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DISTRICT DEPARTMENT OF THE ENVIRONMENT

Stormwater Guidebook

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Chapter 1

Introduction to the Guidebook

1.0 Introduction

On January 1, 1988, the District of Columbia Storm Water Management Regulations became effective. These regulations require that development and redevelopment projects provide a system to manage the quality and quantity control of storm water runoff from those sites. Inadequate management of increased storm water runoff resulting from development increases combined sewer overflows, overloads system transport capacity of streams and storm sewers, and creates adverse impacts downstream such as flash flooding, channel erosion, and surface and groundwater quality degradation.

As a highly urbanized city with runoff patterns no longer gathering to natural drainage routes, the District of Columbia is very sensitive to urban runoff. Point sources are controlled by permits, pretreatment programs, and technologically advanced treatment facilities. Non-point source pollution, especially due to runoff from the city's impervious surfaces, is more difficult to control. This design manual presents the minimum standard criteria to be used by design engineers and planners for the planning, design, and construction of Best Management Practices (BMPs) in order to comply with the District of Columbia Storm Water Management Regulations (District of Columbia Municipal Regulations (DCMR) Title 21, Chapter 5).

1.1 Purpose And Scope of the Guidebook

The purpose of 21 DCMR, Chapter 5 is to promote public health, safety, and welfare by establishing requirements and procedures to control the adverse impacts of increased storm water runoff. The application of the provisions and the procedures stated in 21 DCMR, Chapter 5, together with the specific design criteria stated in the *Storm Water Guidebook*, establishes the District of Columbia's Storm Water Management program. The Watershed Protection Division, within the Department of Health’s Environmental Health Administration (also referred to as the "Department" in this guidebook) shall be responsible for coordinating and enforcing the provisions stated in 21 DCMR, Chapter 5.

This Guidebook is intended only as a guide. The design engineer shall be fully responsible for reviewing, verifying and selecting the applicability of all material presented herein as it pertains to the specific project under design and to submit, as required, all reports, design computations, worksheets, geotechnical studies, surveys, rights-of-way determinations, etc., in a fashion prescribed hereinafter. All such required submittals will bear the seal and signature of the Professional Engineer licensed to practice in the District of Columbia, and who is responsible for that portion of the submitted project.

1.2 Impacts of Urban Runoff

There are two primary impacts that typically result from new development. First, the hydrologic response of the area is changed. This change typically includes increased runoff volumes, flows, velocities, and reduced base flow. Second, increases in human activities, which may range from heavy automobile use to complex chemical processing, generate pollutants which are washed off into District of Columbia surface and groundwater bodies.

1.2.1 Hydrologic Impacts

Urban development can cause significant changes in the rainfall-runoff relationship within a watershed. Typically, the volume and rate of runoff for a given rainfall event increases with urban development (see Figures 1.1 and 1.2). As a result of the transformation from a natural catchment to a typical sewershed, there is an increase in the amount of imperviousness. Also, the natural drainage patterns are modified and the runoff is channeled into storm drains, road gutters, and paved channels. This leads to an increase in runoff velocity, a shorter time for the runoff to travel downstream, and decreased infiltration into groundwater aquifers.

Many small watersheds or small drainage areas in the District of Columbia are rapidly being developed for residential, commercial, and industrial uses. The natural drainage systems or existing storm/combined sewer systems are not capable of adjusting to the dramatic hydrologic changes that occur as a result of such development. A major consequence associated with this phenomenon are the increased magnitude and frequency of flooding often causing flooding of city streets during and after high frequency storm events.

1.2.2 Water Quality Impacts

As land is developed, naturally vegetated areas that once allowed water to infiltrate and purify itself in the soil are replaced with impervious surfaces. These impervious surfaces accumulate pollutants deposited from the atmosphere, leaked from vehicles, or windblown from adjacent areas. During storm events, these pollutants quickly wash off and are rapidly delivered to downstream waters. Some common pollutants found in urban storm water runoff and their sources are profiled in Table 1.1.

Most nonpoint source pollution in the District of Columbia comes from urban runoff, which includes surface runoff, combined sewer overflows, new construction, and land disposal of pollutants. Approximately 65% of the District of Columbia's natural groundcover has been replaced with impervious surfaces. As the percentage of impervious surface increases, the volume and rate of surface runoff increases. With the city receiving an average of 40 inches of precipitation per year, approximately 87,000 ac-ft of overland runoff may potentially reach District of Columbia waterways each year through overland flow, stream flow, and sewer systems. Annual pollutant loads for the District of Columbia were calculated using the Alternative Model for Urban Water Quality (AMUWQ). The model output provides an estimate of the total annual load at the 1982 levels of treatment (Table 1.2).

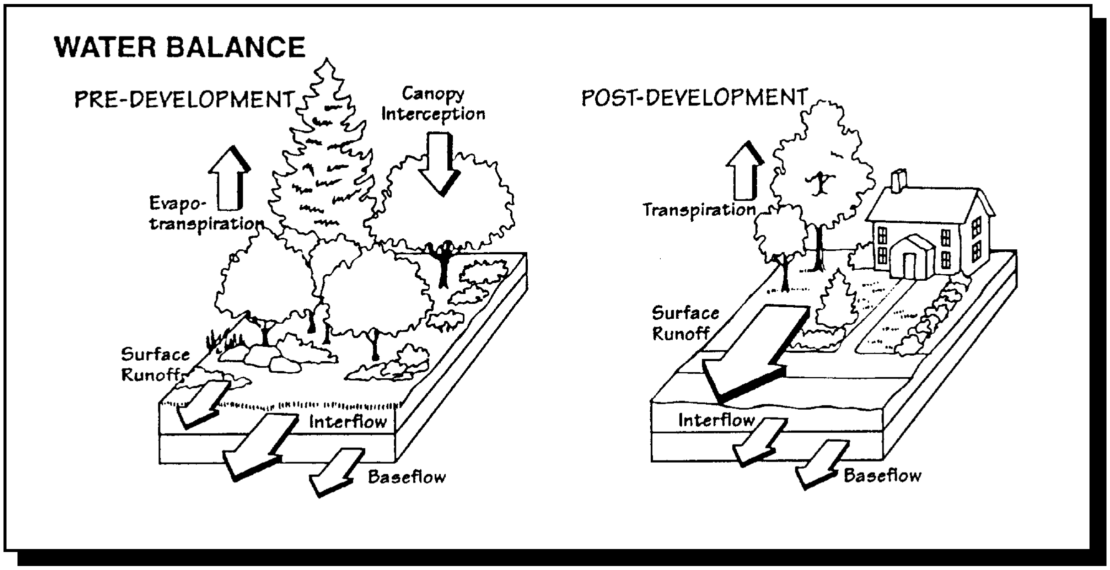


Figure 1.1 Changes in the Water Balance Resulting from Urbanization

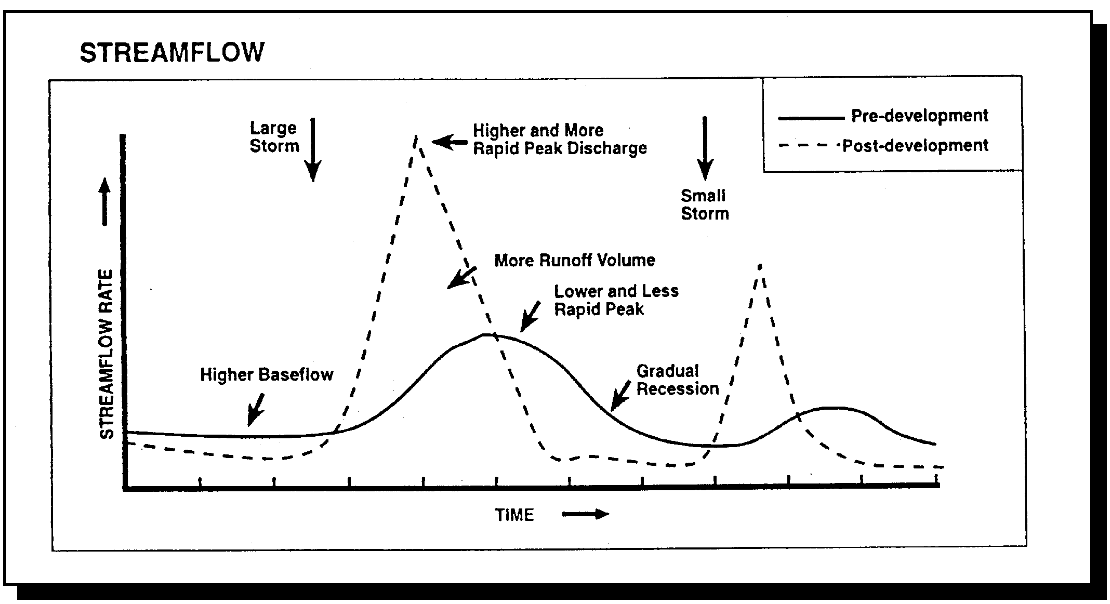


Figure 1.2 Changes in Streamflow Resulting from Urbanization

Table 1.1 Common Pollutants Found in Urban Storm Water Runoff and Their Sources

Pollutant	Automobile/ Atmospheric Deposition	Urban Housekeeping / Landscaping Practices	Industrial Activities	Construction Activities	Non-Storm Water Connections	Accidental Spills & Illegal Dumping
Sediments	X	X	X	X		
Nutrients	X	X	X	X	X	X
Bacteria and Viruses	X	X		X	X	X
Oxygen Demanding Substances		X	X	X	X	X
Oil and Grease	X	X	X	X	X	X
Anti-Freeze	X	X		X	X	X
Hydraulic Fluid	X	X	X	X	X	X
Paint		X		X	X	X
Cleaners and Solvents	X	X	X	X	X	X
Wood Preservatives		X		X	X	X
Heavy Metals	X	X	X	X	X	X
Chromium	X	X	X			
Copper	X	X	X			
Lead	X	X	X			
Zinc	X	X	X			
Iron	X		X			
Cadmium	X		X			
Nickel	X		X			
Magnesium	X		X			
Toxic Materials						
Fuels	X		X	X	X	X
PCBs	X				X	X
Pesticides	X	X	X	X	X	X
Herbicides	X		X	X	X	X
Floatables		X	X	X		

Source: Municipal Handbook, State of California, 1993

Table 1.2 Estimated Total Annual Pollutant Loading to Waterways in the District of Columbia

Pollutant Parameter	**Total Annual Load (million pounds per year)**	**Total Annual Load (pounds per acre per year)**
Biological Oxygen Demand	69.6	1,733
Total Suspended Solids	309.1	7,699
Total Nitrogen	12.0	299
Total Phosphorus	2.95	73
Lead	0.98	24

Source: Young and Danner, 1982

Chapter

2

District of Columbia Minimum Control Requirements for Storm Water Management

2.0 District of Columbia Minimum Control Requirements for Storm Water Management

This chapter presents a unified approach for sizing storm water BMPs in the District of Columbia to meet pollutant removal goals, reduce peak discharges, and pass extreme floods. For a summary, please consult Table 2.1 below. The remaining sections describe the four sizing criteria in detail and present guidance on how to properly compute and apply the required storage volumes.

