less than 2". This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the filtering treatment practice due to an inadequately sized diversion structure or leads to the design of undersized grass channels.

The following procedure can be used to estimate peak discharges for small storm events. It relies on the volume of runoff computed using the Small Storm Hydrology Method and utilizes SCS, TR-55 Graphical Peak Discharge Method.

< Using the water quality volume (WQV), computed using the methods

previously presented, a corresponding Curve Number (CN) is computed utilizing equation 2.3.

$$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25\ QP)^{½}]$$ **Equation 2.3**

where P = rainfall, in inches (use 1.0" for the Water Quality Storm)
and Q = runoff volume, in inches (equal to WQV)

Note: Equation 2.3 above, is derived from the SCS Runoff Curve Number method described in detail in NEH-4, Hydrology (SCS 1985) and SCS TR-55 Chapter 2: Estimating Runoff. The CN can also be obtained graphically (also from TR-55).

< Once a CN is computed, the time of concentration (t_c) is computed (based on the methods identified in TR-55, Chapter 3: "Time of concentration and travel time"). The t_c for small sites is often small based on relatively short flow paths; however, a minimum value of 0.1 hours should be used.

< Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_p) for the Water Quality Storm is computed (based on the procedures identified in TR-55, Chapter 4: "Graphical Peak Discharge Method"). For the Chesapeake Bay Watershed use Rainfall distribution type II.

! Read initial abstraction (I_a), compute I_a/P
! Read the unit peak discharge (q_u) from Exhibit 4-II for appropriate t_c
! Using the water quality volume (WQV), compute the peak discharge (Q_p)

$$Q_p = q_u (A (WQV$$ **Equation 2.4**

where Q_p = the peak discharge, in cfs
q_u = the unit peak discharge, in cfs/mi²/inch
A = drainage area, in square miles
and WQV = Water Quality Volume, in watershed inches

EXAMPLE CALCULATION

Using the previous example:

where WQV = .80"
CN = 1000/[10+5 (1.0"+10 (.80"-10((0.80")^2+1.25 (.80" (1.0")$^{1/2}$]
CN = 98

assume t_c = 10 minutes = .17 hours
I_a = 0.041 for CN = 98, I_a/P = 0.041/1.25" = .03
read q_u = 950 csm/in (TR-55 Exhibit 4-II)
A = 3.0 acres/640ac/mi^2 = .0047mi^2
Q_p = 950 csm/in (.0047mi^2 (.80" =3.6 cfs

For computing runoff volume and peak rate for storms larger than the Water Quality Storm (i.e., 2, 10 and 100 year storms), use the published CN's from TR-55 and follow the prescribed procedure in TR-55.

In some cases the Rational Formula may be used to compute peak discharges associated with the Water Quality Storm. The designer must have available reliable intensity, duration, frequency (IDF) tables or curves for the storm and region of interest. This information may not be available for many locations and therefore the TR-55 method described above is recommended.

CHAPTER 3
SELECTING THE RIGHT FILTER FOR A SITE

This chapter presents guidance for selecting the most appropriate stormwater filter for a particular development site. This information has been condensed in a series of tables that help designers and municipal officials select the most effective stormwater filter for their situation. In addition, the chapter compares stormwater filters against other stormwater practices that also could be applied at the site (e.g., ponds, wetlands, and infiltration systems) Again, a series of tables examine the comparative pollutant removal, feasibility criteria and environmental benefits of the four groups of stormwater BMPs.

3.1 SELECTING THE BEST STORMWATER FILTER DESIGN

Given that there are at least eleven different stormwater filter designs that can be used, what is the best option for each site? Quite simply, three factors need to be considered when making the choice. First, is the filter appropriate for the type of development being considered? Second, do site conditions such as space consumption, available head, cost or maintenance consideration favor the use of the design? Third, how effective is the stormwater filter design in removing the key pollutants of concern? Usually, by the time all three questions are answered, the filtering options are narrowed down to one or two design options. The engineer can then compare the design criteria for the remaining options and select one based on cost and effectiveness.

3.1A MOST APPROPRIATE DEVELOPMENT CONDITIONS FOR STORMWATER FILTERS

Although as a group, stormwater filters can be applied to a diverse range of development conditions, individual designs are limited to a much narrower range. These common development situations include ultra-urban sites, parking lots, road and streets, small residential subdivisions and backyard/rooftop drainage. Table 3.1 is a matrix that illustrates the most economical and feasible filtering designs for each of these five broad categories of development, as well as those that are not applicable.

For example, in *ultra-urban or retrofit* settings where space is at a premium, the underground sand filter is often the most ideal filtering design, although surface, perimeter, and pocket sand filters, as well as gravel filters may also be considered. In most cases, the space requirements of grass channels, swales and filter strips are so great that they can be eliminated from consideration.

Three filtering systems are considered ideal to treat the quality of stormwater runoff from *parking lots*- the surface and perimeter sand filter and bioretention areas.

TABLE 3.1: MOST APPROPRIATE FILTER OPTION FOR DIFFERENT LAND USES

Filtering System	*Ultra-Urban or Retrofit*	*Parking Lots*	*Roads and Highways*	*Residential*	*Pervious*	*Rooftops*
Surface Sand	Yes	Ideal	Depends	Depends	No	Yes
Underground Sand	Ideal	Yes	No	Depends	No	Yes
Perimeter Sand	Yes	Ideal	Depends	Depends	No	Yes
Organic	Depends	Yes	No	Depends	Depends	Yes
Pocket Sand	Yes	Yes	Depends	Yes	No	Yes
Grass Channel	No	No	Depends	Depends	Yes	Depends
Dry Swale	No	Depends	Ideal	Ideal	Yes	Depends
Wet Swale	No	Depends	No	No	Depends	Depends
Bioretention	Depends	Ideal	Yes	Yes	Yes	Yes
Filter Strip	No	Depends	Yes	Yes	Ideal	Yes
Gravel Filter	Yes	Yes	No	No	Depends	Depends

Notes:

Ideal:	**Physically and economically the best alternative for a site**
Depends:	**May be suitable under certain conditions (space, soils, water table)**
Yes:	**Generally suitable for most development projects within category**
No:	**Seldom or never suitable**

While several other filtering options are also feasible at parking lots, these three appear to be the most effective and economical designs. (It should be noted that filter strips may be an excellent choice on smaller parking lots adjacent to stream buffers or open space).

The linear nature of *streets and highways* make the grass channel an excellent choice for this kind of development. A grass channel usually fits within the right of way, and is a relatively simple adaption of the drainage channel that is usually provided anyway. If a greater degree of pollutant removal is desired, a dry or wet swale might be contemplated, or an off-line bioretention area. Generally, sand, gravel or organic filter designs are not practical for roads and highways.

In *residential subdivisions*, the preferred designs are the dry swale, bioretention, and the pocket sand filter. When designed properly, each design blends into the landscape and has relatively low maintenance requirements. Sand filters and gravel filters may not be a very practical option in most small subdivisions because of their high maintenance requirements and appearance. Homeowners also have shown little preference for wet swales that are hard to mow and may cause nuisance problems. Once a residential subdivision exceeds a density of 4 dwelling units per acre, few filtering designs of any kind are practical (due to drainage area limitations). Larger systems, such as ponds and wetlands, are probably a better choice.

The filter strip is considered the most ideal option for dealing with *rooftop* runoff and lawns in residential areas, although bioretention is also a practical solution. These two designs can effectively treat stormwater quality at low cost with only minor changes in site grading, assuming land is available. Rooftop runoff from *industrial and commercial* sites may be effectively treated within sand and organic filters, or gravel filters. A higher level of treatment and a concern for groundwater protection make these three options the best choice.

3.1B KEY FEASIBILITY CRITERIA FOR STORMWATER FILTERS

What site conditions make a stormwater filtering design infeasible at a site? While stormwater filters are subject to fewer site constraints than other types of BMP systems, there are a few key factors that should be screened in the selection process including space consumption, minimum head, maintenance burden, cost and soil conditions. These feasibility factors are compared in a matrix format for each of the eleven stormwater filters in Table 3.2.

Space consumption is probably the most critical factor affecting the selection of a stormwater filter design. To compare the practices on the same basis, space consumption is expressed as a practice surface area as a percentage of the contributing impervious acreage. As can be seen, filtering designs run the full range of space consumption with sand filters at the low end of the range (2 to 3%), grass channels and bioretention in the mid-range (about 5%), swales (10 to 20%), and filter strips (100%) at the high end. Thus, if available space is tight, a practice could very well be eliminated.

Most filtering designs require a *minimum vertical distance* (or fall) be available from the inflow to the filter to its outflow point. This distance, known as head, is necessary to drive water through the entire filtering system by gravity. In nearly every filter design, at least two feet of head are needed. Most sand filter designs require about five feet. If a site has very low relief, a filter design may not be practical for the site.

A third key feasibility factor is the *cost* of constructing the filtering system, and again, the eleven designs exhibit a wide range. The most expensive designs, based on the cost per impervious acre treated, are the underground sand, organic sand, perimeter sand and gravel filters. The pocket sand filter and dry swale are in the mid-cost range, whereas bioretention, wet swales, filter strips and grass channels are very attractive options from a cost standpoint. It should be noted that the construction cost does not include the price of land. If land costs are significant, the rank-order changes dramatically.

TABLE 3.2: COMPARISON OF STORMWATER FILTERING SYSTEM OPTIONS

Filtering System	*Space Consumed*	*Minimum Head*	*Maintenance Burden*	*Cost $$*	*Other Notes*
Surface Sand	2-3%	5-8 feet	Annual	Moderat	Large Sediment
Underground Sand Filter	None	5-6 feet	Semi-Annual Cleanouts	High	OSHA Confined Space
Perimeter Sand Filter	2%	2-3 feet	Annual Cleanouts	Moderate	Located Outside Curbstops or in Travelway
Organic Sand Filter	2-3%	5-8 feet	Annual Cleanouts	High	Filter Replacement
Pocket Sand Filter	2%	5 feet	Cleanouts	Moderate	Level Spreader
Grass Channel	6.5%	2 feet	Mowing	Low	Checkdams
Dry Swale	10-20%	2-6 feet	Mowing	Moderate	Prepared Soil
Wet Swale	10-20%	2-6 feet	Wetland	Low	Standing Water
Bioretention	5.0%	4 feet	Landscaped	Low	Plant Selection
Filter Strip	100%	2 feet	Edge Scraping	Low	Contributing Flow Length 75' or 150' max
Gravel Filter	3-5%	2-4 feet	Wetland Cleanouts	High	Standing Water

*Approximate % of total site impervious area draining to practice

Table 3.2 also compares how each filtering design rates with respect to *maintenance burden* and other important feasibility factors.

3.1C COMPARATIVE POLLUTANT REMOVAL CAPABILITY

How effective are the filtering designs at removing the key pollutants of concern in a watershed? As part of the preparation of this manual, some thirty published and unpublished monitoring studies were consulted on the pollutant removal performance of stormwater filtering systems. Estimated average removal rates for each of the eleven stormwater filter designs are indicated in Table 3.3. The matrix also shows the number of actual performance monitoring studies that were available to assess a given design. Three filtering designs (underground sand filters, pocket sand filters and bioretention) have yet to be monitored, and their potential performance is inferred from monitoring of similar designs, infiltration rates, modeling and other analysis provided in Chapter 4.

Despite their many differences in design, stormwater filters have some similarities with respect to performance. For example, all typically report removal rates of *suspended sediment* in excess of 80%. Although monitoring data for *hydrocarbons* is more limited, removal rates typically ranged from 65% to 90%.

Some differences were seen in the comparative ability to remove *total phosphorus.* The best performers were the surface and perimeter sand filter, dry swale and gravel filter, all of which showed at least a 50% removal. Grass channels, wet swales, filter strips and possibly organic sand filters were less reliable, at 10 to 40% average removal.

Stormwater filtering systems exhibit only a modest capacity to remove *total nitrogen.* only one design was found to remove more than 50% of total nitrogen (gravel filter), and most ranged from 30 to 45%. The bulk of the observed removal was for organic forms of nitrogen; eight of eleven filtering designs had zero or even negative removal rates for soluble nitrate-nitrogen. The latter phenomena reflects the fact that while nitrification is prevalent in the mainly aerobic environment of most filter beds, denitrification is limited (leading to buildup of nitrate in the effluent). Only the gravel filter, dry swale, and wet swale showed a capability to remove nitrate.

While all filtering designs showed at least moderate capacity to remove *trace metals* such as copper, lead, and zinc, most of the removed metals were already attached to particles. Designs that showed promise in removing dissolved metals include the organic sand filter, gravel filter and dry swale.

TABLE 3.3: ESTIMATED POLLUTANT REMOVAL CAPABILITY OF DIFFERENT STORMWATER FILTER SYSTEMS (AVERAGES OF REPORTED MONITORING DATA)

Filtering System	*Monitoring Data?*	*TSS*	*TP*	*TN*	*NO^3*	*Other Pollutants/Comments*
Surface Sand Filter	Yes, 6	85%	55%	35%	Neg	Bacteria: 40-80% Metals: 35-90%
Underground Sand Filter	No Data	Presumed to Comparable to Surface Sand Filter				
Perimeter Sand Filter	Yes, 3	80%	65%	45%	Neg	Hydrocarbons: 80%
Organic Sand Filter	Yes, 1	95%	40%	35%	Neg	Hydrocarbons: 90% Sol. P Negatives Metals: 85%+
Pocket Sand Filter	No Data	Presumed to be Comparable to Surface Sand Filter				
Drainage Channel	Yes, 10	30%	10%	Zero	Zero	Bacteria: Negative
Grass Channel = biofilter	Yes, 1	65%	25%	15%	Neg	Hydrocarbons: 65% Metals: 20-50% Bacteria: Negative
Dry Swale	Yes, 3	90%	65%	50%	80%	Metals: 80-90%
Wet Swale	Yes, 2	80%	20%	40%	50%	Metals: 40-70%
Bioretention	No Data	Presumed to be Comparable to Dry Swale				
Filter Strip	Yes, 1	70%	10%	30%	Zero	Metals: 40-50%
Gravel Filter	Yes, 2	80%	80%	65%	75%	Hydrocarbons: 85% Metals: 50-75%

Control of *fecal coliform bacteria* is important in shellfish areas, beaches and drinking water supplies. The filter designs that showed the best ability to remove bacteria included surface sand filters and gravel filters; drainage channels and grass channels had no effect on bacterial levels, and the remaining practices have yet to be monitored for this important parameter.

It should be noted that pollutant removal rates and mechanisms rely on processes in a generally aerobic environment, as opposed to anaerobic environment. Filters which go anaerobic tend to release previously captured phosphorous as iron phosphates break down.

3.1D COMPARATIVE DESIGN CRITERIA

The sizing criteria for each of the eleven filtering designs are summarized in Table 3.4. Each type of filter design is compared based on the sizing criteria for each of its four standard design components:

- the quantity and method used for flow regulation
- the quantity and method used for pretreatment
- the depth and nature of the filter media and the area of the filter bed, expressed as the percentage of contributing impervious area
- the quantity and method used for overflow

TABLE 3.4: COMPARATIVE DESIGN CRITERIA FOR STORMWATER FILTERS

Filter Type	**Design Criteria by System Component**			
	Flow Regulation Quantity and Method	***Pretreatment Method and Quantity***	***Filter Bed and Media***	***Overflow***
Surface Sand	WQV by volume, within facility Entire treatment system must hold at least ¾ of WQV	Dry Sedimentation for 24 hours, approx. ½ of minimum treatment volume (¾ of WQV)	Size based on Darcy's law, coeff. of permeability (k) = 3.5 ft/day, approx. 1% of impervious drainage area. 18"-24" thick sand	Gravel/pipe underdrain system, overflow weir sized to pass **a** of WQV from filter bed.
Underground Sand	WQV by rate, within drainage system. Entire treatment system must hold at least ¾ of WQV	Wet Retention, approx. **b** of min. treatment volume (¾ of WQV)	Size based on Darcy's law, coeff. of permeability (k) = 3.5 ft/day, approx. 1% of impervious drainage area. 18"-24" thick sand	Gravel/pipe underdrain system, overflow weir sized to pass **b** of WQV from filter bed.
Perimeter Sand	WQV by volume, within facility. Entire treatment system must hold at least ¾ of WQV	Wet Retention, approx. **b** of min. treatment volume (¾ of WQV)	Size based on Darcy's law, coeff. of permeability (k) = 3.5 ft/day, approx. 1% of impervious drainage area. 18" thick sand	Gravel/pipe underdrain system, overflow weir sized to pass 100% of WQV from filter bed.
Pocket Sand	WQV by rate, within drainage system. Entire treatment system must hold at least ¾ of WQV	Vegetated strip and plunge pool, no minimum volume	Size based on Darcy's law, coeff. of permeability (k) = 3.5 ft/day, approx. 1% of impervious drainage area. 18"-24" thick sand	Gravel/pipe underdrain system, overflow weir sized to pass 100% of WQV from filter bed.
Organic Media	WQV by volume, within facility.* Entire treatment system must hold at least ¾ of WQV	Dry Sedimentation for 24 hours, approx. ½ of minimum treatment volume (¾ of WQV)*	Size based on Darcy's law, coeff. of permeability (k) = 4.3 ft/day (peat), 8.7 ft/day (compost), less than 1% of impervious drainage area. 24" thick peat/sand profile consisting of 12" thick peat, 4" thick peat/sand mix, and 8" sand. or 18" thick compost	Gravel/pipe underdrain system, overflow weir sized to pass **a** of WQV from filter bed.*
Bioretention	On-line, up to WQV, then overflow	Pea gravel diaphragm and 10' min. vegetated filter strip, no min. volume.	Size based on Darcy's law, coeff. of permeability (k) = 0.5 ft/day, approx. 5.0 % of impervious drainage area. 4' thick planting soil bed, and overlying mulch layer.	Gravel/pipe underdrain system, overflow drainage catchbasin sized to pass design peak discharge (e.g., Q_{10})

Filter Type	**Design Criteria by System Component**			
	Flow Regulation Quantity and Method	***Pretreatment Method and Quantity***	***Filter Bed and Media***	***Overflow***
Grass Channel	On-line, rate based on Q_p from WQV, velocity #1.5 fps	Pea gravel diaphragm and vegetated filter strip, forebay at inflow, no min. volume.	Rate based design, minimum residence time = 10 min. Depending on slope, treatment area approx. = 6.5% of impervious drainage area. Grass surface/soil interface	On-line flow, sized to treat WQV with velocity # 1.5 fps, 2 year non-erosive velocities (# 4.0 to 5.0 fps), adequate capacity for 10 year storm with 6" freeboard.
Dry Swale	On-line, volume based on WQV	Pea gravel diaphragm and vegetated filter strip, forebay at inflow, no min. volume.	Volume based design to retain WQV. Depending on slope and depth, treatment area approx. = 16% of impervious drainage area. 30" thick planting soil bed, consisting of 50% soil/50% sand mix.	On-line flow, sized to treat WQV, 2 year non-erosive velocities (# 4.0 to 5.0 fps), adequate capacity for 10 year storm with 6" freeboard.
Wet Swale	On-line, volume based on WQV	Pea gravel diaphragm and vegetated filter strip, forebay at inflow, no min. volume.	Volume based design to retain WQV. Depending on slope and depth, treatment area approx. = 16% of impervious drainage area. Grass/wetland vegetation surface/soil interface	On-line flow, sized to treat WQV, 2 year non-erosive velocities (# 4.0 to 5.0 fps), adequate capacity for 10 year storm with 6" freeboard.
Filter Strip	On-line volume based on WQV	Pea gravel diaphragm, no min. volume	Volume based design to retain WQV. Depending on slope and depth, treatment area approx. = 100% of impervious drainage area. Grass surface/soil interface	On-line flow, sized to treat WQV, all other flows overflow berm
Gravel Wetland Biofilter	WQV by volume, within facility	Wet Retention, approx. **b** of min. treatment volume (¾ of WQV)	24" recycled concrete or bank-run gravel	

* assumes same design variation as surface sand filter

3.2 COMPARISON TO OTHER BMPS

Stormwater filtering systems are just one of four groups of stormwater best management practices that can be used to treat the quality of stormwater runoff. The other three groups include:

- stormwater ponds
- stormwater wetlands, and
- stormwater infiltration systems.

Since these practices can also be applied at development sites, it is helpful to compare their advantages and disadvantages with respect to stormwater filtering systems. Again, a series of matrices are used to make generalized comparisons based on physical feasibility, pollutant and environmental restrictions.

3.2A COMPARATIVE FEASIBILITY

A quick glance at Table 3.5 suggests that stormwater filtering systems are subject to far fewer physical restrictions than the other three groups of practices. For example, stormwater filters are feasible option on smaller drainage areas, have no soil restrictions, and have relatively modest head and space requirements. Ponds and wetlands, by contrast, typically require larger drainage areas (ten acres or more) and infiltration practices are often severely restricted by soil conditions.

On the other hand, stormwater filters are generally not a cost-effective option beyond a five acre drainage area, and need frequent cleanouts to maintain the performance of the filter bed (1 to 3 years). In addition, both filtering and infiltration systems are not commonly used to provide stormwater quantity controls for the larger design storms (2, 10 and 100 year events). These must be treated in a downstream detention or retention facility.

TABLE 3.5: FEASIBILITY CRITERIA FOR DIFFERENT STORMWATER BMP OPTIONS

Feasibility Criteria	*Pond Systems*	*Wetland Systems*	*Infiltration Systems*	*Filter Systems*
Soils	Most Soils	Most Soils	Need Infiltration Rate .5"/Yr or	All Soils
Drainage Area	10 Acre Min.	10 Acre Min.	2-5 Acre Max.	2-5 Acre Recommended
Head	3-6 Feet	1-6 Feet	2-4 Feet	1-8 Feet
Space	2-3% of Site	3-5% of Site	2-3% of Site	2-7% of Site
Cost/Acre	Low	Moderate	High	Moderate-High
Water Table	No Restrictions	No Restrictions	4 Feet Below	2 Feet Below Filter Bottom
Cleanout	2-10 Years	2-5 Years	1-2 Years	1-3 Years
SWM Mgmt.	Yes	Yes	No	No
Longevity	20-50 Years	20-50 Years	1-5 Years	5-20 Years?

3.2B COMPARATIVE POLLUTANT REMOVAL

The matrix in Table 3.6 compares the performance of stormwater filtering systems as a group with ponds, wetlands and infiltration. It is worth noting that this comparison is necessarily general, and several designs within each group may have better or worse performance than indicated in Table 3.6. (For more discussion, consult Section 4.4b in Chapter 4). A few of the general trends are outlined below.

All four practices generally display an excellent ability to remove *suspended sediment,* with filtering systems performing slightly better than the other three. Although filtering systems can perform as well as the other groups with respect to total phosphorus removal, they do show greater variability. Filtering systems generally have only a fair ability to remove nitrogen. Of particular concern, many filtering systems show a low ability to remove soluble nutrients that are of greatest concern in the Chesapeake Bay. Their tendency to export or leach nitrate and soluble phosphorus are well documented in Chapter 4, and few stormwater filters have an effective method for biological uptake by algae or wetland plants.

On the positive side, several filtering systems are documented to have a high to excellent ability to remove bacteria, metals, and hydrocarbons, which is equal or greater to that of ponds, wetlands and infiltration (although there are some data gaps).

In summary, the overall pollutant removal capability of stormwater filtering systems is on par with that for the other three systems, with higher removal for some pollutants, and lower removal for others.

TABLE 3.6:COMPARATIVE POLLUTANT REMOVAL CAPABILITY OF FOUR BMP OPTIONS

Pollutant	*Pond Systems*	*Wetland Systems*	*Infiltration System*	*Filter Systems*
Sediment	Excellent	Excellent	Excellent	Excellent
Phosphorus	High	High	Excellent	Fair-High
Nitrogen	Fair	Fair	High	Fair
Soluble Nutrients	High	Fair	High	Low
Bacteria	Low-High	?	?	Low-Fair
Hydrocarbons	High	High	?	Excellent
Trace Metals	Fair-	Fair-Excellent	High	Fair-Excellent

Key: **Lo = 0-25% removal** **Fair = 26-50%**
High = 51-75% **Excellent = 76% or higher**

3.2C ENVIRONMENTAL RESTRICTIONS AND BENEFITS

A key factor in the selection and permitting of urban BMPs are environmental restrictions and benefits. Table 3.7 compares stormwater filters with ponds, wetlands and infiltration systems in regard to ten common environmental restrictions. The matrix suggests that while there is little environmental risk associated with most stormwater filtering systems, they also confer few if any community benefits. For example, stormwater filters as a group generally pose little risk of stream warming, groundwater contamination, wetland impairment and safety hazards. Each of the other practices is subject to one or more of these environmental constraints, which may make it difficult to obtain approval or permits in some subwatersheds. On the other hand, with few exceptions, most stormwater filter designs confer few of the environmental benefits that some of the other practices can provide, such as streambank protection, habitat creation and groundwater recharge. In addition, most stormwater filters are neutral with respect to community benefits (such as flood control, landscaping or increase in property value).

TABLE 3.7: ENVIRONMENTAL BENEFITS AND DRAWBACKS OF BMP OPTIONS

Selection Factor	*Pond Systems*	*Wetland Systems*	*Infiltration System*	*Filter Systems*
Groundwater Quality	Low Risk	Low Risk	Moderate Risk	No Risk
Groundwater Recharge	Moderate Benefit	Low Benefit	High Benefit	No Benefit (a)
Temperature	High Risk	High Risk	No Risk	Low Risk
Wetlands	High Risk	Moderate Risk	No Risk	Low Risk
Safety	High Risk	Low Risk	No Risk	No Risk
Habitat	Moderate Benefit	High Benefit	No Benefit	No Benefit
Flood Control	High Benefit	High Benefit	No Benefit (b)	No Benefit (b)
Streambank Protection	Moderate Benefit	Moderate Benefit	Low Benefit	Low Benefit (b)
Property Value	High Premium	Moderate Premium	No Premium	Unknown
Landscaping	High Benefit	High Benefit	No Benefit	Low Benefit

Notes: (a) Assumes filtering system has underdrain system
(b) Most do not control channel stability design storm events

3.3 SUMMARY

Stormwater filters are most applicable at small development sites, and can generally provide reliable rates of pollutant removal if design improvements are made and regular maintenance is performed. Stormwater filters appear to have particular utility in treating runoff from urban "hotspot" source areas such as commercial parking lots, vehicle service centers, and industrial sites, as well as problematic street and highway sites when other BMPs are not feasible. They are also an appealing alternative to other practices in environmentally-restricted watersheds. Lastly, it should be kept in mind that stormwater filters only treat the quality of runoff, and that a downstream detention or retention facility will be needed for stormwater management and flood control purposes.

CHAPTER 4
POLLUTANT REMOVAL MECHANISMS

This chapter explores the pollutant removal capability of five kinds of stormwater filtering systems, based on an analysis of published performance monitoring studies, engineering theory and basic research conducted around the country. The five groups of stormwater filtering systems that are considered include:

- sand and organic filters
- open channels
- filters strips
- bioretention
- vegetated submerged bed wetlands

The chapter describes the primary pollutant removal pathways associated with each group of practices, reviews reported pollutant removal performance monitoring principles and compares these rates to other stormwater BMP technologies. The discussion culminates in a series of key design principles to enhance pollutant removal performance that are incorporated into the design criteria in succeeding chapters.

4.1 POLLUTANT REMOVAL PATHWAYS

This section reviews the major pollutant removal pathways that occur within stormwater filtering systems. An understanding of these pathways is useful to interpret performance monitoring data, and to design more effective filtering practices. Each filtering practice utilizes a different combination of pollutant removal pathways which are compared in Table 4.1. The six primary removal pathways include:

4.1A PATHWAY NO. 1: SEDIMENTATION

Most urban BMPs rely heavily on gravitational settling as a primary pollutant removal pathway, and filtering systems are no exception. There are upper limits, however, to the amount of pollutant removal that can be achieved in this pathway. This is evident in the data of Stanley (1994) and Grizzard et al. (1983). In Stanley's recent study of an extended detention pond in North Carolina, modest to high removal of particulate pollutants that are prone to settling was observed, although removal of soluble pollutants was negligible (Table 4.2). Grizzard's settling column experiments demonstrate a similar behavior (Figure 4.1), with most removal occurring in the first six to twelve hours.

Most filtering systems incorporate a sedimentation chamber to settle out pollutants before runoff reaches the filter bed. The importance of settling as a pollutant removal pathway increases as more time and volume is provided for settling.

TABLE 4.1
COMPARISON OF POLLUTANT REMOVAL PATHWAYS IN STORMWATER FILTERS

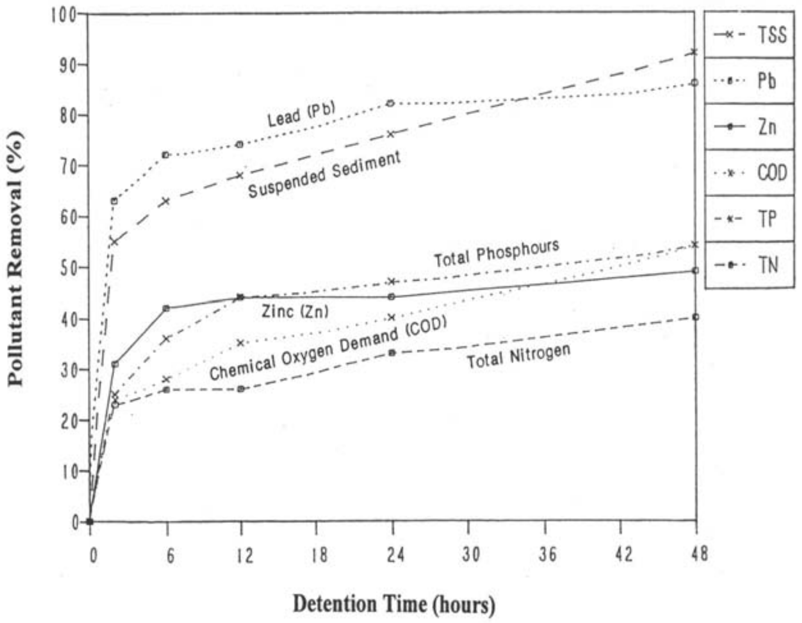
Pollutant Removal	***Sand Filters***	***Organic Filters***	***Open Channel (a)***	***Filter Strip***	***Bio-retention***	***Gravel Filter***
Sedimentation	M, in pretreatment cell		M, surface cells	M, at grass strip interface	M, surface ponding	M, in pre-treatment cell
Straining	M, in filter media		L, surface soil	L, surface soil	H, sand, mulch, grass, soil	L, large pore spaces
Adsorption	by organics on filter surface	peat or sand	soil, thatch	soil, thatch	soil, mulch	biofilms on rocks
Microbe Action	On filter surface		L, soil	L, soil	M, soil/mulch	biofilms on rocks
Plant Uptake	None, unless grass cover crop used		L, by grass mowing	M, by grass or forest	M, by trees/ shrubs	H, by epilithic algae
Infiltration	None, unless designed as open system		H, based on soil Fc	M, based on length	M, if designed as open system	None
TDS Leaching?	Yes	Yes	Yes	No	?	No
Nitrification/ Denitrification?	Nitrification: Yes Denitrification: No		N: Yes D: No	N: Yes D: ?	N: Yes D: ?	N: Yes D: Yes
Key: L - Low M - Moderate H - High						

TABLE 4.2: THE LIMITS OF SEDIMENTATION: PERFORMANCE OF A DRY EXTENDED DETENTION POND IN NORTH CAROLINA (SOURCE: STANLEY, 1994)

Parameter	*All Storms*	*Big Storms**
Total Suspended Solids	71%	25%
Particulate Organic Carbon	45%	19%
Dissolved Organic Carbon	(-6%)	(-5%)
Cadmium	54%	12%
Copper	26%	11%
Lead	55%	19%
Zinc	26%	11%
Ammonia (NH4-N)	9%	20%
Nitrate-N	(-2%)	6%
Particulate Nitrogen	43%	22%
Total Nitrogen	26%	--
Dissolved Phosphorus	(-9%)	6%
Particulate Phosphorus	33%	17%
Total Phosphorus	14%	--

* Removal rate includes pollutants that bypassed the pond through the emergency spillway and were not subject to settling.

FIGURE 4.1: REMOVAL RATE VS. DETENTION TIME FOR SELECTED POLLUTANTS (SOURCE: GRIZZARD, 1983)



4.1B PATHWAY NO. 2: FILTRATION

Many particulate pollutants are physically strained out as they pass through the filter bed of sand, soil or organic matter, and are trapped on the surface or among the pores of the filter media. The effect of filtration can be very strong. For example, Pitt et al. (1995) report that as much as 90% of small particles commonly found in urban runoff (6 to 41 microns) are trapped by an 18 inch layer of sand, and presumably an even greater percentage of larger particles. As might be expected, the filtration pathway is <u>not</u> effective in removing soluble pollutants and the smallest particles upon which pollutants are often attached. In addition, the importance of the filtration pathway is a function of the media used in the filter. Some of the common chemical properties of several filter media are presented in Table 4.3. In relatively tight media, such as soil or sand, filtration is very important, whereas, in more porous media such as compost or peat, the filtration effect is comparatively weak.

TABLE 4.3
COMPARATIVE PROPERTIES OF DIFFERENT FILTERING MEDIA
(ADAPTED FROM GALLI, 1990; STEWART, 1992 AND PITT et al., 1995)

	Sand	*Silt Loam*	*Compost*	*Peat*
Hydraulic Conductivity (cm/hr)	3.3	0.1 - 0.4		.025 - 140
Water Holding Capacity (cm/cm)	0.14	0.07 - 0.1		.01 - 0.20
Bulk Density (gms/cm)	2.65	1.25 -	1 - 2	<0.1 - 0.3
pH	--	5.7	7.8	3.6 - 6.0
Organic Matter (%)	<1	<20	30 - 70	80 - 98
Cation Exchange Capacity	1 - 3	12 - 18	66	183 - 265
Total-Phosphorus (%)	0.0	0.09	<0.1	<0.1
Total Nitrogen (%)	0.0	0.15	<1.0	<2.5
Filtration Efficiency after 18" (%)	93	94	16	47

4.1C PATHWAY NO. 3: ADSORPTION

The ability of a filtering system to remove soluble nutrients, metals, and organic pollutants is often due to the adsorption pathway, in which ions and other molecules attach to binding sites on filter media particles. In general, the adsorption potential of a filtering system increases when the filtering media has a high content of organic matter or clay, a high cation exchange capacity (CEC) and a neutral to alkaline pH. Once again, each of the media used for filtering systems exhibit sharply different adsorption potentials. Pure sand, for example, initially has little or no organic matter, clay or cation exchange capacity, and therefore, little potential for adsorption (Table 4.3). Over time, however, most sand filters develop a thin layer of organic matter and fine particles at the surface layer of the filter media as a result of sediment deposition thereby increasing the adsorption potential. Organic filter media such as soil, peat and compost, on the other hand, have a much greater potential for adsorption, if the pH of the media is in the optional range.

4.1D PATHWAY NO. 4: INFILTRATION

The bottom of many filtration systems is impermeable so that the filtered runoff can be collected in perforated pipes and returned to the channel. This may be desirable if the contributing site is a hotspot that could contaminate groundwater. If the site is not a hotspot, and underlying soils have a reasonable infiltration rate, however, it is possible to utilize infiltration as a major removal pathway. Both runoff and entrained pollutants can migrate downward into the soil layer (which can provide additional filtering or adsorption of pollutants), and may eventually reach the water table. Runoff infiltration is a major removal pathway for several filtering systems, such as dry swales, filter strips and some bioretention designs. In many cases, the total mass removal of these systems is proportional to the mass of pollutants that are infiltrated into the soil.

4.1E PATHWAY NO. 5: MICROBIAL ACTION

Filter media is inevitably colonized by microbes that break down organic pollutants, and transform nutrients. Optimal growth of microbes is achieved when organic matter is plentiful, temperatures are warm and the filter media moist. "Biofilms" develop around filter particles, providing an ideal surface area for microbial growth. Microbial action is a significant pollutant removal pathway in most filtering systems, and two microbial processes in particular, are very important in explaining their nitrogen dynamics—nitrification and denitrification.

NITRIFICATION

Nitrification is an important nitrogen removal pathway as organic matter is gradually decomposed. Microbes break down organic nitrogen into ammonia, which is then transformed into soluble nitrate-nitrogen. The nitrification process generally requires an anaerobic (oxygen-rich) environment which is characteristic of many filtering systems. As a result, nitrification occurs rapidly in many filtering systems, resulting in the export of low concentrations of ammonia.

DENITRIFICATION

The final step in the nitrogen cycle is the conversion of soluble nitrate into nitrogen gas that is returned to the atmosphere. To proceed, the denitrification process requires a moist, anaerobic environment (zero-oxygen), an abundant supply of both organic carbon and nitrate, and the presence of denitrifying bacteria. These conditions are not always met in most filtering systems. Consequently, most filtering systems actually export more soluble nitrate than they receive. In recent years, designers have attempted to create suitable conditions for denitrification within filtering systems, and have demonstrated a capability to remove nitrate.

4.1F PATHWAY NO. 6: PLANT RESISTANCE AND UPTAKE

Several filtering systems incorporate plants, such as algae, emergent wetlands or grass to improve removal rates. Examples included vegetated open channels (grass), sand or organic filters (that have a grass cover crop), bioretention, filter strips, and gravel wetland filters (algae, wetland plants). Plants can increase pollutant removal in several ways. During periods of stormflow, for example grass and emergent wetland plants provide resistance to flow, thereby reducing runoff velocities. Slower runoff velocities translate into more time for other pollutant pathways to work (such as settling, filtering, infiltration and adsorption). In addition, the roots of grass and emergent plants help bind up the filter media, preventing loss of sediments and attached pollutants via erosion.

The growing plants also create a continual supply of thatch or detritus, which provide the organic matter needed for greater adsorption. During periods of growth, the plants also take up nutrients and metals from the filter bed and incorporate it into their biomass. If plant biomass is harvested or mowed, pollutants are removed. Taken together, however, the use of plants in a filtering system is usually of secondary importance as a pollutant removal pathway in comparison to the other five pathways.

4.2 PERFORMANCE MONITORING STUDIES

In this section, nearly forty performance monitoring studies of stormwater filtering systems are reviewed, in order to extract general principles with regard to pollutant removal that can be used in design. As with any broad review of monitoring studies, it is important to keep in mind some important caveats with respect to their interpretation. The monitoring studies were not carefully controlled replicates. Instead, they encompassed a wide geographic and climatic range, reflect considerable differences in basic designs, utilized different methods to compute pollutant removal, and exhibited wide differences in inflow and outflow capture, storm bypass, and number of storms sampled. Even with these many differences among the studies, however, several important generalizations can be made with respect to pollutant removal performance of filtering systems.