Table 2.1 Summary of the District of Columbia Storm Water Criteria

Sizing Criteria	Description of Storm Water Sizing Criteria
Water Quality Volume (V_w) (ft^3)	$V_w = \frac{R*I_a}{12}$ Where: V_w = water quality volume to be treated (ft^3) R = runoff depth (in), see Table 2.2 I_a = impervious area (ft^2) 12 = conversion factor
2 Year Storm Control (Qp_2)	The peak discharge rate from the 2- year storm event controlled to the pre-development rate.
15 Year Storm Control (Qp_{15})	The peak discharge rate from the 15-year storm event controlled to the pre-development rate.
Extreme Flood Requirements (Q_f)	When storm water runoff from a planned development will increase the downstream discharge into an area designated as a flood hazard watershed, an analysis of the downstream peak discharge for a 100 year frequency storm event must be completed, and appropriate controls to avoid exceeding this peak discharge must be installed.

This chapter also presents a list of acceptable BMP options that can be used to comply with the sizing criteria.

All storm water management administration, including review(s) and modification(s) of appurtenances design plans, and sheet flow storm water runoff controls shall be the sole responsibility of the Department. All storm water runoff controls shall conform to 21 DCMR, Chapter 5 as well as the criteria set forth in this chapter. All of the requirements in Chapter 2 may be altered by the Department if it determines that alternative approaches may better control flood damage, mitigate accelerated stream erosion and sedimentation, and improve surface water quality.

2.1 Water Quality Requirements (V_w)

By EPA definition, the first half-inch of runoff should contain 85 - 90% of the pollutants in the initial runoff volume. To meet water quality standards, the District of Columbia requires that the first flush runoff be treated by filter media, natural percolation, detention or extended detention or an equivalent process within 48 hours, then released.

The District of Columbia's management strategy for treating storm water is to capture and isolate the first-flush runoff from impervious surfaces within the contributing drainage area. The following equation is used to determine the water quality volume, V_w (in ft^3 of storage):

$$V_w = \frac{R * I_a}{12} \tag{2.1}$$

Where: V_w = water quality volume to be treated (ft^3)
R = runoff depth (in), see Table 2.2
I_a = impervious area (ft^2)
12 = conversion factor

In the District of Columbia, the post-development land use characteristics and the projected future activities of the impervious area determine the depth of runoff that must be held for water quality treatment (Table 2.2).

Table 2.2 Runoff Depth to Be Treated Based on Post-Development Land Use

Runoff Depth (R)	Land Use
0.5 inches	Parking lots, city streets (with or without on-street parking), highspeed roads
0.3 inches	Rooftops, sidewalks, pedestrian plaza areas

Capturing the first flush runoff is essential to removal of the majority of pollutants. In the District of Columbia, the first flush runoff volumes have been separated into two categories based on land use: 0.5" runoff depth for parking lots, city streets, and high speed roads; and 0.3" runoff depth for rooftops, sidewalks and pedestrian plaza areas. This grouping is based typical pollutant loads from the different land uses.

Pollutants accumulate on impervious areas, then are at least partially washed away by subsequent storm events. This phenomenon is commonly referred to as the first flush of runoff and is characterized with the highest load of pollutants. This is significant because the majority of storm events produce 0.5" or less of runoff. In one study, pollutant loads removed by the first 0.5" of runoff averaged about 52 and 39 % of the total storm load averages (Chang, 1990).

First flush pollutant contributions are typically higher from parking lots, city streets, and highspeed roads (within the 0.5" runoff depth category). Rooftops, sidewalks, and pedestrian areas represent lower concentrations of sediment and nutrients compared to parking lots and streets (Steuer et al., 1997; Bannerman et al., 1993; Waschbusch et al., 2000). PAHs, oil, and grease concentrations also tend to increase in commercial and industrial areas (Sturm, 2000); these areas are characterized by extensive parking lots and streets.

2.2 Quantity Control Requirements (Qp_2 and Qp_{15})

To meet quantity control and peak discharge requirements, the District of Columbia requires the following:

2-Year Storm Control (Qp_2) Maintain the post-development peak discharge for a 24-hour, 2-year frequency storm event at a level that is equal to or less than the 24-hour, 2-year pre-development peak discharge rate through storm water management practices that control the volume, timing, and rate of flows. The rainfall intensity - duration - frequency curve for the District of Columbia is provided in Appendix A.

15-Year Storm Control (Qp_{15}) Maintain the post-development peak discharge for a 24-hour, 15-year frequency storm event at a level that is equal to or less than the 24-hour, 15-year pre-development peak discharge rate through storm water management practices that control the volume, timing, and rate of flows. The rainfall intensity - duration - frequency curve for the District of Columbia is provided in Appendix A.

All storm water facilities and conveyance systems shall be designed using the 15-year design frequency with ultimate land use conditions. If another higher storm frequency is needed, the review engineer will require that all of the computations and assumptions be submitted for detailed evaluation. Where the storm water management facility discharges into a closed conduit system, the release rate of the structure must be designed so as not to adversely affect the downstream hydraulic gradient. See Appendix B for details and guidance on the design of storm water conveyance systems. See Appendix C for details and guidance on the design of flow control structures.

2.3 Extreme Flood Requirements (Q_f)

Where a development is planned in which the storm water runoff will increase the downstream discharge into an area designated as a flood hazard watershed, as delineated on the National Flood Insurance Flood Hazard Boundary Maps (FHBM), the developer shall complete an analysis of the downstream peak discharge for a 100-year frequency storm event, and shall install the appropriate controls to avoid exceeding this peak discharge.

The final release rate of the facility should be modified if any increase in flooding or stream channel erosion would result at a downstream structure, highway, or natural point of restricted streamflow. The release rate of the structure shall:

1. Be reduced to a level that will prevent any increase in flooding or stream channel erosion at the downstream control point;

2. Be not less than the 1-year pre-development peak discharge rate; and when deemed necessary by the Department, the developer shall submit an analysis of the impacts of storm water flows downstream in the watershed. The analysis should include hydrologic and hydraulic calculations necessary to determine the impact of the hydrograph timing modifications of the proposed development upon any control structure, highway, or natural point of restricted streamflow, established with the concurrence of the Department, downstream of a tributary of the following size:

    - The first downstream tributary whose drainage area equals or exceeds the contributing area to the facility; or
    - The first downstream tributary whose peak discharge exceeds the largest designed release rate of the facility.

For on-line designs, the limits of the recorded 100-year floodplain easement or surface water easement sufficient to convey the 100-year flow must be shown. The easement must be acceptable to the District of Columbia floodplain review authority.

The minimum horizontal clearance between a residential structure and the 100-year floodplain is 25 feet. Structure locations, existing and proposed, are to be shown when it is not absolutely clear whether 25-foot setback from the floodplain can be met.

2.4 Additional Storm Water Management Requirements

Any storm water runoff discharge facility which may receive storm water runoff from areas which may be potential sources of oil and grease contamination in concentration exceeding 10 milligrams per liter (mg/l) shall include a baffle, skimmer, grease trap or other mechanism which prevents oil and grease from escaping the storm water discharge facility in concentrations that would violate or contribute to the violation of applicable water quality standards in the receiving water of the District of Columbia.

Any storm water discharge facility which receives storm water runoff from areas used to confine animals and which discharges directly into receiving waters shall be designed to prevent at least eighty-five percent (85%) of the organic animal wastes from escaping the storm water discharge facility. The discharge from the facility shall not violate the water quality standards of the District of Columbia.

2.5 Hydrology Methods

The following are the acceptable methodologies and computer models for estimating runoff hydrographs before and after development. These methods are used to predict the runoff response from given rainfall information and site surface characteristic conditions. The design storm frequencies used in all of the hydrologic engineering calculations will be based on design storms required in this guidebook unless circumstances make consideration of another storm intensity criteria appropriate.

- Rational Method & Modified Rational Method
- Natural Resource Conservation Service TR-55
- TR-20, HEC-1, and SWMM computer models

These methods are given as valid in principle, and are applicable to most storm water management design situations in the District of Columbia. Other methods may be used when the Department approves their application.

The use of the Natural Resource Conservation Service storage indication routing method or an equivalent acceptable method may be required to route the design storms through storm water facilities.

See Appendix A for further details and guidance.

2.6 Pollutant Load Calculations

For all development sites, the following calculations must be performed and certified by a professional engineer (civil or environmental engineer) licensed to practice in the District of Columbia.

1. Estimate the post-development pollutant export of total nitrogen (TN), total phosphorus (TP), and total suspended solids (TSS)
2. Estimate the annual TN, TP, and TSS loads which should be removed by the application of approved BMP(s).

All new development is required to provide these calculations by using the methods outlined in

Appendix D. The loading calculation sheets should be submitted at the 85% project design completion stage to be reviewed by the Department before the submission for final approval.

2.7 Acceptable Urban BMP Options

This section sets forth five acceptable groups of BMPs that can be used to meet the storm water water quality (V_w) criteria.