4.2A POLLUTANT REMOVAL PERFORMANCE OF SAND FILTERS

Presently, performance monitoring data for sand filters consists of nine studies conducted in Austin TX, Seattle WA, Orlando FL, and Alexandria VA. In addition, one compost filter has been extensively monitored (Table 4.4).

The initial monitoring results suggest that sand filters are very effective in removing particulate pollutants, such as total suspended solids, lead, zinc, organic carbon and organic nitrogen, but exhibit rather mediocre removal of soluble pollutants. Removal rates for coliform bacteria, ammonia, ortho phosphorus and copper were moderate, and quite variable—ranging from 20 to 75% in the ten sand filters tested.

SUSPENDED SEDIMENT

Sand filters uniformly demonstrated an excellent ability to remove suspended sediment. Mean TSS removal rates of 75 to 90% were reported at most sites. The high sediment removal is expected, given the effectiveness of filtration and settling in removing particulates. It is not entirely clear what proportion of the removal occurs in the filter bed (filtration) or in the pretreatment chamber (settling), but the two pathways appear complimentary. Pitt et al. (1995), however, reported in settling column studies indicated that some very fine sediment particles (10 microns or less) may not be captured in the sand filter, and pass through.

ORGANIC CARBON

Sand filters were found to be effective in removing various forms of organic carbon (BOD, COD and TOC) with mass removal in most sand filters ranging from 45 to 65%. Settling and filtration again were the dominant pollutant removal pathways for organic carbon. It should be noted, however, that some particulate organic carbon deposited or trapped on the filter bed decomposes, and may be actually exported from the filter as dissolved organic carbon.

NUTRIENTS

Most sand filters showed a moderate capability to remove total nitrogen, with an average removal rate of about 35%. Of the nitrogen forms that comprise total nitrogen, the greatest removal is noted for TKN (organic nitrogen). A sizeable fraction of TKN is in particulate form, making it susceptible to settling and filtration. Removal of soluble nitrate-nitrogen is usually negative, indicating that while nitrification is occurring in the filter media, denitrification is not.

The nine sand filters consistently exhibited a moderate to high potential to remove total phosphorus, with six filters exceeding 60%. Not surprisingly, much of the removal rate can be ascribed to the settling and filtration of the particulate fraction of phosphorus (which is often about 50 or 60% of total phosphorus). Removal data on the more biologically available forms of phosphorus (ortho or soluble reactive) are more limited. Bell et al. (1995) reports high levels of removal (Table 4.5).

TABLE 4.5: POLLUTANT REMOVAL OF THREE DELAWARE SAND FILTERS
(SOURCE: BELL et al. 1995, HORNER AND HORNER 1995)

Parameter	Alexandria, VA Bell et al., 1995 Mass Removed (a)	Seattle, WA Horner and Horner, 1995 Mean Removal (b)	Seattle, WA Horner and Horner, 1995 Mean Removal (b)
Number of Storms	20	14	6
Total Suspended	79%	83%	8% (c)
Oil and Grease	NA	84%	69%
Petroleum	ND	84%	55%
Total Organic Carbon	66%	NA	NA
BOD (five-day)	78%	NA	NA
Total Phosphorus	63% (d)	41%	20%
Ortho-Phosphorus	68% (d)	NA	NA
Total Nitrogen	47%	NA	NA
Nitrate+Nitrite	(-53.3%)	NA	NA
TKN	70.6%	NA	NA
Zinc	91%	33%	69%
Copper	25% (b)	22%	31%

Notes:
(a) fraction of total incoming pollutant load retained in filter over all storms
(b) average of storm pollutant concentration reduction, all storms
(c) poor removal due to very low TSS inflow concentrations (4 - 24 mg/l)
(d) removal rates were higher if four anaerobic events are excluded
NA parameter not analyzed during monitoring study
ND parameter not detected in runoff during sampling study

TRACE METALS

Sand filters were usually capable of removing trace metals, such as lead and zinc with an average rate of about 70%. High removal is generally expected for these metals, since they are often attached to particles that easily settle or filter out. Sand filters showed less ability to remove metals predominantly found in soluble form. For example, Bell et al. (1995), Horner and Horner (1995) and Austin ERM generally reported copper removal on the order of 20 to 35%. The fact that any soluble metals were removed was surprising given that pure sand has virtually no adsorptive capacity.

BACTERIA

Five sand filters were analyzed to determine their ability to remove fecal coliform bacteria. Bacteria removal ranged from 37 to 83%. The relatively modest bacteria removal noted in the five Texas filters was surprising, given that sand filters have been used extensively to treat drinking water, and typically remove 90 to 95% of incoming bacteria (Ellis, 1987).

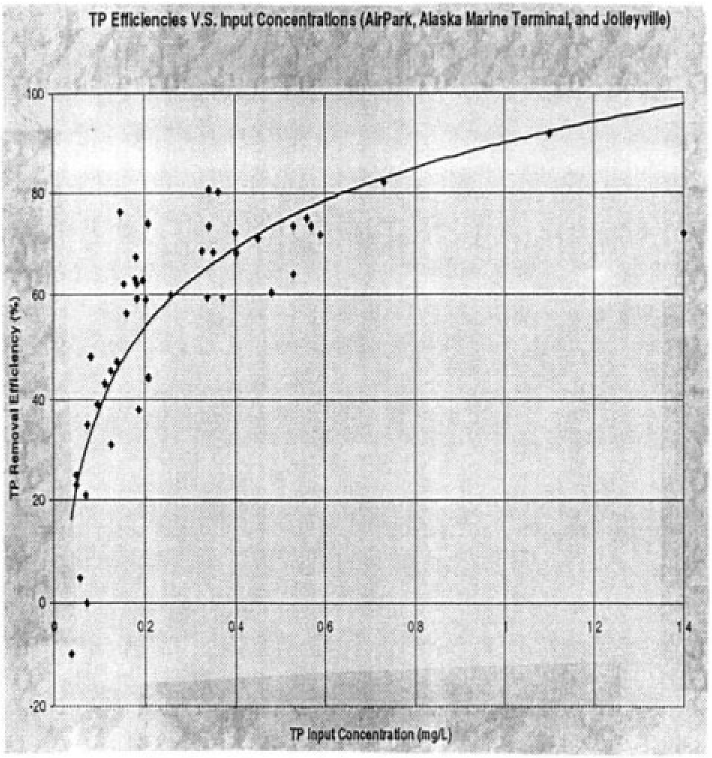
PETROLEUM HYDROCARBONS

Horner and Horner (1995) examined the ability of two sand filters to remove petroleum hydrocarbons and oil and grease at a Seattle marine terminal, and reported removal rates that ranged from 55 to 84%. He also detected a strong dose response relationship. When incoming hydrocarbon concentrations were low, for example, removal rates were relatively low. However, when incoming hydrocarbon concentrations surpassed 3 mg/l, removal rates consistently exceeded 90%. This suggests that sand filters may be an effective practice for controlling hydrocarbons at stormwater pollution hotspots.

DOSE-RESPONSE RELATIONSHIP

Bell conducted a detailed analysis of the relationship between inflow concentration and pollutant removal on sand filters that were monitored in Alexandria, Seattle and Texas. He detected a strong relationship between inflow concentration and removal efficiency for sediment, phosphorus, organic nitrogen, zinc, and total petroleum hydrocarbons. Simply put, removal efficiency sharply increased when the concentration of particulate pollutants entering the sand filter was high, and dropped when incoming pollutant concentrations were low (and of less water quality significance). The dose-response relationship for total phosphorus is depicted in Figure 4.2.

FIGURE 4.2: DOSE-RESPONSE RELATIONSHIP FOR TOTAL PHOSPHORUS IN SAND FILTERS (SOURCE: BELL et al. 1995)



IRREDUCIBLE CONCENTRATIONS FROM SAND FILTERS

After analyzing the effluent quality of many monitored BMPs, Schueler (1996) has shown that there is an apparent minimum pollutant concentration that is always discharged from sand filters, as well as other BMPs. Pollutant levels cannot be reduced below this rather low level using existing technology. The irreducible concentration computed for six to ten sand filter systems that reported outflow concentrations are provided in Table 4.6. The approximate limits to sand filter treatment for some common urban pollutants are shown below.

< TSS 20 mg/l
< TN 2.0 mg/l
< TP 0.15 mg/l

The irreducible concentration may represent leaching or flushing of pollutants that had been trapped on the filter surface over time. It should be noted that the irreducible concentration values for sand filters are surprisingly comparable to those derived at pond and wetland systems (Schueler, 1996).

TABLE 4.6: IRREDUCIBLE CONCENTRATIONS OF SAND AND ORGANIC FILTERS (SOURCE: SCHUELER, 1996)

Parameter	*N*	*Concentration (mg/l)*
Total Suspended Solids	10	19.3 ± 10.1
Total Phosphorus	10	0.14 ± 0.13
Ortho-Phosphorus	ND	--
Total Nitrogen	6	1.93 ± 1.02
Total Kjeldahl Nitrogen	6	0.90 ± 0.52
Nitrate-Nitrogen	6	1.13 ± 0.55
Data Sources: Horner and Horner (1995), City of Austin (1990), Bell et al. (1995), CSF (1994)		

NITRIFICATION EFFECT

Nitrate export was observed in five out of seven sand filters monitored for parameter. This behavior suggests that nitrification is taking place within the filter bed. During the nitrification process, microbial bacteria converts ammonia-nitrogen into the nitrate form of nitrogen. The apparent loss of ammonia through the filter bed, coupled with the production of excess nitrate, strongly suggests that nitrification is taking place. Nitrate export has also been observed in other stormwater filtering systems that do not rely on sand (i.e., compost and grass channels). Apparently, sand filter conditions do not allow for significant denitrification to occur (that converts nitrate into nitrogen gas). Bell has speculated that denitrification could be more pronounced if the bottom of the sand filter is allowed to become anaerobic (i.e., designing lengthy periods of water saturation in the bottom layer) and presents some evidence that denitrification did occur at several microsites in his Alexandria test filter.

LEACHING EFFECT

Negative removal rates were frequently reported for total dissolved solids (TDS) and nitrate-nitrogen, and occasionally, for soluble phosphorus and metals. The negative TDS rate may be due to the preferential leaching of cations from organic matter trapped on the surface of sand filter. The leaching effect was observed regardless of whether the filter medium was sand or compost. Huang and Petrovic (1994) noted that a layer of zeolites could retain nitrate and other cations from leaching through the sand layer below a golf course green.

ALTERNATIVE MEDIA

Limited monitoring data are presently available to assess whether organic media are more effective than sand in trapping pollutants. Pitt et al. (1995) noted that experimental sand columns did not always securely retain small sediment particles that contained toxicants and metals, but often flushed them through the filtering column, and has investigated whether peat, compost, activated carbon or soil would increase retention of smaller particles. He notes that organic media in combination with sand have considerable potential to increase removal of a sand filter.

At this time, the pollutant removal performance of only one alternative media filter system has been monitored in the field (CSF, 1994). The compost filter system provided excellent removal of sediment, particulate nutrients, organic carbon, hydrocarbons and some trace metals (Table 4.7). Total dissolved solids, however, increases, which appears to reflect the exchange and/or leaching of cations within the compost. Similarly, while particulate nutrient forms are trapped within the compost, the system exports soluble forms of nutrients, such as nitrate and soluble phosphorus. The organic matter in the compost has a high cation exchange capacity and therefore have a greater potential to adsorb soluble metals and organics.

TABLE 4.7: POLLUTANT REMOVAL PERFORMANCE OF A COMPOST FILTER (SOURCE: CSF, 1994)

Parameter	*% Removed*
Total Suspended Solids	95%
Total Dissolved Solids	(-37%)
COD	67%
Total Phosphorus	41%
Soluble Phosphorus	(negative)
Organic Nitrogen	56%
Nitrate	(-34%)
Cadmium	ND
Lead	ND
Zinc	88%
Hydrocarbons	87%
Copper	67%
Boron, Calcium,	(negative)

COMPARISON WITH WASTEWATER SAND FILTER PERFORMANCE

The pollutant removal behavior of stormwater sand filters is generally comparable to that reported for sand filters used in wastewater treatment (Ellis, 1987). Wastewater sand filters typically contain finer sand, are cleaned more frequently, and are subject to more uniform and controlled flow than their stormwater counterparts. Consequently, wastewater filters exhibit slightly higher removal rates for sediment, phosphorus and organic carbon (often in excess of 90%), but seldom can achieve more than 20% removal of nitrate (again, due to the lack of denitrification). They do show greater capability to remove fecal coliform bacteria.

4.2B OPEN VEGETATED CHANNELS

Few best management practices exhibit such a great variability in pollutant removal performance as open grass channels. Sixteen historical performance monitoring studies of "grass swales" were re-analyzed based on the open channel classification presented earlier to try to explain this variability. Ten of the open channels could be classified as "drainage channels" based on two criteria—they were designed only to be non-erosive for the two year storm, and their particular combination of soil and slope did not allow significant infiltration of runoff into the soil profile. Site data and pollutant removal data for these drainage channels are shown in Table 4.8. The poor performance of drainage channels is due to the fact that they do not act as an effective filter (i.e., very little runoff actually filters through the soil media). Since the soil filter is not used, drainage channels can only rely on sedimentation and adsorption pathways for removal. During most storms, runoff passes through the channel in just a few minutes, thereby greatly reducing the effectiveness of those removal pathways.

One open channel was explicitly designed as a grassed channel (Seattle METRO , 1992). The 200 foot long grass channel, termed a biofilter, was found to be reasonably effective in removing many pollutants contained in urban stormwater. The performance monitoring data for the biofilter is summarized in Table 4.9. In general, high rates of removal were reported for sediment, hydrocarbons, and particulate trace metals. Nutrient removal was much more mixed.

Five open channels were either explicitly designed as a dry or wet swale, or had a combination of soils, slope and water table so that they effectively functioned like one (Table 4.10). Given the small number of open channels that met these criteria, they were lumped together as a single group. The swales demonstrated a much greater and more consistent capability to remove pollutants conveyed in urban stormwater. In nearly every case, most of the mass removal could be accounted for by the infiltration or retention of runoff into the soil profile during storms (i.e., actual pollutant concentration did not change appreciably as they passed through the channel). As a group, the swales showed excellent removal of suspended sediment, nitrogen, organic carbon and trace metals.

Table 4.8
Pollutant Removal Performance of Ten Drainage Channels

No	Reference	State	YR	N	M	S	L	A	SOIL	TSS	OC	TP	SP	TN	NO3	Cu	Pb	Zn	Other
1	OWML	VA	83	33	M	1.8	260	9.5	SL	NEG	NEG	NEG	-	NEG	-	-	NEG	NEG	-
2	OWML	MD	83	50	M	4.1	445	19.0	SL	NEG	NEG	NEG	-	NEG	-	-	NEG	NEG	-
3	OWML	MD	83	8	M	5.1	425	12.0	SL	31	NEG	NEG	-	37	-	-	33	NEG	-
4	DORMAN	VA	89	9	M	4.7	185	1.3	SL	65	76	41	-	-	11	28	48	49	TKN=17
5	DORMAN	MD	89	4	M	3.2	193	1.3	SL	NEG	23	12	-	-	NEG	14	55	9	TKN=9
6	YU	VA	89	4	M	5A	200	1.5	-	68	-	60	-	-	-	-	-	74	-
7	YOUSEF	FL	85	6	C	1.0	550	-	Sa	-	-	8	26	13	11	14	27	29	TKN (-20)
8	OAKLAND	NH	83	11	C	>2%	100	-	-	33	-	NEG	NEG	-	-	48	57	50	COLI=NSD
9	WELBORN	TX	87	19	C	-	200	2.9	-	NSD	NEG	NEG	NEG	NSG	NEG	NSD	NSD	NSD	COLI=NSD
10	PITT	ONT	86	50	C	-		-	-	NSD	-	-	-	NSD	-	NSD	NSD	NSD	COLI=NSD

Notes, N= number of samples, M=mass or concentration method, S=slope, L=length, A=contributing area (acres), SOIL (SL=silt loam, Sa=sandy), COLI-fecal coliforms, NEG=negative removal efficiency reported, NSD=no statistically different concentration between control (usually pipe flow)

TABLE 4.9: POLLUTANT REMOVAL PERFORMANCE OF A GRASS CHANNEL (BIOFILTER) OF TWO LENGTHS IN WASHINGTON (SOURCE: SEATTLE METRO , 1992)

Pollutant	*100 Foot Biofilter*	*200 Foot Biofilter*
Suspended Sediment	60%	83%
TPH (Hydrocarbons)	49%	75%
Total Zinc	16%	63%
Dissolved Zinc	negative	30%
Total Lead	15%	67%
Total Copper	2%	46%
Total Phosphorus	45%	29%
Bioavailable P	72%	40%
Nitrate-N	negative	negative
Bacteria	negative	negative

TSS

Only four out of nine drainage channels had a positive removal rate for suspended sediment, suggesting that neither settling, filtration or infiltration occurred to any great degree as it passed through the channels. By contrast, sediment removal rates for dry swales, wet swales and the grass channel all exceeded 80%.

ORGANIC CARBON

Drainage channels showed little ability to remove organic carbon, with four of six tested showing negative removal rates. Both dry swales and wet swales on the other hand, had carbon removal rates in excess of 50%. While no data was available for grass channels it would appear reasonable that settling and filtration pathways would be effective for this primarily particulate pollutant.

Table 4.10
Pollutant Removal Performance of Six Water Quality Swales

No.	Reference	State	YR	N	M	S	L	A	SOIL	TSS	OC	TP	SP	TN	NO3	Cu	Pb	Zn	Other
1	DORMAN	FL	89	8	M	1.0	185	0.6	Sa	98	64	18	-	-	45	65	81	81	TKN=48
2	HARPER	FL	88	16	M	1.0	210	0.8	Sa	87	69	83	-	84	80	89	90	90	-
3	HARPER	FL	88	11	M	1.8	210	1.2	WET	81	48	17	-	40	52	56	50	69	-
4	KERCHER	FL	83	13	M	>2.0	-	14.0	Sa	99	99	99	-	99	99	-	99	99	
5	METRO	WA	92	6	C	4.0	200	16.0	till	83	-	29	72	-	NEG	46	67	73	HC=75 COLI=NEG
6	WANG	WA	81	8	M	-	200	-	-	80	-	-	-	-	-	70	80	60	-

Notes. N=number of samples, M=mass or concentration method, S=slope, L=length, A=contributing area (acres), SOIL (SL=silt loan, Sa=sandy), HC=hydrocarbons, COLI=fecal coliforms, NEG=negative removal efficiency reported, NSD=no statistically different concentration between control (usually pipe flow)

NUTRIENTS

Drainage channels provided negligible removal of nutrients. In most sites, nitrogen and phosphorus removal was either consistently low or non-existent. Nutrient removal in the grassed channel, in contrast, was somewhat higher, with about 30% of total phosphorus and 70% of soluble phosphorus effectively removed (Seattle METRO, 1992). The grass channel was also a net exporter of nitrate.

Dry and wet swales showed better ability to remove nitrogen, with the mass removal rates ranging from 40 to 99%. Phosphorus removal was more variable, with the two swales experiencing the most infiltration recording phosphorus removal rates greater than 80%, and three reporting with minor infiltration capability showing removal rates of 30% or less. Phosphorus removal may be limited in any open channel system. Monitoring has shown that open channels have high phosphorus levels stored in the thatch and surface soil layer. Some of the stored phosphorus may recycle back into the water column, or be eroded during larger storms. In addition, the high phosphorus levels in channel soils may be too high to allow meaningful adsorption.

TRACE METALS

While some drainage channels did exhibit a moderate ability to remove trace metals attached to particles (i.e, lead and zinc), an equal number showed no metal removal capability whatsoever. By contrast, trace metal removal rates for grass channels, dry swales and wet swales were uniformly high. It should be noted that most metal removal is due to settling and filtering of metals attached to particles. Removal of soluble metals, however, was only 20 to 50% (Yousef et al., 1985).

Most monitoring studies only report removal of total trace metals, and do not independently measure the fraction of metals found in soluble form. This can be significant as soluble metals usually exert the greatest impact or toxicity to aquatic life. Many trace metals are primarily found in soluble forms (cadmium, copper and zinc), while others are mostly attached to sediment particles (iron and lead). Yousef et al. (1985) found that swales were not very effective at adsorbing soluble metal species. Adsorption requires that a metal be present in runoff as a positively charged cation that can be adsorbed to a negatively charged particle in the soil or organic layer. Metals, however, can be found in a complex number of ion species depending on the prevailing acidity (pH) of runoff. Some metals such as zinc readily adsorb to soil at pH levels typical of stormwater runoff 6.5 to 8.0, but many others (aluminum, cadmium, copper, chromium and lead) show little tendency to adsorb to soils within this pH range. Consequently, the ability of swale soils to remove many soluble trace metals tends to be rather low.

BACTERIA

The three studies that examined the ability of drainage channels to remove fecal coliform bacteria found no significant change in the counts of this key human health indicator after channel treatment. Oakland (1983), Welborn and Veenhuis (1987), Pitt and McLean (1986) all reported that drainage channels had no effect in reducing bacterial concentrations as they traversed through the swale. Seattle METRO (1992) also reported that a grass channel actually tended to increase the level of fecal coliform bacteria as runoff passed through it. This increase was thought to be due pet droppings and possible bacterial multiplication within the biofilter itself.

PETROLEUM HYDROCARBONS

The only study that examined hydrocarbon removal in grass channels found they were very effective at removing both hydrocarbons and oil and grease (Seattle METRO, 1992).

CHLORIDES

Open channels appear to have no capability to trap soluble chlorides (Harper, 1988, Demers and Sage, 1990).

METAL AND NUTRIENT ACCUMULATION IN SOILS

A number of researchers have found that both metals and nutrients tend to be higher in surface soils of open channels than adjacent upland soils. (Wiggington et al. 1983, Dorman et al. 1989, Harper 1988, WCC 1994, Lind and Karro 1995). A summary of the average concentration of metals and nutrients in twelve open channel systems in the U.S. can be found in Table 4.11. The higher levels appear to suggest that swales are accumulating metals and nutrients. One interpretation from the data might be that open channels are trapping and retaining these pollutants, but it can also be argued that swales are simply a better depositional environment. Since swales are a depression in the landscape, they represent an excellent depositional site for aerosols and dust generated by vehicles on adjacent roads, and this factor may well explain the higher levels.

Another interesting aspect of Table 4.11 is the surprising consistency in phosphorus, organic nitrogen, copper and zinc levels in surface soils among the many geographically diverse sites. The only pollutant that exhibits great variability is lead. The lead variability may be due to the declining rates of lead deposition in recent years associated with the gradual introduction of unleaded gasoline, and localized differences in airborne lead deposition due to traffic factors.

According to Lind and Karro (1995), soil type is very important factor for metal accumulation in open channels. Those that have a high content of clay or organic matter in surface soils are able to adsorb metals better.

TABLE 4.11: SEDIMENT POLLUTANT LEVELS IN TWELVE GRASS DRAINAGE CHANNELS (ALL VALUES IN MG/KG)

Reference	*State*	*TP*	*TKN*	*NO_3*	*Copper*	*Lead*	*Zinc*
Dorman	VA	1057	947	0.5	39	100	106
Dorman	MD	1135	1794	10	32	419	251
Dorman	FL	1112	1900	12	11	143	144
Wiggington	VA	-	-	-	4	42	101
Wiggington	MD	-	-	-	10	17	70
Wiggington	VA	-	-	-	23	936	106
Harper	FL	748	1524	-	75	1378	680
Harper	FL	571	1971	-	22	325	157
WCC	CA	-	-	-	36	262	225
WCC	CA	-	-	-	37	43	142
WCC	CA	-	-	-	43	82	179
WCC	CA	-	-	-	20	11	85
Mean		**924**	**1627**	**8**	**29**	**313**	**187**

CULVERT LEACHING

Wiggington et al. (1986) discovered that galvanized metal culverts that are often used for driveway crossings in residential swale systems can be a source of some trace metals. Under some conditions, the metal coating of these pipes leach trace metals, particularly when runoff is slightly acidic. The leaching effect was most pronounced for zinc, but was also observed for copper and cadmium.

INFILTRATION

One of the key benefits of dry swales is their ability to reduce the volume of runoff through soil infiltration. Pitt and McLean (1986) noted that while pollutant concentrations did not change through open channels in metropolitan Toronto, they did produce 25% less annual runoff. This effect was particularly evident for storms smaller than a half inch. Anderson (1982) and Yu et al. (1992) also observed that swales seldom produced measurable runoff during storms, although adjacent curb and gutter systems did. The importance of the infiltration pathways in dry swales is evident in the work of Yousef et al. (1985). As can be seen in Table 4.12, the total mass removal through the test channel was roughly proportional to the mass of runoff that fully infiltrates through the bottom of the channel. Again, pollutant concentrations in runoff that did not infiltrate through the channel bottom did not change appreciably in Yousef's study.

TABLE 4.12: NUTRIENT REMOVAL IN SIX EXPERIMENTAL SWALES IN FLORIDA AS A FUNCTION OF RUNOFF VOLUME INFILTRATED (SOURCE: YOUSEF ET AL., 1985)

Site Number	*Infiltration Volume*	*Nitrate-N*	*Organic-N*	*Total N*	*Diss. P*	*Total P*
M-6	26%	(-2%)	22%	27%	26%	31%
E-4	38%	48%	41%	39%	43%	45%
E-5	50%	(-21%)	41%	24%	40%	27%
M-1	57%	57%	64%	61%	62%	63%
M-2	60%	67%	63%	73%	79%	79%
M-3	100%	100%	100%	100%	100%	100%

SOLUBLE NUTRIENTS

The channels experiments conducted by Yousef et al. (1985) indicated that most swales showed little capability to adsorb or filter soluble forms of nitrogen and phosphorus as they passed through the swale. Little or no reduction in soluble nutrient concentration was observed. The bulk of the mass nutrient removal in the channel could be accounted for by simple infiltration of runoff through the bottom of the swale. Indeed, a cursory glance at Table 4.12 shows that total removal rates and the fraction of total runoff infiltrated into the swale bottom were essentially identical. This implies that the major pollutant removal pathway in dry swales is an underground one (infiltration) and not necessarily a surface one (settling, filtering or adsorption).

IRREDUCIBLE CONCENTRATIONS

Only a small number of drainage channels and dry swales reported outflow data from which the irreducible concentration could be computed (Table 4.13). The provisional values for the limits of open channel treatment for some common pollutants are provided below. Please note that these values have a considerable standard deviation.

<	TSS	40	mg/l
<	TP	0.30	mg/l
<	TN	1.75	mg/l

With the exception of total nitrogen, open channels appear to have a higher "irreducible concentration" for sediment, total phosphorus and soluble phosphorus than other BMP systems (ponds, wetlands, and sand filters).

TABLE 4.13: ESTIMATED IRREDUCIBLE CONCENTRATION OF OPEN CHANNEL SYSTEMS (SOURCE: SCHUELER, 1996)

Parameter	*N*	*Concentration*
Total Suspended Solids	5	43.4 ± 47.0
Total Phosphorus	5	0.33 ± 0.15
Ortho-Phosphorus	3	0.16
Total Nitrogen	5	1.74 ± 0.71
Total Kjeldahl Nitrogen	5	1.19 ± 0.41
Nitrate-Nitrogen	5	0.55 ± 0.29
Data Sources: Harper (1988) and Dorman et al. (1989)		

LENGTH/CONTACT TIME EFFECT

Dorman et al. (1989) concluded that channel length alone was not a reliable predictor of the removal efficiency in drainage channels. Although the ten drainage channels ranged in length from 100 to 550 feet, there was no relationship between length and removal efficiency. A quick calculation illustrates why channel length, by itself, is not useful parameter. Given a typical stormflow velocity of 1.5 feet per second, it takes just over a minute to travel 100 feet of a channel (which allows very little time to utilize adsorption, settling or infiltration pathways).

The grass channel design alters the geometry of the channel to decrease the speed of runoff. Seattle METRO (1992) reported that a 10 minute residence time in a grass channel is needed to attain reliable pollutant removal for most storms. Their monitoring indicated that a 200 foot grass channel did perform better than a 100 foot grass channel.

SOIL TYPE

Soil type is an important design factor for three reasons. First, the soil type governs the rate of infiltration that can occur. A sandy soil, for example, often allows for substantial infiltration of runoff, whereas a clay soil does not. Consequently, many dry swales utilize natural or prepared "sandy" soils to infiltrate significant runoff volumes. Second, soil type is influential in determining the rate of adsorption. Soils with a high clay or silt content and soils with a high organic matter content have a higher adsorption potential than sandier ones (See Table 4.2). Third, the underlying soil type often determines the density and vigor of grass cover in the swale. Extremely clayey or sandy soils often make it difficult to establish the vigorous grass cover needed to provide flow resistance and prevent channel erosion.

4.2C VEGETATED FILTER STRIP

Our current knowledge about the pollutant removal capability of urban vegetated filter strips is confined to a single study. Yu et al. (1993) analyzed a grass filter strips to treat urban stormwater runoff from a large parking lot (Table 4.14). Yu reported moderate to high removal rates for a 150 foot grass strip, and mediocre pollutant removal performance in a shorter, 75 foot strip.

Most of the research on the pollutant removal capability of filter strips has been conducted in agricultural areas. Desbonette et al. (1994) has conducted an excellent review of nearly 35 different agricultural monitoring studies. The buffer studies can be grouped into two categories: those that utilize grass filter strips to treat sediment and nutrient laden surface runoff from row crops, and those to employ forested strips that remove nutrients in subsurface flows from crop and pastureland.

Moderate removal rates were consistently reported for sediment, nitrogen and phosphorus for filters that treat surface runoff (Desbonette et al., 1994). Typically, a fifteen foot wide grass buffer can achieve a 50% reduction in all three pollutants in surface runoff. Further increases in the removal rate, however, require substantially higher filter widths. For example, an average of 70% removal for all three pollutants was attained when the strip length is increased to 100 feet. Impractically long strips (300 to 600 feet) are needed to attain a consistent 90% removal rate for these pollutants.

TABLE 4.14: POLLUTANT REMOVAL OF AN URBAN VEGETATED FILTER STRIP IN VIRGINIA (SOURCE: YU ET AL., 1993)

Parameter	Removal Performance of LS/VBS System	
	75 Foot Filter Strip	150 Foot Filter Strip
Total Suspended Solids	54%	84%
Nitrate+Nitrite	(-27%)	20%
Total Phosphorus	(-25%)	40%
Extractable Lead	(-16%)	50%
Extractable Zinc	47%	55%

The ability of grass and forested buffers to remove nutrients in subsurface flow has been mixed. When conditions are ideal, very high removal rates for nitrate have been reported. These conditions include the combination of poorly drained and highly organic soils, trees with deep roots systems, and lateral groundwater movements within four to six feet of the surface. These conditions actively promote the denitrification process, which converts nitrate-nitrogen into nitrogen gas. Where such conditions persist, forest buffers can effectively reduce the nitrogen content of septic system effluent in rural residential areas.

In general, most researchers consider agricultural filter strips to be a useful BMP, but only when they are combined with other practices (Magette et al. 1989). It is also widely recognized that many agricultural filter strips fail to perform as designed after they are installed in the field (Dillaha et al., 1989). Field surveys indicate that many filter strips lack good vegetative cover, are subject to excessive sediment deposition, or are short-circuited by channels formed by concentrated flow. This is particularly true for filter strips employed in urban areas, where runoff concentrates very quickly.

4.2D BIORETENTION

No monitoring data is presently available to assess the pollutant removal capability of bioretention areas. Based on the number and redundancy of possible removal pathways, it is very likely that the removal rate will be high. In many ways bioretention areas function in the same manner as a dry swale (i.e., both filter ponded runoff through a filter bed of prepared soil), so it is presumed that their pollutant removal capability would also be similar to the dry swale.

4.2E SUBMERGED GRAVEL FILTERS

Three submerged gravel or rock filters have been monitored to determine their ability to remove pollutants in stormwater runoff (Egan et al.,1995, Horsley,1995 and Reuter et al., 1992). Although the both the design and site conditions associated with each filter were very different, the performance of the submerged gravel filters as a group appears to be very promising.

Egan designed and constructed an experimental "stormwater treatment train" to treat runoff from a 121 acre industrial subwatershed in Central Florida. The off-line system featured packed bed filter cells. Each packed bed filter cell was excavated into the soil, and had dimensions of 80 feet wide by 30 feet long and three feet deep. The bottom of each cell was sealed with a plastic liner, and then filled with either crushed concrete or granite rock. Eight filter cells were planted with one or more of the following emergent wetland plant species: maidencane, giant bulrush, fireflag. Two cells were not planted to serve as controls (i.e., to test the pollutant removal capability of the rock media itself). The overall pollutant removal performance of Egan's packed bed filter system is summarized in Table 4.15.

Horsley's design is termed the StormTreat system and consists of a circular tank. Runoff passes through internal sedimentation chambers for pretreatment, and then is diverted into a outer ring that is filled with a gravel/sand media in which wetland plants are rooted. A schematic of the StormTreat system is provided in Figure 4.3, and recent performance monitoring data for an experimental site in coastal Massachusetts is supplied in Table 4.16.

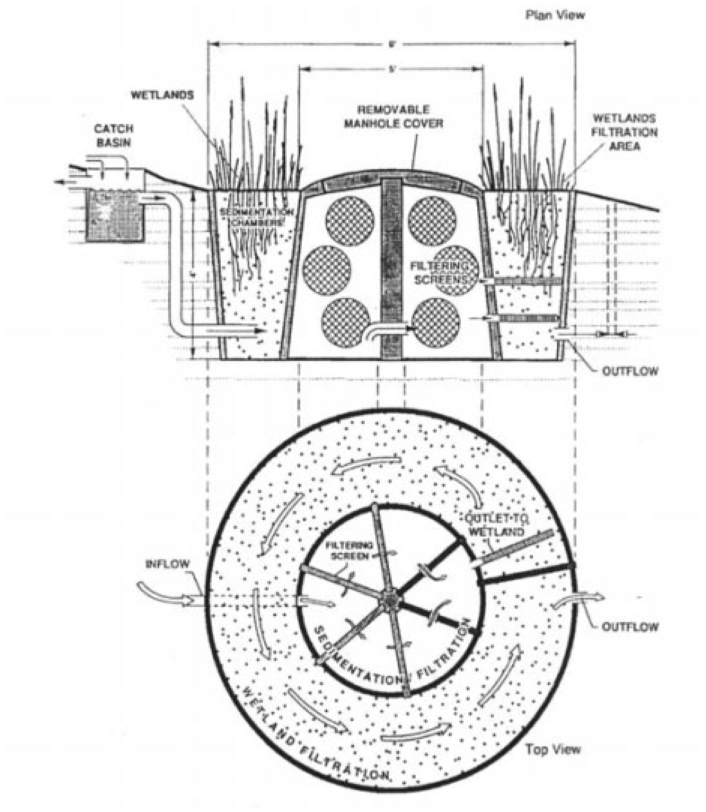
TABLE 4.15: POLLUTANT REMOVAL PERFORMANCE OF A ROCK WETLAND CELL FILTER SYSTEM IN FLORIDA (SOURCE: EGAN et al., 1995)

Parameter	*Mass Removal Rate (%)*
Total Suspended Solids-TSS	81
Total Dissolved Solids-TDS	8
Total Organic Carbon-TOC	38
Total Kjeldahl Nitrogen TKN	63
Nitrate-Nitrogen	75
Total Nitrogen	63
Ortho-phosphorus	14
Total Phosphorus	82
Cadmium	80
Chromium	38
Copper	21
Lead	73
Zinc	55
Fecal Coliforms	78

TABLE 4.16: POLLUTANT REMOVAL OF THE STORM TREAT SYSTEM (SOURCE: HORSLEY, 1995)

Parameter	*Stormwater Influent*	*Percentage Removed*
Fecal Coliform (no./100ml)	690	97
Total Suspended Solids	93	99
Chemical Oxygen Demand	95	82
Phosphorus (ug/l)	300	89
Dissolved N (ug/l)	1638	44
Total Petro HC (mg/l)	3.4	90
Lead (ug/l)	6.5	77
Chromium (ug/l)	60	98
Zinc (ug/l)	590	90

FIGURE 4.3: CROSS SECTION AND PLAN VIEW OF STORMTREAT SYSTEM



Reuter and his colleagues designed a simple submerged gravel filter in a high altitude, cold climate region in California. The filter treated the runoff produced from a 2.5 acre recreational area, most of which was fertilized ballfields (i.e., no impervious cover). The filter was a rather small 0.16 acres in size, composed of transplanted cattails that had not become fully established during the course of study. The bottom of the wetland was sealed with a liner, and filled with a three foot deep layer of fine gravel. Runoff was introduced into the gravel layer in a perforated pipe; outflow was collected by means of perforated pipe located in a standing well. Thus, runoff had to pass through the entire gravel filter before leaving the wetland. In general, the gravel layer was anaerobic (no oxygen), except for the top few inches. The bottom of the gravel layer was "innoculated" with muck from an adjacent wetland to introduce denitrifying bacteria into the system (Table 4.17).

TABLE 4.17: PERFORMANCE OF LAKE TAHOE GRAVEL-BASED STORMWATER WETLAND (SOURCE: REUTER et al., 1992)

Water Quality Parameter	*Mean Storm Removal (%)*
Suspended Sediment	80 to 88
Particulate Phosphorus	44 to 47
Soluble Reactive Phosphorus	-28 to -41
TKN	-3 to -58
NH4	-53 to -58
Nitrate	85 to 87
Soluble Iron	72 to 78

While the basic design of each gravel filter was somewhat different, each used a rock or gravel media, had standing water and had difficulty in getting wetland plants to colonize the media.

SUSPENDED SEDIMENT

All three gravel filter systems were able to remove at least 80% of the incoming suspended sediment concentrations, with may reflect the excellent settling environment with the gravel media (Wegelin, 1983). Removal of various forms of organic carbon ranged from 38 to 82%, which may be due to the export of algal detritus.

NITROGEN

The removal of organic forms of nitrogen in most gravel filters was generally high, ranging from 60 to 75%. Of even greater interest, each of the gravel filters were very effective in removing nitrate (or inorganic nitrogen), with Egan, Reuter and Horsley reporting 75%, 86% and 44% removal, respectively. As noted before, the high nitrate removal rate is unusual among filtering systems, and may indicate that both nitrification and denitrification may be occurring in the aerobic and anaerobic environment present in the rock and gravel filter cells.

PHOSPHORUS

The gravel filters also exhibited a strong potential to remove total phosphorus, ranging from 44 to 89%. It should be noted that the two studies that actually measured soluble phosphorus removal recorded low or even negative rates, which may reflect "leakage" of internal biological production.

TRACE METALS

Egan reported variable removal of trace metals, with low to moderate removal for metals often found in soluble form (copper and chromium), and moderate to high removal for metals found primarily in particulate form (cadmium, lead and zinc). Horsley, on the other hand, reported removal rates ranging from 80 to 90% for chromium, lead and zinc.

BACTERIA AND PETROLEUM HYDROCARBONS

Monitoring of the StormTreat system indicated that this version of the gravel filter was able to remove an average of 97% of fecal coliform bacteria and 90% of incoming petroleum hydrocarbons.

RELATIVE IMPORTANCE OF GRAVEL MEDIA AND WETLAND PLANTS UPTAKE

The growth of algae and microbes among the gravel media was found to be the dominant removal pathway in the gravel filter, clearly outdistancing the effect of wetland plant uptake. Egan noted that unplanted rock filter cells performed better than any other planted cells, suggesting that wetland vegetation had no discernable influence on pollutant removal. He concluded that the rock surfaces themselves were important for pollutant removal, by creating a large substrate area for growth of epilithic algae and microbes, reducing flow rates, and providing more contact surfaces. The same basic conclusion was reached by Reuter et al. (1992) and Horsley (1995), since their gravel filter cells also never achieved extensive wetland plant coverage during the monitoring period.

In general, the pollutant removal performance of the packed bed filter was similar to those reported for sand and organic sand filters, with the notable exception of consistently higher rates for inorganic nitrogen.

4.3 COMPARATIVE POLLUTANT REMOVAL CAPABILITY

Several generalizations can be made about the overall performance of stormwater filtering systems. In general, they exhibit a high capability to remove suspended sediments, organic carbon and hydrocarbons, a moderate ability to remove total phosphorus and nitrogen (although low or negative with respect to soluble nutrient forms, and a moderate to high ability to remove trace metals pollutants (although, again, some designs are less effective at removing soluble forms). The one stormwater pollutant whose performance cannot easily be generalized is fecal coliform with some designs showing a high capability to remove bacteria, and others showing none. The average reported removal rates for the eleven stormwater filtering designs are compared in Table 3.5 in the last chapter.

How do the different stormwater filtering designs compare with respect to pollutant removal capability? Table 4.18 provides a general comparison of expected pollutant removal rates, based on monitoring data, theory and best professional judgement. As can be seen, most filtering designs have a high capability to remove sediment and hydrocarbons. Phosphorus removal rates range more widely, with the highest rates reported for gravel filters, dry swales and perimeter sand filters, and the lower rates for grass channels, wet swales and filter strips. Nitrogen removal typically ranges from 30 to 50%. Most filtering systems; however, have a zero or negative removal rate for soluble nitrate (with the exception of dry swales, wet swales and gravel filters). Most filtering systems have a high capability to remove bacteria, with the exception of open channel options such as drainage channels and grass channels. Metal removal rates are variable, but most designs appear capable of removing 50 to 75% of the total metal load delivered to them.

How does the performance of filtering systems, as a group, compare to other BMP systems, such as stormwater ponds, wetlands and infiltration systems? Table 4.19 presents a very generalized comparison of the comparative pollutant removal capability of these four groups of BMPs (important caveat: actual removal rates for a particular design within a BMP group, however, may be higher or lower than those shown in the Table, and are presented only for rough technology comparison).

When the four groups of BMP systems are compared, it is evident that there is not a great deal of difference in their capability to remove sediment, hydrocarbons or total phosphorus. Greater differences in pollutant removal are noted for nitrogen (especially nitrate), organic carbon, and trace metals. There is not enough data available to assess if their are any differences in bacteria removal among the four groups of BMPs. It should also be noted that the removal rates indicated for infiltration BMPs are projections only, since very few of these systems have actually been monitored. In summary, it appears that the removal capability of most BMP systems is similar for most pollutants of concern, *when they are designed and maintained properly and incoming pollutant levels are higher than the irreducible concentration.*

TABLE 4.18: ESTIMATED POLLUTANT REMOVAL CAPABILITY OF DIFFERENT STORMWATER FILTER SYSTEMS (AVERAGES OF REPORTED MONITORING DATA)

Filtering System	*Monitoring Data?*	*TSS*	*TP*	*TN*	*NO_3*	*Other Pollutants/Comments*
Surface Sand Filter	Yes, 6	85%	55%	35%	Neg	Bacteria: 40-80% Metals: 35-90%
Underground Sand Filter	No Data	Presumed to Comparable to Surface Sand Filter				
Perimeter Sand Filter	Yes, 3	80%	65%	45%	Neg	Hydrocarbons: 80%
Organic Sand Filter	Yes, 1	95%	40%	35%	Neg	Hydrocarbons: 90% Sol. P Negatives Metals: 85%+
Pocket Sand Filter	No Data	Presumed to be Comparable to Surface Sand Filter				
Drainage Channel	Yes, 10	30%	10%	Zero	Zero	Bacteria: Negative
Grass Channel = biofilter	Yes, 1	65%	25%	15%	Neg	Hydrocarbons: 65% Metals: 20-50% Bacteria: Negative
Dry Swale	Yes, 3	90%	65%	50%	80%	Metals: 80-90%
Wet Swale	Yes, 2	80%	20%	40%	50%	Metals: 40-70%
Bioretention	No Data	Presumed to be Comparable to Dry Swale				
Filter Strip	Yes, 1	70%	10%	30%	Zero	Metals: 40-50%
Gravel Filter	Yes, 2	80%	80%	65%	75%	Hydrocarbons: 85% Metals: 50-75%

TABLE 4.19
COMPARATIVE POLLUTANT REMOVAL CAPABILITY OF FOUR TYPES OF BMP SYSTEMS

Stormwater Pollutant	*Pond Systems**	*Wetland Systems*	*Infiltration Systems*	*Filtering Systems*
Suspended Sediment	80	75	90**	85
Organic Carbon	65	15	90**	50
Total Nitrogen	35	25	50**	35
Nitrate-N	60	60	50**	Negative
Total P	65	50	60**	60
Ortho-P	70	40	50**	50
Copper	50	30	60**	45
Lead	85	75	90**	85
Zinc	65	50	90**	75
Bacteria	1-2 Log	1-2 Log	1-2 Log**	2 Log
Hydrocarbons	80**	80**	?	85

Notes: * Does not include dry extended detention ponds
**** Projected**
The removal rates shown are for comparison purpose only
Actual removal for each system can vary widely depending on design
Sources: Current Assessment of Urban BMPs, Design of Stormwater Wetlands

4.4 DESIGN FACTORS TO ENHANCE FILTERING SYSTEM PERFORMANCE

In this section, practical design techniques are presented to consistently enhance the pollutant removal performance of stormwater filtering systems. These key design principles have been incorporated into the engineering methods presented in succeeding design chapters. Some general design principles that apply to all filtering systems include:

4.4A TYPE AND VOLUME OF PRETREATMENT

A pretreatment cell is not only needed to protect a filter from clogging, but also to temporarily store diverted runoff for subsequent treatment. Consequently, the pretreatment volume is usually significantly greater for filtering systems than in other

BMP systems. Where possible, some fraction of the pretreatment cell should be "wet" (i.e,. a permanent pool) to reduce incoming runoff velocities and reduce the potential for re-suspension of pollutants.

4.4B ADEQUATE CAPTURE VOLUME

It is important to capture and store a relatively large water quality volume (WQV) prior to treatment, since most filters are an off-line practices and will bypass some runoff during larger storm events (which is not treated). Based on consideration of rainfall/runoff statistics, monitoring data and pollutant removal pathways, it is practical to capture 90% of the average annual rainfall volume within or before the filter. In most regions of the Chesapeake Bay, this volume is equivalent to 1.0 inch of rainfall multiplied by the volumetric runoff coefficient (Rv) and the site area (acres). The capture volume for each filter can be temporarily stored in either the pretreatment cell or over the filter bed surface.

4.4C OFF-LINE FILTER DESIGN

Since filtering designs are intended to treat the water quality volume, they should be designed as off-line practices wherever possible. This usually involves constructing a flow-splitter or other device to divert the WQV into the filter bed. In cases where the filtering system must be designed on-line (e.g., grass channels, and dry and wet swales, it is important to ensure that the channel will not be subject to erosive runoff velocities during the 2 year design event (usually 4 or 5 feet per second).

4.4D SIZING OF FILTER BED

Each filtering system utilizes a slightly different area and depth for the filter bed. In most cases, the surface area of the filter bed is a direct function of the impervious area treated, and the depth of the filter bed ranges from one to two and a half feet (with the exception of bioretention areas, which are typically 4 feet deep). In most cases, the bulk of the filtration occurs with the top few inches of the filter media.

4.4E IMPROVED FILTER MEDIA

A common design approach has been to add a more organic media to the filter bed to enhance its removal capability. A series of organic media can be used for this purpose: peat, compost, organic soils to name a few. The limited data on organic media suggest that they may be superior in removing hotspot pollutants such as hydrocarbons, metals, and organics (but also may be a net source of some nutrients due to leaching).

4.4F MULTIPLE POLLUTANT REMOVAL PATHWAYS

The key to improving the performance of any filter design is to maximize the value of settling, straining, infiltration, uptake or adsorption pathways within the system. Where possible, multiple pollutant removal pathways should be utilized to create a redundant treatment system.

4.4G PROMOTE PARTIAL EXFILTRATION

Filtering systems should be designed to exfiltrate runoff into the soil where insitu conditions allow, rather than collecting it in a pipe (groundwater contamination is not considered a risk). The partial exfiltration of runoff allows for additional pollutant removal by the soil layer. Infiltration of runoff can be a very important pollutant removal pathway, particularly for open channel designs.

4.4H IMPROVING NITROGEN REMOVAL

Many filtering systems have been found to have a poor ability to remove soluble nitrogen from urban runoff. If greater nitrogen removal is desired, it is important to promote greater denitrification within the filtering system. Usually, this is done by creating a wet cell or zone within the filter to maintain an anaerobic condition and a high organic matter content. This permanently saturated and anaerobic zone at the bottom of the filter bed creates favorable conditions for denitrifying bacteria, which might substantially improve the rate of nitrate removal. It is important to maintain an aerobic portion of the filter to avoid phosphorous leaching.

4.4I OPEN CHANNELS

To be effective, open channels should be explicitly designed to increase the volume of runoff that is retained or infiltrated within the channel, or at least lengthen the contact time through the channel during a storm. For best removal, open channels should be designed to retain/infiltrate the full water quality volume during a storm event.

4.4J INTERNAL FILTER GEOMETRY

The hydraulics of each filter system should be carefully evaluated to ensure that incoming runoff does not "short-circuit" through either the pretreatment cell or the filter bed. In particular, a "drop-off" is often needed for many filtering systems to prevent an accumulation of sediment at the entry point to the filter. In addition, the designer should carefully evaluate whether the filter design meets minimum criteria for length, area or slope from the contributing drainage area or contact time.

CHAPTER 5
KEY DESIGN ELEMENTS:
SAND AND ORGANIC FILTERS

5.1 INTRODUCTION

The following three chapters follow the same general format adapted for the design of sand and organic filters, bioretention and open channel filters. Each practice has four major components: flow regulation, pretreatment, filter bed, and overflow. In addition, material specifications, construction considerations and maintenance elements are presented.

5.2 ALTERNATIVE CONFIGURATIONS

The five most common sand and organic filter alternatives, presented in Chapter 1, are reviewed again for clarity. They each were developed and adapted by various governments and engineers to serve different water quality treatment goals or to accommodate different physical constraints. Other alternative configurations may prove useful for different land use applications or climatic conditions and should be encouraged.

5.2A SURFACE SAND FILTER

The City of Austin, Texas first developed the sand filter technology for treatment of urban stormwater runoff in the early 1980's. The surface sand filter (or Austin sand filter, as it has often been called) is usually supported by a concrete shell, although earthen embankments are equally acceptable (Figure 5.1). The system is divided into a sedimentation chamber, for pretreatment to collect diverted runoff and settle out course sediments and a filter bed chamber, consisting of a flow distribution cell and the sand filter bed. The filter bed has an 18" - 24" sand layer which traps or strains pollutants before runoff is collected in an underdrain system (gravel and perforated pipe) and conveyed to the receiving stream, channel or pipe.

5.2B UNDERGROUND SAND FILTER

The underground sand filter (or District of Columbia sand filter) was developed for the intensely developed area within the inner city. This system is placed underground but maintains essentially the same components as the surface sand filter (Figure 5.2). The practice consists of a three chamber vault. A three feet deep wet sedimentation chamber is hydraulically connected by an underwater opening with the second chamber. This element is designed to dissipate energy and to provide pretreatment by trapping grit and floating organic material. The second chamber contains an 18" - 24" sand filter bed and an underdrain system including inspection/cleanout wells. A layer of plastic filter cloth with a gravel layer can be placed on top of the sand bed to act as a pre-planned failure plane which can be replaced when the filter surface becomes clogged. The third chamber collects the flow from the underdrain system and directs flow to the downstream receiving drainage system (Truong, 1989).

FIGURE 5.1: SURFACE SAND FILTER

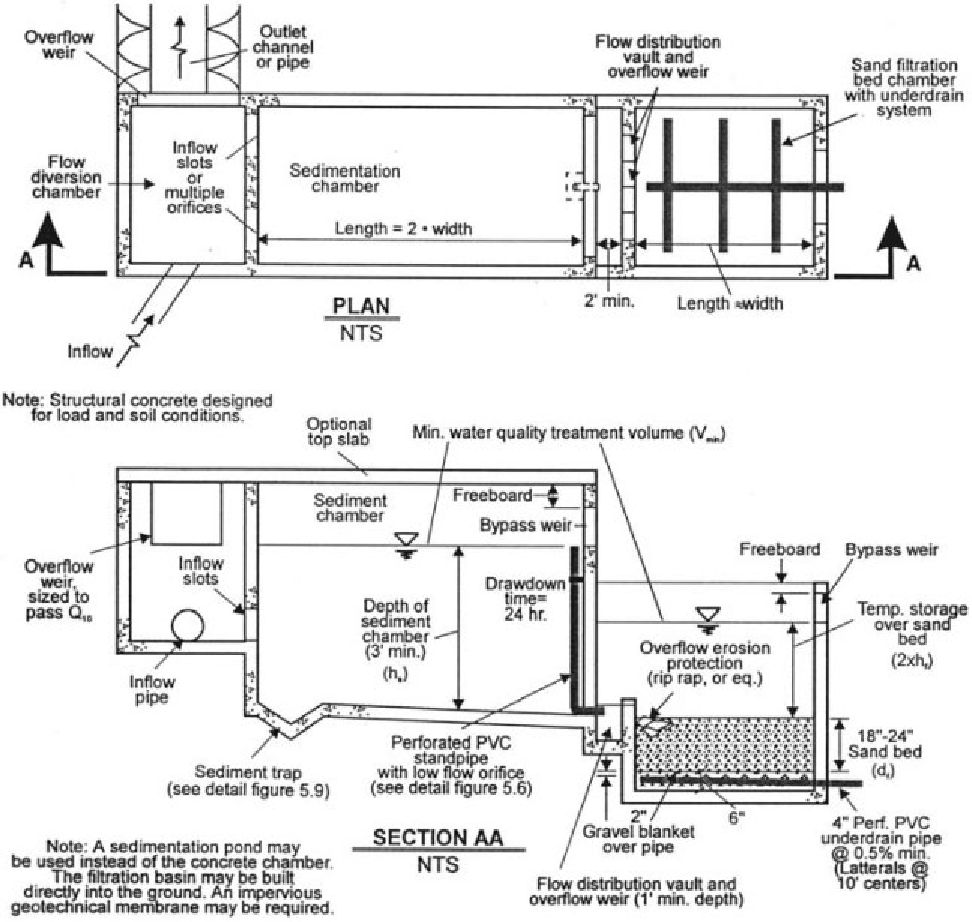
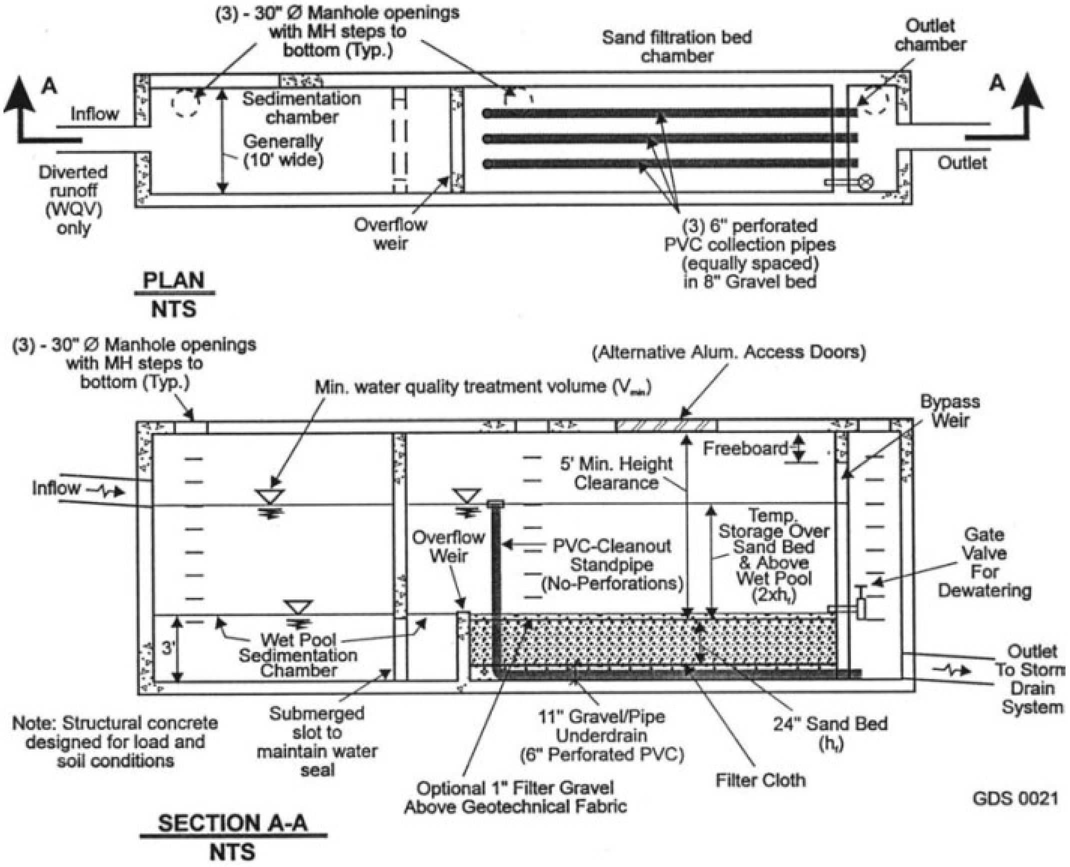


FIGURE 5.2: UNDERGROUND SAND FILTER

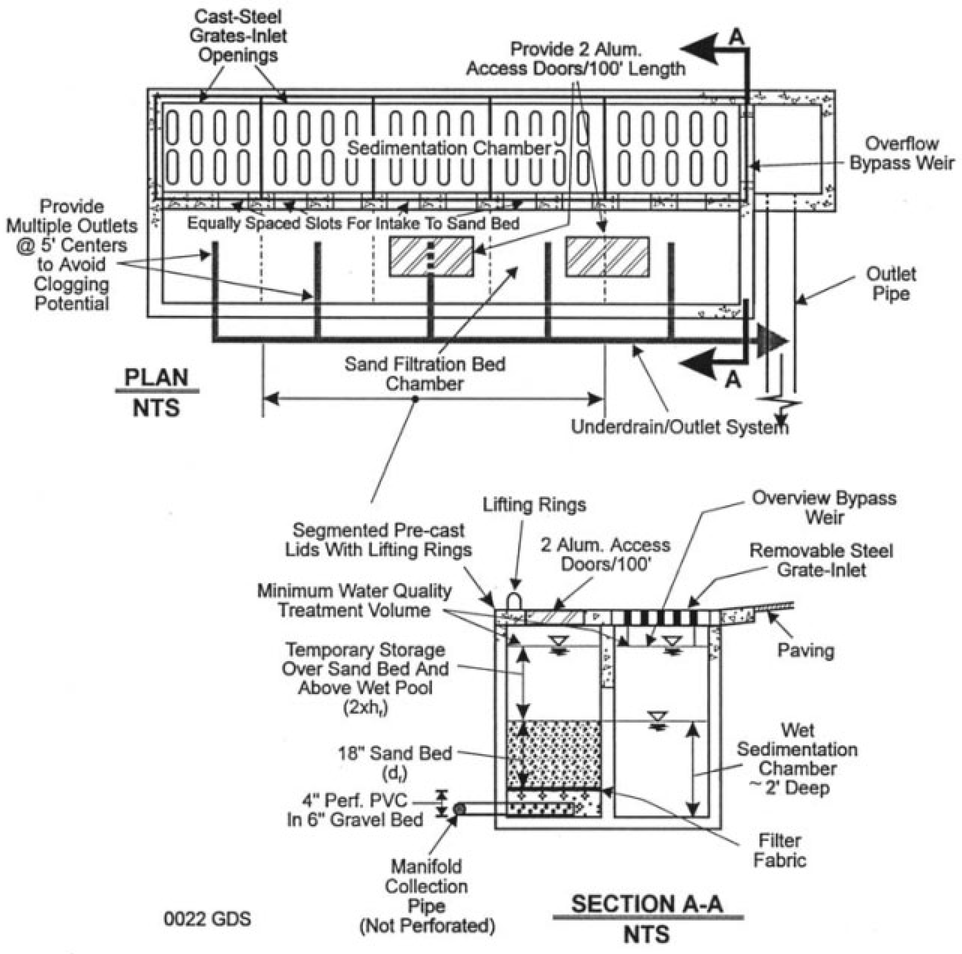


5.2C PERIMETER SAND FILTER

The perimeter sand filter was originally developed in Delaware by Earl Shaver and conceived as an on-line facility which treats all stormwater entering the system up to the overflow limit, originally set at the first one inch of runoff. The City of Alexandria, Virginia modified this system to incorporate a flow-splitter to isolate and treat only the "water quality volume," (WQV). Figure 5.3 illustrates the modified Delaware sand filter. The system consists of a grated steel inlet to the sedimentation chamber, the filter bed itself, and an outlet chamber. The

WQV flows into the filter bed via distribution slots or multiple orifices, while larger storm volumes bypass the filter chamber through an overflow weir. The filter bed chamber consists of an 18" sand bed over a gravel/perforated pipe underdrain system with several outlets which discharges into a manifold pipe collection system.

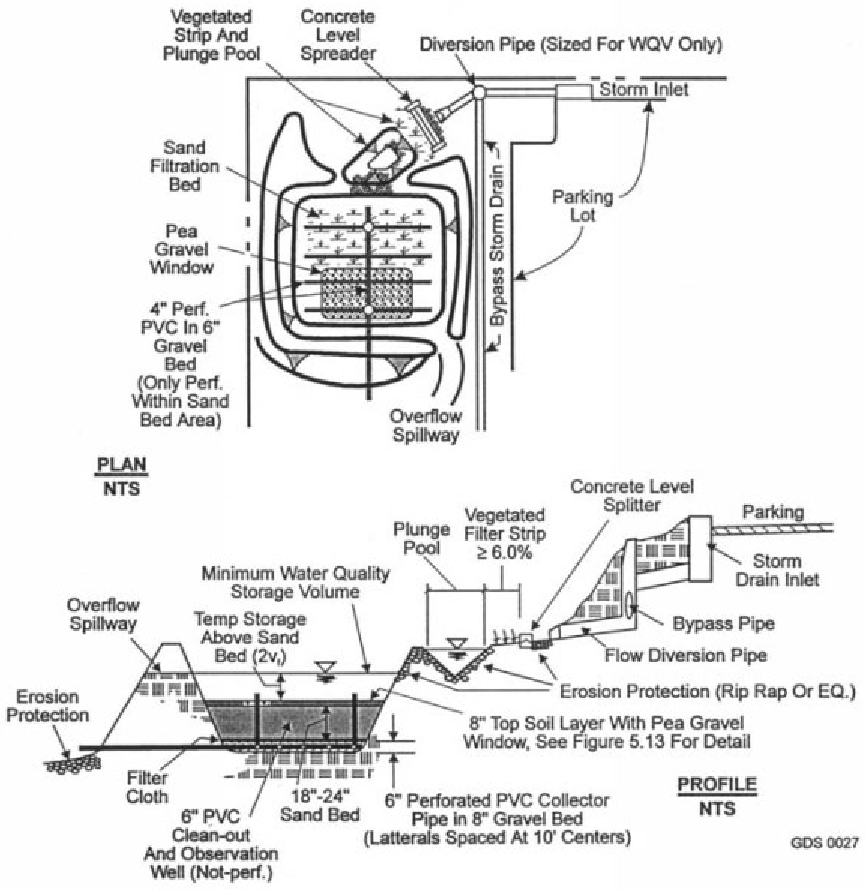
FIGURE 5.3: PERIMETER SAND FILTER



5.2D POCKET SAND FILTER

The pocket sand filter system is intended to provide an inexpensive solution for utilizing sand filter technology for those small sites where anticipated sediment loads do not warrant a sedimentation chamber and can suffice with vegetative pretreatment practices. The pocket sand filter (Figure 5.4) consists of a flow splitter inlet structure to capture the WQV, a vegetative filter strip or suitable alternative (such as a small stilling basin at a storm drain pipe outfall) and an above ground sand filter bed (18"-24") over a gravel underdrain system. The filter bed chamber may require an impermeable liner for areas where groundwater contamination is a critical concern. The pocket sand filter may also be constructed on-line for very small drainage areas (say, less than one acre). In these cases, a conventional inlet and discharge structure is necessary to accommodate both the WQV and larger storms.

FIGURE 5.4: POCKET SAND FILTER



5.2E ORGANIC FILTER MEDIA

The use of an organic media within a filtering system may offer increased pollutant removal efficiencies over sand filters alone (particularly with respect to nutrients). At least two principal types of organic media have been utilized recently for the treatment of urban runoff. These are peat (partially decomposed organic material of geological origin) and leaf compost.

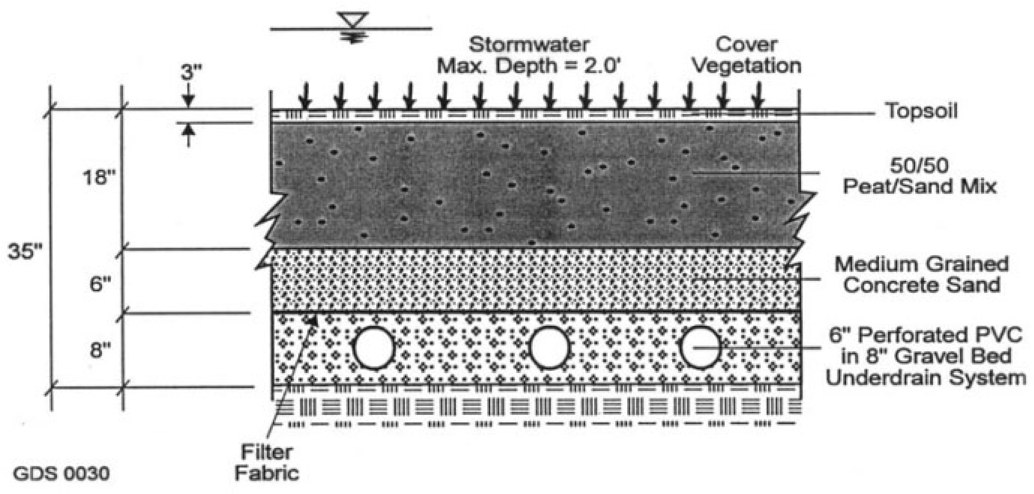
Peat has been utilized in conjunction with sand as an alternative wastewater treatment system for several years. Recently there have been attempts to adapt this practice for stormwater applications. John Galli, of the Metropolitan Washington Council of Governments, prepared an analysis paper of peat-sand practices for stormwater applications (Galli, 1990). The City of Alexandria has incorporated the work of Galli and others into a design criteria (City of Alexandria, 1992).

A proprietary leaf compost system has been developed by CSF Treatment Systems, Inc. of Portland Oregon for treating stormwater runoff from smaller drainage areas. Approximately 30 compost systems have been installed in the Pacific Northwest over the past several years.

PEAT-SAND FILTER SYSTEM

The peat-sand filter system was originally developed in the Pacific Northwest and consists of a fabricated soil filtration bed which combines the benefits of peat with those of a grass cover crop and a sand underlayer. The type of peat utilized in the filtration bed is extremely important. A fibric peat, where the undecomposed fibrous organic matter is easily identifiable is the preferred type. A hemic peat, where more material is decomposed, may also be utilized. Under no circumstances should a sapric peat, made up of mostly decomposed matter, be used. Figure 5.5 illustrates a typical peat-sand filtration bed cross section. The surface sand filter application can be modified to incorporate the peat-sand filtration bed.

FIGURE 5.5: PEAT-SAND CROSS SECTION

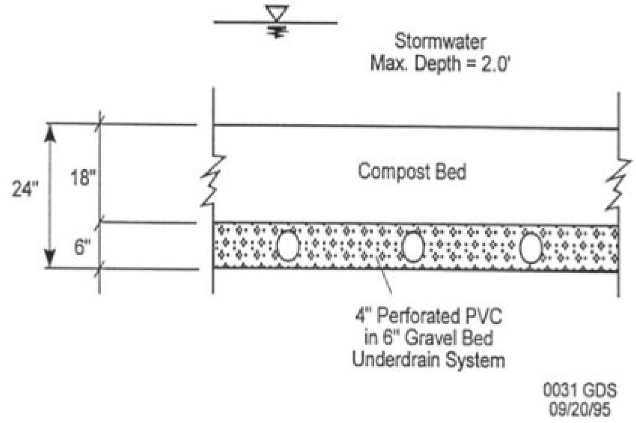


COMPOST FILTER SYSTEM

The compost filter system consists of a fabricated leaf compost filtration bed overlying a gravel/pipe underdrain system. The key to the system is through the proper selection of compost. The compost should be mature and humic (the organic material is no longer rapidly degrading), have low contaminant levels, have a high permeability, and be locally available at a reasonable cost. A leaf compost medium, as opposed to a yard waste compost mixture is necessary. A high quality leaf compost is prepared by ensuring weekly turning which promotes good size reduction, aeration and rapid maturation. Some road gravel is often included in the compost which helps afford good flow permeability. Recently, a pelletized compost is being employed to maintain higher filtration rates (CSF, 1996). Figure 5.6 illustrates a typical compost filtration bed cross section. The surface sand filter application can be adapted for the compost filter media.

CSF Treatment Systems, Inc. (1994) has a proprietary system which incorporates a design size based on a compost bed surface area requirement of 200 ft^2/cfs, a filter media thickness if 18 inches, a forebay of unspecified size, a gravel/pipe underdrain system, and a discharge structure.

FIGURE 5.6: COMPOST FILTER CROSS SECTION



5.3 FLOW REGULATION

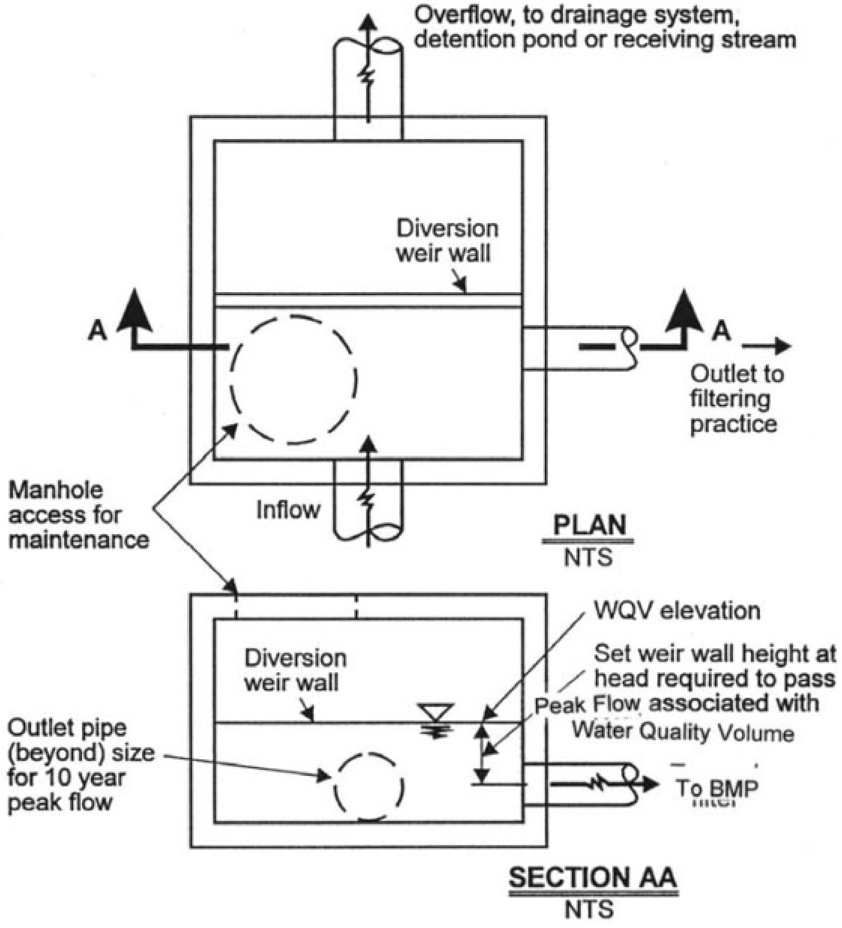
Since sand filters are designed to provide treatment for the "water quality volume" (WQV) only, they should be located off-line from the primary conveyance/detention system. Sand filters should be located where they can intercept as much of the site impervious area as possible and where discharge to the primary conveyance system is feasible.

Offline designs are recommended for sand filter systems to avoid mixing with larger storm events which are likely to resuspend settled solids within the sedimentation chamber, scour the filter bed, or otherwise compromise the pollutant removal effectiveness of these facilities.

The design objective is to capture and divert the water quality volume (WQV) to the sand filter and “bypass” larger storms to the downstream storm drainage system or receiving water. WQV is computed based on the methods identified in Chapter 2. In most Chesapeake Bay drainage jurisdictions, the enclosed conveyance systems are sized for the 10 year storm event. Open channel systems may be sized for larger events. A flow diversion structure must be able to accommodate these larger flows as well as the water quality storm.

Two methods for diverting the WQV include: Computing a peak discharge (Q_p) for the "water quality storm" and (1) utilizing an isolation/diversion structure upstream and within the drainage network or, (2) incorporating the isolation/diversion structure within the treatment practice itself. Figure 5.7 illustrates an application of the first method. See Figure 5.1 for application of the second method.

FIGURE 5.7: ISOLATION/DIVERSION STRUCTURE



The preferred method for accomplishing a diversion is within the treatment practice itself, where the overflow (or bypass) weir elevation is set equal to the design WQV elevation within the adjacent practice. This method ensures larger inflows will overflow the bypass weir, thus minimizing mixing within the BMP. It is also a more reliable capture technique, than reliance on a computed peak rate of discharge (Qp) to size the diversion structure.

It is still necessary to compute the Qp to size the intake slots or openings. The openings directing runoff to the treatment practice should be slightly oversized to ensure that the entire WQV is treated. The design example at the end of this chapter illustrates the methodology for doing this.

In many cases, however, it is not possible to maintain the necessary geometry and elevations to locate the isolation/diversion structure within the treatment practice itself. Therefore, an alternative technique for isolation/diversion within the drainage network should be utilized. The methodology for doing this is described in Table 5.1:

TABLE 5.1
DESIGN PROCEDURE FOR DIVERSION TECHNIQUE WITH THE DRAINAGE NETWORK

1. Peak Discharge (Qp) for WQV is computed based on the methods presented in Chapter 2.
2. Qp for the "bypass storm" is computed (most jurisdictions utilize the 10 year frequency storm). Utilize the Rational Formula or SCS TR-55.
3. Size diversion slots/openings or pipe utilizing the orifice equation: $Q=CA(2gh)^{½}$
4. Size overflow weir for "bypass storm" using the Weir Equation: $Q = C_W LH^{3/2}$, size the outfall pipe, if provided, using the orifice equation (to check inlet condition flow capacity) and Manning's equation to check friction losses.

5.4 PRETREATMENT

Pretreatment is necessary for stormwater filters to remove excessive sediment which contributes to premature failure of the practices. Pretreatment may be in the form of sedimentation basins, vegetative filter strips, grass swales, storm drain structures with sumps, or water quality (oil/grit separator) inlets.

5.4A SEDIMENTATION BASINS

Sedimentation basins, also called pre-settling basins, are the preferred method of pretreatment for stormwater filters because the basins are constructed in conjunction with the filter bed and maintenance requirements are relegated to one location. In addition, the performance and sizing criteria for sedimentation basins are reasonably well established. Sedimentation basins can also be constructed underground in high density areas where space is limited.

The water quality volume, computed in Chapter 2, is used as the basis for sizing the pretreatment chamber for all types of stormwater filters, except the “pocket system.” According to an extensive literature review conducted by the City of Austin, TX, removal of discrete particles by gravity settling is primarily a function of surface loading (the rate of outflow divided by the basin surface area) and is independent of basin depth (Washington State Department of Ecology, 1992). However, a minimum basin depth of 3 feet is recommended to minimize particle resuspension and turbulence effects. Therefore, surface area is the primary design parameter for sedimentation affecting removal efficiency (E). E is also a function of particle size distribution. Silt sized particles are used as the target particle size for sedimentation basin design (i.e., . 20 microns).

For sites with imperviousness $ 75%, which have a higher percentage of coarse grained sediments (Shaver and Baldwin, 1991), the target capture partical is approximately 40 microns.

The following equation is used to size pretreatment settling basin surface area. It was derived by the Washington State Department of Ecology from the Camp-Hazen equation (Washington State Department of Ecology, 1992 and Chen, 1975).

$A_s = -(Q_o/w)\ (\ Ln(1-E)$ where: **Equation 5.1**

A_s = Sedimentation basin surface area (ft^2)

E = Trap efficiency; which is the target removal efficiency of suspended solids (set equal to 90%)

w = Particle settling velocity; for target particle size (silt) use settling velocity = 0.0004 ft/sec (0.0033 ft/sec for I $ 75%, where I is percentage impervious area)

Q_o = rate of outflow from the basin; which is equal to the water quality volume (WQV) divided by the detention time (t_d); use 24 hours.