The dozens of different BMP designs currently used in the District of Columbia are assigned into five general categories for storm water quality control:

BMP Group 1	filtering systems
BMP Group 2	infiltration practices
BMP Group 4	storm water ponds
BMP Group 5	storm water wetlands
BMP Group 6	open channels

A sixth group is set forth to explicitly provide storm water detention to meet Qp_2, Qp_{15}, and / or Q_f requirements:

BMP Group 3	storage practices

Within each BMP group, detailed performance criteria are presented that govern feasibility, conveyance, pretreatment, treatment, environmental/landscaping and maintenance requirements (see Chapter 3).

To be considered an effective BMP, a design shall be capable of:

1. capturing and treating the full water quality volume (V_w),
2. having an acceptable longevity rate in the field.

Guidance on selecting the most appropriate combination of BMPs is provided in Chapter 4.

BMP Group 1. Filtering Systems

Practices that capture and temporarily store the V_w and pass it through a filter bed of sand, organic matter, soil or other media are considered to be filtering practices. Filtered runoff may be collected and returned to the conveyance system. Design variants include:

F-1 surface sand filter
F-2 one-chamber underground sand filter

F-3 three-chamber underground sand filter
F-4 perimeter sand filter
F-5 vertical sand filter
F-6 organic filter
F-7 bioretention areas
F-8 roof downspout system

BMP Group 2. Infiltration Practices

Practices that capture and temporarily store the V_w before allowing it to infiltrate into the soil over a two day period include:

I-1 infiltration trench
I-2 infiltration basin

BMP Group 3. Storage Practices

Storage practices are explicitly designed to provide storm water detention. Storage practices are not considered an acceptable practice to meet the water quality volume requirement (V_w). Design variants include:

S-1 underground vault
S-2 dry pond
S-3 rooftop storage

Design guidance and criteria for rooftop storage practices is provided in Appendix H.

BMP Group 4. Storm Water Ponds

Practices that have a combination of a permanent pool, extended detention or shallow marsh equivalent to the entire V_w include:

P-1 micropool extended detention pond
P-2 wet pond
P-3 wet extended detention pond
P-4 pocket pond

BMP Group 5. Storm Water Wetlands

Practices that include significant shallow marsh areas to treat urban storm water but often may also incorporate small permanent pools and/or extended detention storage to achieve the full V_w include:

W-1 shallow wetland
W-2 Extended Detention (ED) shallow wetland
W-3 pocket wetland

BMP Group 6. Open Channel Practices

Vegetated open channels that are explicitly designed to capture and treat the full V_w within dry or wet cells formed by checkdams or other means include:

O-1 dry swale
O-2 wet swale

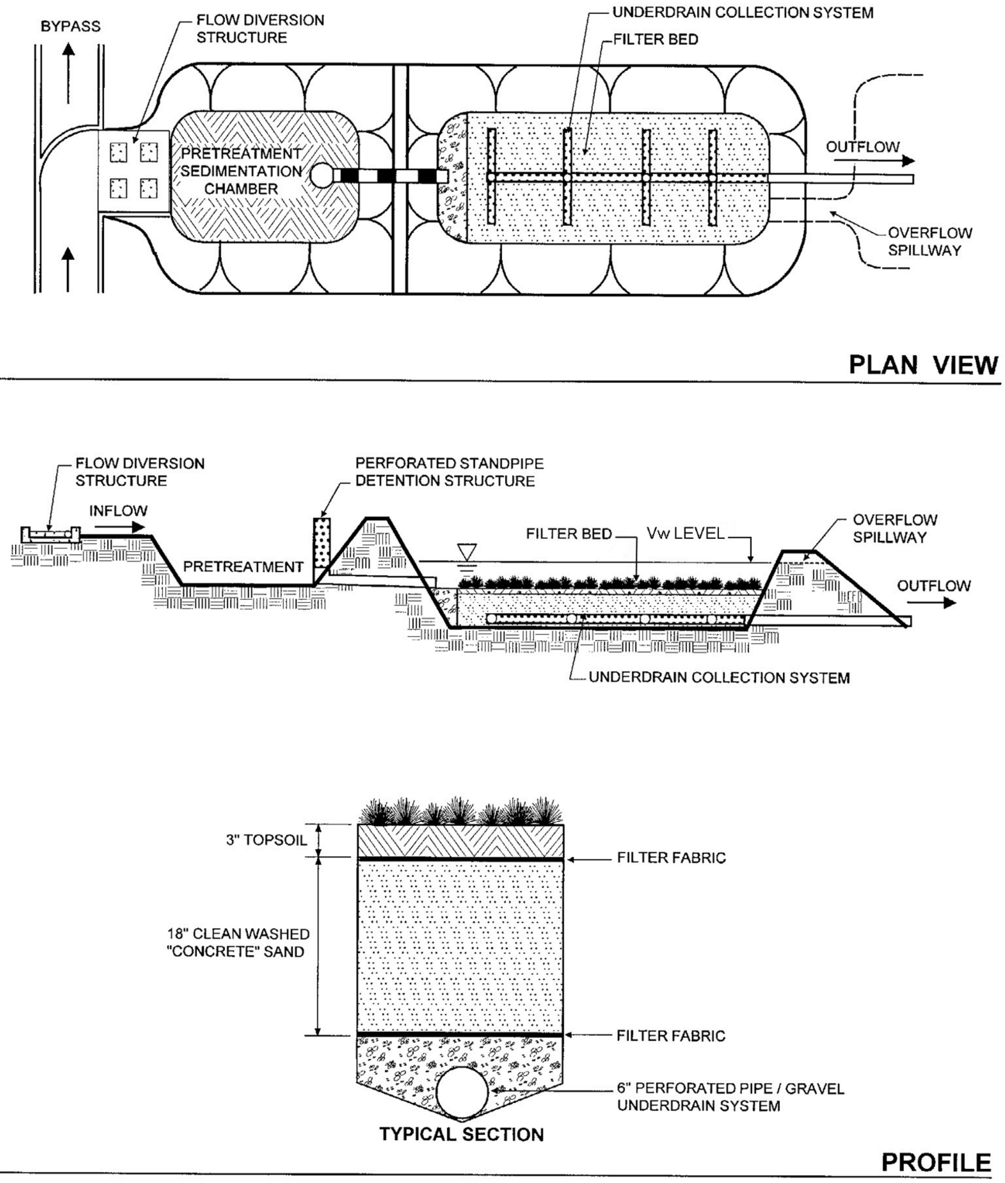
Section 3.1 Storm Water Filtering Systems

Definition: Practices that capture and temporarily store the V_w and pass it through a filter bed of sand, organic matter, soil or other media. Filtered runoff may be collected and returned to the conveyance system, or allowed to partially exfiltrate into the soil. Design variants include:

- F-1 surface sand filter
- F-2 one-chamber underground sand filter
- F-3 three-chamber underground sand filter
- F-4 perimeter sand filter
- F-5 vertical sand filter
- F-6 organic filter
- F-7 bioretention
- F-8 roof downspout system

Filtering systems are typically not to be designed to provide storm water detention (Qp_2, Qp_{15}, and / or Q_f), but they may be in some circumstances. Filtering practices shall generally be combined with a separate facility to provide those controls. However, in combined sewer areas, the three-chamber underground sand filter can be modified by expanding the first or settling chamber, or adding an extra chamber between the filter chamber and the clear well chamber to handle the detention volume, which is subsequently discharged at a pre-determined rate through an orifice and weir combination.

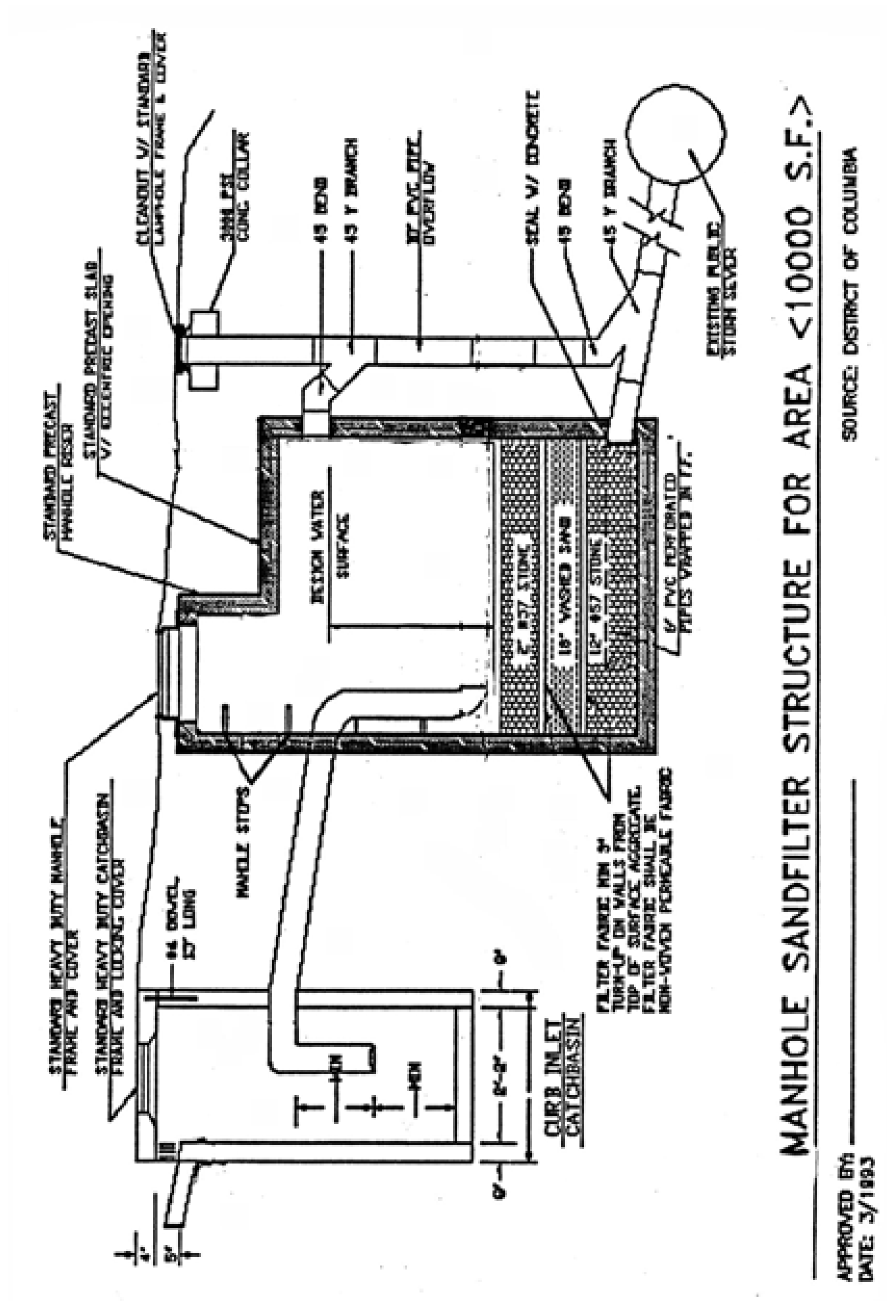
F-1	**Figure 3.1** Example of Surface Sand Filter	F-1