$Q_o = WQV/t_d$ therefore:

$A_s = -WQV / [(24\ hr)(3600\ sec/hr)(0.0004\ ft/sec)]\ (\ Ln(0.1)$

$A_s = \underline{0.066\ (\ (WQV)\ ft^2}$ **Equation 5.2**

$A_s = \underline{0.0081\ (WQV)\ ft^2}$ for I $ 75% **Equation 5.2.1**

As discussed in Chapter 1, the WQV is used as a basis for sizing all filtering practices. However, for sand and organic media filters, where pervious areas are intentionally limited, the runoff for the WQV can be a sizable quantity and complete storage of the WQV is often not feasible or is cost prohibitive. Therefore, although the WQV is used to size minimum surface areas for both the sedimentation and filter bed chambers, **a volume of three-quarters of the WQV is maintained as the minimum storage volume required.**

$V_{min} = ¾\ (\ WQV$ **Equation 5.3**

Storing three-quarters of the WQV versus 100% of WQV is justified because the sedimentation chamber is continually draining into the filter bed during the course of a storm event. Only short duration, high intensity storms are likely to exceed the three quarters WQV threshold.

The length to width ratio of the basin should be 2:1 or greater. Inlet and outlet structures should be located at extreme ends of the basin. Baffles may be used to mitigate short-circuiting and/or dead storage problems. The basin bottom shall have a minimum depth of 3 feet to minimize resuspension and turbulence. The basin bottom shall be nearly level to facilitate sedimentation. The design elements for pretreatment which are specific to the different design variations are presented in Table 5.2.

TABLE 5.2: PRETREATMENT COMPONENTS FOR FOUR DESIGN VARIATIONS

Surface Sand Filter:

- Dry detention basin.
- Minimum volume = ¾ (WQV: split between volume within filter bed (voids), volume above filter bed, and volume within pretreatment chamber.
- Perforated standpipe with orifice sized to release volume (within sedimentation basin) over 24 hour duration (Figure 5.8). Note: The size and number of perforations depends on the release rate needed to achieve 24 hour detention.
- Overflow weir within the sedimentation chamber is set at design treatment volume, sized to pass 2/3 of WQV peak flow. Overflow weir within sand bed chamber set at design treatment volume, sized to pass 1/3 of WQV peak flow. This ensures at least partial treatment for flows exceeding ¾ (WQV.
- Permanent sediment trap: Since the sedimentation basin is dry, a permanent sediment trap is recommended. This consists of a small storage area to trap incoming sediment and remove this from the basin flow regime. It is recommended that the sediment trap volume be equal to ten (10) percent of the sedimentation basin volume. Water collected in the trap is conveyed directly to the flow distribution vault (Figure 5.9).

Underground Sand Filter:

- Wet retention basin.
- Wet volume (Vw) = A_s (depth (3' deep, minimum permanent pool storage).
- Total minimum volume = ¾ (WQV: Split between volume within filter bed (voids), wet volume within sedimentation chamber, volume above wet volume, and volume above sand bed.
- Overflow weir elevation (in filter chamber) set at design treatment volume, sized to pass 2/3 of WQV peak flow.

Perimeter Sand Filter:

- Wet retention basin.
- Wet volume (Vw) = A_s (depth (2' minimum depth permanent pool storage).
- Total minimum volume = ¾ (WQV: Split between volume within filter bed (voids), wet volume within sedimentation chamber, volume above wet volume, and volume above sand bed.
- Elevation of overflow weir to outlet chamber set at top of dry storage elevation (¾ (WQV), sized to pass 100% of incoming 10 year design flow.

Organic Filter Media

- The pretreatment technique for organic media is the same as with the surface sand filter, or it can incorporate a wet retention component as well (as with the perimeter and underground sand filter).

FIGURE 5.8: PERFORATED STANDPIPE WITH DRY SEDIMENTATION BASIN

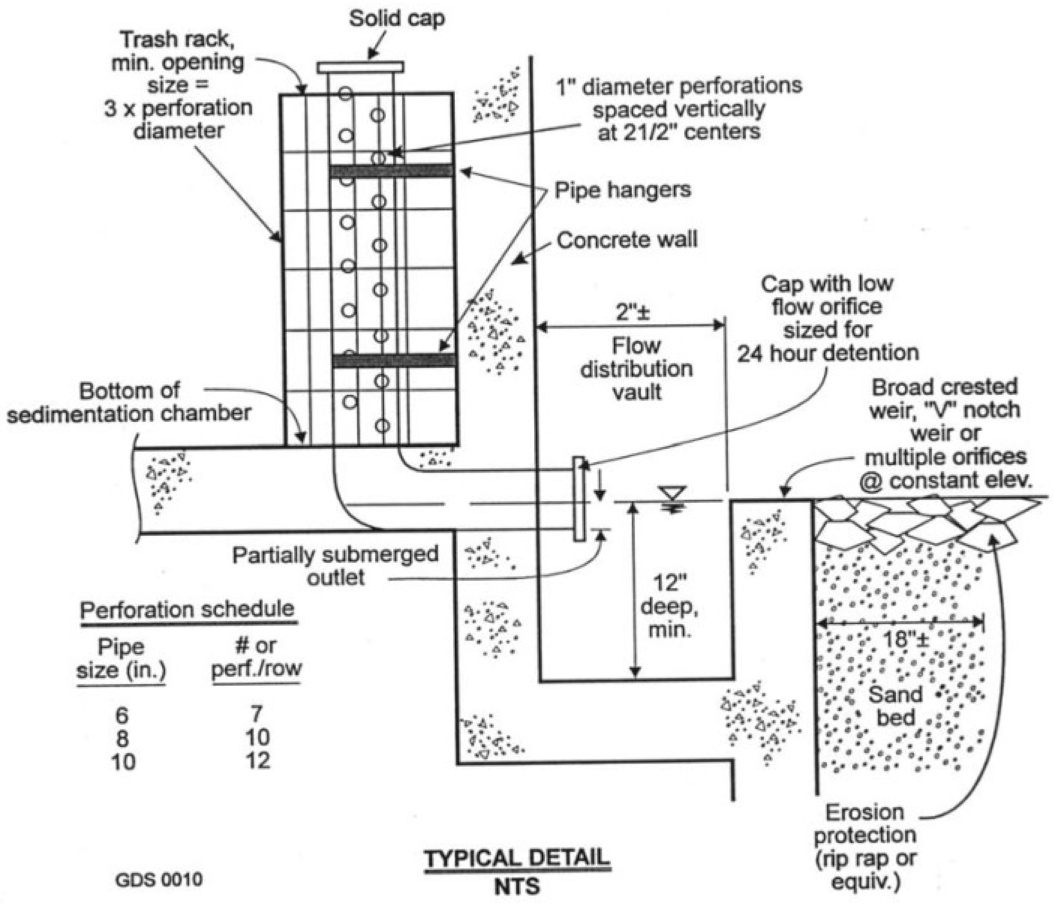
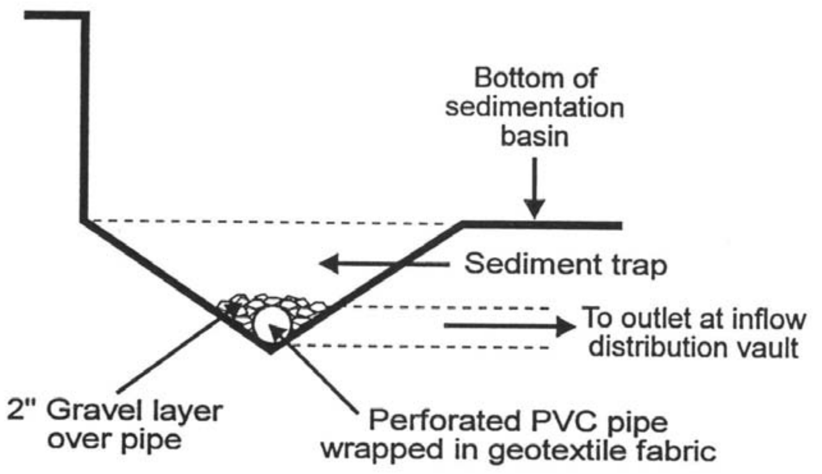


FIGURE 5.9: SEDIMENT TRAP DETAIL



5.4B VEGETATIVE PRACTICES

Vegetative filter strips and grass swales may be useful pretreatment methods where adequate space and conditions permit. Design parameters for these practices are reviewed in detail in Chapter 7. If the practice is being used for pretreatment for sand or organic filters, the design length or volume can be reduced to **10% of the required "stand-alone" design length.** The principal pretreatment components for the pocket sand filter are presented in Table 5.3.

TABLE 5.3
PRINCIPAL PRETREATMENT COMPONENTS FOR POCKET SAND FILTER

< Concrete level spreader for facilitating sheet flow
< Vegetated filter strip for partial pre-treatment (5' minimum)
< Plunge pool stilling basin for partial pre-treatment

5.4C STORM DRAINAGE SUMP INLETS

Storm drain structures with sumps can provided some reduction in incoming sediment loads but require frequent cleaning to avoid resuspension of solids. If a jurisdiction does not have a strenuous maintenance program with frequent storm drain system cleaning, this method is not encouraged.

5.4D WATER QUALITY INLETS

Water quality inlets have been shown to be a marginal method for removing particulate matter according to a study by the Metropolitan Washington Council of Governments (Schueler and Shepp,1993), and are therefore not recommended for sand filter pretreatment.

5.5 FILTER MEDIA

5.5A GENERAL SIZING GUIDANCE

The principles of Darcy's Law are used for sizing the sand filter bed area as derived by the City of Austin, TX, Environmental and Conservation Services Department (City of Austin, TX 1988).

The primary design parameter for filtration basins is surface area. The necessary surface area is a function of the permeability of the filter medium, the bed depth, the hydraulic head (height of water above the bed), and sediment loading. The following equation can be used to size all types of filter media presented in this manual.

$A_f = WQV\ ((d_f)/[k((h_f + d_f)(t_f)]$ where: **Equation 5.4**

A_f = Surface area of the sand filter bed (ft^2)
WQV = Water quality treatment volume (ft^3)
d_f = Sand filter bed depth (ft)
k = Coefficient of permeability for sand bed (ft/day)
h_f = Average height of water above the sand bed (ft); $h_f = ½ (h_{max}$
t_f= Time required for the Water Quality Treatment Volume (WQV) to filter through the sand bed

< WQV is computed using the procedures outlined in Chapter 2.
< d_f can vary depending on the site conditions but should not be more 24" (18" is the standard).
< h_f will also vary depending on the site conditions, but should not exceed 6 feet.
< A value of 40 hours is recommended for the filter bed draw-down time (t_f).

K VALUES FOR SAND FILTERS

k values for sand were computed by the City of Austin staff based on field observation and actual performance of previously installed sand filters. The values ranged form approximately 0.5 to 2.7 ft/day, with an average value of 1.5 ft/day. These values are substantially lower than those quoted in textbooks (Hwang, 1981) but allow for clogging associated with accumulated sediments. With an appropriately sized sedimentation basin (as described above), a value of **k ' 3.5 ft/day** is recommended (City of Austin, TX, 1988).

K VALUES FOR PEAT-SAND FILTERS

A composite coefficient of permeability is used based on the 50/50 mixture thickness of the different media.

$k = k_1 + k_2/2$

For peat, k can range from as high as 110 ft/day to as low as .02 ft/day depending on whether the peat is fibric, hemic, or sapric (Galli, 1990). Galli (1990) and Bell (1993) use a 2 ft/day coefficient of permeability for surface area sizing considerations (for a mixture of fibric and hemic peat).

Based on the broad range of peat permeability and its superior water holding capacity, a **coefficient of permeability of 2 ft/day** is recommended for design.

Using a k of 2 ft/day for peat, 3.5 ft/day for sand, and the typical section illustrated in Figure 5.5, an average coefficient of permeability of 2.75 ft/day is recommended.

K VALUES FOR COMPOST FILTERS

CSF Treatment Systems, Inc. (1994) recommends using permeability of 2.25 gpm/ft^2 for compost, based on laboratory tests. This translates to a permeability coefficient of 433 ft/day. Stewart (1992), presented data that showed that while the initial permeability of compost was always high, it tapered off after approximately 24 hours to generally 20% of the initial rate. Based on these results, the initial permeability rate should be reduced by 80%

The coefficient of permeability should reflect design conditions prevalent locally. In the Chesapeake Bay watershed, where rainfall characteristics differ substantially from the Pacific Northwest, it is important to capture and treat the high intensity rainfall events which form a significant portion of the annual runoff. In addition, since surface clogging does occur on filtering practices, the design permeability rate should reflect a percentage of the laboratory results after several hours of rainfall have occurred. A rate equivalent to 10% of the diminished rate should be used. Therefore the recommended design k value for compost should be as follows:

433 ft/day (0.20 (0.10 ' 8.66 ft/day, use **k ' 8.7 ft/day**

TABLE 5.4: COEFFICIENT OF PERMEABILITY VALUES FOR STORMWATER FILTERING PRACTICES

Filter Media	*Coefficient of Permeability (k, ft/day)*
Sand	3.5
Peat/Sand	2.75
Compost	8.7

5.5B SIZING PROCEDURES FOR DESIGN VARIATIONS

Sizing sand and organic filters is in most cases a straightforward process. Listed below is the sizing procedure for each design variation. Items 1 through 6 are the same for all variations, which include computing the water quality volume and flow regulation to the facility. Identified separately is the process for each design variation.

1. Compute the Water Quality Treatment Volume (WQV).
2. Calculate the peak discharge (Q_p) utilizing the 90% Rule (from Chapter 2), 1.0" rainfall for Chesapeake Bay Watershed.
3. Size the flow diversion structure to divert the WQV to the sand filter.
4. Using Darcy's Law, size the sand filter bed surface area (A_f).
5. Using the Camp-Hazen equation, size the sedimentation basin surface area (A_s)(except pocket sand filter).
6. Compute the required minimum storage within the practice ($V_{min} = ¾$ (WQV)

SURFACE SAND FILTER (FIGURE 5.10)

< Compute the water volume within the filter bed (V_f) = A_f (depth of bed & gravel (d_f) (porosity (n) (use n = 0.4 for sand/gravel/perforated pipe).

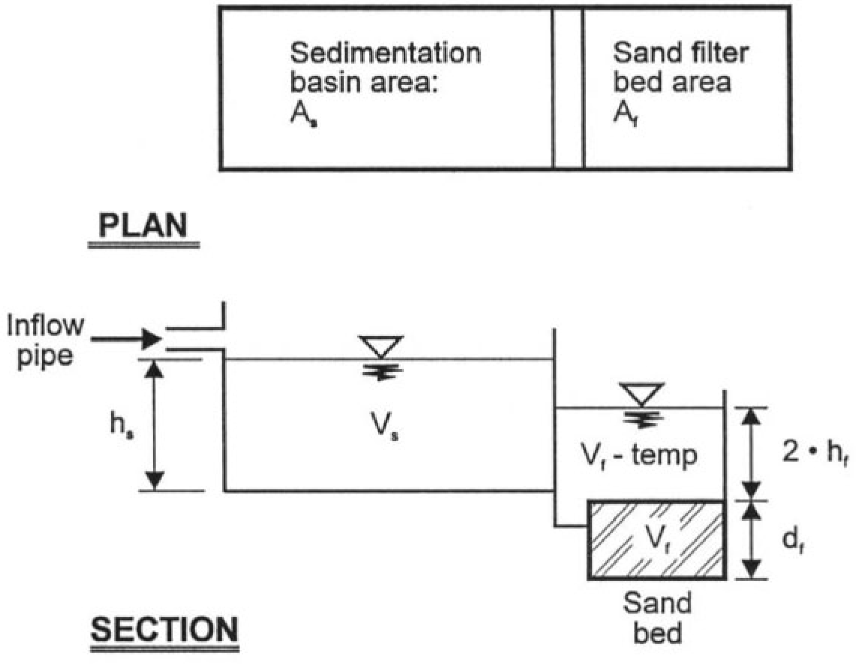
< Compute the temporary storage volume above the filter bed ($V_{f\text{-temp}}$)= 2 (h_f (A_f.

< Compute the remaining volume required for the dry settling basin (V_s) = V_{min} - (V_f + $V_{f\text{-temp}}$). Note: V_s should be approximately to 50% of V_{min}. If not, decrease h_f, and recompute.

< Compute height (h_s) in settling basin chamber = V_s/A_s.

< Check to make sure h_s > 2 (h_f, and h_s\$ 3', if not adjust h_f and repeat procedure.

FIGURE 5.10: SCHEMATIC-SURFACE SAND FILTER



UNDERGROUND SAND FILTER (FIGURE 5.11)

< Compute the water volume within the filter bed (V_f) = A_f (depth of bed & gravel (d_f) (n.

< Compute the minimum wet pool volume in the settling basin (V_w) = A_s (3' minimum.

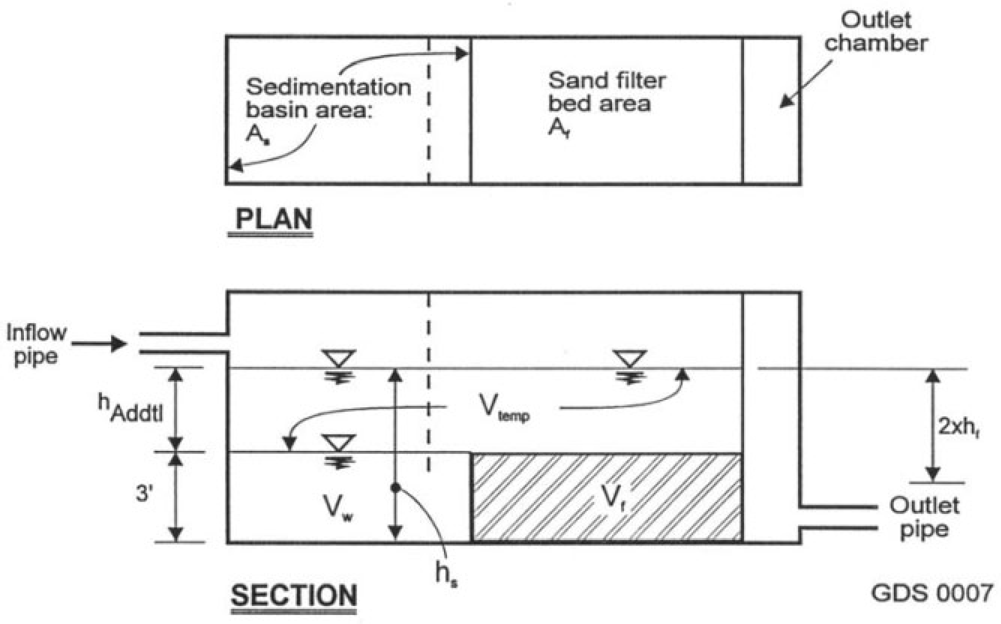
< Compute the temporary storage volume required within both chambers (V_{temp}) = V_{min} - (V_f +V_w).

< Compute the total surface area of both chambers (A_f + A_s).

< Compute additional temporary storage height (h_{addtl}) = V_{temp} /(A_f + A_s).

< Check to make sure $h_{addtl} \$ 2 (h_f$ (from Darcy's Law), if not, decrease h_f and recompute.

FIGURE 5.11: UNDERGROUND SAND FILTER



PERIMETER SAND FILTER (FIGURE 5.12)

< Compute the water volume within the filter bed (V_f) = A_f (depth of bed & gravel (d_f) (n.

< Compute the minimum wet pool volume in the settling basin (V_w) = A_s (2' minimum.

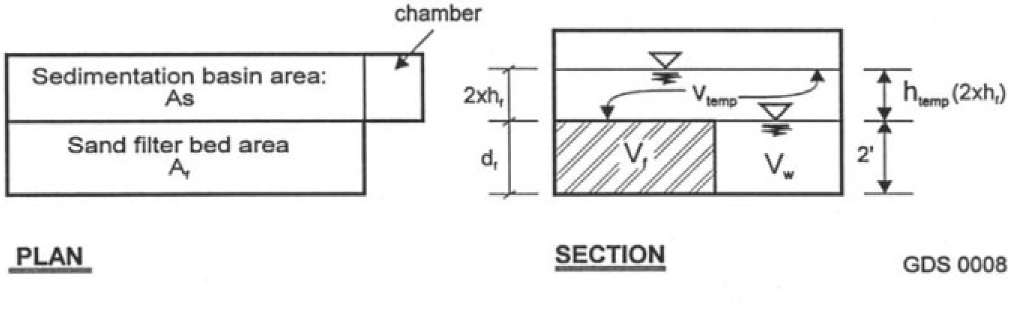
< Compute temporary storage volume required (V_{temp}) = V_{min} - (V_f +V_w).

< Compute the total surface area of both chambers (A_f + A_s).

< Compute temporary storage height (h_{temp}) = V_{temp}/(A_f + A_s).

< Check to make sure h_{temp} \$ 2 (h_f (from Darcy's eq.), if not, decrease h_f and recompute.

FIGURE 5.12: PERIMETER SAND FILTER



POCKET SAND FILTER

< Compute the water volume within the filter bed (V_f) = A_f (depth of bed & gravel (d_f) (n.

< Compute the temporary storage volume required (V_{temp}) = V_{min} - V_f.

< Compute the temporary storage height (h_{temp}) = V_{temp}/A_{avg}, where A_{avg} is the average area of the pocket sand filter.

< Set overflow spillway elevation = h_{temp}.

ORGANIC FILTER MEDIA

< Compute the water volume within the filter bed (V_f) = A_f (depth of bed & gravel (d_f) (porosity (n) (porosity of organic layer, plus gravel/pipe system will vary depending on the practice filter medium, from approximately 0.33 to 0.4).

< Compute the temporary storage volume above the filter bed ($V_{f\text{-}temp}$) = 2 (h_f (A_f.

< Compute the remaining volume required for the settling basin (V_s) = V_{min} - (V_f + $V_{f\text{-}temp}$). Note: V_s should be approximately to 50% of V_{min}. If not, decrease h_f, and recompute.

< Compute height (h_s) in settling basin chamber = V_s/A_s.

< Check to make sure h_s > 2 (h_f, if not adjust h_f and repeat procedure.

5.6 OVERFLOW

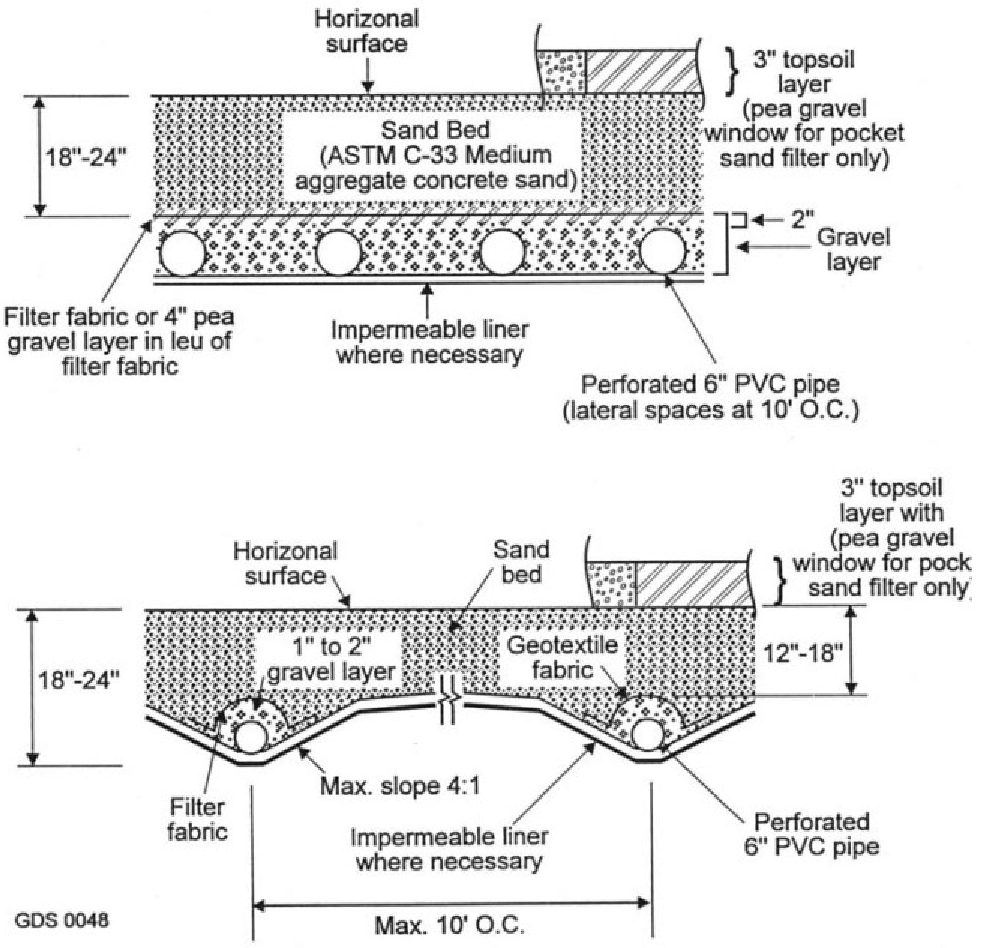
The overflow elements of the filter chamber consist of a flow distribution vault, a sand or organic media filter bed, underdrain piping, a basin liner (whenever necessary to prevent groundwater contamination), and a high flow overflow structure. In some applications the filter bed will have a cover of either vegetation, gravel or a synthetic geotextile-matrix matting.

The flow distribution vault should be designed to spread the flow uniformly across the surface of the filter bed. V-notch weirs, a level broad crested weir or multiple orifice openings are alternatives. Flow should be dispersed in a non-scouring way. The height of the inlet structure should be equal to the filter bed elevation. Rip rap or other suitable erosion protection should be installed immediately below the inlet structure discharge location where necessary.

The filter bed profile (Figure 5.5, 5.6, and 5.13) generally consists of a cover vegetation planted in a 3" topsoil layer above an 18" layer of 0.02"-0.04" diameter, clean concrete sand or organic media, and over a 6" to 11" gravel underdrain system with a 6" perforated pipe collection system.

The sand bed depth recommended above should be considered the final consolidated depth. Depending on moisture content and compactive effort, a 5% increase in depth should be considered. The sand and gravel should be separated by a layer of permeable geotextile fabric, meeting the recommended specifications. Some authorities recommend not using geotextile fabrics to separate layers. Four inches of pea gravel may be substituted for the filter cloth. This allows for an integrated sand/gravel boundary with a higher matrix potential which allows easier water flow from the sand to the gravel. An alternative sand bed profile consists of a top layer of 12" to 18" of 0.02"-0.04" diameter sand. Lateral pipes are placed in trenches and covered with gravel and geotextile fabric. The laterals are underlain by a layer of drainage matting which provides for adequate vertical and horizontal flow.

FIGURE 5.13: SAND BED CROSS SECTIONS



The underdrain system consists of perforated collector and lateral pipe system. Perforations should be 3/8" diameter and should be spaced approximately 6" on-center. The lateral pipes should be spaced at a maximum distance of 10' on-center. Pipes should be adequate to accommodate the weight of the sand and gravel above. Pipes should be 6" PVC, Schedule 40 or greater. The entire underdrain system should have positive drainage, a design slope of at least .5% is recommended. A vertical standpipe should be provided for inspection and cleanout.

The outlet structure may simply be a direct connection of the underdrain piping system to a downstream storm drainage system, channel or waterway. In many cases the outlet structure will be a separate chamber, into which the underdrain system flows. This chamber will then discharge directly to the receiving waters. The outlet chamber may also act to collect overflow drainage associated with larger storm events, as applied in the underground sand filter and the perimeter sand filter.

5.7 SAND AND ORGANIC FILTER SYSTEM MATERIAL SPECIFICATIONS

Specifications are listed in Table 5.5 for many materials frequently used in sand and organic filters. These are typically specifications for materials required for filter practices, alternative localized specifications are available in different jurisdictions.

TABLE 5.5: MATERIAL SPECIFICATIONS

Parameter	*Specification*	*Size*	*Notes*
Sand	Clean AASHTO M-6/ASTM C-33 medium aggregate concrete sand	.02" - .04"	
Peat	Ash content: < 15% pH range: 5.2 - 4.9 Loose bulk density: .12 - .15g/cc		The material must be Reed-Sedge Hemic Peat, shredded, uncompacted, uniform, and clean.
Leaf Compost			Refer to CFS Treatment Systems, Inc (Stewart, 1992).
Underdrain Gravel	AASHTO M-43	½" - 2"	
Geotextile Fabric Between Layers	ASTM D-751 (Puncture Strength - 125 lbs.) ASTM D-1117 (Mullen Burst Strength-400psi) ASTM D-1682 (Tensile Strength - 300 lbs.)	0.8" Thick Equiv. Opening Size - # 80 U.S. Sieve	Maintain 125 gal/min per sq. ft. flow rate.
Impermeable Liner	ASTM D 751 (Thickness) ASTM D 412 (Tensile Strength - 1100 lbs, elongation - 200%) ASTM D 624 (Tear Resistance - 150 lbs/in) ASTM D 471 (Water Absorption - +8 to -2% mass)	30 mil thickness	Liner should be ultraviolet resistant. A geotextile fabric should be used to protect the liner from puncture.
PVC Piping	AASHTO M-278	6" - Rigid Schedule 40	⅜" perf. @ 6" centers, 4 holes per row .

5.8 CONSTRUCTION ELEMENTS

Several specific considerations are important for the construction of sand and organic filtering practices. These include the following:

< Sufficient access to the basin for construction and maintenance is necessary. An access ramp should be provided with a maximum slope of 10% for vegetated ramps, 15% if the slope is stabilized with crushed stone or, 25% if paved.

< Provisions must be made for the removal of sediment (both from the sedimentation basin and filter bed chambers) either on-site in a pre-established location or off-site at an approved and permitted location.

< No runoff should enter the sand filter bed until the upstream drainage area is completely stabilized and site construction is completed. The sedimentation basin may serve as a temporary sediment control basin during site construction with the provision that overflows will bypass the filtration bed. The erosion and sediment control plan must be carefully designed and sequenced to allow for the construction of the filter bed while maintaining erosion and sediment control.

< The top of the filter bed must be constructed **completely level**. Allowance for settlement after initial construction is also required. A geotechnical engineer should specify a minimum and maximum compactive effort based on material (sand, peat, or compost) gradation, moisture content, thickness of the filter bed and design permeability.

< Materials used for construction should meet the specifications outlined in Table 5.5. Materials which might be damaged during construction (such as perforated PVC piping, geotextile liners, etc.) should be stored in a safe location and handled carefully. Exposed piping and accessories should be constructed out of durable, strong materials to avoid susceptibility to damage by vandalism.

< Underground sand filters, facilities within sensitive groundwater aquifers, and filters designed to serve urban hotspots should be tested for water tightness prior to placement of filter layers. Entrances and exits should be plugged and the system completely filled with water to demonstrate water tightness.

< Overflow weirs, multiple orifices and flow distribution slots must be constructed completely level to ensure adequate distribution of design flows.

< Access manholes and/or grates to underground and below grade structures should be provided for each subsurface chamber. Manholes should be in compliance with the standard specifications of the relevant jurisdiction. Manhole diameters should be 30" to meet confined space access criteria (and not be too heavy to manually remove). Aluminum and steel louvered doors provide

excellent access, light and ventilation for routine maintenance operations. Manhole steps should be placed to allow maintenance personnel easy access to structure bottoms. A 5' minimum height clearance (from the top of the sand layer to the bottom of slab) is required for all fixed permanent underground structures. Lifting rings or other suitable element should be provided to lift and replace structure top slabs.

< The main collector pipe for underdrain systems should be constructed at a minimum slope of 0.5%. Observation and clean-out pipes must be provided for all underdrain piping.

< The underground sand filter should be constructed with a dewatering gate valve located just above the top of the sand filter bed. Should the filter bed and/or underdrain system clog completely, the gate valve can be opened to dewater the filter chamber for needed maintenance.

< To help extend the design life of the sand filter bed for the underground sand filter a wide mesh geotextile screen should be placed on the surface of the filter bed to trap the large quantities of trash, litter and organic detritus associated with highly urban areas. During maintenance operations the screen is rolled up, removed and cleaned, and reinstalled.

< Designers specifying a grass cover crop for sand or organic filter beds should choose an appropriate species which will develop a root system which does not inhibit infiltration. Appendix B describes several characteristics of grass. To help ensure that the filter bed will resist clogging on the pocket sand filter, a pea gravel “window” is recommended to cover approximately 10% of the sand bed surface area.

< Many of the alternatives call for the use of filter fabric to separate different layers of filter medium. These filter fabric layers are often the first place to clog with fine sediments. A 4" pea gravel layer may be substituted for filter cloth to separate layers of different materials.

< Whenever possible, sand filters should be visible so that they are easily recognizable as BMPs and can be quickly located for routine inspections. Perhaps the biggest concern with underground facilities is that they are often forgotten and inspections and maintenance are rarely performed.

5.9 MAINTENANCE

Several maintenance considerations are provided below. Table 5.6 presents a recommended inspector’s checklist for stormwater sand filters.

5.9A GENERAL MAINTENANCE ELEMENTS

SEDIMENTATION BASIN

< The sedimentation basin should be cleaned out when the sediment depth exceeds 12". Removal of accumulated paper, trash and debris should be conducted every six months or after major storms.

< Vegetation growing within the sedimentation basin should be limited to 18" in height.

< Corrective maintenance is required for draw-down times exceeding 36 hours (24 hours is the design value). The perforated standpipe or low flow orifices should be checked and cleaned as necessary.

< Corrective maintenance is necessary for the sediment trap, when provided, if it does not drain within 96 hours.

< Access manholes, gate valves, flumes and other facilities shall be kept clean and ready for use.

FILTRATION BED COMPONENTS

< Grass clippings and other organic debris from landscape areas on the catchment should be bagged and removed from the site to prevent them from washing into and contaminating the sediment and filter chambers.

< Removal of silt should be conducted when accumulation exceeds approximately one-half (1/2) inch. When the filter layer will no longer draw down within the design period, the top layer of sand or organic media, sacrificial failure zone, or ballast gravel must be removed and replaced with new materials conforming to the original specifications. Any discolored or contaminated material, below the surface shall also be removed and replaced.

< Each sand or organic media filter should be inspected in accordance with the guidance in Table 5.6. Materials deposited on the surface of the filter chamber (e.g., trash and litter) should be removed manually . When the capacity of the filter bed begins to diminish due to surface clogging, manual removal of the top few inches of discolored material should be done. In some cases, manual manipulation or roto-tilling of the surface may restore filtration capacity. Removed material should be replaced with fresh sand or organic media meeting the original design specifications. The contaminated material should be dewatered and disposed of at a pre-approved and permitted location.

< Urban hotspot land uses, particularly automotive uses with heavy oil/grease loadings, should conduct semi-annual clean-out of the sedimentation chamber and more frequent inspection of the filter bed.

< Vegetation growing within the basin should not exceed 18" in height.

5.9B SPECIFIC MAINTENANCE ELEMENTS

UNDERGROUND SAND FILTER

< The water level in the filter chamber should be monitored on a quarterly basis and after large storms for the first year of service. A log should be maintained documenting the results of the rate of dewatering and water depth of each observation. After the first year, monitoring may be reduced to a semiannual basis.

< The sedimentation chamber must be pumped out when the sediment depth reaches 12". Oil on the surface should be removed separately and recycled, the remaining material may be removed by vacuum pump and disposed of at an approved and permitted site.

PERIMETER SAND FILTER

< During the first year of operation, the system should be inspected after each major storm to ensure that the system is functioning properly. Inspections may be reduced to a semiannual basis afterwards.

< Trash collected on the grates protecting the inlets should be removed on a regular basis to preserve the inlet capacity of the facility.

PEAT-SAND FILTER SYSTEM

< Periodic mowing is required for the grass cover crop of the peat-sand filter bed. Grass clippings should be removed. Mowing frequency is largely up to the owner of the system, lower cutting height (less than 6") can be achieved by a conventional rotary lawn mower with a grass catcher. Mowing may need to be as frequent as weekly during the peak growing season. Higher cutting levels can be achieved with a sickle mower, but grass raking will be required. Mowing using this method may only be required 3 to 4 times per year (Galli, 1990).

< Regular inspection should be conducted, particularly in the first year of operation, to ensure that the filter surface is not scouring or otherwise failing. Reseeding of areas sparsely covered with grass may be required.

COMPOST FILTER SYSTEM

< Annual maintenance of the compost filter bed consists of removing an accumulated sediment layer from the surface of the filtration bed and roto-tilling the compost media itself.

< The compost bed should be replaced with fresh compost every 3 to 4 years, or as heavy metal concentrations within the compost media exceed EPA's 503 Sewage Sludge Regulations for "clean sludge."

TABLE 5.6: RECOMMENDED INSPECTION CHECKLIST FOR STORMWATER SAND FILTERS (ADAPTED FROM SHAVER AND BELL, 1996)

Inspection Item	*Inspection frequency*	*Disposition*
Debris Cleanout Inlets and outlets clear of debris? Filtration facility clear of debris?	Quarterly	Identify areas requiring cleanout and severity of buildup.
Vegetation Drainage area to facility stable? Area mowed, and clippings removed? Cover vegetation less than 18"?	Monthly during growing season, Quarterly during non-growing season.	Identify evidence of erosion, vegetation needing mowing, or unstabilized areas.
Filter Bed Chamber Evidence of filter bed surface clogging? Drainage area to facility clear of oil/grease sources? Sediment buildup on surface less than 1 inch?	Semi-annual	Identify clogged filter bed, source area contributions, and actions required.
Sedimentation Chamber Permanent pool wet? Evidence of leaking? Sediment buildup less than 12 inches?	Semi-annual	Identify leaking chamber and sediment level, specify actions required.
Structural Components Evidence of structure deterioration? Inlet grates, pipes, etc in good condition? Evidence of spalling or cracking of concrete?	Annual	Identify problems, specify actions required.
Outlets/Overflow Spillway Evidence of clogging of outlet pipe? Evidence of downstream erosion? Evidence of underdrain piping failure?	Annual	Identify problems, specify actions required.

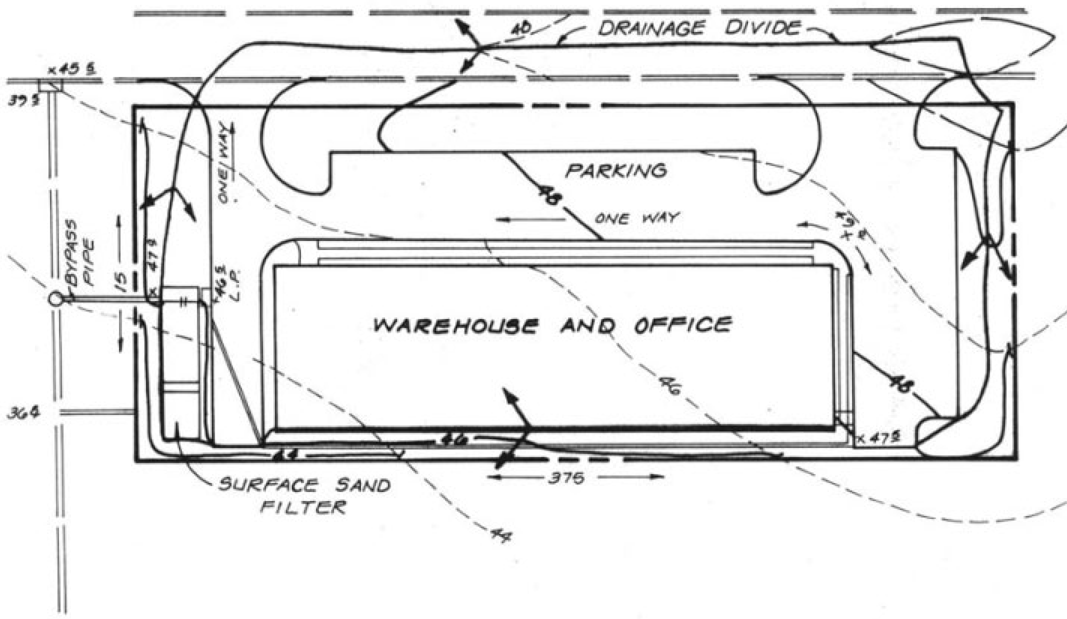
5.10 DESIGN EXAMPLE

Given: 1.3 acre site (see Figure 5.14)
Light Industrially zoned (predominately building & parking)
0.56 ac - paving
0.03 ac - sidewalk
0.39 ac - flat roof
0.02 ac - filter practice
0.35 ac - pervious

% Impervious = (0.56 + 0.35 + 0.39 + 0.02)1.35] (100 = 74% *

* Note: impervious area < 75%, use equation 5.2 to size sedimentation chamber area (for I $ 75% use equation 5.2.1).

FIGURE 5.14: DESIGN EXAMPLE - SITE PLAN



1. Compute WQV

1.0" Rainfall (from Chapter 2, Section 2.7)
From Table 2.13

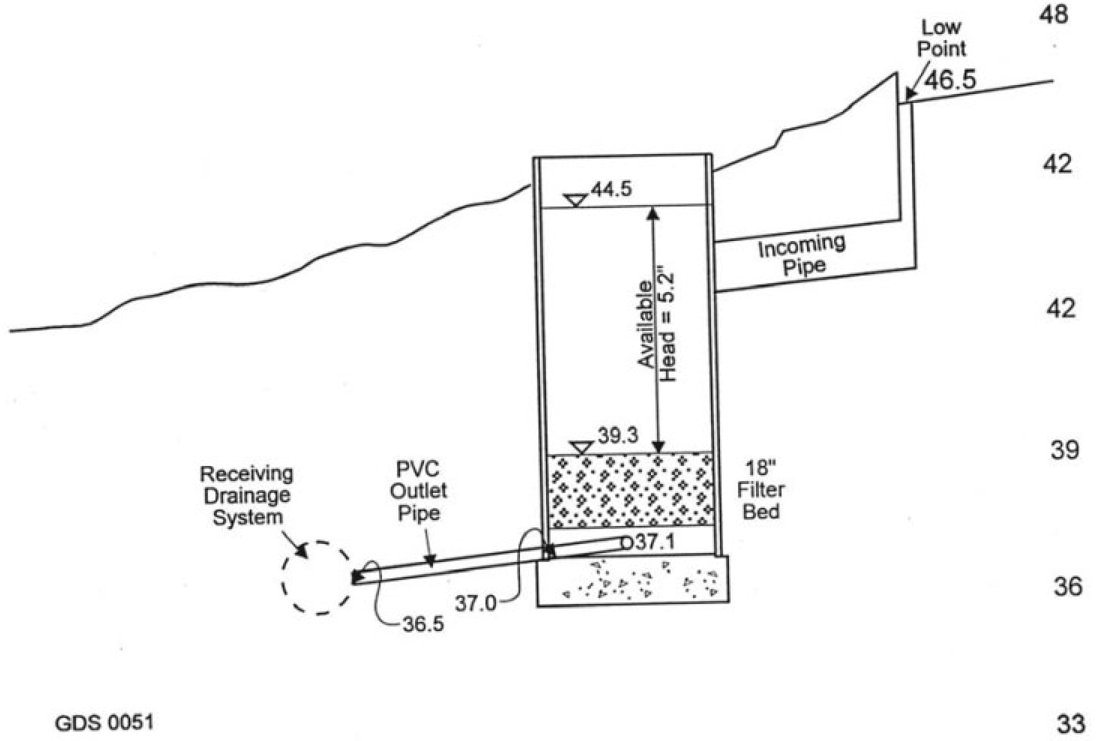
Flat roofs: $R_v = 0.84$
Large impervious area: $R_v = 0.97$
Small imp. area (streets): $R_v = 0.70$
Filter surface area $R_v = 1.00$
Pervious areas (silty soils) $R_v = 0.11$

Weighted R_v = [0.56 (.97) + 0.03 (.70) + 0.39 (.84) + 0.02 (1.0) + 0.35 (.11)] / 1.35
$R_v = 0.70$

From Equation 2.2 WQV = 1.0" (0.70 = 0.70"
WQV = 0.70" ((1.35 ac/ 12"/ft) (43,560 ft^2/ac = 3,430 ft^3

Compute maximum head available (Figure 5.15)
Low point in street = el. 46.5 (subtract 2' to pass Q_{10} discharge) el. 44.5
Inv. @ storm drain system = el. 36.5
Inv. out of filter bed = el. 37.0
Top of filter bed = el. 39.3
allowable depth (2 (h_f) = 44.5 - 39.3 = 5.2 ft. Use 2 (h_f = 5 ft

FIGURE 5.15: AVAILABLE HEAD DIAGRAM



2. Compute WQV peak discharge (Q_p)

From Chapter 2, Equation 2.3 (modified TR-55 methodology)

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

$$CN = 1000 / [10 + 5\ (1.0") + 10\ (.70") - 10\ ((.70")^2 + 1.25\ (.70")\ (1.0"))^{1/2}]$$

CN = 96.93 Use <u>CN = 97</u>

Use $t_c = 0.1$ hr.

From TR-55, Chapter 4: I_a = 0.062, I_a/P = 0.092 / 1.25 = 0.05
From Exhibit 4 - II (see Figure 5.16) q_u = 1040 csm/in
For 1.0" rainfall
Q_p = 1040 csm/in (.70") (1.35 ac / 640 ac/mi^2) = 1.5 cfs

Compute 10 year discharge for bypass
(conventional TR-55 methodology)

For 74% impervious, B soils (CN = 98 for Imp., CN = 61 for open space)
CN = .74 (98) + .26 (61) = 88.4 Use CN = 88
Use t_c = 0.1 hr.
From TR-55, Chapter 4: I_a = 0.273, P = 5.0", I_a/P = 0.273/5.0 = 0.055
From Exhibit 4-II, q_u = 1040 csm in
For 5.0" rainfall
Q_{10} = 1040 csm/in (3.67") (1.35 ac/640 ac/mi^2) = 8.0 cfs

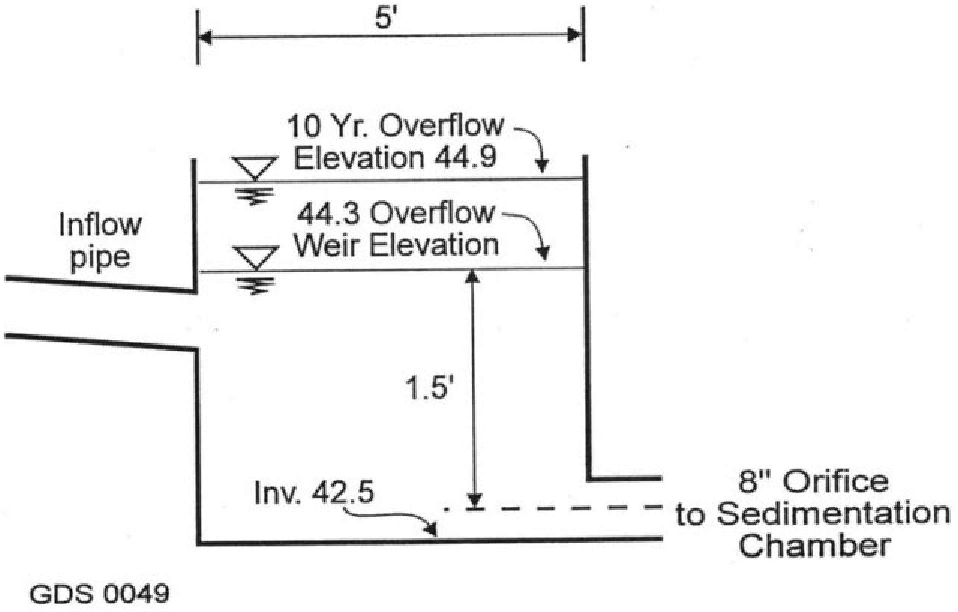
3. Size flow diversion structure
(see Figure 5.16)
Size low flow pipe to pass 1.5 cfs with 1.5' of head
$Q = C (A (2gh)^{1/2}$
1.5 cfs = 0.6 (A (2 (32.2 ft/sec^2 (1.5')$^{1/2}$
A = 0.25 ft^2 = (d^2/4: d = 0.57' Use 8" (over sized)

10 year overflow elevation = 44.3
Set low flow orifice inv. el. @ 44.3 -[1.5' + (½ (8" (1 ft/12")] = 42.47
Set at el. 42.5

Compute overflow elevation in diversion structure (weir equation)
(10 year overflow = 8.0 cfs)
$Q = CLh^{3/2}$
8.0 cfs = 3.1 (5.0 ft ($h^{3/2}$
h = 0.64 ft Elevation = 44.3 + 0.6 = 44.9

Size outlet pipe: with 2.0' of head.
$Q = C (A (2gh)^{1/2}$
8.0 cfs = 0.6 A (2 (32.2 ft/sec^2 (2.0')$^{1/2}$
A = 1.17 ft^2 = (d^2/4: d = 1.22' Use 15" RCP outlet
Set invert @ elev. 44.9 -[2.0' + (½ (15" (1 ft/12") = 42.28: Use 42.3

FIGURE 5.16: FLOW DIVERSION STRUCTURE



4. Size sand filter bed

From Equation 5.4: $A_f = WQV\ ((d_f)/k((h_f + d_f)(t_f)$
$A_f = 3{,}430\ ft^3\ (\ (1.5')\ /\ [3.5\ (\ (2.5 + 1.5)\ (\ (40\ hr/24\ hr/day)]$
$A_f = 220.5\ ft^2 = 12'$ by 18.4': Use <u>12' by 20' (= 240 ft^2)</u>
where: $d_f = 1.5'$
$h_f = 2.5'$
k = 3.5 ft/day (Table 5.4)
t = 40 hr

5. Size sedimentation chamber

From Equation 5.2 (Camp-Hazen equation) $A_s = 0.066\ (\ (WQV)$
$A_s = 0.066\ (\ (3{,}430\ ft^3) = 301\ ft^2$
for 12' width $226\ ft^2 / 12' = 18.9$ ft: Use <u>12' by 20 ft (= 240 ft^2)</u>

6. **Compute V_{min} = ¾ (WQV**
   (Equation 5.3)

   V_{min} = ¾ ((3,430 ft^3) = 2,573 ft^3

7. **Compute volume within practice**
   Compute volume within the filter bed (V_f): $V_f = A_f$ ((d_f) ((n)
   V_f = 240 ft^2 (2.0 (0.4 = 192 ft^3
   Compute temporary storage above filter bed ($V_{f\text{-}temp}$): $V_{f\text{-}temp}$ = 2 (h_f (A_f
   $V_{f\text{-}temp}$ = 2 (2.5' (240 ft^2 = 1,200 ft^3
   Compute remaining volume for sedimentation chamber (V_s):
   $V_s = V_{min} - (V_f + V_{f\text{-}temp})$
   V_s = 2,573 ft^3 - (192 ft^3 + 1,200 ft^3) = 1,181 ft^3 (note: V_s is approx 50% of V_{min})
   Compute height in sedimentation chamber (h_s): $h_s = V_s/A_s$
   h_s = 1,181 ft^3 / 240 ft^2 = 4.9 ft set h_s = 5.0 ft (h_s>2 (h_f, and h_s > 3')
   (5.0' is less than available head of 5.2', OK)

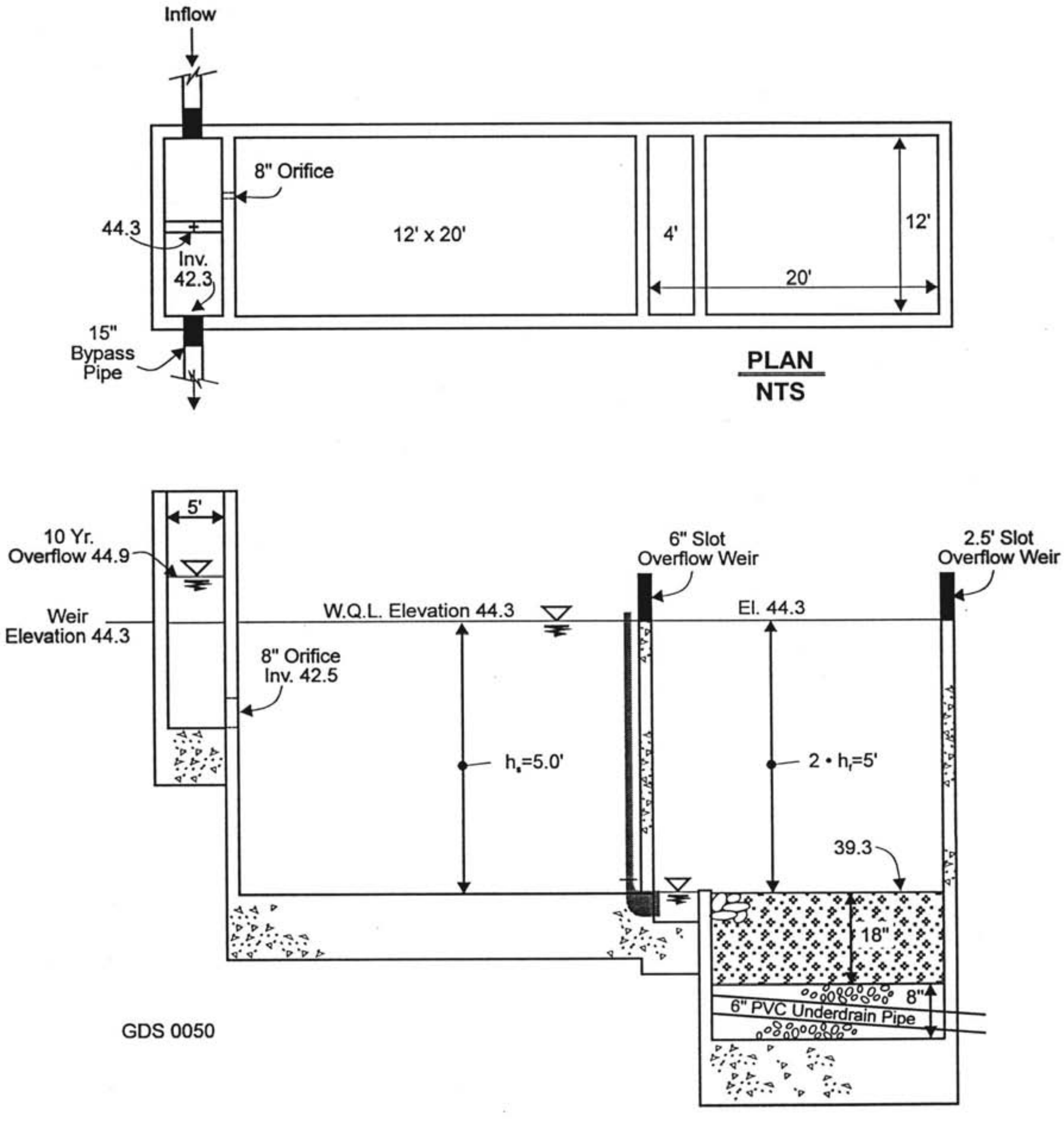
   See Figure 5.17

8. **Compute overflow weir sizes:**

From sedimentation chamber (size to pass ⅔ of WQV peak discharge)
   Q_p = 2.0 cfs
   Weir equation: $Q_w = CLh^{3/2}$
   ⅔ (2.0 cfs = 3.1 (L $(1.0)^{3/2}$, L = 0.4': Use <u>L = 0.5 ft</u>

From filter bed chamber (size to pass ⅓ of WQV peak discharge)
   ⅓ (2.0 cfs = 3.1 (L $(0.2)^{3/2}$, L = 2.4': Use <u>L = 2.5 ft</u>

FIGURE 5.17: PLAN/PROFILE: SURFACE SAND FILTER

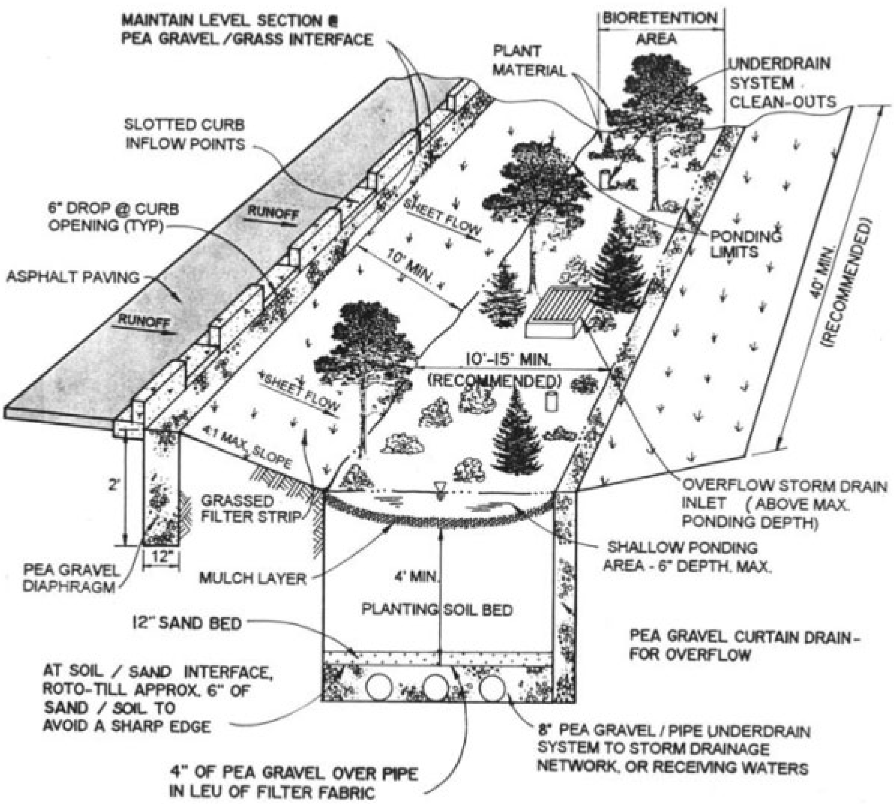


CHAPTER 6
KEY DESIGN ELEMENTS OF BIORETENTION SYSTEMS

6.1 INTRODUCTION

The bioretention concept was originally developed by the Prince George's County, Maryland, Department of Environmental Resources in the early 1990's as an alternative to traditional BMP structures (ETAB, 1993). Bioretention is a practice to manage and treat stormwater runoff using a conditioned planting soil bed and planting materials to filter runoff stored within a shallow depression. The method combines physical filtering and adsorption with biological processes. The system consists of a flow regulation structure, a pretreatment filter strip or grass channel, a sand bed, pea gravel overflow curtain drain, a shallow ponding area, a surface organic layer of mulch, a planting soil bed, plant material, a gravel underdrain system, and an overflow system (Figure 6.1).

FIGURE 6.1: THE BIORETENTION CONCEPT SYSTEM COMPONENTS



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DESIGN MANUAL FOR THE USE OF BIORETENTION
IN STORMWATER MANAGEMENT, 1993

Bioretention is intended as a water quality control practice only and therefore should generally be located off-line. Several methods are presented for diversion of the Water Quality Volume (WQV) into these facilities. In some instances online applications may be appropriate where the drainage area is limited or where insufficient room is available to accomplish a diversion of the WQV. In these instances, the designer must accommodate the larger storms with sufficient erosion protection measures and adequate overflow provisions.

6.2 ALTERNATIVE APPLICATIONS

Bioretention can be applied to almost all development situations except perhaps the ultra-urban condition (where pervious surfaces are likely to be limited to 5% or less). The concept is applicable for residential land uses, either on private lots or within common open space, and is certainly applicable for treating parking lot runoff. The practice is also applicable for roadways where adequate space is available for off-line implementation. Finally, bioretention facilities are good candidates for pervious surface treatment, such as golf courses (See Chapter 3 and Figure 6.2).

6.3 SYSTEM COMPONENTS

Each component of the bioretention system is integral to the long term success of the practice and must be evaluated carefully in the overall design.

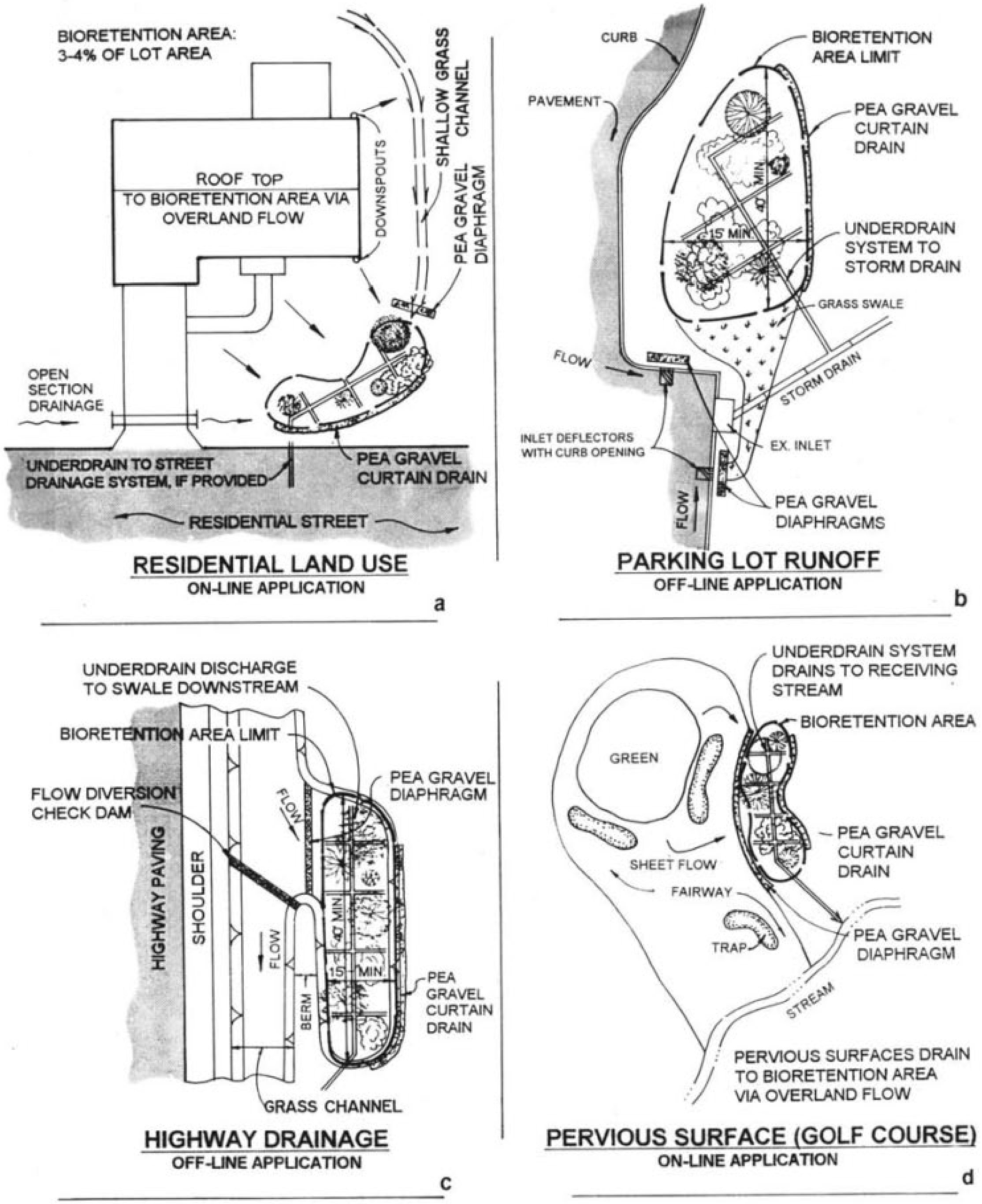
6.3A FLOW REGULATION AND/OR INTAKE STRUCTURE

For off-line applications, this element is responsible for ensuring that the WQV is captured and diverted to the practice for treatment. The isolation/diversion technique within the drainage system (described in Chapter 5) is one method for diverting the WQV to the bioretention system. Other principal techniques are described later in this chapter. The intake structure is equally important for both off-line and on-line applications to insure non-erosive velocities with adequate protection against clogging.

6.3B PRETREATMENT FILTER STRIP

This component is necessary to aid in reducing incoming velocities as well as capturing coarser sediment particles to extend the design life and reduce replacement maintenance of the bioretention system. The pretreatment method may incorporate other techniques, such as a sand or gravel diaphragm to aid in extending the design life of the practice.

FIGURE 6.2: THE BIORETENTION CONCEPT SYSTEM COMPONENTS



6.3C PEA GRAVEL OVERFLOW CURTAIN DRAIN

This element provides an overflow feature to help augment infiltration into the planting soil bed. This allows a greater portion of the WQV to be treated by the facility.

6.3D SHALLOW PONDING AREA

The shallow ponding area just above the mulch layer and vegetation root zone provides surface storage for a percentage of the WQV. This area also allows for particulate settling during the detention period allowing finer particles to settle on the surface of the mulch layer.

6.3E SURFACE MULCH LAYER

The mulch layer provides an environment for plant growth by maintaining moisture and allowing for the decomposition of organic matter. The surface layer acts as a filter for finer particles still in suspension and maintains an environment for the microbial community to help breakdown urban runoff pollutants.

6.3F PLANTING SOIL BED

The planting soil bed provides the region for water and nutrients for the planting material above. The voids within the soil provide additional storage for the WQV. The soil particles can adsorb various pollutants through cation exchange.

6.3G PLANTING MATERIAL

The plant material takes up some nutrients and other pollutants, and available water through evapotranspiration. The use of native plant material, combined with a minimum planting area size provides cover for wildlife and creates a micro-environment within the urban landscape.

6.3H SAND BED

The sand bed is provided to keep finer soil particles from washing out through the underdrain system, and it provides an aerobic sand filter as a final “polishing” treatment media.

6.3I GRAVEL UNDERDRAIN SYSTEM

This component is utilized to collect and distribute treated excess runoff. A properly designed underdrain system helps keep the soil from becoming saturated. The underdrain system consists of a gravel layer with a 4" or 6" perforated piping system (maintaining a 2" cover of gravel over the pipe).

6.3J OVERFLOW SYSTEM

The overflow system provides a means to convey larger storm flow volumes to the downstream receiving waters or drainage system. This component usually consists of a conventional drainage catchbasin, inlet, or overflow channel located slightly above the shallow ponding limit.

6.4 FLOW REGULATION

An off-line design is recommended for most bioretention applications. Larger storms are likely to cause erosion problems at the inflow points, disrupt the mulch layer, and otherwise negatively affect the plant material. For situations where it is not possible to separate the WQV from the larger storms an on-line design may be utilized. For these applications, it is imperative that adequate precautions are taken to protect the inlet, mulch layer, and plant material (e.g., stone stabilization or synthetic erosion protection materials). On-line designs should only be considered for very small drainage areas.

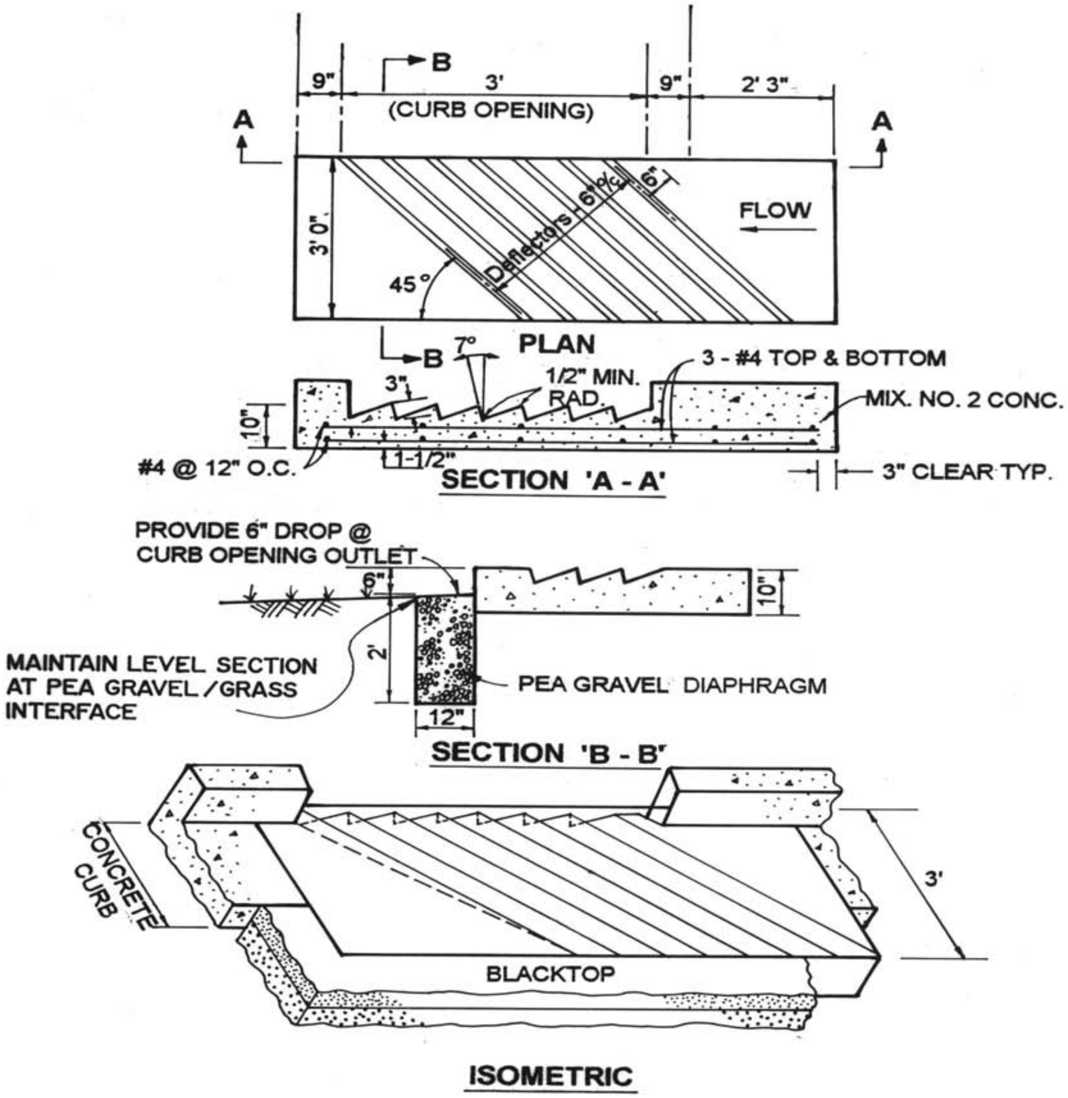
The basic flow regulation design objective is to capture and divert the WQV to the bioretention area and "bypass" the larger storms to the downstream storm drainage system, detention pond or receiving water. In some cases, utilizing bioretention structures for treating the WQV throughout a site or subcatchment may also provide significant runoff attenuation to effectively manage smaller “quantity control” storms as well, and therefore, the need for downstream detention facilities may be eliminated. Refer to the Chapter 2 discussion of the Rainfall Frequency Spectrum and stormwater management control points for more information. Table 6.1 presents several alternative techniques for diversion of the WQV.

Table 6.1: Four Methods for WQV Diversion to Bioretention facilities

Divert runoff from an enclosed storm drain system using the peak flow rate (Q_p) methodology described in Chapter 5 (Figure 5.7).
Divert runoff from curbed pavements using a curb opening with slotted deflector grooves in the gutter pan (ETAB, 1993). Utilize a 6" drop below the curb, with a pea gravel diaphragm, as illustrated in Figures 6.2b and 6.3.
Divert runoff from curbed parking lots utilizing a slotted curb with limited width and design the parking area grades to divert the WQV into the bioretention area. Once the capacity of the slotted curb is exceeded, additional runoff bypasses the facility to flow into a downstream storm drain inlet or channel. This method utilizes a portion of a parking area for temporary ponding and may not be acceptable for areas with limited parking (Figure 6.4).
Divert runoff from an open conveyance channel into the bioretention area. A log, concrete curb stop or other structural measure in the form of a check dam backs-up flowing water to a 6" maximum depth which then flows into the adjacent bioretention area. Once the ponded water reaches the design capacity, the water overflows the checkdam and proceeds downstream. (See Figure 6.2c for application of this technique).

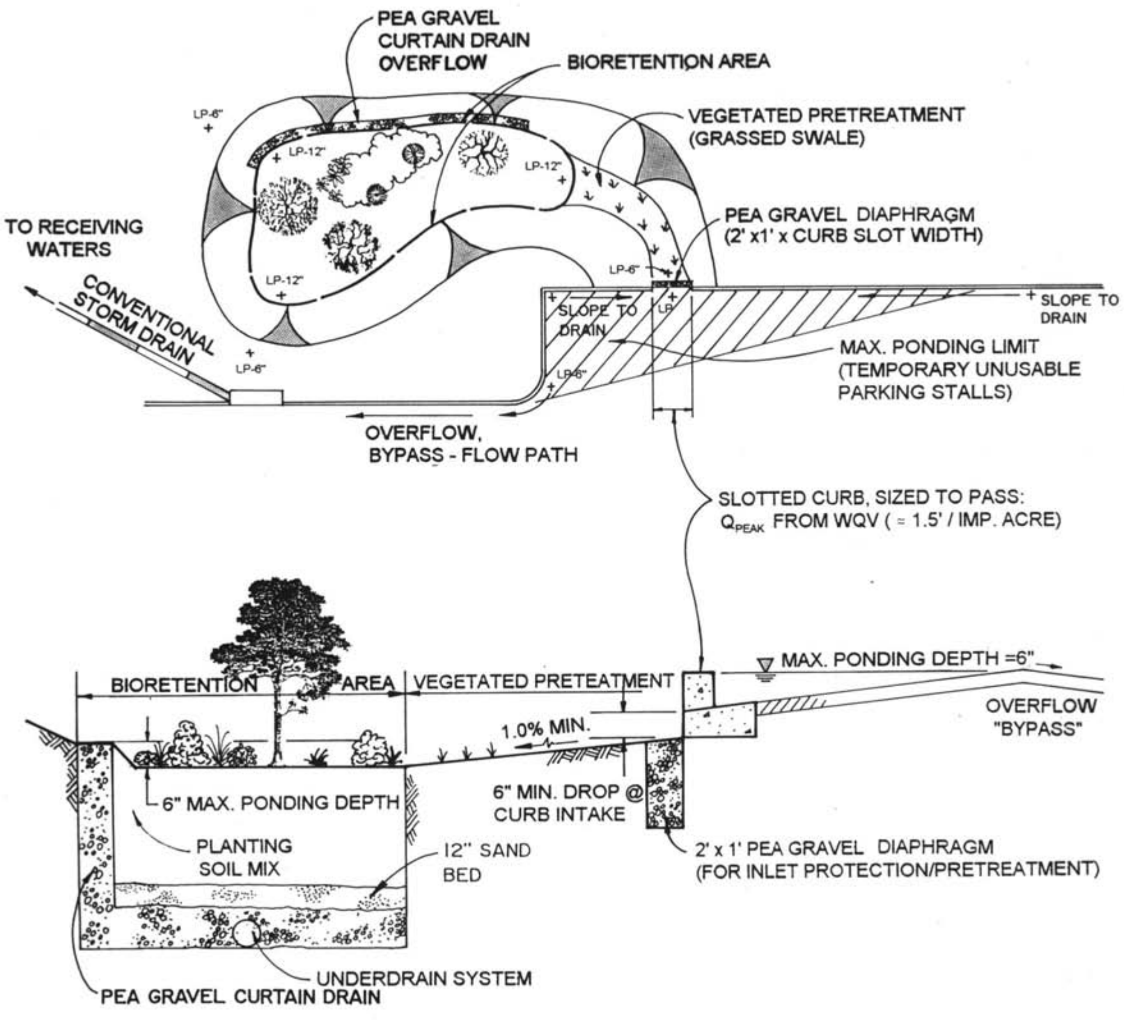
On-line design applications should be limited to a maximum drainage area of 0.5 acres (See Figure 6.2a for an example of this application). The designs should incorporate adequate overflow measures to accommodate larger flows. A yard inlet storm drainage structure, with the throat opening held 6" above the top of the mulch layer is one technique for handling overflow.

FIGURE 6.3: INLET DEFLECTOR DETAIL



SOURCE: ADAPTED FROM PRINCE GEORGE'S COUNTY DESIGN MANUAL FOR THE USE OF BIORETENTION IN STORMWATER MANAGEMENT, 1993

FIGURE 6.4: BIORETENTION AREA WITH SLOTTED CURB FLOW DIVERSION SYSTEM



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6.5 PRETREATMENT

The primary pretreatment technique for bioretention facilities is through the use of a grass filter strip or grass channel. For applications where runoff enters the bioretention system through sheet flow, such as from parking lots, or residential back yards, a grass filter strip with a pea gravel diaphragm is the preferred pretreatment method. The length of the filter strip depends on the drainage area, imperviousness, and the filter strip slope. Table 6.2 gives some sizing guidelines as a function of inflow approach length, land use, and slope. The minimum filter strip length should be 10 feet.

TABLE 6.2: PRETREATMENT FILTER STRIP SIZING GUIDANCE

Parameter	*Impervious Parking Lots*				*Residential Lawns*				*Notes*
Maximum inflow approach length (feet)	35		75		75		150		
Filter strip slope	#2%	$2%	#2%	$2%	#2%	$2%	#2%	$2%	Maximum slope=6%
Filter strip minimum length	10'	15'	20'	25'	10'	12'	15'	18'	

For applications where concentrated (or channelized) runoff enters the bioretention system, such as through a slotted curb opening, a grassed channel with a pea gravel diaphragm is the preferred pretreatment method. The length of the grass channel depends on the drainage area, land use, and channel slope. Table 6.3 gives sizing for grass channels leading into a bioretention facility for a one acre drainage area. These values are based on approximately 10% of the stand alone BMP design criteria (see Chapter 7, Section 7.5: Grass Channel Design Procedure). The minimum grassed channel length should be 20 feet.