Source: Adapted from MDE, 2000

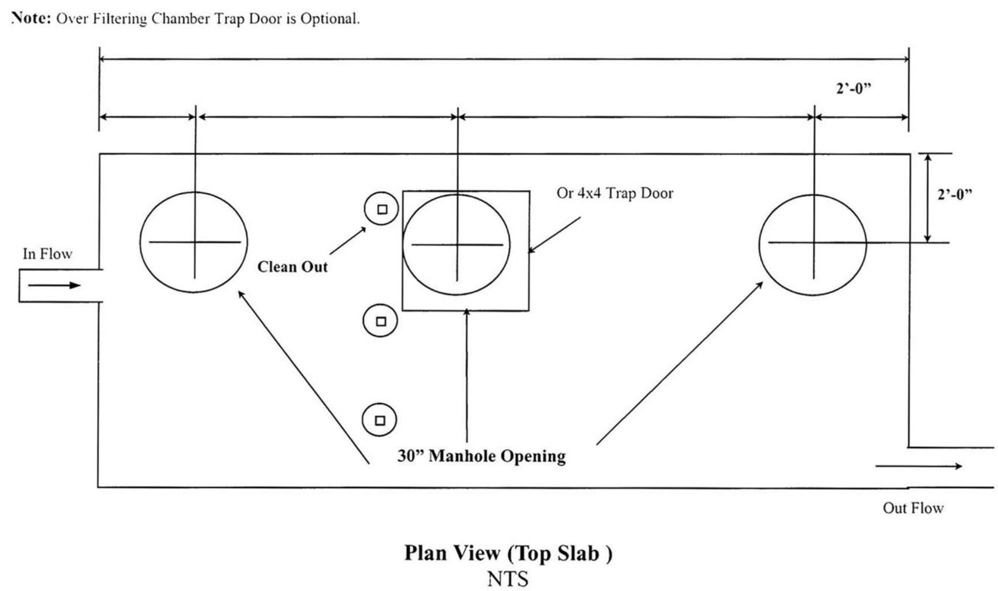
Surface sand filters can serve the largest drainage area of all the filtering systems.

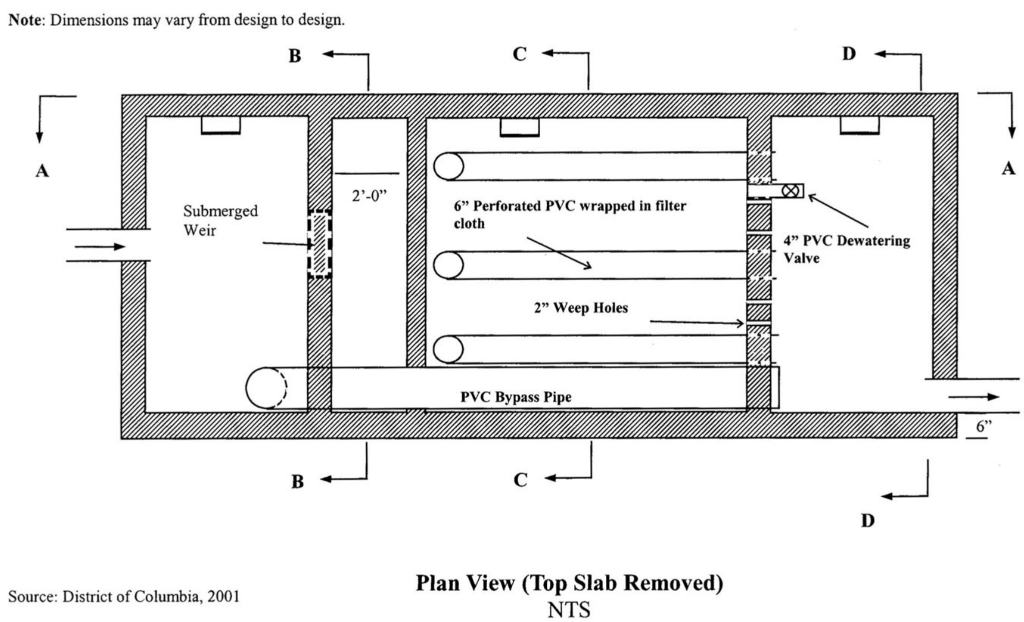
F-2	**Figure 3.2** Example of One-Chamber Underground SF	F-2



The one-chamber underground sand filter is an option for providing V_w on sites with less than 10,000 ft^2 of impervious area where space or land prices are at a premium.

F-3	**Figure 3.3a** Example of Three-Chamber Underground Sand Filter for Separate Sewer Areas	F-3



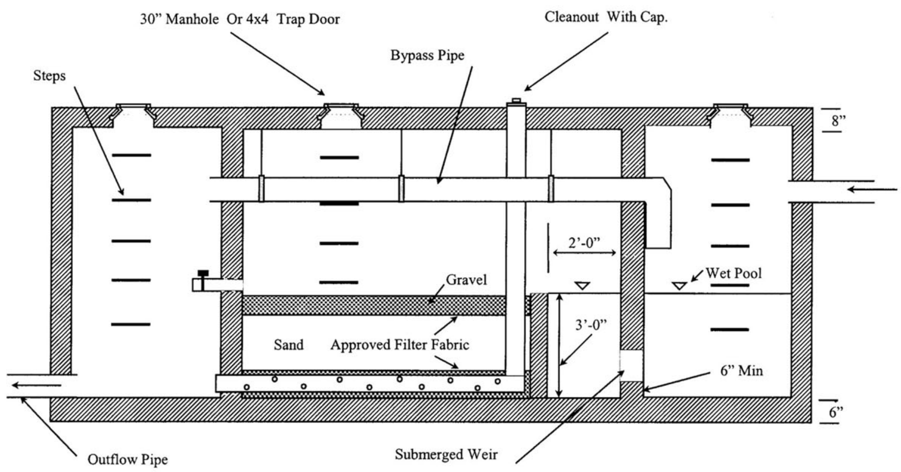


Source: District of Columbia, 2001

The three-chamber underground sand filter is an option for providing V_w where space or land prices are at a premium.

F-3	**Figure 3.3b** Example of Three-Chamber Underground Sand Filter for Separate Sewer Areas	F-3

Note: Dimensions may vary from design to design.



Section A-A
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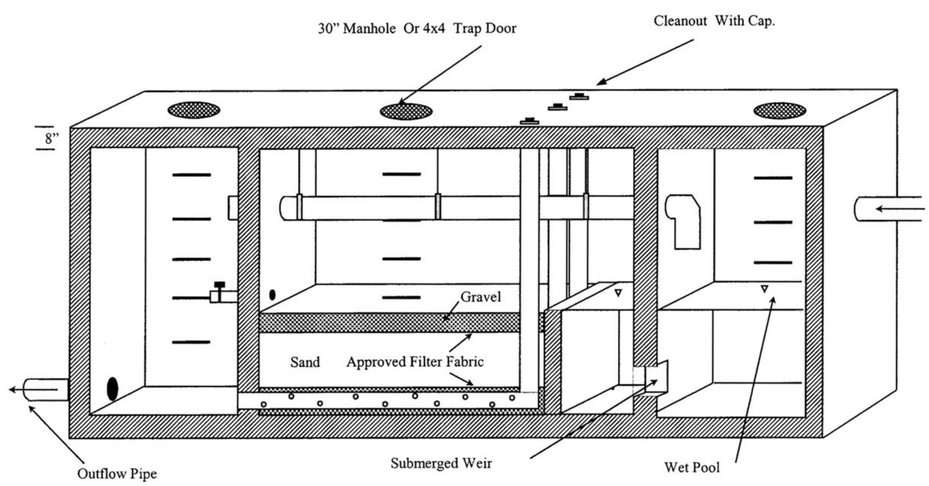
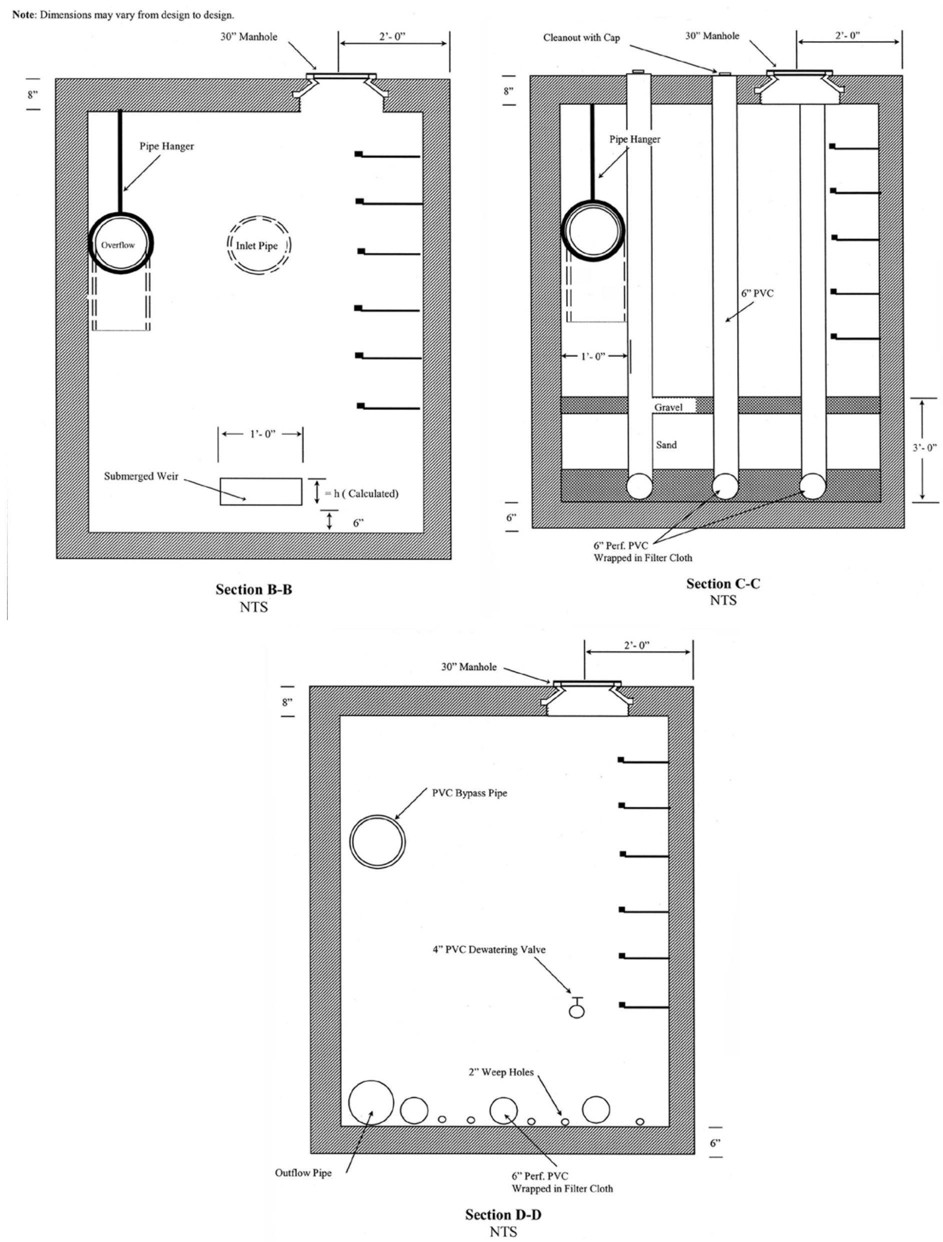


Section A-A (3D)
NTS

Source: District of Columbia, 2001

The three-chamber underground sand filter is an option for providing V_w where space or land prices are at a premium.

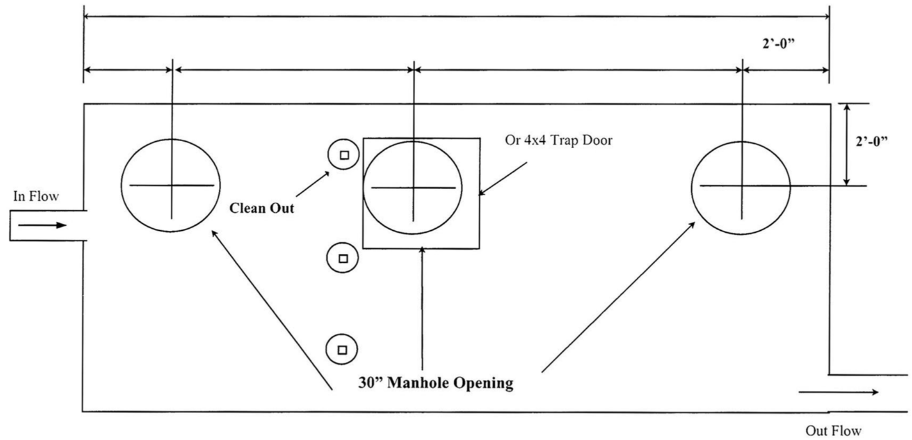
F-3	**Figure 3.3c** Example of Three-Chamber Underground Sand Filter for Separate Sewer Areas	F-3



The three-chamber underground sand filter is an option for providing V_w where space or land prices are at a premium.

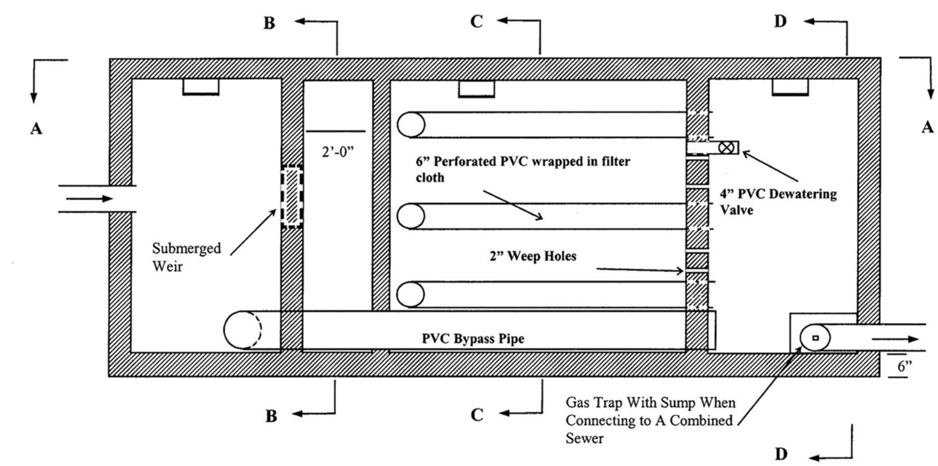
F-3	**Figure 3.4a** Example of Three-Chamber Underground Sand Filter for Combined Sewer Areas	F-3

Note: Over Filtering Chamber Trap Door is Optional.



Plan View (Top Slab)
NTS

Note: Dimensions may vary from design to design.

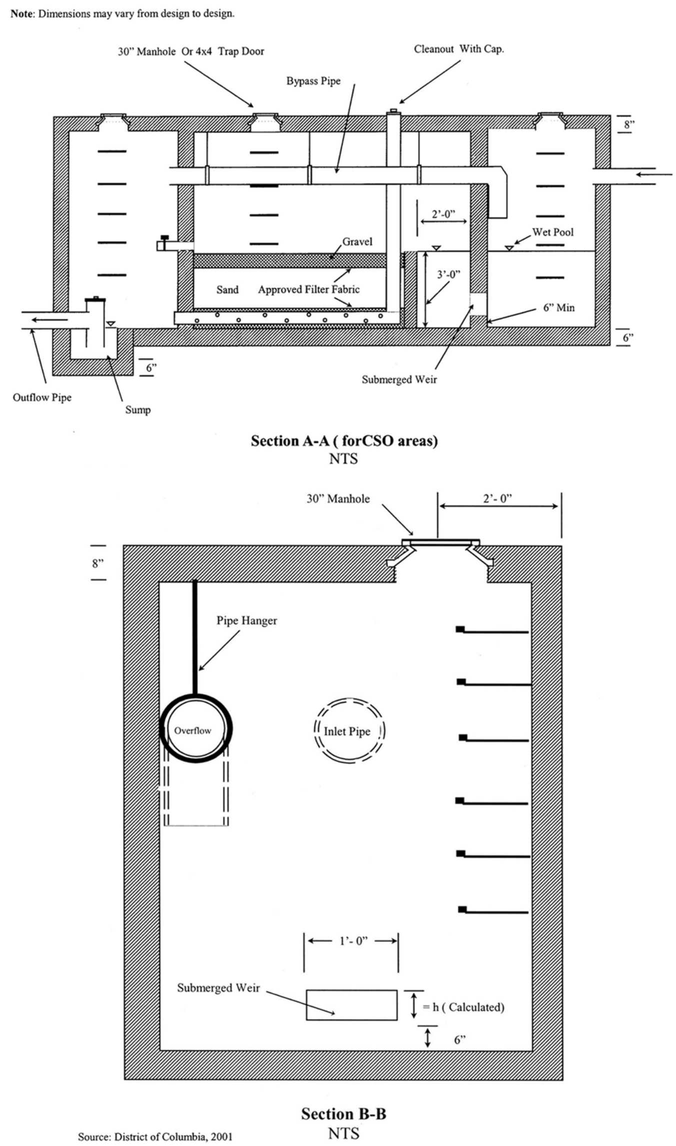


Source: District of Columbia, 2001

Plan View (Top Slab Removed)
NTS

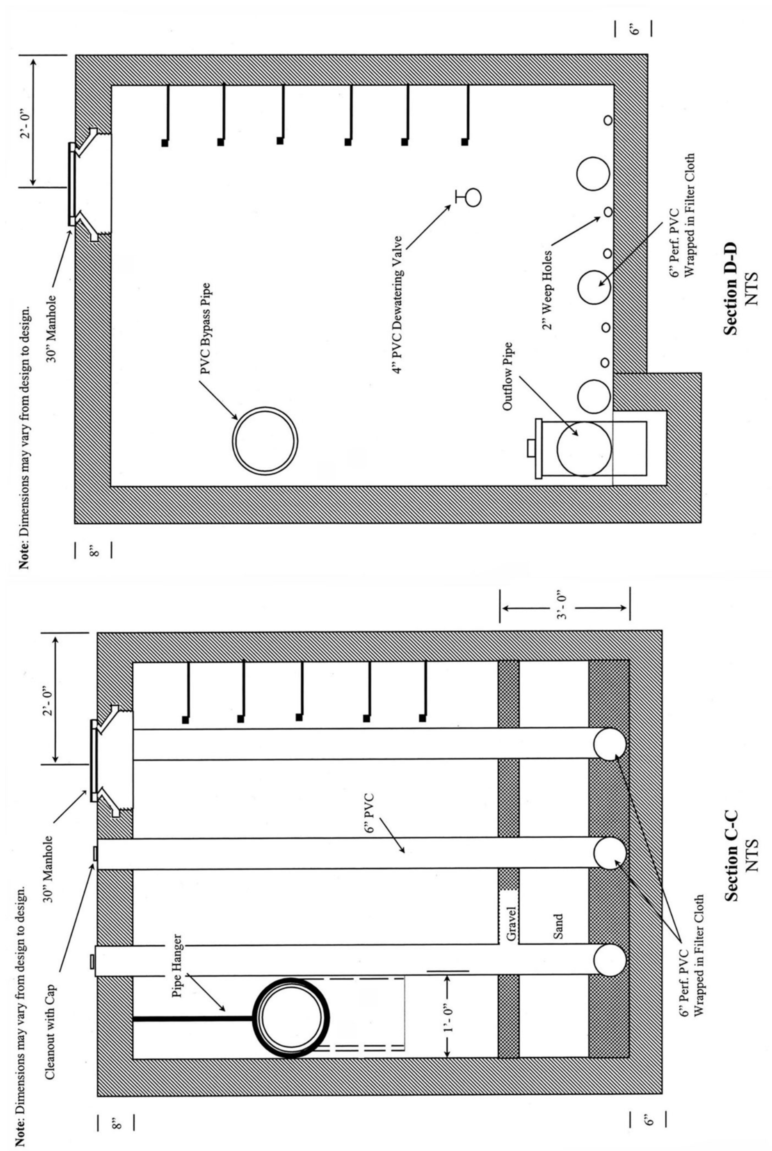
The three-chamber underground sand filter is an option for providing V_w where space or land prices are at a premium.