TABLE 6.3
PRETREATMENT GRASS CHANNEL SIZING GUIDANCE FOR A 1.0 ACRE DRAINAGE AREA

Parameter	*# 33% Impervious*		*Between 34% & 66% Impervious*		*$ 67% Imperious*		*Notes*
Slope	#2%	$2%	#2%	$2%	#2%	$2%	Maximum slope = 4%
Grassed channel min. length (feet)	25	40	30	45	35	50	Assumes a 2' wide bottom width

The pea gravel diaphragm is designed to slow the velocity and aid in spreading out the flow entering the practice. In addition, this component captures the coarser-grained sediments. It is anticipated that the pea gravel diaphragm will exhibit clogging within the first three to four years after installation and may require periodic flushing and/or replacement. The maintenance schedule of the pretreatment measures are discussed further in this chapter.

6.6 FILTER MEDIA

Bioretention facilities are sized based on the consistent sizing criteria reviewed in Chapter 5 as derived from Darcy's Law by the City of Austin TX. (City of Austin, TX, 1988).

6.6A FILTER BED SURFACE AREA

Since the bioretention concept incorporates a gravel underdrain system and a porous soil filter medium and sand bed, runoff entering the shallow ponding area will slowly percolate through the soil bed in a fashion similar to other filter practices.

Equation 5.4 is utilized to estimate the minimum surface area and then the volume capacity is checked against that required to treat the WQV:

From Chapter 5:

$A_f = WQV\ ((d_f)/k((h + d_f)(t_f)$

where:

A_f = Surface area of the bioretention planting bed (ft^2)

WQV = Water quality treatment volume (ft^3)
d_f = Planting soil bed depth (ft)
k = Coefficient of permeability for planting soil bed (ft/day)
h = Average height of water above the bioretention bed (ft); h_{avg} = ½ (h_{max}
t_f= Time required for the Water Quality Treatment Volume (WQV) to filter through the planting soil bed

< WQV is computed using the procedure outlined in Chapter 2
< d_f is four(4) feet
< k = 0.5 ft/day: Median value of a silt loam (Hwang, 1981) h is equal to 3", assuming a maximum ponding depth of 6" above the planting soil bed
< A value of 72 hours is recommended for the filter drawdown time (t_f)

Derivation of bioretention facility sizing criteria:

For a one (1) acre site which is 100% Impervious (R_v = 0.95)
WQV = [1.0" ((0.95)/ (12"/ft)] ((43,560 ft^2/ac) = 3,449 ft^3
k = 1.0 ft/day
d_f = 5' (4' soil + 1' sand bed)
h = 3" = 0.25'
t_f = 2 days
A_f = 3,449 ft^3 (5' /[(0.5 ft/day) ((5.25 ft) ((3 days)] = 2,190 ft^2
% of site area = 2,190/43,560 (100 = 5.0

Therefore, use the following equation for sizing the bioretention surface area:

A_f = D.A. (5.0% (R_v **Equation 6.1**
where,

A_f is the required surface area of the bioretention facility, D.A. is the drainage area and R_v is the volumetric runoff coefficient, which is computed using the methods outlined in Chapter 2.

6.6B FILTER BED WATER BALANCE EVALUATION

A water balance calculation was conducted to check the surface area sizing criteria stated above. The purpose of the calculation is to see how much runoff the system can accommodate through temporary ponding, infiltration and evaporation verses how much runoff will by-pass the system. The calculation was evaluated over a 72 hour duration. 72 hours is the generally established maximum ponding time within infiltration practices in the Mid-Atlantic region (Maryland DNR, 1984). The water balance simulation is based on a spreadsheet computer program originally developed by Engineering Technologies Associates, Inc for Prince George's County for the development of the original bioretention design manual (ETAB, 1993).

The water balance computations are run at one-hour intervals for the 72 hour duration. The simulation infiltrates runoff from the ponded area to the planting soil/sand bed, and then into the gravel/pipe underdrain system. Because an underdrain is provided, the infiltration limitations of the in-situ soils are not evaluated. Runoff by-passes the bioretention area once the surface ponding volume is exceeded.

Percent of WQV captured by bioretention facility:

For a one (1) acre, 100 % impervious site, a bioretention facility should have a surface area as follows:

A_f = (.95) ((43560) ((0.05) ' 2,069 ft^2
Using a bioretention area with dimensions: 25' x 83' x 5'
8" gravel/pipe underdrain
12" wide gravel overflow curtain drain
6" deep shallow ponding surface area
12" sand bed
.5 ft/day (.25"/hr) infiltration rate for the planting soil bed
32.0 ft/day (16"/hr) infiltration rate for underdrain system
6 hour rainfall event

Results:
Approximately 70% of the WQV is accommodated by the bioretention facility (through surface ponding and infiltration). This corresponds well with the minimum volume required for sand and organic filter practices (V_{min} = ¾ (WQV). The ponded surface volume infiltrates within 30 hours, but there is residual moisture within the soil planting bed after 72 hours.

In order to maintain a suitable micro-environment and to help simulate conditions which exist within an existing forest community, bioretention facilities must have a minimum area coverage. The sizing criteria presented above ensures the necessary treatment area and volume to accommodate the WQV, but additional criteria (Table 6.4) are necessary to assure the survival and success of the planted material.

TABLE 6.4 : RECOMMENDED MINIMUM SIZING GUIDANCE FOR BIORETENTION FACILITIES (ADAPTED FROM ETAB, 1993)

- Minimum width of 10 feet
- Minimum length of 15 feet
- For widths greater than 10 feet, maintain a length to width ratio of 2:1
- Maximum shallow ponding depth of 6 inches
- Minimum planting soil bed depth of 4 feet (with 12" sand bed)

The minimum width allows for random spacing of trees and shrubs, it also permits planting densities which help create a micro-environment where stresses from urban stormwater pollutants are minimized. The 2:1 length to width ratio maintains a longer flowpath for the settlement of particulates and maximizes the edge-to-interior ratio. The maximum ponding depth of 6" provides surface storage for stormwater runoff (approximately 40% of WQV) but is not so deep as to adversely affect plant health. The 6" depth also will dissipate within a reasonable time (less than 3 days) which maintains flexibility in species selection, and minimizes the likelihood that the bioretention area will become a breeding ground for mosquitoes. The four foot planting soil bed depth is sized to provide adequate storage for the WQV, suitable capacity for root system growth and adequate moisture in the soil during dryer periods (ETAB, 1993).

6.6C PLANTING SOIL BED CHARACTERISTICS

The characteristics of the soil for the bioretention facility are perhaps as important as the facility location, size, and treatment volume. The soil must be permeable enough to allow runoff to filter through the media, while having characteristics suitable to promote and sustain a robust vegetative cover crop. In addition, much of the nutrient pollutant uptake (nitrogen and phosphorus) is accomplished through adsorption and microbial activity within the soil profile. Therefore, the soils must balance soil chemistry and physical properties to support biotic communities above and below ground.

The planting soil should be a sandy loam, loamy sand, loam (USDA), or a loam/sand mix (should contain a minimum 35 to 60% sand, by volume). The clay content for these soils should by less than 25% by volume (EQR, 1996; ETAB, 1993). Soils should fall within the SM, ML, SC classifications or the Unified Soil Classification System (USCS). A permeability of at least 1.0 feet per day (0.5"/hr) is required (a conservative value of 0.5 feet per day is used for design). The soil should be free of stones, stumps, roots, or other woody material over 1" in diameter. Brush or seeds from noxious weeds, such as Johnson Grass, Mugwort, Nutsedge, and Canadian Thistle should not be present in the soils. Placement of the planting soil should be in lifts of 12 to 18", loosely compacted (tamped lightly with a dozer or backhoe bucket). The specific characteristics are presented in Table 6.5.

TABLE 6.5: PLANTING SOIL CHARACTERISTICS (ADAPTED FROM EQR, 1996; ETAB, 1993)

Parameter	*Value*
pH range	5.2 to 7.00
Organic matter	1.5 to 4.0%
Magnesium	35 lbs. per acre, minimum
Phosphorus (P_2O_5)	75 lbs. per acre, minimum
Potassium (K_2O)	85 lbs. per acre, minimum
Soluble salts	# 500 ppm
Clay	10 to 25%
Silt	30 to 55%
Sand	35 to 60%

6.6D MULCH LAYER

The mulch layer plays an important role in the performance of the bioretention system. The mulch layer helps maintain soil moisture and avoids surface sealing which reduces permeability. Mulch helps prevent erosion, and provides a micro-environment suitable for soil biota at the mulch/soil interface. It also serves as a pretreatment layer, trapping the finer sediments which remain suspended after the primary pretreatment.

The Mulch layer should be standard landscape style, single or double, shredded hardwood mulch or chips. The mulch layer should be well aged (stockpiled or stored for at least 12 months), uniform in color, and free of other materials, such as weed seeds, soil, roots, etc. The mulch should be applied to a maximum depth of three inches. Grass clippings should not be used as a mulch material.

6.6E PLANTING PLAN GUIDANCE

Plant material selection should be based on the goal of simulating a terrestrial forested community of native species. Bioretention simulates an ecosystem consisting of an upland-oriented community dominated by trees, but having a distinct community, or sub-canopy, of understory trees, shrubs and herbaceous materials. The intent is to establish a diverse, dense plant cover to treat stormwater runoff and withstand urban stresses from insect and disease infestations, drought, temperature, wind, and exposure.

The proper selection and installation of plant materials is key to a successful system. There are essentially three zones within a bioretention facility (Figure 6.5). The lowest elevation supports plant species adapted to standing and fluctuating water levels. The middle elevation supports a slightly drier group of plants, but still tolerates fluctuating water levels. The outer edge is the highest elevation and generally supports plants adapted to dryer conditions.

Appropriate plant materials for bioretention facilities are included in Appendix C. This list was adapted from the work by Prince George's County, Department of Environmental Resources, and their consultants (ETAB, 1993).

The layout of plant material should be flexible, but should follow the general principals described in Table 6.6. The objective is to have a system which resembles a random and natural plant layout, while maintaining optimal conditions for plant establishment and growth.

FIGURE 6.5: PLANTING ZONES FOR BIORETENTION FACILITIES

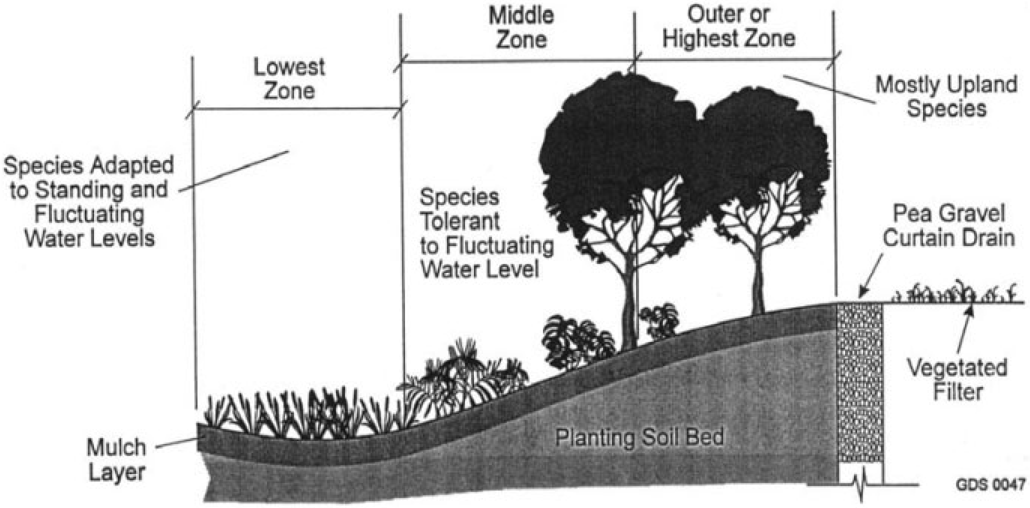


TABLE 6.6: PLANTING PLAN DESIGN CONSIDERATIONS

Native plant species should be specified over exotic or foreign species.
Appropriate vegetation should be selected based on the zone of hydric tolerance (see Figure 6.5).
Species layout should generally be random and natural.
A canopy should be established with an understory of shrubs and herbaceous materials.
Woody vegetation should not be specified in the vicinity of inflow locations.
Trees should be planted primarily along the perimeter of the bioretention area.
Urban stressors (e.g., wind, sun, exposure, insect and disease infestation, drought) should be considered when laying out the planting plan.
Noxious weeds should not be specified.
Aesthetics and visual characteristics should be a prime consideration.
Traffic and safety issues must be considered.
Existing and proposed utilities must be identified and considered.

6.6F PLANT MATERIAL GUIDANCE

Plant materials should conform to the American Standard Nursery Stock, published by the American Association of Nurserymen, and should be selected from certified, reputable nurseries. Planting specifications should be prepared by the designer and should include a sequence of construction, a description of the contractor's responsibilities, a planting schedule and installation specifications, initial maintenance, and a warranty period and expectations of plant survival. Table 6.7 presents some typical issues for planting specifications.

TABLE 6.7: PLANTING SPECIFICATION ISSUES FOR BIORETENTION AREAS

Specification Element	*Elements*
Sequence of Construction	Describe site preparation activities, soil amendments, etc.; address erosion and sediment control procedures; specify step-by-step procedure for plant installation through site clean-up.
Contractor's Responsibilities	Specify the contractors responsibilities, such as watering, care of plant material during transport, timeliness of installation, repairs due to vandalism, etc.
Planting Schedule and Specifications	Specify the materials to be installed, the type of materials (e.g., B&B, bare root, containerized); time of year of installations, sequence of installation of types of plants; fertilization, stabilization seeding, if required; watering and general care.
Maintenance	Specify inspection periods; mulching frequency (annual mulching is most common); removal and replacement of dead and diseased vegetation; treatment of diseased trees; watering schedule after initial installation (once per day for 14 days is common); repair and replacement of staking and wires.
Warranty	Specify the warranty period, the required survival rate, and expected condition of plant species at the end of the warranty period.

6.7 OVERFLOW

The overflow component of the bioretention system consists of the gravel underdrain system, pea gravel overflow curtain drain and a high flow overflow structure. The underdrain system should be designed in accordance with the principals reviewed in Chapter 5. These include: a 6" minimum perforated pipe system within an 8" gravel bed. The pipe should have ⅜" perforations, spaced at 6" centers, with a minimum of 4 holes per row. The pipe should be spaced at a maximum of 10' on-center and a minimum grade of 0.5% should be maintained. At least one cleanout per run should be provided. The underdrain system should be connected to the conventional drainage system, or should daylight to a suitable, non-erosive outfall.

The high flow overflow system usually consists of a yard drain catchbasin (see Figure 6.1), but any number of conventional drainage practices may be used, including an open vegetated or stabilized channel. The system should be designed to convey the peak discharge (Q_p) for the WQV, if the system is located off-line, and should be set above the shallow ponding limit. If the facility is located on-line, the high flow overflow should be designed as a conventional storm drainage structure, or channel. The overflow system should be connected to the site drainage system, or should outfall to a suitable, non-erosive location.

6.8 MATERIAL SPECIFICATIONS

Table 6.8 and 6.9 identify many of the material specifications necessary for bioretention facilities. Designers should refer to their local landscape specifications.

TABLE 6.8: MATERIAL SPECIFICATIONS

Parameter	Specification	Size	Notes
Planting Soil	Refer to Table 6.5	N/A	
Plantings	Refer to Table 6.9	Varies	Refer to Appendix C for specific information, by species.
Mulch	Shredded hardwood	N/A	Aged 2 to 12 months, minimum.
Pea gravel diaphragm and curtain drain	ASTM D 448 size no. 6	Varies (approximately⅛" - ¼")	Use clean bank-run river pea gravel.
Underdrain gravel	AASHTO M-43	½" - 2"	Use clean bank-run river pea gravel.
PVC piping	AASHTO M-278	6" - Rigid Schedule 40	⅜" perf. @ 6" centers, 4 holes per row.

TABLE 6.9: BIORETENTION PLANTING SPECIFICATIONS
(ADAPTED FROM EQR, 1996; ETAB, 1993)

Root stock of the plant material shall be kept moist during transport form the source to the job site.
Planting pits should follow LCA planting guidelines.
The diameter of the planting pit must be six inches larger than the diameter of the ball.
The planting pit shall be deep enough to allow ⅛th of the ball to be above existing ground. Tamp loose soil at the bottom of the pit by hand.
Set and maintain the plant straight during the entire planting process.
Backfill the pit with existing soil.
Trees shall be braced using 2" by 2" stakes only as necessary and for the first growing season only. Stakes are to be equally spaced on the outside of the tree ball.
Planting non-grass ground cover: < Dig holes through the mulch with hand trowel, shovel, bulb planter, or hoe. < Split biodegradable pots and remove non-biodegradable pots < Surround the roots with soil below the mulch. Set potted plants so that the top of the pot is even with existing grade. Cover bare root plants to the crown. < Thoroughly water the entire ground cover bed.
Grasses and legume seed shall be tilled into the soil to a depth of at least one inch. Grass and legume plugs shall be planted following the non-grass ground cover planting specifications.
No fertilization is necessary.

6.9 MAINTENANCE GUIDELINES

The following general maintenance guidance is recommended for bioretention systems. Although these systems are designed to simulate some of the functions of a natural forested plant community, the fact is, that these facilities are located within an urban setting and will be exposed to a wide array of conditions, many of which will tend to compromise the effectiveness of the system. Bioretention facilities will require a reasonable amount of routine maintenance (not too different from conventional landscaping maintenance) to ensure that the system both functions well as a stormwater BMP, and maintains an aesthetic element compatible with the surrounding land uses.

Inspections are an integral part of any maintenance program. Bioretention facilities should be inspected on a semi-annual basis for the first year, and after major storm events. After the first year annual inspections should be sufficient. Since the practice is relatively new, longer term maintenance issues may become apparent which are currently not well understood. There are, however, several maintenance objectives common to all filtering practices, plus some common sense issues specific to bioretention facilities. The following is recommended:

6.9A PLANTING SOIL BED

- The soils of the planting bed should be tested on an annual basis 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 iron sulfate plus sulfur can be added to reduce the pH.

- The soil bed may experience some erosion, particularly at the inflow points, periodic inspection and correction of erosion may be necessary.

- The surface of the bed may become clogged with fine sediments over time. Core aeration or cultivating of unvegetated areas may be required to ensure adequate filtration.

6.9B MULCH LAYER

- Bi-annual mulching, as part of a regular landscape contract, is recommended. The previous mulch may be removed and discarded to an appropriate disposal area or retained if it is decayed. The mulch should be placed to depths not to exceed 3". Seeded ground cover or grass areas should not receive mulching.

6.9C PLANTING MATERIALS

- Annual inspection of plant materials is necessary. Dead or severely diseased species should be replaced. Replacement of particular species should be considered for species which fail to establish.
- Woody vegetation may require periodic pruning, depending on the adjacent land uses, to avoid conflicts with overhead utilities, or hazards with adjacent people and property. Pruning shall follow the standard

pruning practices (ANSI A300, National Arborist Association, Inc., 1995).

- Remove plant stakes after the first growing season

6.9D PRETREATMENT, INFLOW LOCATIONS, AND OVERFLOW

- The pea gravel diaphragm should be inspected annually for clogging. Sediment build-up should be removed, as needed. Replacement of the diaphragm after three to four years may be warranted (or when the voids are obviously filled with sediment and water is no longer infiltrating).

- The vegetated filter strip or grassed channel should be inspected for erosion rill or gulleys and corrected, as needed. Bare areas should be seeded, or sodded, as necessary.

- The inflow location should be inspected annually for clogging. Sediment build-up is common problem with many practices where runoff leaves an impervious surface and enters a vegetative or earthen surface. Any built-up sediment should be removed to avoid runoff by-passing the facility.

- The overflow structure should be inspected annually to ensure that it is functioning. Accumulated trash and debris should be removed, as necessary.

CHAPTER 7
KEY DESIGN ELEMENTS OF OPEN VEGETATED CHANNELS AND FILTER STRIPS

7.1 INTRODUCTION

This chapter explores the design principles of four different grass vegetative filter practices. Each of the four practices incorporate the four major design components discussed in Chapter 1: flow regulation, pretreatment, filtering, and overflow. Vegetative practices have been called a whole suite of names in the past. These include, grassy swales, bio swales, filter strips, grass buffers, and grass channels, to name a few. This chapter consolidates many of these past naming conventions and design principles into a unified approach for the design of water quality treatment using vegetative filters.

This chapter also reviews applicable material specifications and maintenance elements. Appendix B provides a detailed chart of various grasses and provides information to help assess the viability of each species for different design intents.

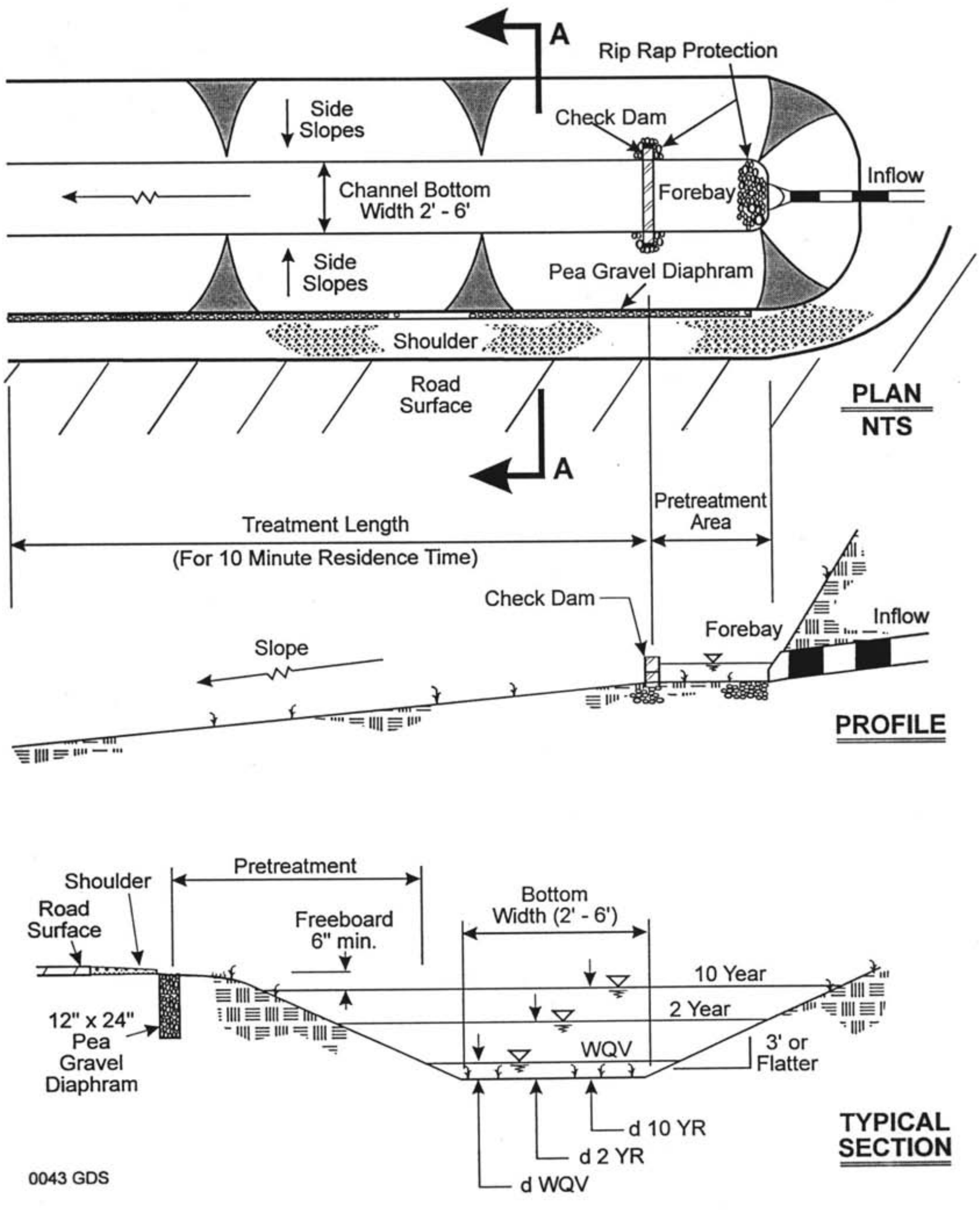
7.2 DESIGN VARIATIONS

Four basic design variations are presented here. The simplified design approaches and criteria, have been adapted from principles governing the design of open channels for conveyance purposes, and more recently, from principles governing the design of vegetative swales for water quality treatment (Horner, 1988). These practices are intended for application to smaller sites where the primary design objective is water quality treatment. The selection of the appropriate design variations are discussed in Chapter 3. The four basic design variations are briefly discussed below.

7.2A GRASS CHANNEL

The grass channel consists of a broad, mildly sloped open channel designed to maintain a minimum residence time of 10 minutes for the "water quality storm" (see Figure 7.1). Grass channels have traditionally been utilized only for stormwater conveyance purposes. In the past, designs ensured adequate capacity to carry a larger storm, usually the 10 year frequency storm and protection against erosion for smaller, more frequent storms, usually the 2 year event. Water quality treatment for the smallest, most frequent storms has only recently been a design consideration. The grass channel design is the only practice presented in this manual which uses a flow rate as the principle design criteria variable. This is referred to as a **rate based** design.

FIGURE 7.1: GRASS CHANNEL



7.2B DRY SWALE

The dry swale consists of an open channel capable of temporarily storing the water quality treatment volume (WQV) and a filtering medium consisting of a soil bed with an underdrain system. The dry swale uses a **volume based** sizing criteria. The dry swale is designed to drain down between storm events within approximately one day. The water quality treatment mechanisms are similar to bioretention practices except that the pollutant uptake is likely to be more limited since only a grass cover crop is available for nutrient uptake. Figure 7.2 illustrates the design components of the dry swale.

7.2C WET SWALE

The wet swale also consists of a broad open channel capable of temporarily storing the WQV (also a **volume based** sizing criteria), but does not have an underlying filtering bed. The wet swale is constructed directly within existing soils and may or may not intercept the water table. Like the dry swale, the WQV within the wet swale should be stored for approximately 24 hours. The wet swale has water quality treatment mechanisms similar to stormwater wetlands which rely primarily on settling of suspended solids, adsorption, and uptake of pollutants by vegetative root systems. Figure 7.3 illustrates the design components of the wet swale.

7.2D FILTER STRIP

Filter strips are grassed practices which accept sheet flow runoff from adjacent surfaces. Filter strips function by slowing runoff velocities and filtering out sediment and other pollutants. The design approach for filter strips involves site design techniques to maintain prescribed maximum sheet flow distances as well as checking to ensure adequate temporary storage for the WQV for a 24 hour period. Filter strips are also designed using a **volume based** sizing criteria.

As discussed in Chapter 1, it is doubtful that runoff can be maintained as sheet flow over distances beyond 150 feet for pervious surfaces, and 75 feet for paved surfaces. Once runoff concentrates, filtering is reduced or eliminated through short-circuiting, preventing effective treatment. Therefore, the use of filter strips to treat stormwater runoff is primarily a function of limiting the flow path to the filter. One of the main abuses of the past has been draining too much area through the filter strip. In most cases the sheet flow distance limitations will be the controlling factor. Figure 7.4 illustrates the primary design components of the filter strip.

FIGURE 7.2: DRY SWALE

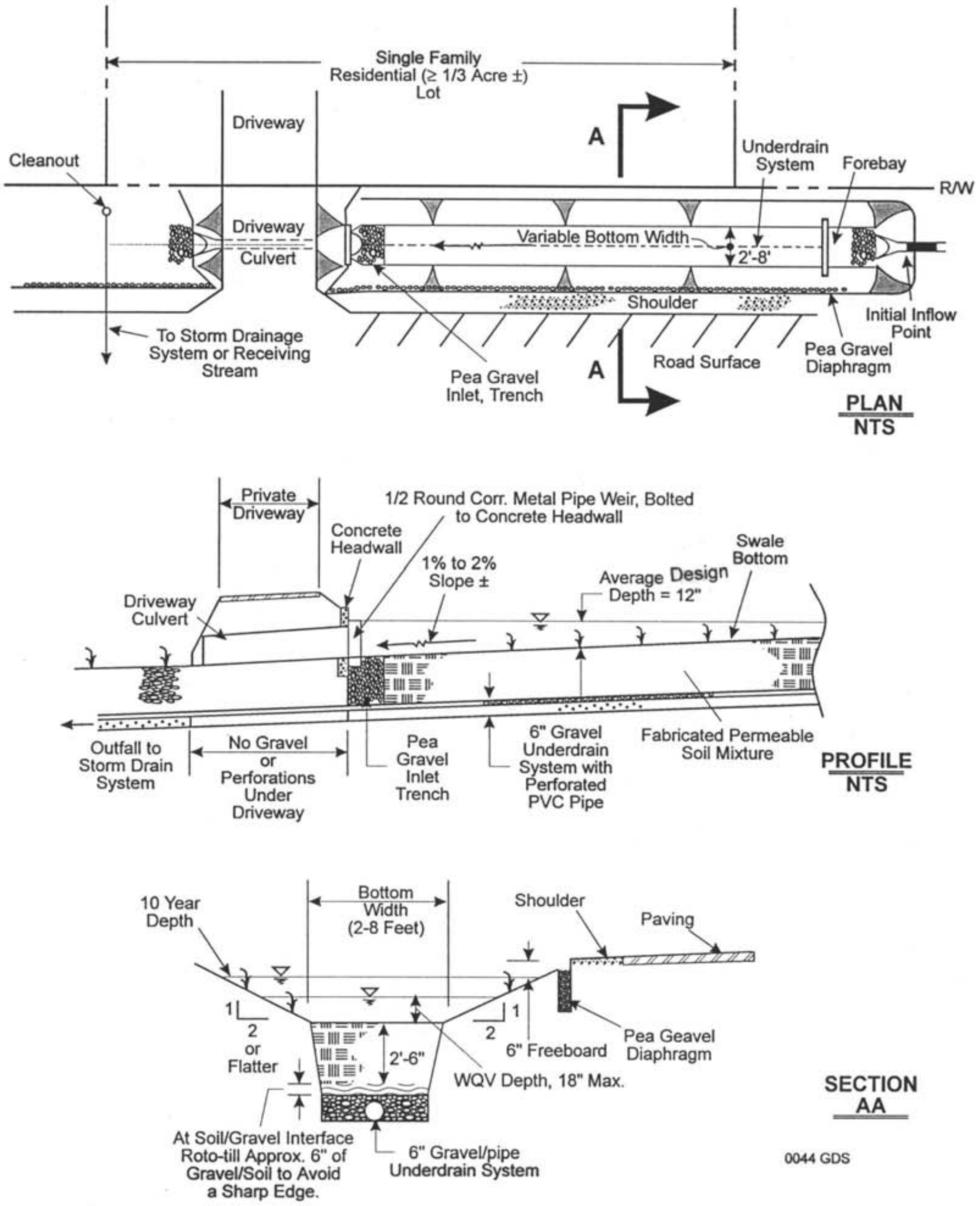


FIGURE 7.3: WET SWALE

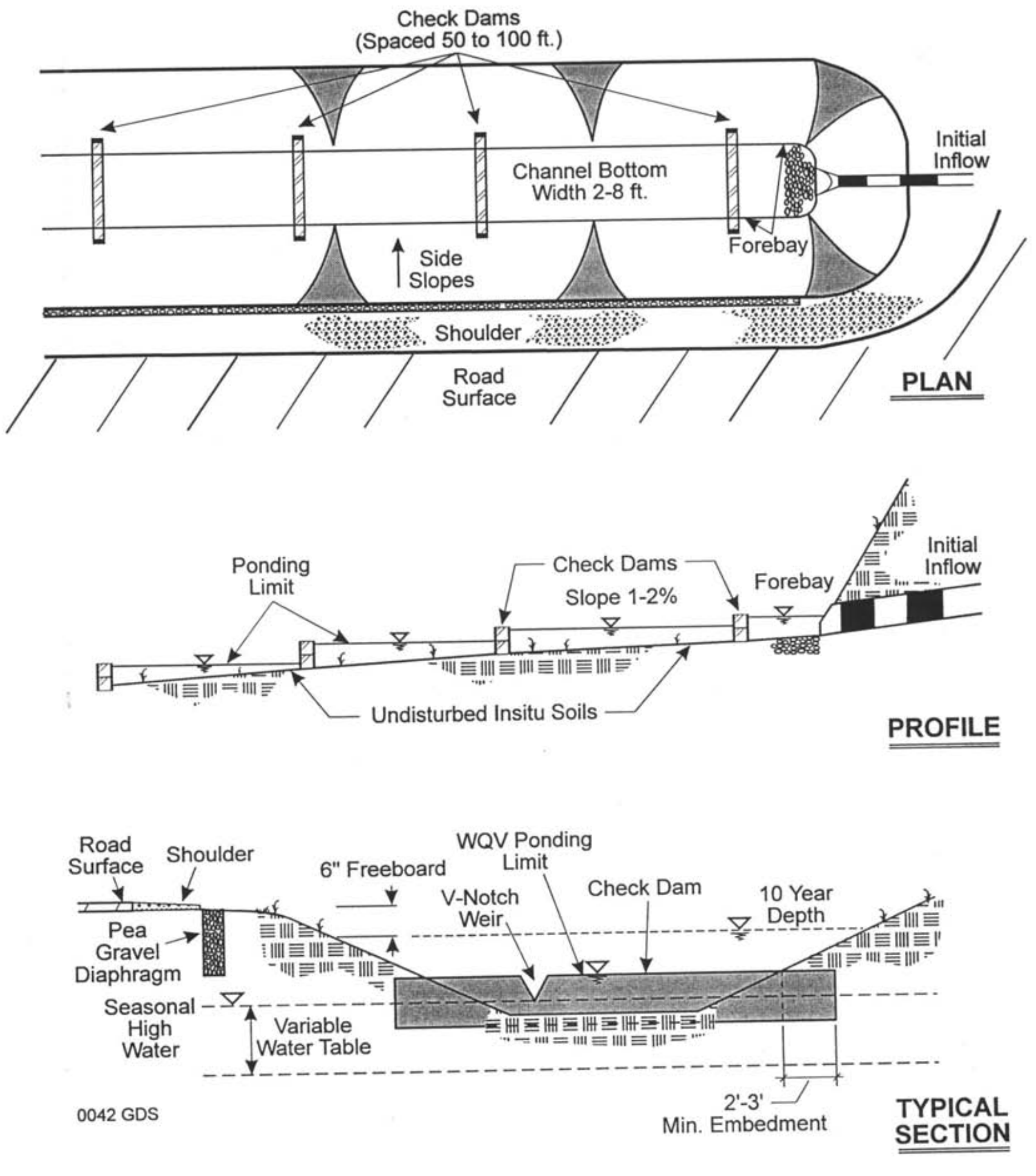
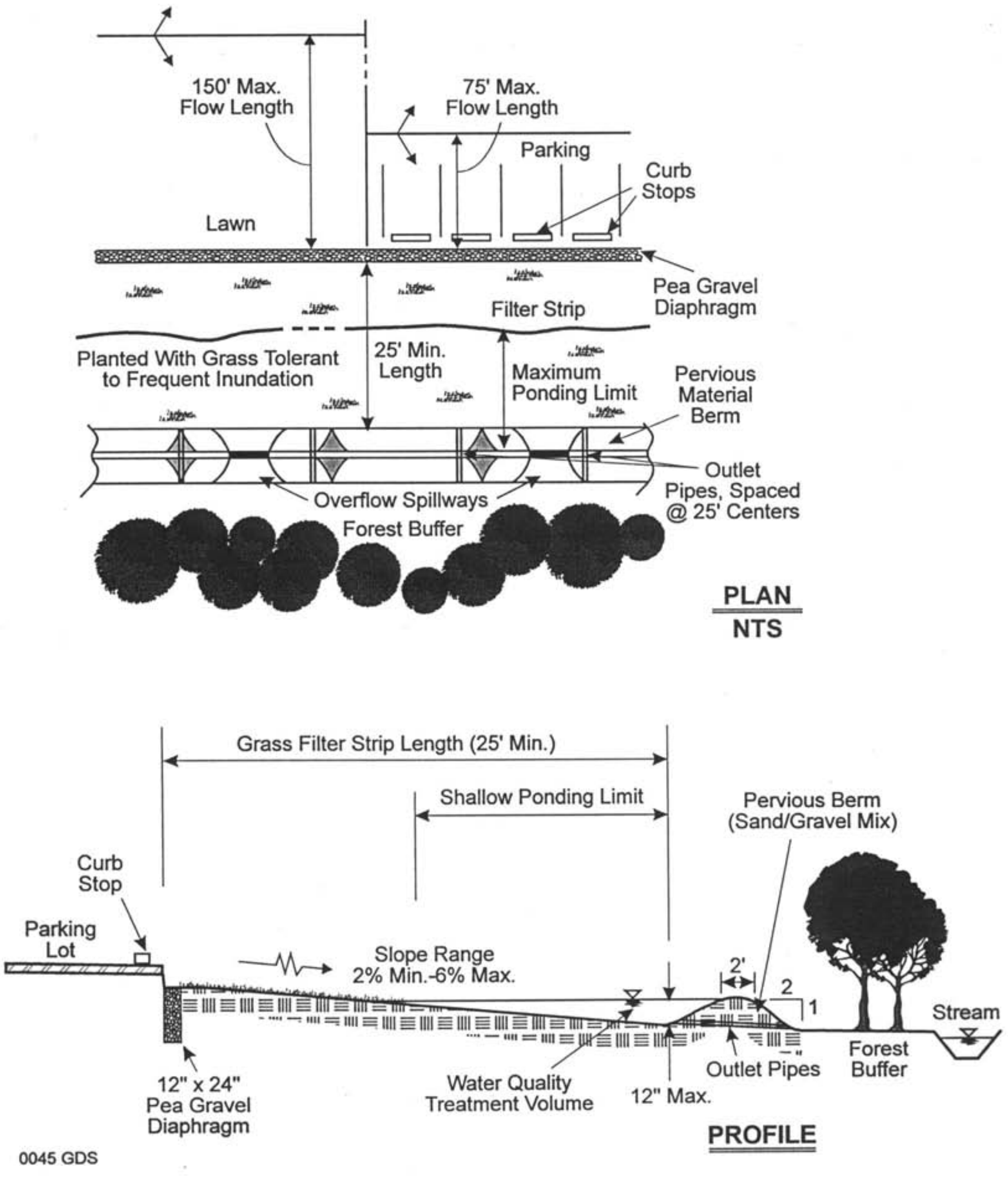


FIGURE 7.4: FILTER STRIP



7.3 FLOW REGULATION

The four design variations presented here are all primarily on-line stormwater treatment practices. The inherent nature of the practice and their applications for use do not lend themselves to many off-line applications. Clearly, it is still best to divert the WQV into the practice wherever possible, and bypass the larger storms around the facility. The grass channel, dry, and wet swales can receive runoff from concentrated sources (pipe outfalls), as well as from lateral sheet flow along the length of the practice. The isolation/diversion structure within the drainage network, reviewed in Chapter 5, is the preferred method for diverting concentrated flows, prior to entering these treatment practices.

The filter strip, which receives runoff through sheet flow from impervious or pervious surfaces is most commonly designed as an on-line practice. It may be possible, through site grading and other design techniques, to provide an overflow diversion which bypasses larger flows around the facility. However, since the filter strip drainage area is limited by the flow path, the volume of high flow runoff will not generally be excessive, and there should be little need to design the system as an off-line practice.

7.4 PRETREATMENT

As with all other filtering practices, pretreatment is necessary to extend the practice's functional life, as well as to increase the pollutant removal capability. All four design variations have incorporated nominal pretreatment as a component of the system design. The difference with these practices from other filtering practices is that the pretreatment component is more qualitative in nature and is an integral part of the practice itself (e.g., the side slopes of the grass channel). The design components for pretreatment which are specific to the four design variations are presented in Table 7.1. With the exception of sizing a forebay at the initial inflow point, there are no specific, quantitative sizing criteria for these pretreatment components.

TABLE 7.1: PRETREATMENT COMPONENTS FOR VEGETATIVE FILTERING PRACTICES

Grass Channel, Dry Swale and Wet Swale: < A shallow forebay is provided at the initial inflow point of the channel. The volume of this forebay should equal approximately .05" per impervious acre of drainage. < A pea gravel diaphragm is recommended along the top of the channel to provide pretreatment for lateral flows entering the practice. < Mild side slopes (# 3:1) provide additional pretreatment for lateral flows.
Filter Strip: < A pea gravel diaphragm is recommended along the top of the slope. < The uphill area, above the shallow ponding limit provides additional pretreatment.

7.5 FILTER MEDIA

7.5A CHARACTERISTICS

The four vegetative filtering practices described in this chapter differ from the sand, organic, and bioretention practices because filtering is primarily through lateral or linear processes, as opposed to vertical filtering (with the exception of the dry swale which has a vertical component). For this reason, the sizing criteria is based on open channel design principles, for the grass channel; and volume detention principles for the remaining practices. Darcy's Law is not particularly applicable to this linear filtering process.

7.5B SIZING GUIDANCE

The grass channel, as previously stated, is a flow rate design based on open channel flow hydraulic characteristics. The principles of small storm hydrology, presented in Chapter 2, and the water quality pollutant removal processes, presented in Chapter 4, are used to design the channel. The dry swale, wet swale, and filter strip are all designed based on detention of the WQV.

GRASS CHANNEL DESIGN CONSIDERATIONS

The following design approach presents a three part criteria for sizing grass channels for stormwater quality treatment, while also accommodating larger storms. The channel is initially designed based on the treatment principles of Small Storm Hydrology for the Water Quality Storm (see Chapter 2), and then checked against the larger 2 year storm to ensure a non-erosive condition. Finally, the capacity for conveyance of the 10 year frequency storm is checked and a minimum freeboard is applied. The design procedure is a **rate based** sizing criteria which uses Manning's equation to compute velocities and depths based on specified channel geometry and slope.

The design application is predominately for highway drainage, but may also be appropriate for some residential applications and for treating small impervious areas (refer to Chapter 3). Figure 7.1 illustrates the design components of the grass channel.

The specific design considerations are presented below and summarized in Table 7.2.

Shape: The channel should be trapezoidal or parabolic in shape. The trapezoidal cross section is the easiest to construct and a more efficient hydraulic configuration. However, since channels tend to become parabolic in shape over time, a channel originally designed as a trapezoidal section should also be checked against parabolic sizing equations as a long term functional assessment. The criteria presented in this chapter assumes a trapezoidal cross section. Note that the same design principles will govern parabolic cross sections except for the cross sectional geometry.

Bottom width: For a trapezoidal cross section, size the bottom width between two and six feet. The two feet minimum allows for construction considerations and ensures a minimum filtering surface for water quality treatment. The six feet maximum prevents shallow flows from concentrating and potentially gullying, thereby maximizing the filtering by grass blades. Widths up to 12 feet may be used if separated by a dividing berm or structure to avoid braiding.

Manning's *n* value: The roughness coefficient, *n*, varies with the type of vegetative cover and flow depth. At very shallow depths, where the vegetation height is equal to or greater than the flow depth, the *n* value should be approximately 0.15. This value is appropriate for flow depths up to approximately 4 inches. For higher flow rates and flow depths, the *n* value decreases to a minimum of 0.03 for grass channels at a depth of approximately 12 inches. The *n* value must be adjusted for varying flow depths between 4" and 12" (see Figure 7.5 for variable *n* values with varying depths).

Side slopes: The sides slopes should be flat as possible to aid in providing pretreatment for lateral incoming flows and to maximize the channel filtering surface. Steeper side slopes are likely to have erosion gullying from incoming lateral flows. A maximum slope of 3:1 is recommended (33%), a 4:1 slope is preferred where space permits.

Channel longitudinal slope: The slope of the channel should be steep enough to ensure uniform flow and which can be constructed using conventional construction equipment without ponding, but not steeper than 4.0%. A minimum slope of 1.0% is recommended.

Flow depth: The maximum flow depth for water quality treatment should be approximately the same as the height of the grass. Since most channels will be mowed relatively infrequently the vegetation may reach heights of 6" or more. However, since higher grass is prone to fallover during higher flows, a maximum flow depth of 4" is required for water quality design. The flow depth for the 2 year and 10 year storms will depend on the flow rate and channel geometry.

Flow velocity: The maximum flow velocity for water quality treatment should be sufficiently low to provide adequate residence time within the channel. A maximum flow velocity of 1.0 feet per second for water quality treatment is required. The maximum flow velocity for the 2 year storm should be non-erosive (a rate of 4.0 to 5.0 feet per second is generally recommended). The permissible velocities of several grass species are listed in Appendix A. Velocity values are purely guidelines and may not always be representative of field conditions. The 10 year permissible velocity may be somewhat higher due to the low frequency of occurrence. A permissible maximum rate of approximately 7.0 feet per second for this event is recommended.

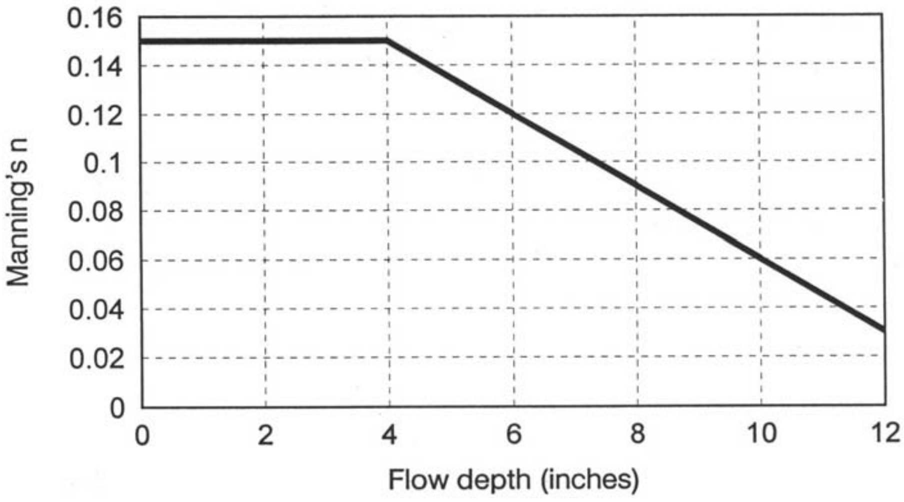
Length of channel: Generally grass channel length (for conveyance) is a function of site drainage constraints and a required length is not necessary. However, for water quality treatment, a minimum residence time of 10 minutes should be obtained to facilitate filtering. The minimum length required for water quality treatment grass channels is equal to the velocity, in feet per second multiplied by the minimum residence time of 600 seconds.

TABLE 7.2: DESIGN CRITERIA FOR TRAPEZOIDAL GRASSED CHANNELS FOR WATER QUALITY TREATMENT

Parameter	***Design Criteria***
Bottom Width	2 feet minimum, 6 feet maximum *
Side Slopes	3:1 or flatter
Channel Longitudinal Slope	1.0% minimum, 4.0% maximum
Flow Depth	4" for water quality treatment
Manning's *n* Value	0.15 for water quality treatment (depths # 4") varies from 0.15 to 0.03 for depth between 4" and 12" 0.03 minimum for depths $ 12 inches (see Figure 7.5)
Flow Velocity	1.0 fps for water quality treatment 4.0 fps to 5.0 fps for 2 year storm 7.0 fps for 10 year storm
Length	Length necessary for 10 minute residence time

*Widths up to 12' are allowable with a dividing berm or structure.

FIGURE 7.5: VARIABLE MANNING'S N WITH VARYING FLOW DEPTH



GRASSED CHANNEL DESIGN PROCEDURE

< Use the 90% Rule to select rainfall for the Water Quality Storm (refer to Chapter 2, Section 2.5)

< Compute the peak rate of discharge (Q_p) for the Water Quality Storm based on the procedures identified in Chapter 2, Section 2.8, Small Storm Hydrology.

< Utilize Q_p to size the channel, maintain design criteria parameters noted in Table 7.2
  - Utilize the design charts (Figures 7.6-7.8) for channel widths 2, 4, and 6 feet, or
  - Utilize computer model which solves Manning's equation, or other open channel flow equations.

< Compute 2 year and 10 year frequency storm event peak discharges using SCS, TR-55.

< Check 2 year velocity for erosive potential (adjust geometry, if necessary and re-evaluate WQV design parameters).

< Check 10 year depth and velocity for capacity (adjust geometry, if necessary and re-evaluate WQV and 2 year design parameters).

< Provide minimum freeboard above 10 year storm water surface elevation (6 inches minimum, recommended).

The design charts provided (Figures 7.6 - 7.8) solve Manning's equation for various slopes and discharges. The charts were adapted from the U.S. Department of Transportation, Federal Highway Administration's, Hydraulic Design Series "Design Charts for Open-Channel Flow," reprinted 1980.

EXAMPLE CALCULATION

Given: Roadway draining 1.0 acre drainage area
0.6 acre narrow paved surface
0.4 acre grassed pervious surface, silty soils

from Chapter 2, Small Storm Hydrology
$R_v = [0.6(0.70) + 0.4(0.11)]/1.0 = 0.46$
for 1.0" rainfall
WQV = 1.0"(0.46)*1.0=0.46"

Using modified TR-55 methodology
(from equation 2.3)
$CN = 1000/[10 + 5(1.0) + 10(.46) - 10[(.46)^2 + 1.25(.46)(1.0)]^{½}]$
CN = 93.2 use 93
for time of concentration (t_c) = 6 minutes, = 0.1 hour

and
$I_a = 0.151$, $I_a/P = .151/1.0 = 0.15$
q_u = 1000 csm/in
Qp = 1000 csm/in(1.0 ac/640ac/mi²)(.46 in) = .72 cfs, use 0.7 cfs

See chart (Figure 7.7) for 4' bottom width
for n = 0.15, Qn = (0.7)(0.15) = 0.11, and 4' bottom width, read depth = .29, and Vn = 0.08 for 2.0% slope, V = 0.08/0.15 = 0.5 fps
minimum length for 10 minute residence time, L = 0.5 ft/sec(600 sec) = 300 feet

Using traditional TR-55 methodology
for 2 and 10 year storm (2 year rainfall = 3.0 inch and 10 year rainfall = 5.0)
CN = [0.6(98) +0.4(61)]/1.0 = 83.2 use 83
t_c = 0.1 hour
$I_a = .41$, $I_a/P = .41/3.0 = .14$
volume of runoff, 2 year = 1.45 inches
10 year = 3.17 inches
q_u = 1000 csm/in
2 year Q_p = 1000 csm/in(1.0 ac/640ac/mi²)(1.45 in) = 2.3 cfs
10 year Q_p = 1000 csm/in(1.0 ac/640ac/mi²)(3.17 in) = 5.0 cfs

Figure 7.7: 2 year, use n =0.10, Qn = (2.3cfs)(0.1) = 0.23, slope = 2.0%; depth = .45 feet
and Vn = .10, therefore V = 1.0 fps
10 year, use n = 0.08, Qn = (5.0cfs)(0.08) = 0.4, slope = 2.0%; depth = 0.6 feet
and Vn = .12, therefore V = 1.5 fps

Use 4 foot wide channel, with 3:1 side slopes, 2.0% slope, with a minimum depth of 0.6 + 0.5 = 1.2 feet, note n is lower for the 2 and 10 year events as the depth increased (once the depth exceeds approximately 12 inches use n = 0.03).

FIGURE 7.6: DESIGN CHART FOR TRAPEZOIDAL CHANNEL (2' BOTTOM WIDTH)

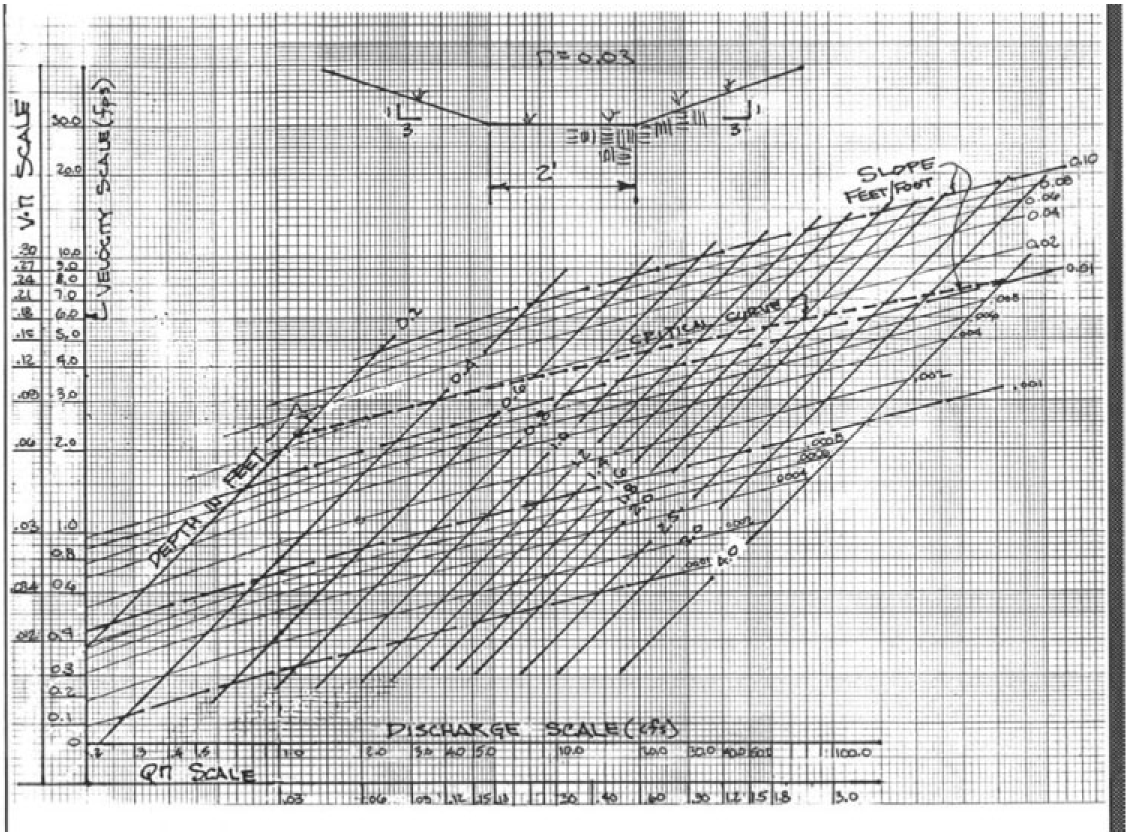


FIGURE 7.7: DESIGN CHART FOR TRAPEZOIDAL CHANNEL (4' BOTTOM WIDTH)

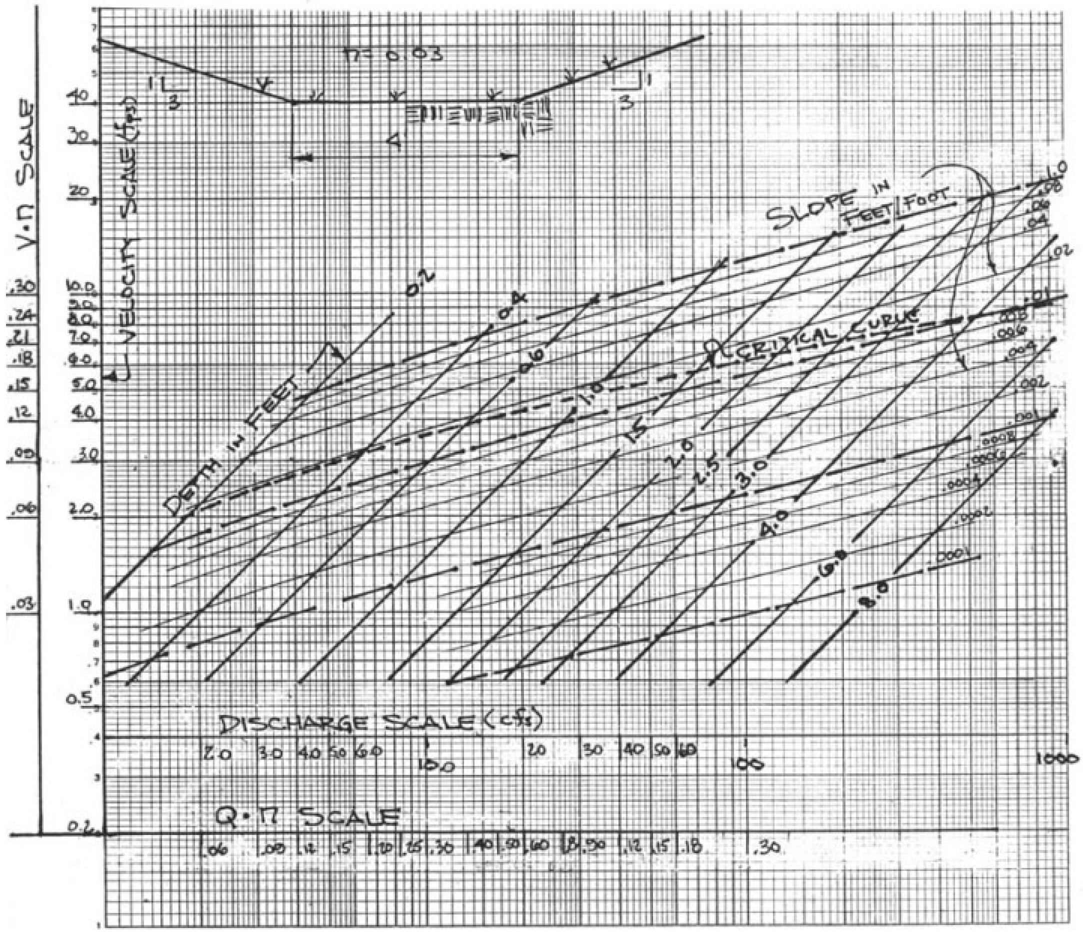
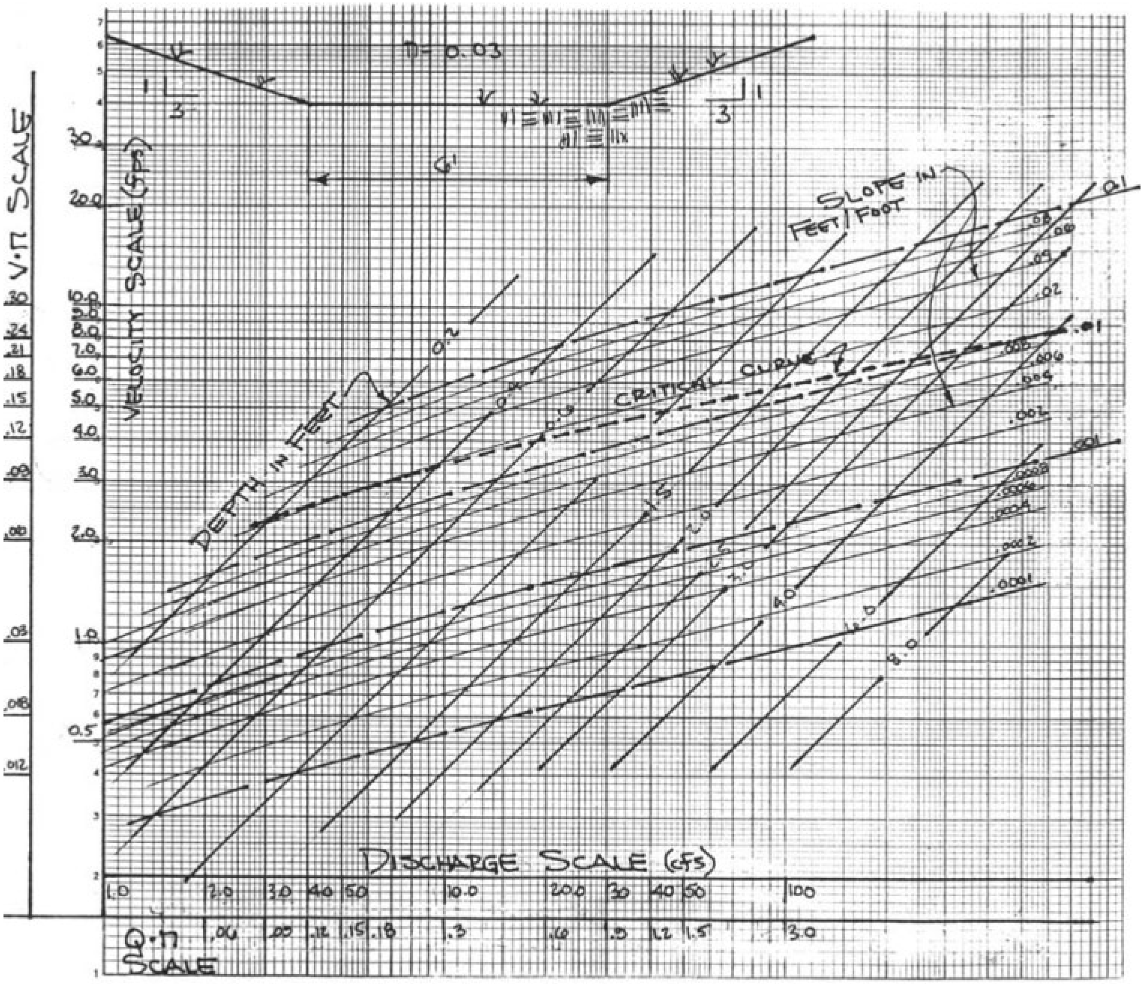


FIGURE 7.8: DESIGN CHART FOR TRAPEZOIDAL CHANNEL (6' BOTTOM WIDTH)



DRY SWALE AND WET SWALE DESIGN CONSIDERATIONS

The design approach for sizing dry and wet swales is based on temporarily storing the WQV within a shallow ponding area. This methodology incorporates a **volume based** sizing criteria for the WQV, and a rate based criteria for checking the erosive potential for the 2 year frequency storm and capacity for the 10 year frequency storm.

The dry swale is mainly applied to moderate to large lot residential land uses. Small impervious areas (small parking lots and rooftops) and rural highway runoff can be accommodated by the dry swale. Wet swales are predominately used for highway runoff applications, but can also be used to filter water from small parking lots, rooftops and pervious areas (see Chapter 3).

The specific design considerations are presented below, and summarized in Table 7.3.

Shape: The swales should generally be trapezoidal in shape, although a parabolic shape is also acceptable (provided the underlying soil bed design width, for dry swales, is equal to or greater than, the design bottom width for a trapezoidal cross section). The criteria presented in this section assumes a trapezoidal cross section.

Bottom width: For the trapezoidal cross section, size the bottom width between two and eight feet. The two feet minimum allows for construction considerations and ensures a minimum filtering surface for water quality treatment. The eight feet maximum reduces the likelihood of flow channelization within a portion of the bottom of the swale. Eight feet is allowed (versus the six feet specified for grass channels) to accommodate additional storage for WQV. Widths up to 16 feet may be used if separated by a dividing berm or structure to avoid braiding.

Side slopes: The side slopes of the channel should be no steeper than 2:1 for maintenance considerations (mowing). Flatter slopes are encouraged where adequate space is available to aid in providing pretreatment for lateral flows. The steeper maximum side slope for the dry/wet swales is permitted because these practices are designed to retain a storage volume versus being designed for a minimum residence time.

Swale longitudinal slope: The slope of the swale should be moderately flat to permit the temporary ponding of the WQV within the channel without having excessively deep water at the downstream end. A slope between 1.0% and 2.0% is recommended. When natural topography necessitates, steeper slopes may be acceptable if check dams (vertical drops of 6 to 12 inches) are used. These structures will require additional energy dissipating measures and should be placed no closer than 50 to 100 feet intervals (see Figure 7.3).

TABLE 7.3: DESIGN CRITERIA FOR DRY AND WET SWALE SYSTEMS

Parameter	***Swale Design Criteria***
Pretreatment volume	.05" per impervious acre, at initial inflow point.
Preferred shape	Trapezoidal or parabolic.
Bottom width	2 feet minimum, 8 feet maximum widths up to 16 feet are allowable if a dividing berm or structure is used.
Side slopes	2:1 maximum, 3:1, or flatter preferred.
Longitudinal slope	1.0% to 2.0% without, check dams.
Sizing criteria	Length, width, depth, and slope needed to provide surface storage for WQV. Outlet structures sized to release WQV over 24 hours.
Underlying soil bed	Equal to swale width <u>Dry Swale</u>: Moderately permeable soils (USCS ML, SM, or SC) 30" deep with gravel/pipe underdrain system <u>Wet Swale</u>: Undisturbed soils, No underdrain system
Depth and capacity	< Surface storage of WQV with a maximum < depth of 18 inches for water quality treatment (12" average depth). < Safely convey 2 year storm with non-erosive velocity (# 4.0 to 5.0 ft/s) < Adequate capacity for 10 year storm with 6" of freeboard

<u>Design sizing criteria</u>: The detention/retention capacity of both dry and wet swales is governed by the runoff associated with the "water quality storm." The swale length, width, depth, and slope should be designed to temporarily accommodate the WQV through surface ponding. For the dry swale, all of the surface ponding should dissipate within a maximum 24 hour duration. The outlet structure (half-round pipe in Figure 7.2) is sized to release the WQV over 6 hours. Using perforations in the bottom 6" of pipe. The soil media will have an infiltration capacity of at least a foot/day.

For wet swales, the WQV volume is still retained for 24 hours, but ponding may continue indefinitely depending on the depth and elevation to the water table. The WQV for high density residential, commercial and industrial land uses will most likely be too great to be accommodated with most swale designs. However, swales may be appropriate for pretreatment in association with other practices for these higher density land uses or may be acceptable solutions for watershed retrofit projects (see Chapter 3).

Underlying soil bed: The soil bed below the dry swale should consist of a moderately permeable soil material with a high level of organic matter (USCS ML, SM, or SC). The soil bed should be 30 inches deep, and should be accompanied by a gravel/pipe underdrain system. This soil mixture is necessary in residential areas to ensure drainage of the swale system within a moderately short time period to avoid safety and nuisance concerns. The soil/gravel interface should be roto-tilled to have an approximate six inch mixing zone of fine sand, soil, and gravel to augment the filtering capability of the practice.

The soil bed below the wet swale should consist of undisturbed soils. This area may be periodically inundated and remain wet for long periods of time, and is therefore not appropriate for residential land uses. (Filter fabric is not recommended for this section).

Swale depth and capacity: Swales should be designed to provide a shallow ponding depth for the WQV (a maximum depth of 18" for the WQV is recommended), safely convey the 2 year storm with design velocities less than 4.0 to 5.0 feet per second, and provide adequate capacity for the 10 year storm with a minimum of 6 inches of freeboard.

DRY SWALE AND WET SWALE DESIGN PROCEDURE

< Use the 90% Rule to select rainfall for the Water Quality Storm (refer to Chapter 2, Section 2.5).

< Compute the Water Quality Treatment Volume (WQV) for the given land surfaces, based on the procedures identified in Chapter 2, Section 2.7, Small Storm Hydrology.

< Identify the required swale bottom width, depth, length, and slope necessary to store the WQV within a shallow ponding depth (# a maximum depth of 18").

< Compute the WQV drawdown time to ensure that it is less than 24 hours.

- Compute the 2 year and 10 year frequency storm event peak discharges using SCS, TR-55.

- Check the 2 year velocity for erosive potential (adjust swale geometry, if necessary and re-evaluate WQV design parameters.

- Check the 10 year depth and velocity for capacity (adjust swale geometry, if necessary and re-evaluate WQV and 2 year design parameters).

- Provide minimum freeboard above 10 year storm water surface profile (6 inches minimum recommended).

- Specify vegetation required to meet design conditions (see Appendix B).

- For dry swales, specify grasses resistant to periodic inundation and periodic drought.

- For wet swales, specify grasses resistant to sustained inundation and/or water table at or near the surface, wetland species are appropriate for swale bottom.

- For all swales, check permissible velocities of selected vegetation to ensure the 2 year frequency storm velocity is non-erosive.

EXAMPLE CALCULATION

Given: Four, approximately ⅓ acre lots
4 driveways and a 26 foot wide open section road
A 1.55 acre total drainage area
(See Figure 7.9)

Houses = 0.22 acres
Driveways = 0.10 acres
Street = 0.15 acres
Pervious surface (lawns) = 1.08 acres

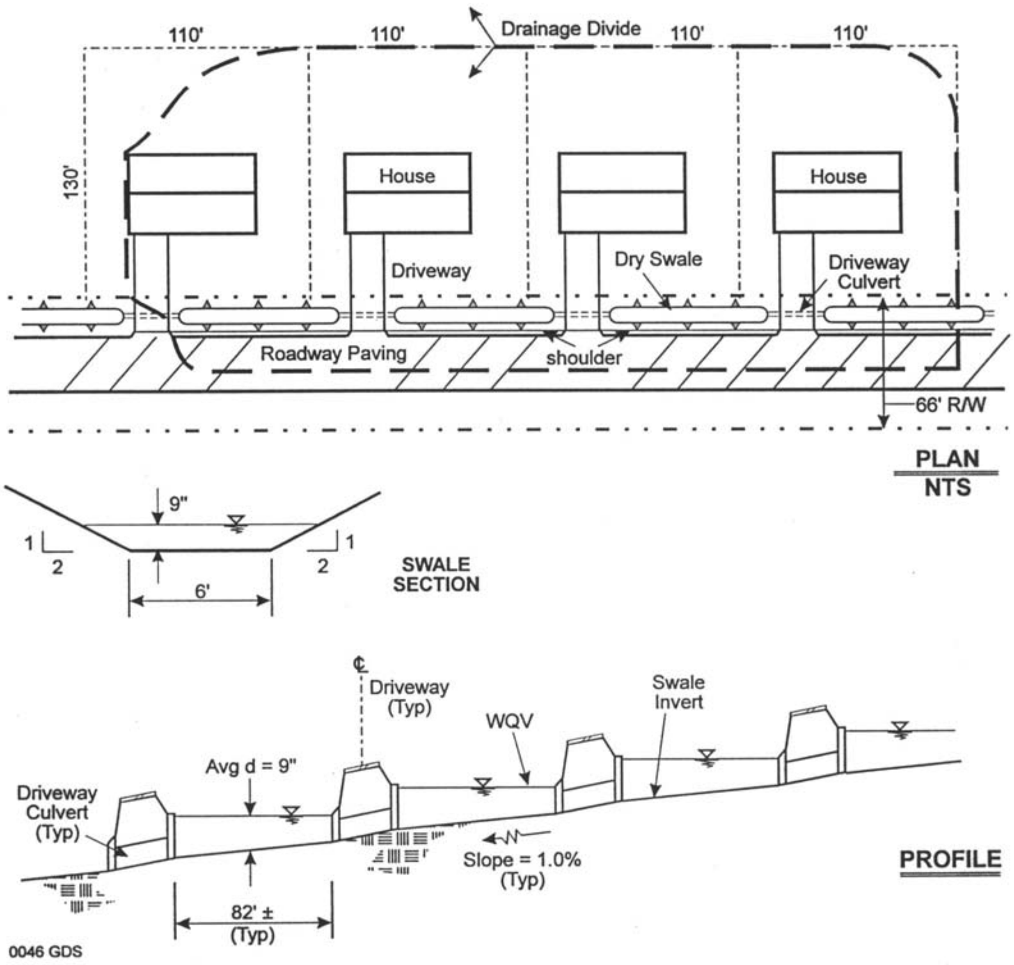
from Chapter 2, Small Storm Hydrology
R_v = .97(0.22) + .70(0.25) + .11(1.08) / 1.55 = 0.33
for 1.0" rainfall
WQV = 1.0"(0.33) = 0.33"
WQV = 0.33"(1 ft/12" (1.55 ac (43,560 ft^2 / ac = 1,857 ft^3

Size dry swale to provide minimum storage = 1,857 ft^3
from Figure 7.9, length available = 328 ft (4 * 82')

try 6 ft bottom width, 9" depth, 2:1 side slopes swale
A = (6.0)(.75) + 2 (½(1.5)(.75) = 5.63 ft^2
Vol. provided = 328 ft (5.6 ft^2 = 1,845 ft^3 which is . 1,857 ft^3

Note: Swale must have a minimum 1.0% slope, and must have approximately 82 feet separating each driveway, so as to maintain an average 9" depth, set depth = 9" ft at ½ distance between driveways, therefore, use a 14" maximum depth at culvert inlets and 4" at culvert outlets.

FIGURE 7.9: DRY SWALE - EXAMPLE CALCULATION



FILTER STRIP DESIGN CONSIDERATIONS

The design approach for sizing filter strips is based on temporarily storing the WQV within a shallow ponding area. This methodology also utilizes a volume based sizing criteria for the WQV.

This practice is primarily designed for pervious surfaces and rooftops (rear yard runoff from single and multi-family residential). Filter strips may also be appropriate for some small parking lots and residential land uses where adequate treatment space is available (see Chapter 3).

The specific design considerations are presented below and summarized in Table 7.4.

TABLE 7.4: DESIGN CRITERIA FOR FILTER STRIPS

Parameter	***Filter Strip Design Criteria***
Sizing criteria	Length, depth and slope necessary to provide surface storage for WQV. Width equal to area draining to filter. Minimum length = 25 feet
Slope	Minimum slope = 2.0% Maximum slope = 6.0%
Treatment drainage area	Maximum overland flow lengths: pervious surfaces = 150 feet impervious surfaces = 75 feet

<u>Size</u>: The size of the filter strip is determined by the required treatment volume, however the minimum length must be 25 feet. The width of the filter strip is equal to the width of the area draining to it.

<u>Slope</u>: The maximum slope should be no more than 6.0% and the minimum slope should be no less than 2.0%. Steeper slopes will increase velocities and lead to concentration of runoff and likelihood for erosion. In addition, as the slope increases, the treatment volume per cross sectional area decreases. Slopes flatter than 2.0% may be appropriate for some geographic regions, but are discouraged in residential areas due to the tendency for surface ponding to create potential nuisance conditions.

<u>Drainage Area</u>: The maximum drainage area to filter strips is limited by the overland flow limits of 150 feet for pervious surfaces and 75 feet for impervious surfaces.

FILTER STRIP DESIGN PROCEDURE

< Prepare site grading design to limit length of overland flow entering filter strip.

< Design filter strip to receive runoff through sheet flow along a level plain. For paved surfaces, provide multiple entry points (utilizing curb stops, instead of concrete curb and gutter). For pervious surfaces, ensure that the entry point is level at the top of the filter strip. A concrete "level spreader" may be necessary for some applications.

< Compute the WQV using the 90% Rule (refer to Chapter 2, Section 2.5 and Section 2.7).

< Check available storage within shallow ponding limits, based on slope, height of pervious berm and width of filter strip.

< Size outlet pipes to ensure release of the WQV drawdown time is less than 24 hours.

< Layout filter strip to maintain 25 feet minimum length.

< Maintain the toe of the filter strip as level as possible.

< Construct the filter strip outside of the boundary of a natural stream buffer area.

< Provide overflow or bypass for storms larger than WQV (e.g., 2 and 10 year events).

< Specify vegetation required to meet design conditions (see Appendix B):

- Specify grasses resistant to frequent inundation within shallow ponding limit.

- Specify grasses with high retardance or high permissible velocities for sloping area leading to shallow ponding area.

EXAMPLE CALCULATION

Single family lot, 150' deep x 100' wide
Drainage area = 15,000 sq. ft. or 0.34 acres
Single family house (60' x 40') = 0.06 ac
Pervious surface (lawn) = 0.28 ac

From Chapter 2, Small Storm Hydrology
$R_v = [.97(.06) + .11(.28)]/.34 = 0.26$
for 1.0" rainfall
WQV = 1.0"(0.26) = 0.26"
WQV = 0.26" 1 ft/12 in (15,000 sq. ft. = 325 ft^3

Assume filter strip slope = 2.0%
Pervious berm height = 6"
for 25' minimum length
cross sectional area = ½ (25' (0.5' = 6.25 ft^2

Required filter strip width: = 325 ft^3/6.25 ft^2 = 52 ft
100 foot width provided, volume acceptable

Use 25' long x 100' wide filter strip with 6" high pervious berm

7.6 OVERFLOW

The overflow element of the four vegetated filter design variations consist of safely conveying the high flow events (storms greater than the WQV) through the systems. In the case of the grass channel, dry swale, and wet swale, this involves ensuring that the velocities of more frequent high flow events (e.g., 2 year storm) are non-erosive, and that the less frequent high flow events (e.g., 10 year, or in some cases, the 100 year storm) are contained within the channel, and do not pose a flooding condition to adjacent areas. In the case of the filter strip, this involves either diverting the higher flows to by-pass the practice, or providing an overflow spillway to ensure a non-erosive condition.

7.7 MATERIAL SPECIFICATIONS

Table 7.5 identifies many of the material specifications necessary for the design of the four vegetated filter design variations. Specific information on the use of various grasses is presented in Appendix A. Detailed seeding specifications are not included. Designers should refer to their local landscaping specifications.