F-3	**Figure 3.4b** Example of Three-Chamber Underground Sand Filter for Combined Sewer Areas	F-3



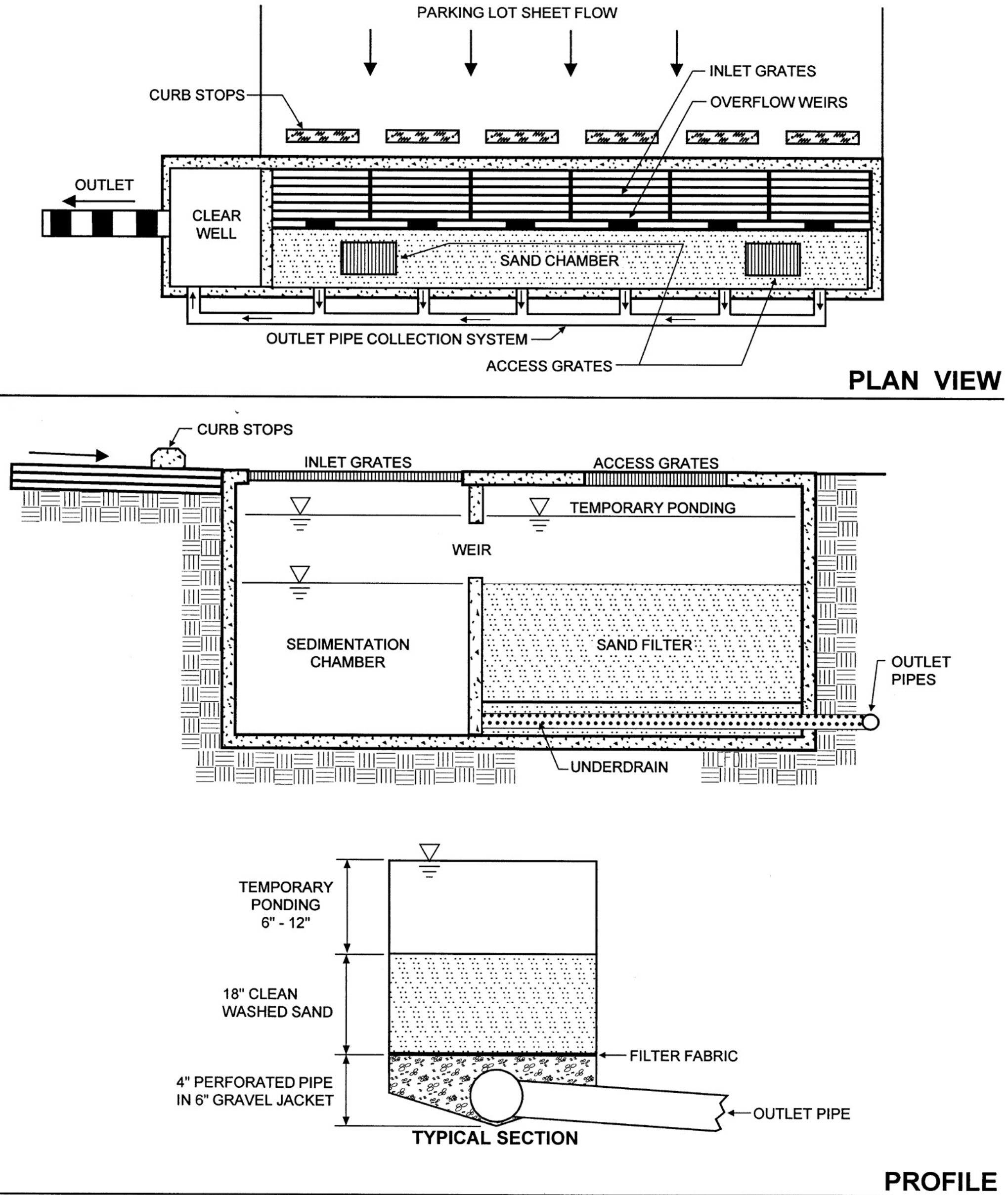
The three-chamber underground sand filter is an option for providing V_w where space or land prices are at a premium.

F-3	**Figure 3.4c** Example of Three-Chamber Underground Sand Filter for Combined Sewer Areas	F-3



The three-chamber underground sand filter is an option for providing V_w where space or land prices are at a premium.

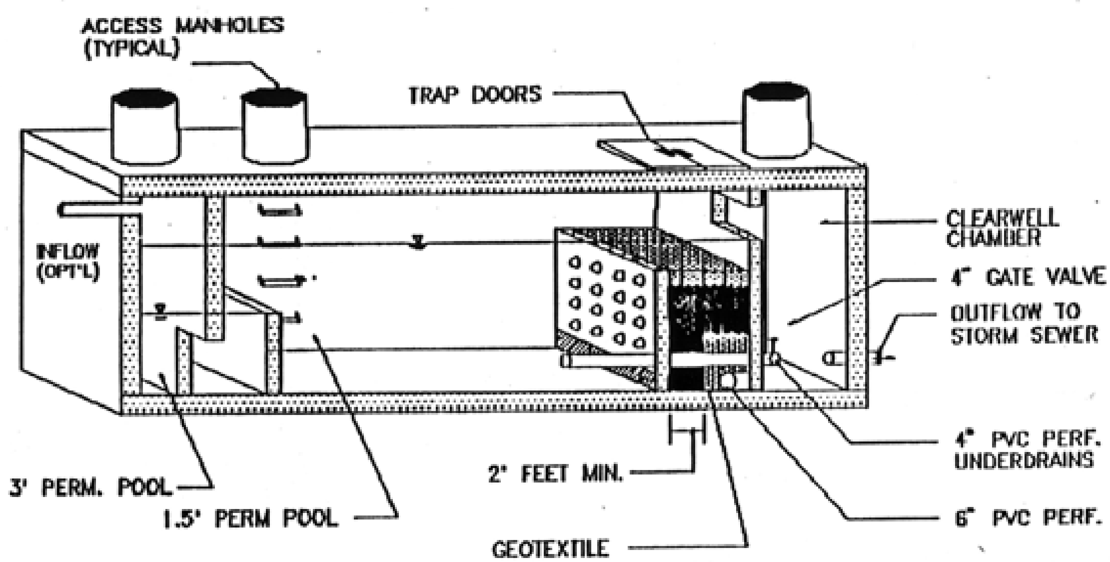
F-4	**Figure 3.5** Example of Perimeter Sand Filter	F-4



Source: Adapted from MDE, 2000

The perimeter sand filter is most practical for small sites with flat terrain or a high water table.

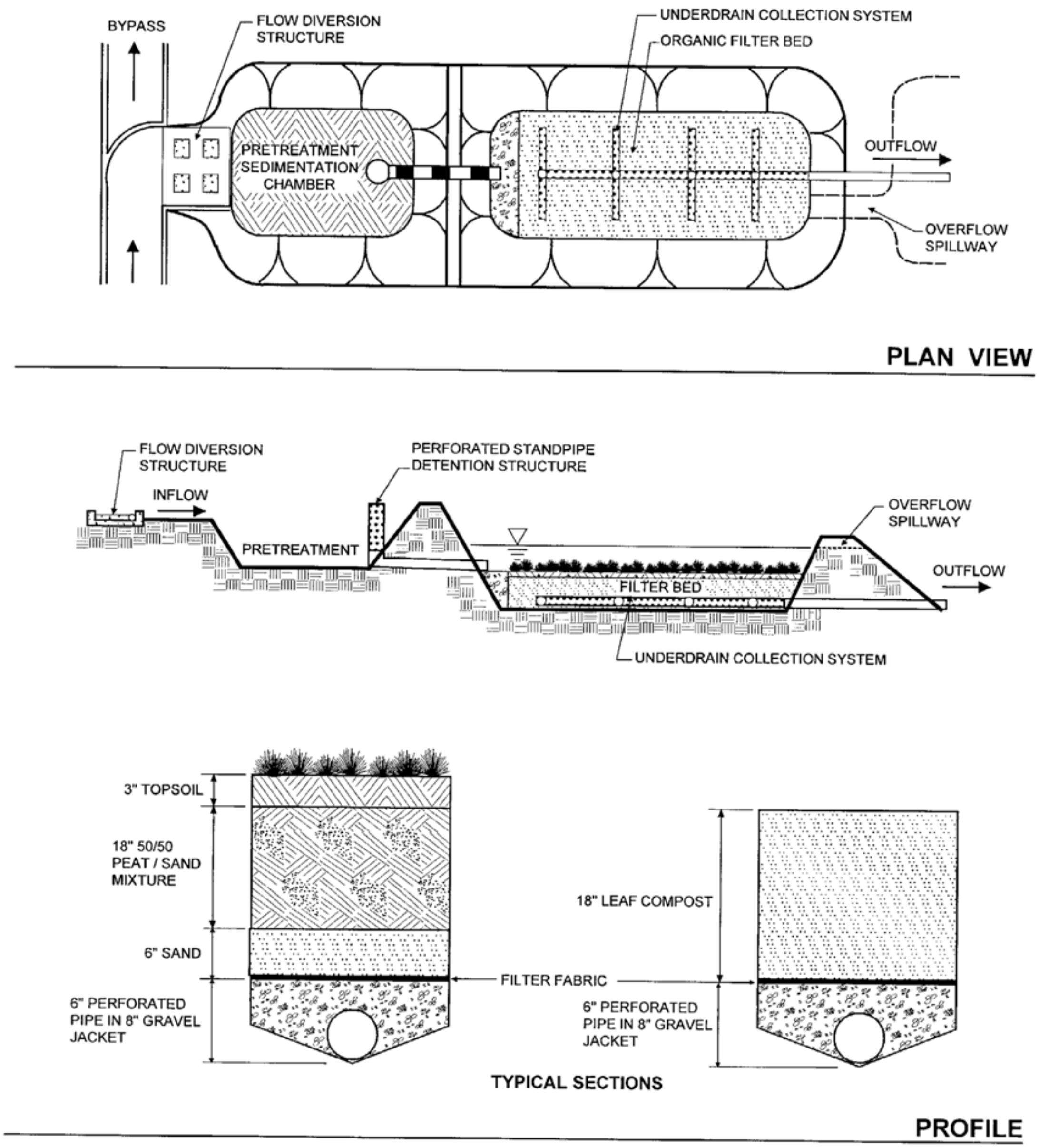
F-5	**Figure 3.6** Example of Vertical Sand Filter	F-5



Source: District of Columbia, 1988

The vertical sand filter is an option for providing V_w where space or land prices are at a premium.

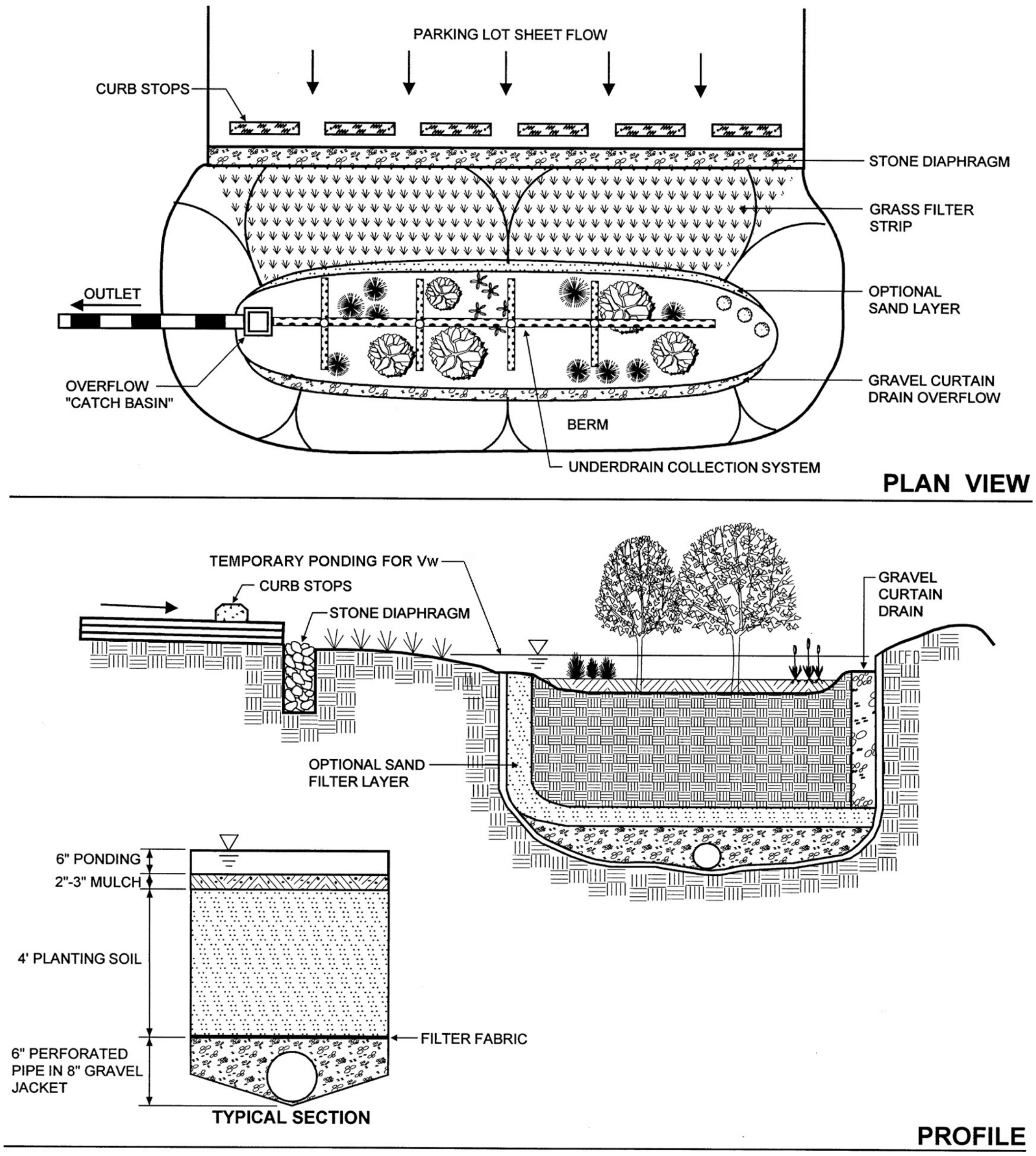
F-6	**Figure 3.7** Example of Organic Filter	F-6



Source: Adapted from MDE, 2000

The organic filter is used when maximum nutrient or trace metal removals are desired.

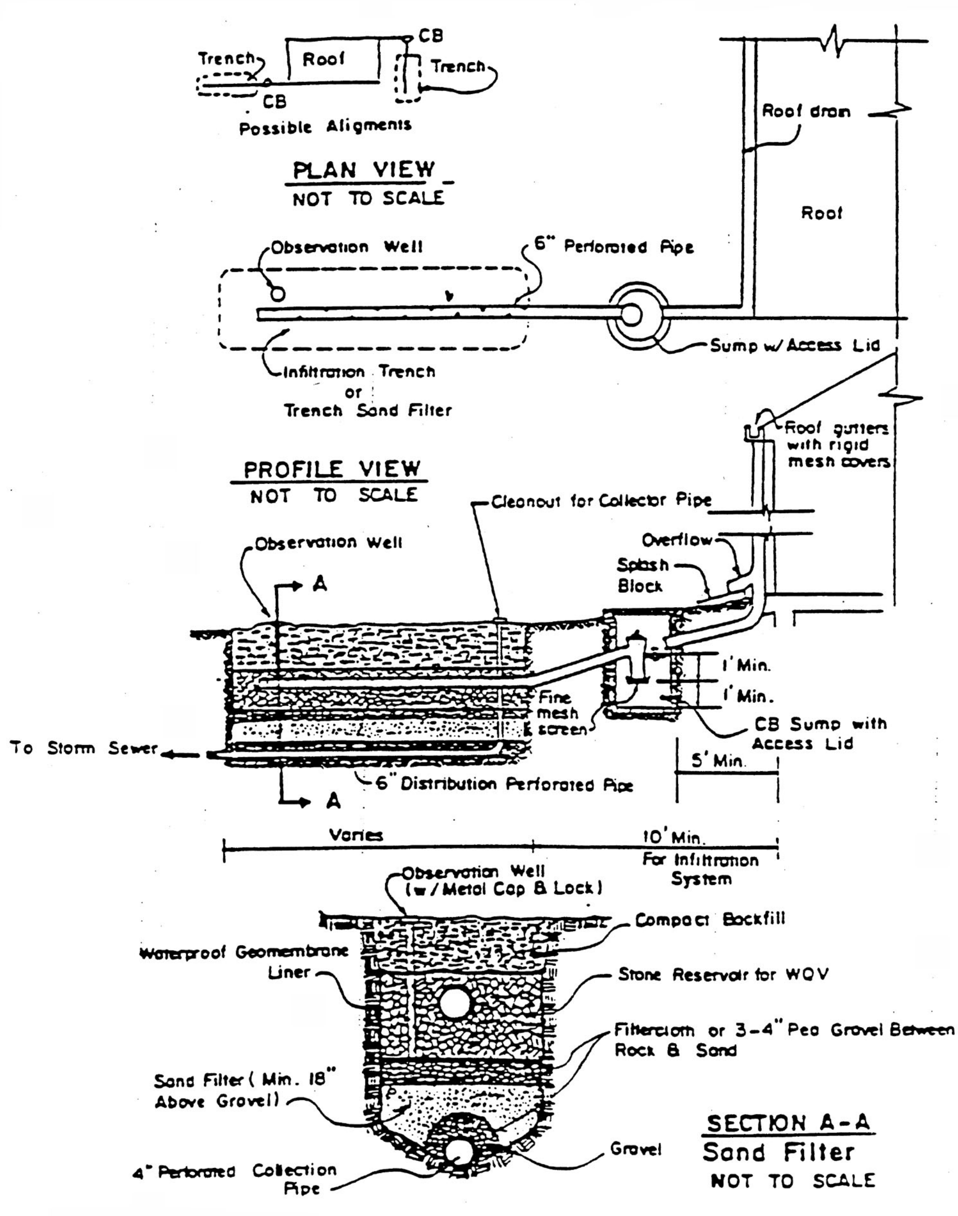
F-7	**Figure 3.8** Example of Bioretention	F-7



Source: Adapted from MDE, 2000

Bioretention combines open space with storm water treatment.

F-8	**Figure 3.9** Example of Roof Downspout System	F-8



A roof downspout system is a trench sand filter system that may be used to treat runoff from roofs.