TABLE 7.5: MATERIAL SPECIFICATIONS

Parameter	Specification	Size	Notes
Dry Swale Soil	Sand: ASTM C-33 fine aggregate concrete sand Soil: USCS: ML, SM, or SC	Sand: .02" - .04"	Soil with a higher percent organic matter is preferred
Check Dam	Pressure treated or equiv. rot resistant wood (e.g., black locust)	6" x 6"	Embed 2-3' into side slopes
Filter Strip Sand/Gravel Pervious Berm	Sand: ASTM C-33 fine aggregate concrete sand Gravel: AASHTO M-43	Sand: .02" - .04" Gravel: ½" - 1"	Mix with approx. 25% loam soil to support grass cover crop
Pea Gravel Diaphragm and Curtain Drain	ASTM D 448 size no. 6	Varies (approximately ⅛" - ⅜")	Use clean bank-run river pea gravel
Underdrain Gravel	AASHTO M-43	½" - 2"	
PVC Piping	AASHTO M-278	6" - Rigid Schedule 40	⅜" perf. @ 6" centers, 4 holes per row

7.8 MAINTENANCE GUIDELINES

The following general maintenance elements are applicable to all of the four vegetative practice design variations. Inspections are an integral part of any maintenance program. These four vegetative filtering practice variations should be inspected on a semi-annual basis for the first year, and after major storm events. After the first year, annual inspections should be sufficient.

7.8A PRETREATMENT

< The pea gravel diaphragm should be inspected annually for clogging. Sediment build-up should be removed, as needed. Replacement of the diaphragm may be warranted when the voids are obviously filled with sediment and water no longer percolates into the stone.

< The grass vegetation along the side slopes should be inspected for erosion rills or gullys, and corrected, as needed. Bare areas should be seeded, or sodded, as necessary.

< The initial inflow forebay should be inspected annually for sediment build-up. Any excessive sediment, trash, and debris should be removed and disposed of in an appropriate location.

7.8B DRY SWALE SAND AND SOIL BED

< The sand/soil bed may experience some erosion, particularly at the inflow point. Periodic inspection and correction of erosion areas may be necessary.

< The surface of the bed may become clogged with fine sediments over time. If the swale does not drain within 48 hours, roto-tilling, or cultivation of the top of the soil bed may be required to ensure adequate filtration.

7.8C VEGETATION

< Grass should be mowed on a regular basis. What is considered a regular basis will depend on the location, the type of practice (e.g., wet or dry), climate, and type of grass selected. In general, in order to maintain the optimal filtering capability, grass levels should not exceed 3 to 4 inches. This may require mowing as frequently as bi-weekly during the peak growing season. Wet swales, which incorporate wetland vegetation, do not require mowing at the same frequency as the other practices.

- The grasses should be selected based on the hydric conditions anticipated (refer to Appendix B). However, water table conditions vary from season to season, and from location to location, and the specified grass may not establish itself sufficiently. Annual inspection of the vegetation condition is necessary. An alternative grass species should be considered for species which fail to establish.

- Sediment build-up within the bottom or the channel, swale or filter strip should be removed when it has accumulated to approximately 25% of the original design volume or channel capacity.

APPENDIX A.

STORMWATER POLLUTANT CONCENTRATIONS FROM DIFFERENT URBAN SOURCE AREAS AND HOTSPOTS

Mean concentrations in the table are unweighted averages of published mean concentrations in the indicated reference. Some authors had multiple sites of an individual source area or hotspot.

Source	***N***	***References***
Resid Roof	3	Bannerman (1994), Pitt and McLean (1986)
Comm Roof	2	Bannerman (1994), Pitt and McLean (1986)
Indust Roof	5	Pitt et al. (1994), Good (1993), Bannerman (1994), Thomas and Greene (1993)
C/R Parking	9	Bannerman, Rabinal, Pitt et al., Pitt and McLean, Bell, Schueler
Ind Parking	4	Bannerman (1994), Pitt et al. (1994) Horner (1994)
Res Street	4	Bannerman (1994), Pitt and McLean (1986), Schueler and Shepp (1993)
Comm Street	4	Bannerman (1994), Pitt et al. (1994)
Rural Highway	6	FHWA (1990)
Urban Highway	8	FHWA (1990)
Lawns	2	Bannerman (1994), Pitt et al. (1994)
Driveway	1	Bannerman (1994)
Gas Stat/VMA	3	Pitt et al. (1994), Schueler (1994), Rabanal & Grizzard (1995)
Auto Recycler	2	Swamikannu (1994)
Heavy Indus	1	Leersnyder (1993)
Landscaping	1	Pitt et al. (1994)
NURP-DC	-	MWCOG (1983)
NURP-US		USEPA (1983)
NOTES: NURP US lead value is considered unrepresentative as it reflects deposition conditions of leaded gas.		

STORMWATER POLLUTANT CONCENTRATIONS FROM DIFFERENT URBAN SOURCE AREAS AND HOTSPOTS

Shaded cells indicate mean concentrations twice the national or regional average.

Source	N	TSS	F Coli	Cd	Cu	Pb	Zn	PP	O/G
Resid Roof	3	19	.26	3	20	21	312	7	NA
Comm Roof	2	9	1.1	0.3	7	17	256	16	NA
Indust Roof	5	17	5.8	2	62	43	1390	6	NA
C/R Parking	9	27	1.8	8	51	28	139	13	8.5
Ind Parking	4	228	2.7	2	34	85	224	0	15
Res Street	4	172	37	1	25	51	173	1	2
Comm Street	4	468	12	6.7	73	170	450	13	NA
Rural Highway	6	41	NA	NA	22	80	80	NA	NA
Urban Highway	8	142	NA	1	54	400	329	NA	NA
Lawns	2	602	24	ND	17	17	50	NA	NA
Driveway	1	173	34	0.5	17	ND	107	NA	NA
Gas Stat/VMA	3	31	NA	9	88	80	290	19	14
Auto Recycler	2	335	NA	8.5	103	182	520	NA	25
Heavy Indus	1	124	NA	NA	148	290	1600	NA	NA
Landscaping	1	37	NA	0.3	94	29	263	14	NA
NURP-DC	-	100	NA	0.5	10	18	37	2	NM
NURP US		150	NA	0.7	34	140***	160	3	3

NOTES: TSS (mg/l), F. coliforms (1000 counts/ml), Cd, Cu, Pb and Zn (ug/l), PP= number of non-metal and non-pesticide priority pollutants detected in stormwater. O/G= oil and grease (mg/l), N= number of monitoring studies used to characterize a source area. Values are an un-weighted mean of each monitoring study. Total number of studies for a given parameter may be less than N.

Grass Type	Availability	Salt Tolerance	Resistance to Frequent Inundation	Permissible Velocity ft/s	Native or Introduced	Growth Season /	Germination Rate / Growth Rate	Drought Resistance	Bunch / Sod	Root Network Density	Height at Maturity	Optimal Soil Conditions	Regional Adaptation	Shade Tolerance	Notes:
Bermuda grass *(Cynodon dactylon)*	Yes	Good	Excellent	6	N	Warm	Rapid, highest growth rate of warm season grasses	Good	Sod	Rhizomes form tight sod, fibrous extensive roots	.75-2 ft.	Fine textured, fertile, well drained soil, pH 5.5-7.5	Southeast, most of southern US with irrigation	Low	Good tolerance to heat. Poor tolerance to cold. Many varieties grown from sprigs
Bahiagrass *(Paspalum notatum)*	Yes	Good	Good	5	I	Warm	Slow establishment	Fair		Rhizomatous	1.5 ft.	Droughty, sandy ,fine, poorly drained, slightly acidic soils	Southeast	Good	Germination enhanced by scarification. Intolerance to low temperatures
Big Bluestem *(Andropogon gerardii)*	Yes	Fair	Fair		N	Warm	Slow est.	Fair	Sod	Dense sod	4-6 ft.	Silt-clay	Midwest, Great Plains, with irrigation.	Low	Low maintenance, low nitrogen requirement. Seeded in "warm-season" mixtures
Creeping Bentgrass *(Agrostis palustris)*	Yes	Good	Excellent		I	Cool		Poor	sod	Dense fibrous, medium to shallow in depth, vigorous stolons	1-12"	Best adapted to fertile, fine textured, slightly acidic soils. pH5.5-6.5	Northeast, Northern Great Plains, Midwest	Fair	Hardy cool season turf grasses. Tolerent to low temperature extremes
Buffalograss *(Buchloe dactyloides)*	Yes	Fair	Excellent	5	N	Warm	Fast est.	Excellent	Bunch	Shallow, good soil and sand binder	.33-1 ft.	Fine textured soils. Tolerates alkaline soils, silt-clay	Great Plains	Low	Excellent hot temp. hardiness. Seed only in mixtures. Seeded by stolons
Chewings Fescue *(Festuca rubra* var. *Commutata)*			Good			Cool	Good establishment rate	Excellent	Bunch			Acidic, infertile, sandy soils	Northern Great Plains	Good	Tolerant to cold temperatures, does well in full sun, may have endophytes
Russian wild rye *(Elymus junceas)*					I	Cool		Good	Bunch		13-24"				

GRASS SPECIES SELECTION GUIDE

Grass Type	Availability	Salt Tolerance	Resistance to Frequent Inundation	Permissible Velocity ft/s	Native or Introduced	Growth Season /	Germination Rate / Growth Rate	Drought Resistance	Bunch / Sod	Root Network Density	Height at Maturity	Optimal Soil Conditions	Regional Adaptation	Shade Tolerance	Notes:
Crownvetch *(Coronilla varia)*	Yes	Fair	Low		I		Slow germination. Rapid growth	Good		Deep		Well drained, pH 6.5-7.0	Northeast, Midwest, Great Plains		Requires Liming
Standard Crested Wheatgrass *(Agropyron desertorium)*	Yes	Fair	Poor		I	Cool	Easily established, slower Growth	Excellant	Bunch	Extremely fibrous, deep	12-30"	Fertile soils from clayey to a sandy loam	Pacific Coast, Southwast, Northern Great Plains		Poorly adapted to hot temperures. Seeded alone
Hard Fescue *(Festuca ovina* var. *duriuscula)*	Yes	Poor	Poor		N	Cool	Fair to estabilish, good growth	Fair	Bunch		12-24"		Pacific Coast, Northern Great Plains	Good	Used frequently in erosion control
Kentucky Bluegrass *(Poa pratensis)*	Yes	Poor	Fair	5	I	Cool	Good est. rate, good growth rate	Fair	Sod	Dense	12-25"	Moist, well drained, fertile, medium texture, pH 6-7	Midwest, Northeast	Fair	Medium hot temp. tolerance, but subject to summer dormancy. Good cold Temperture hardiness
Red Fescue *(Festuca rubra)*	Yes	Poor	Poor	2.5	I	Cool		Good	Sod		12-30"				Fair heat and cold tolerance
Reed Canarygrass *(Phalaris arundinacea)*	Yes	Poor	Excellent	5	N-I	Cool	Harder to establish, good growth rate	Fair	Sod	Shallow, rhizomatous	5'	Silty, clayey soils	Northeast, Southeast and Midwest, other areas with irrigation	Poor	High wildlife value. Not recomended for foot traffic. Seeded or spread by sod

GRASS SPECIES SELECTION GUIDE

Grass Type	**Availability**	**Salt Tolerance**	**Resistance to Frequent Inundation**	**Permissible Velocity ft/s**	**Native or Introduced**	**Growth Season /**	**Germination Rate / Growth Rate**	**Drought Resistance**	**Bunch / Sod**	**Root Network Density**	**Height at Maturity**	**Optimal Soil Conditions**	**Regional Adaptation**	**Shade Tolerance**	**Notes:**
Red Top *(Agrostis alba)*	Yes	Fair	Good	2.5	I	Cool	Fair establishment, good growth rate	Poor	Loose sod	Short rhizomes do not form tight soils	25"	Wet. poorly drained, low fertility, acidic soils. ph 5.5 - 5.6	Northeast, Midwest, other areas with irrigation	Good	Fair heat tolerance, good cold temperature tolerance
Ryegrass Perennial *(Lolium perenne)*	Yes	Fair	Fair		I	Cool	Excellent establishment, fair growth rate	poor	Bunch		13-25"	Sandy, silty soils. pH 6 - 7.5	Midwest, Pacific coast, other areas with irrigation	Poor to fair	Poor to fair hot temperatue tolerance. Poor cold temperature hardiness
Annual rye *(Lolium multiflorum)*	Yes	Fair	Good		I	Cool, short lived	Highest germination rate of Cool season grasses	Poor			25"	Neutral to slightly acidic, medium to good fertility	Most of northern US	Fair to good	Tolerates wet soils, good nurse crop
Side Oats Grama' *(Bouteloua curripendula)*		Fair	Poor		N	Warm	Fast est.	Inferior to Blue Grama, but tolerant	Bunch		2-3 ft.	Variety of soils, provided adequate moisture is available		Poor	Blue Grama preferred over Side Oat in soil stabilization applications
Smooth Brome *(Bromus inermis)*	Yes	Fair	Fair		I	Cool	Fast est.	Fair	Sod	Extensive, rhizomatous	3-4 ft.	Deep well drained, fertile, fine texture, upland soils. Silts-clays	Northeast, Southeast, Midwest, other areas with irrigation	Good	Does well in sandy coarser texture soils also, if supplied w/ N
St. Augustine		Good			N		Fair to good	Fair	Sod			Moist organic well drained soils, pH 6.5		Good	Excellent hot temperture Hardiness, poor cold tolerance
Switchgrass *(Panicum virgatum)*	Yes	Fair	Good		N	Warm	Fast establisment, good growth rate	Fair	Sod	Shallow, rhizomatous	4-5 ft.	Lowland and upland, sandy and silty-clay soils	Southeast, Midwest, and Southern Great Plains	Poor	Hot temp. tolerant. Usually seeded with other warm-season grasses

GRASS SPECIES SELECTION GUIDE

Grass Type	Availability	Salt Tolerance	Resistance to Frequent Inundation	Permissible Velocity ft/s	Native or Introduced	Growth Season /	Germination Rate / Growth Rate	Drought Resistance	Bunch / Sod	Root Network Density	Height at Maturity	Optimal Soil Conditions	Regional Adaptation	Shade Tolerance	Notes:
Tall Fescue *(Festuca arundinacea0)*	Yes	Good	Excellent	5	I	Cool	Fast establishment rate, good growth rate	Excellent, among the highest of the turf grasses	Bunch	Extensive, coarse, deep	3-4 ft.	Tolerates low fertility, pH 5.7 Silts-clays preferred	Northeast, Southeast, Midwest, other areas with irrigation	Fair	Doesn't blend well with other cool season turfs. Usually seeded in pure stands. Good hot temp. tolerance, fair cold hardiness
Timotyhy *(Phleum pratense)*	Yes	Poor	Poor		I	Cool	Fast est.	Fair	Bunch		12-24"	Highly fertile, moist, fine grained soils. pH 6-7	Northeast, Midwest, Pacific Coast		Cold temperature hardy. Slow recovery from hot temperatures and water stress. Seeded in mixtures
Weeping Lovegrass *(Eragrostis curvula)*		Fair	Fair		I	Warm	Fast est	Fair	Bunch	Fibrous	3-4 ft.	Upland, sandy soils	Southwest, Southern Great Plains		Seeded in pure stands
Western Wheatgrass *(Andropyron spicatum)*	Yes	Good	Good		N	Cool	Slow est. Good growth rate	Fair to good	Tight Heavy Sod	Fibrous roots and strong creeping rhizomes	2-3 ft.	Fine textured soils, silts-clays	Midwest through most of western US		Cold temp. hardy. Seeded in mixtures or pure stands. Tolerates silting
Ladino White Clover *(Trifolium repens)*	Yes	Poor	Fair to good		I	Cool, humid	Fast est. Fair growth rate	Poor		Shallow roots		Moist, fine textured soils	Northeast, Southeast, Midwest, other locations with irrigation		Creeping by stolons
Zoysiagrass	Yes	Good	Poor		I	Warm	Slow est.	Excellent	Sod	Fibrous roots and strong creeping rhizomes		Well drained, fine textured, fertile soils. pH 6-7		Good	Japanese variety is the most cold temp. tolerant with most rapid growth

GRASS SPECIES SELECTION GUIDE

Grass Type	***Availability***	***Salt Tolerance***	***Resistance to Frequent Inundation***	***Permissible Velocity ft/s***	***Native or Introduced***	***Growth Season /***	***Germination Rate / Growth Rate***	***Drought Resistance***	***Bunch / Sod***	***Root Network Density***	***Height at Maturity***	***Optimal Soil Conditions***	***Regional Adaptation***	***Shade Tolerance***	***Notes:***
Blue Grama *(Bouteloua gracilis)*	Yes	Fair	Fair		N	Warm	Slow est. Good growth rate	Excellent	Sod	Dense fibrous extensive, but shallow	1-2 ft.	Fine textured, upland Soils. Tolerates alkaline soils, silts-clays	Midwest and most of western US		Good cold temp. tolerance. Seeded in mixtures with other warm-season grasses
Orchardgrass *(Dactylis glomerata)*	Yes	Poor	Poor		I	Cool - Warm	Fast Est. Fair growth rate	Poor to fair	Sod			Silty and clayey soils	Northeast, Southeast, Midwest, most other areas with irrigation	Good	Seeded in mixtures

Appendix C Recommended Plant Species For Use In Bioretention - Herbaceous Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name Common Name	Indicator Status	Habitat	Ponding (days)	Salt	Oil/ Grease	Metals	Insects/ Disease	Exposure	Form	Height	Root System	Native	Wildlife	
Agrostis alba redtop	FAC	Mesic-Xeric	1-2	H	–	H	H	Shade	Grass	2-3'	Fiberous Shallow	Yes	High	–
Andropogon gerardi bluejoint	FAC	Dry Mesic-Mesic	1-2	–	–	–	–	Sun	Grass	2-3'	Fiberous Shallow	Yes	High	–
Andropogon virginicus broomsedge	–	Wet meadow	1-2	L	–			Full sun	Grass	1-3'		Yes	High	Tolerant of fluctuating water levels and drought.
Carex vulpinoidea fox sedge	OBL	Freshwater marsh	2-4	L	–			Sun to partial sun	Grass	2-3.5'	Rhizome	Yes	High	–
Chelone glabra														
Deschampsia caespitosa tufted hairgrass	FACW	Mesic to wet Mesic	2-4	H	–	H	H	Sun	Grass	2-3'	Fiberous Shallow	Yes	High	May become invasive.
Glyceria striata fowl mannagrass, nerved mannagrass	OBL	Freshwater marsh, seeps	1-2	L	–			Partial shade to full shade	Grass	2-4'	Rhizome	Yes	High	–
Hedera helix English Ivy	FACU	Mesic	1-2	–	–	–	H	Sun	Evergreen ground cover	–	Fiberous Shallow	No	Low	–
Hibiscus palustris														
Iris kaempferi														
Iris pseudacorus yellow water iris	OBL	Mesic to wet Mesic	2-4	L	–	H	H	Sun	Thin broad leaves	1-4'	Bulb	Yes	Med	–

H	High Tolerance
M	Medium Tolerance
L	Low Tolerance

FACU	Facultative Upland - Usually occur in non-wetlands, however, occasionally found in wetlands.
FAC	Facultative - Equally likely to occur in wetlands and non-wetlands.
FACW	Facultative Wetland - Usually occur in wetlands, however, occasionally found in non-wetlands.
OBL	Obligate Wetland - Occur almost always in wetlands

Adapted from the Prince George's County Design Manual for use of Bioretention in Stormwater Management

Appendix C Recommended Plant Species For Use In Bioretention - Herbaceous Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name **Common Name**	**Indicator Status**	**Habitat**	**Ponding (days)**	**Salt**	**Oil/ Grease**	**Metals**	**Insects/ Disease**	**Exposure**	**Form**	**Height**	**Root System**	**Native**	**Wildlife**	
Lobelia siphilitica														
Lotus Corniculatus birdsfoot-trefoil	FAC	Mesic-Xeric	1-2	H	L	H	H	Sun	Grass	2-3'	Fiberous Shallow	Yes	High	Member of the legume family.
Onoclea sensibilis sensitive fern, beadfern	FACW							Shade		1-3.5'			H	
Pachysandra terminalis Japanese pachysandra	FACU	Mesic	1-2	–	–	–	M	Shade	Evergreen ground cover	–	Fiberous Shallow	No	Low	–
Panicum virgatum switch grass	FAC to FACU	Mesic	2-4	H	–	–	H	Sun or Shade	Grass	4-5'	Fiberous Shallow	Yes	High	Can spread fast and reach height of 6'.
Vinca major large periwinkle	FACU	Mesic	1-2	–	–	–	H	Shade	Evergreen ground cover	–	Fiberous Shallow	No	Low	Sensitive to soil compaction and pH changes.
Vinca minor common periwinkle	FACU	Mesic	1-2	–	–	–	H	Shade	Evergreen ground cover	–	Fiberous Shallow	No	Low	–
Indian grass														
Little bluestem														
Deer tongue														
Green coneflower														

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BIORETENTION SPECIES SELECTION GUIDE

Appendix C: Recommended Plant Species For Use In Bioretention - Shrub Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name **Common Name**	**Indicator Status**	**Habitat**	**Ponding (days)**	**Salt**	**Oil/ Grease**	**Metals**	**Insects/ Disease**	**Exposure**	**Form**	**Height**	**Root System**	**Native**	**Wildlife**	
Aronia arbutifolia *(Pyrus arbutifolia)* red chokeberry	FACW	Mesic	1-2	H	–	H	M	Sun to partial sun	Deciduous shrub	6-12'	–	Yes	High	Good bank stabilizer. Tolerates drought.
Berberis thunbergii Japanese barberry	FAC	Mesic	2-4	H	H	H	M	Sun	Rounded, broad dense shrub	5-7'	Shallow	No	Med	–
Clethra alnifolia sweet pepperbush	FAC	Mesic to wet Mesic	2-4	H	–	–	H	Sun to partial sun	Ovoid shrub	6-12'	Shallow	Yes	Med	Coastal plain species.
Cornus Stolonifera *(Cornus sericea)* red osier dogwood	FACW	Mesic-Hydric	2-4	H	H	H	M	Sun or shade	Arching, spreading shrub	8-10'	Shallow	Yes	High	Needs more consistent moisture levels.
Cornus amomum silky dogwood	FAC	Mesic	1-2	L	–	–	M	Sun to partial sun	Broad-leaved	6-12'	–	Yes	High	Good bank stabilizer
Euonymous alatus winged euonymous	FAC	Mesic	1-2	H	H	H	M	Sun or shade	Flat, dense horizontal branching shrub	5-7'	Shallow	No	No	–
Euonymous europaeus spindle-tree	FAC	Mesic	1-2	M	M	M	M	Sun to partial sun	Upright dense oval shrub	10-12'	Shallow	No	No	–
Hamamelis virginiana witch hazel	FAC	Mesic	2-4	M	M	M	M	Sun or shade	Vase-like compact shrub	4-6'	Shallow	Yes	Low	–
Hypericum densiflorum common St. John's wort	FAC	Mesic	2-4	H	M	M	H	Sun	Ovoid shrub	3-6'	Shallow	Yes	Med	–
Ilex glabra inkberry	FACW	Mesic to wet Mesic	2-4	H	H	–	H	Sun to partial sun	Upright dense shrub	6-12'	Shallow	Yes	High	Coastal plain species.
Ilex verticillata winterberry	FACW	Mesic to wet Mesic	2-4	L	M	–	H	Sun to partial sun	Spreading shrub	6-12'	Shallow	Yes	High	–

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FAC	Facultative - Equally likely to occur in non-wetlands and wetlands.
FACW	Facultative Wetland - Usually occur in wetlands, however, occasionally found in non-wetlands.
OBL	Obligate Wetland - Almost always occur in wetlands.

Adapted from the Prince George's County Design Manual for use of Bioretention in Stormwater Management

Appendix C: Recommended Plant Species For Use In Bioretention - Shrub Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name **Common Name**	**Indicator Status**	**Habitat**	**Ponding (days)**	**Salt**	**Oil/ Grease**	**Metals**	**Insects/ Disease**	**Exposure**	**Form**	**Height**	**Root System**	**Native**	**Wildlife**	
Itea virginica tassel-white, Virginia sweetspire	OBL	Mesic	1-2	M	–	–	M	Sun or shade	Broad-leaved, deciduous shrub	6-12'	–	Yes	Low	–
Juniperus communis "compressa" common juniper	FAC	Dry Mesic-Mesic	1-2	M	H	H	M-H	Sun	Mounded shrub	3-6'	Deep taproot	No	High	Evergreen
Juniperus horizontalis "Bar Harbor" creeping juniper	FAC	Dry Mesic-Mesic	1-2	M	H	H	M-H	Sun	Matted shrub	0-3'	Deep taproot	No	High	Evergreen
Lindera benzoin spicebush	FACW	Mesic to wet Mesic	2-4	H	–	–	H	Sun	Upright shrub	6-12'	Deep	Yes	High	–
Myrica pennsylvanica bayberry	FAC	Mesic	2-4	H	M	M	H	Sun to partial sun	Rounded, compact shrub	6-8'	Shallow	Yes	High	Coastal plain species.
Physocarpus opulifolius ninebark	FAC	Dry Mesic to wet Mesic	2-4	M	–	–	H	Sun	Upright shrub	6-12'	Shallow	Yes	Med	May be difficult to locate.
Viburnum cassinoides northern wild raisin	FACW	Mesic	2-4	H	H	H	H	Sun to partial sun	Rounded, compacted shrub	6-8'	Shallow	Yes	High	–
Viburnum dentatum arrow-wood	FAC	Mesic to wet	2-4	H	H	H	H	Sun to partial sun	Upright, multi-stemmed shrub	8-10'	Shallow	Yes	High	–
Viburnum lentago nannyberry	FAC	Mesic	2-4	H	H	H	H	Sun to partial sun	Upright, mlti-stemmed shrub	8-10'	Shallow	Yes	High	–

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Appendix C Recommended Plant Species For Use In Bioretention - Tree Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name **Common Name**	**Indicator Status**	**Habitat**	**Ponding (days)**	**Salt**	**Oil/ Grease**	**Metals**	**Insects/ Disease**	**Exposure**	**Form**	**Height**	**Root System**	**Native**	**Wildlife**	
Acer rubrum red maple	FAC	Mesic-Hydric	4-6	H	H	H	H	Partial sun	Single to multi-stem tree	50-70'	Shallow	Yes	High	-
Amelanchier canadensis shadbush	FAC	Mesic	2-4	H	M	-	H	Partial sun	Single to multi-stem tree	35-50'	Shallow	Yes	High	Not recommended for full sun.
Betula nigra river birch	FACW	Mesic-Hydric	4-6	-	M	M	H	Partial sun	Single to multi-stem tree	50-75'	Shallow	Yes	High	Not susceptible to bronze birch borer.
Betula populifolia gray birch	FAC	Xeric-Hydric	4-6	H	H	M	H	Partial sun	Single to multi-stem tree	35-50'	Shallow to deep	No	High	Native to New England area.
Fraxinus americana white ash	FAC	Mesic	2-4	M	H	H	H	Sun	Large tree	50-80'	Deep	Yes	Low	-
Fraxinus Pennsylvanica green ash	FACW	Mesic	4-6	M	H	H	H	Partial sun	Large tree	40-65'	Shallow to deep	Yes	Low	-
Ginko biloba Maidenhair tree	FAC	Mesic	2-4	H	H	H	H	Sun	Large tree	50-80'	Shallow to deep	No	Low	Avoid female species-offensive odor from fruit.
Gleditsia triacanthos honeylocust	FAC	Mesic	2-4	H	M	-	M	Sun	Small caopled large tree	50-75'	Shallow to deep variable taproot	Yes	Low	Select thornless variety.
Juniperus virginiana eastern red cedar	FACU	Mesic-Xeric	2-4	H	H	-	H	Sun	Dense single stem tree	50-75'	Taproot	Yes	Very high	Evergreen
Liquidambar styraciflua sweet gum	FAC	Mesic	4-6	H	H	H	M	Sun	Large tree	50-70'	Deep taproot	Yes	High	Edge and perimeter; fruit is a maintenance problem.
Nyssa sylvatica black gum	FACW	Mesic-Hydric	4-6	H	H	H	H	Sun	Large tree	40-70'	Shallow to deep taproot	Yes	High	-

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L Low Tolerance

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Appendix C Recommended Plant Species For Use In Bioretention - Tree Species

Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name Common Name	Indicator Status	Habitat	Ponding (days)	Salt	Oil/ Grease	Metals	Insects/ Disease	Exposure	Form	Height	Root System	Native	Wildlife	
Platanus acerifolia London plane-tree	FACW	Mesic	2-4	H	-	-	M	Sun	Large tree	70-80'	Shallow	No	Low	Tree roots can heave sidewalks.
Platanus occidentalis sycamore	FACW	Mesic-Hydric	4-6	M	M	M	M	Sun	Large tree	70-80'	Shallow	Yes	Med	Edge and perimeter; fruit is a maintennance problem; tree is also prone to windthrow.
Populus deltoides eastern cottonnwood	FAC	Xeric-Mesic	4-6	H	H	H	L	Sun	Large tree with spreading branches	75-100'	Shallow	Yes	High	Short lived.
Quercus bicolor Swamp white oak	FACW	Mesic to wet Mesic	4-6	H	-	H	H	Sun to partial sun	Large tree	75-100'	Shallow	Yes	High	One of the faster growing oaks.
Quercus coccinea scarlet oak	FAC	Mesic	1-2	H	M	M	M	Sun	Large tree	50-75'	Shallow to deep	Yes	High	-
Quercus macrocarpa bur oak	FAC	Mesic to wet Mesic	2-4	H	H	H	M	Sun	Large spreading tree	75-100"	Taproot	No	High	Native to Midwest.
Quercus palustris pin oak	FACW	Mesic-Hydric	4-6	H	H	H	M	Sun	Large tree	60-80'	Shallow to deep taproot	Yes	High	-
Quercus phellos willow oak	FACW	Mesic to wet Mesic	4-6	H	-	-	H	Sun	Large tree	55-75'	Shallow	Yes	High	Fast growing oak.
Quercus rubra red oak	FAC	Mesic	2-4	M	H	M	M	Sun to partial sun	Large spreading tree	60-80'	Deep taproot	Yes	High	-
Quercus shumardii Shumard's red oak	FAC	Mesic	2-4	H	H	H	M	Sun to partial sun	Large spreading tree	60-80'	Deep taproot	No	High	Native to Southeast.

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Species	Moisture Regime		Tolerance						Morphology			General Characteristics		Comments
Scientific Name Common Name	Indicator Status	Habitat	Ponding (days)	Salt	Oil/ Grease	Metals	Insects/ Disease	Exposure	Form	Height	Root System	Native	Wildlife	
Robinia pseudo-acacia black locust	FAC	Mesic-Xeric	2-4	H	H	H	M	Sun	Typically tall and slender	30-50'	Shallow	Yes	Low	Edge and perimeter; fruit is a maintenance problem; tree is also prone to windthrow.
Sophora japonica Japanese pagoda tree	FAC	Mesic	1-2	M	M	–	M	Sun	Shade tree	40-70'	Shallow	No	Low	Fruit stains sidewalk.
Taxodium distichum bald cypress	FACW	Mesic-Hydric	4-6	–	–	M	H	Sun to partial sun	Typically single stem tree	75-100'	Shallow	Yes	Low	Not well documented for planting in urban areas.
Thuja occidentalis arborvitae	FACW	Mesic to wet Mesic	2-4	M	M	M	H	Sun to partial sun	Dense single stem tree	50-75'	Shallow	No	Low	Evergreen
Zelkova serrata Japanese zelkova	FACU	Mesic	1-2	M	M	–	H	Sun	Dense shade tree	60-70'	Shallow	No	Low	Branches can split easily in storms.

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GLOSSARY

BIOFILTERS:
Grass depression areas such as engineered channels or swales that are used to collect and filter urban stormwater. This term was developed in the Pacific Northwest.

BIORETENTION:
A water quality practice that utilizes landscaping and soils to treat urban stormwater runoff by collecting it in shallow depressions, before filtering through a fabricated planting soil media.

BEST MANAGEMENT PRACTICE (BMP):
A structural device designed to temporarily store or treat urban stormwater runoff in order to mitigate flooding, reduce pollution and provide other amenities.

COEFFICIENT OF PERMEABILITY:
An engineering constant value which is used to measure the capability of a filter media to pass liquid through a given surface area.

DISCONNECTED IMPERVIOUS SURFACES:
Discontinuous impervious surfaces that allow for the infiltration and filtration of precipitation. An example of this is a residential subdivision in which each dwelling's roof tops drain through a vegetative strip before reaching the road surface.

DRY SWALE:
An open drainage channel or depression, explicitly designed to detain and promote the filtration of stormwater runoff into an underlying soil media.

EDGE EFFECT:
Extensive, well-defined edges between the impervious and pervious surfaces.

EXFILTRATE:
The downward movement of runoff through the bottom of a treatment system into the soil layer.

FIBRIC PEAT:
Organic material, usually derived from wetland vegetation, in which the undecomposed fibrous organic materials are easily identifiable. Also characterized by low bulk density and a highly porous structure.

FILTER BED CHAMBER:
The section of a constructed filtration device that houses the filter material and the outflow piping.

FILTER STRIPS:
A vegetated area that treats sheetflow and/or interflow by removing sediment and other pollutants. The area may be grass-covered, forested or of mixed vegetative cover (e.g. wildflower meadow).

FOREBAY:
Additional storage space located near a stormwater BMP inlet that serves to trap incoming coarse sediments before they accumulate in the main treatment area.

FREEBOARD:
The space from the top of an embankment or a channel bank to the highest water elevation expected for the largest storm designed to be stored or conveyed. The space is required as a safety margin in a pond, basin or channel.

GEOTEXTILE FABRIC:
A synthetic textile of relatively small mesh or pore size that is used to (a) allow water to pass through while keeping sediment out (permeable), or (b) prevent both runoff and sediment from passing through (impermeable). Also known as filter fabric.

HEAD:
Height or water elevation above a given location and the pressure exerted by it due to gravity.

HEMIC PEAT:
An organic material, usually derived from wetland vegetation that is moderately decomposed, has a moderate bulk density and modest porosity.

HERBICIDES:
Chemicals developed to control or eradicate plants.

HIGH-INPUT LAWN:
A heavily irrigated lawn subject to high usage of chemicals: fertilizers, pesticides and fungicides.

HUMIC:
A soil or other material characterized by a high organic content.

IMPERVIOUS:
The characteristic of a material which prevents the infiltration or passage of liquid through it. This may apply to roads, streets, parking lots, rooftops and sidewalks.

INFLOW REGULATION:
The control, usually by an engineering device, of the inflow into a BMP.

INSECTICIDES:
Chemicals developed to control or eradicate insects.

LOW-INPUT LAWN:
A lawn that is regularly mowed but is not subjected to a high usage of chemicals and irrigation.

MICRO-ENVIRONMENT:
This term refers to the conditions created under which a separate, smaller environment exists distinct from the dominant one, which can affect and be affected by the immediate surroundings.

NPDES:
Acronym for the National Pollutant Discharge Elimination System, which regulates point source and stormwater discharge. NPDES standards were promulgated by the EPA in accordance with the Clean Water Act.

OFF-LINE:
A water quality system designed by diverting stormwater from a stream or storm drainage system.

ON-LINE:
A water quality system designed to retain stormwater in its original stream channel or storm drainage system.

OPEN VEGETATED CHANNELS:
Also known as swales, grass channels, and biofilters. These systems are used for the conveyance, retention, infiltration and filtration of stormwater runoff.

PEA GRAVEL CURTAIN DRAIN:
A thin wall of small, river-run gravel used to convey water to the sides/bottom of bioretention practices.

PEA GRAVEL DIAPHRAGM:
A stone trench filled with small, river-run gravel used as pretreatment and inflow regulation in stormwater filtering systems.

PEAK DISCHARGE (FLOW RATE):
The maximum instantaneous rate of flow during a storm, usually in reference to a specific design storm event.

PERMISSIBLE VELOCITY:
The maximum rate of flow allowable for vegetated open channels, before erosive channel conditions occur.

PERVIOUS:
Any material that allows for the passage of liquid through it.

PUBLIC TURF:
Pervious land held in the public domain. Examples include parks, golf courses, cemeteries, median strips and school grounds.

RUNOFF PRETREATMENT:
Technique employed in a stormwater BMP to retain storage volumes or prevent clogging by trapping coarse materials before they enter the system.

RAINFALL FREQUENCY SPECTRUM:
The frequency distribution of cumulative rainfall volume generated by all storm events. This analysis is used to determine how much rain can be treated in a stormwater filter, and how much may be bypassed.

RATE-BASED DESIGN:
BMP design which uses the discharge in volume per unit of time as a basis for sizing the practice.

RESIDENTIAL SUBDIVISION:
A large land area divided into smaller parcels for the purpose of housing.

RETROFIT:
The installation of a new BMP or the improvement of an existing one in a previously developed area.

RUNON:
The flow of stormwater from impervious cover to pervious cover.

SAND FILTER:
A stormwater quality treatment practice, whereby runoff is diverted into a self-contained bed of sand. The runoff is then strained through the sand, collected in underground pipes and returned back to the stream or channel.

SARA:
An acronym for Congressional legislation referring to the Superfund Amendments and Reauthorization Act of 1980.

SEDIMENTATION CHAMBER:
This is a section of a BMP that provides for the settling out of large particles from suspension.

SHEET FLOW:
Stormwater flowing sheet-like over pervious or impervious surfaces.

SHORT-CIRCUITING:
The passage of runoff through a BMP in a timespan or flowpath without adequate treatment.

STORMWATER HOT SPOTS:
Land-uses or activities that generate highly contaminated runoff. Examples include fueling stations and airport de-icing facilities.

STORMWATER INFILTRATION SYSTEMS:
BMP's which are designed to percolate runoff into the underlying soil.

STORMWATER PONDS:
A land depression created for the detention or retention of stormwater runoff.

STORMWATER WETLAND:
A shallow, constructed pool that captures stormwater and allows for the growth of characteristic wetland vegetation.

SUBMERGED GRAVEL FILTERS:
A filtering BMP which uses a gravel based substrate, supporting a wetland vegetation cover crop, to treat urban runoff.

TIME OF CONCENTRATION:
An engineering term representing the travel time of runoff through a watershed, subwatershed or catchment.

ULTRA-URBAN:
A region dominated by highly developed areas in which very little pervious surfaces exist.

UNDERDRAIN SYSTEM:
A perforated pipe system in a gravel bed, installed on the bottom of filtering BMP's, which are used to collect and remove filtered runoff.

VOLUME-BASED DESIGN:
A BMP design which uses the volume of runoff as a basis for sizing the practice.

VOLUMETRIC RUNOFF COEFFICIENT (RV):
A value that is applied to a given rainfall volume to yield a corresponding runoff volume.

WET SWALE:
An open drainage channel or depression, explicitly designed to retain water or intercept groundwater for water quality treatment.

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