3.1.1 Filtering Feasibility Criteria

All utilities shall have a minimum 5' horizontal clearance from the facility.

The maximum contributing area to an individual storm water filtering system is usually less than 10 acres.

Filtering systems are typically not to be designed to provide storm water detention (Qp_2, Qp_{15}, and / or Q_f), but they may be in some circumstances. Filtering practices shall generally be combined with a separate facility to provide those controls. However, in combined sewer areas, the three-chamber underground sand filter can be modified by expanding the first or settling chamber, or adding an extra chamber between the filter chamber and the clear well chamber to handle the detention volume, which is subsequently discharged at a pre-determined rate through an orifice and weir combination.

All filtering systems shall be located in areas where they are accessible for inspection and for maintenance (by vacuum trucks).

The seasonally high groundwater table and bedrock shall be located at least 4 feet below the footing of the structure.

A geotechnical report is required for all underground BMPs, including filtering systems. Geotechnical testing requirements are outlined in Appendix E.

Since sand filters are gravity flow systems that normally require 2 to 6 feet of head, sufficient vertical clearance between the inverts of the inflow and outflow pipes shall be provided. Whenever there is insufficient hydraulic head for a three-chamber underground sand filter, a well pump may be used to discharge the effluent from the third chamber into the receiving storm or combined sewer.

For three-chamber sand filters in combined-sewer areas, a water trap shall be provided in the third chamber to prevent the back flow of odorous gas.

The one-chamber sand filter is only applicable for impervious area less than 10,000 ft^2 (1/4 acre).

3.1.2 Filtering Conveyance Criteria

If runoff is delivered by a storm drain pipe or is along the main conveyance system, the filtering practice shall be designed off-line.

An overflow shall be provided within the practice to pass storms greater than the V_w storage to a stabilized water course.

3.1.3 Filtering Pretreatment Criteria

Dry or wet pretreatment shall be provided prior to filter media.

Adequate pretreatment for bioretention systems (F-7) is provided when all of the following are provided: (a) grass filter strip below a level spreader, (b) gravel diaphragm and (c) a mulch layer.

3.1.4 Filtering Treatment Criteria

The entire treatment system (including pretreatment) shall temporarily hold at least 75% of the V_w *prior to filtration.*

The filter bed typically has a minimum depth of 18". The perimeter filter may have a minimum filter bed depth of 12".

The filter media shall consist of a medium sand (meeting ASTM C-33 concrete sand). Media used for organic filters (F-6) may consist of peat/sand mix or leaf compost. Peat should be a reed-sedge hemic peat.

Bioretention systems (F-7) shall consist of the following treatment components: A four foot deep planting soil bed, a surface mulch layer, and a 6" deep surface ponding area.

Design Procedure for Storm Water Filtering Systems

Recommended steps to be considered when designing a storm water filtering system are as follows:

1. Examine the topographical conditions of the site and select possible outfalls from the existing drainage or sewer map.

2. Review the final grading plans and determine the maximum head available between the proposed inflow and outflow pipes.

3. Determine the total connected impervious area.

4. Select the runoff depth to be treated based on land use characteristics (Table 2.2). Calculate the water quality volume (V_w) to be treated.

5. Estimate the storage volume and the release rate.

6. Select design storm(s): This should be based on the storm frequencies selected by the Department.

7. Determine the size of the inflow, outflow and emergency release pipes: These should be sized to pass the highest selected storm frequency permitted by the Department. The District of Columbia uses the 15-year storm (with t_c = 5 min) for post-development runoff.

8. Determine detention time: All filter systems should be designed to drain the first-flush runoff from the filter chamber within 5 to 24 hours after each rainfall event.

9. Determine structural requirements: A licensed structural engineer should certify the structural integrity of the design in accordance with local building codes.

10. For underground structures, provide sufficient headroom for maintenance: A minimum head space of 5 feet above the filter is recommended for maintenance of the structure. However, if 5 feet headroom is not available, a removable top should be installed.

Specific Design Methodology: Three-Chamber Underground Sand Filter

The three-chamber underground sand filter is a gravity flow system. The facility may be precast or cast-in-place. The first chamber acts as a pretreatment facility removing any floating organic material such as oil, grease, and tree leaves. It should have a submerged orifice leading to a second chamber and it should be designed to minimize the energy of incoming storm water before the flow enters the second chamber (filtering or processing chamber).

The second chamber is the filter chamber. It should contain three feet of filter material consisting of gravel, geotextile fabric, and sand, and should be situated behind a three foot weir. Along the bottom of the structure should be a subsurface drainage system consisting of a parallel PVC pipe system in a gravel bed. A dewatering valve should be installed at the top of the filter layer for safety release in cases of emergency. A by-pass pipe crossing the second chamber to carry overflow from the first chamber to the third chamber is required.

The third chamber is the discharge chamber. It should also receive the overflow from the first chamber through the bypass pipe when the storage volume is exceeded.

Water enters the first chamber of the system by gravity or by pumping. This chamber removes most of the heavy solid particles, floatable trash, leaves, and hydrocarbons. Then the water flows to the second chamber and enters the filter layer by overtopping a weir (3 feet above the bottom of the structure). The filtered storm water is then picked up by the subsurface drainage system that empties it into the third chamber.

1. Determine Design Invert Elevations

Determine the final surface elevation, invert in, invert out and bottom invert elevation of the structure, see Figure 3.10.

$$D_t = \left(Inv_{in} - Inv_{out}\right) + D_b + 1 \tag{3.1}$$

Where:
- D_t = total depth of structure (ft)
- Inv_{in} = final invert elevation of inflow pipe (ft)
- Inv_{out} = final invert elevation of outflow pipe (ft)
- D_b = diameter of bypass pipe (ft)
- 1 = freeboard constant (ft)

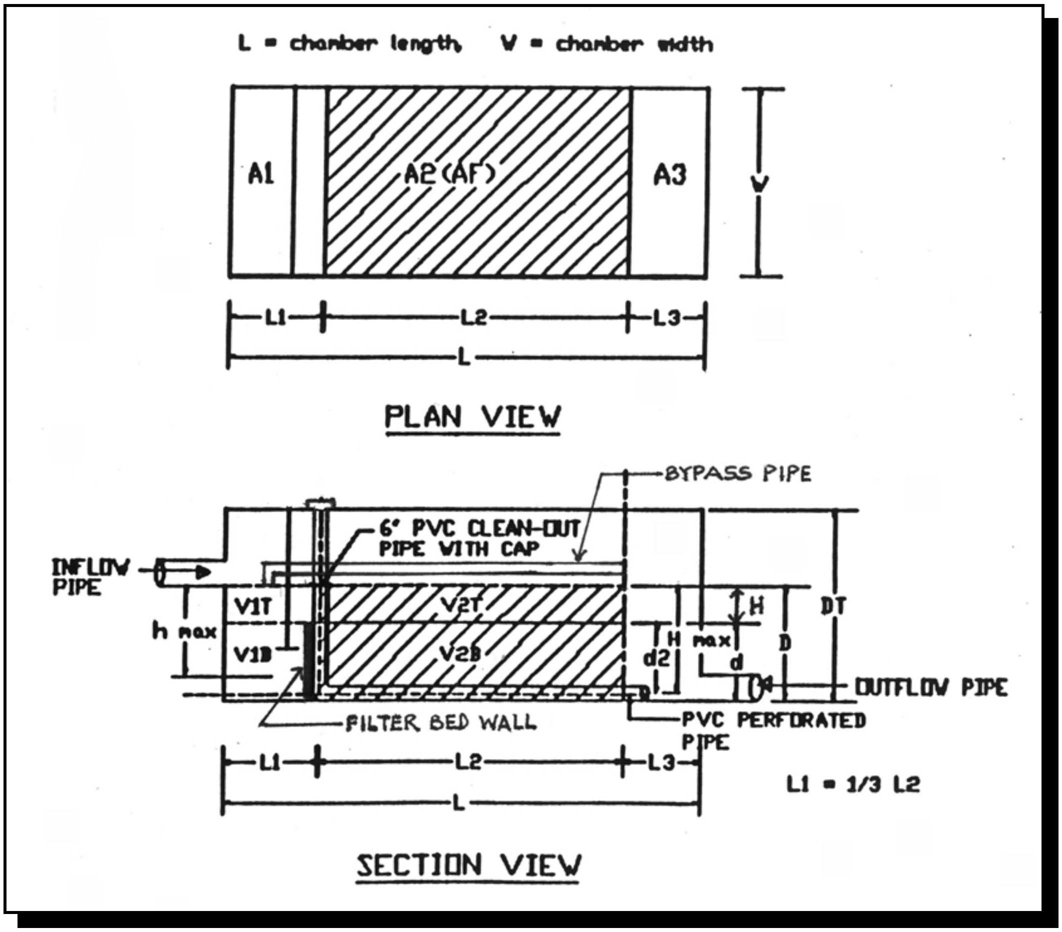


Figure 3.10 Design Guide for the Standard Sand Filter

2. Peak Discharge Calculation for Bypass Flow

The bypass pipe is sized to pass the 15-year event. Using the Rational Method:

$$Q_p = C * I * A \tag{3.2}$$

Where:
Q_p = bypass peak flow (cfs)
C = runoff coefficient (dimensionless)
I = rainfall intensity (in/hr)
A = drainage area (ac)
t_c = time of concentration (minute or hr)
= t_d, storm duration, use in selecting rainfall intensity (minute or hr)

Note: The District of Columbia uses the 15-year storm (with t_c = 5 minutes) for post-development runoff. The District of Columbia Rainfall Intensity - Duration - Frequency curve is provided in Appendix A.

3. Determine Sand Filter Area A_f

Use Figure 3.11 or the following equation:

$$A_f = 50 + \left(I_a - 0.1\,\text{ac}\right) * \left(167\ \text{ft}^2 / \text{ac}\right) \tag{3.3}$$

Where:
A_f = surface area of filter layer (second chamber) (ft^2)
I_a = impervious area (ac)

4. Determine Storage Volume Needed

$$V_s = V_w - \left(F * T * A_f\right) \tag{3.4}$$

Where:
V_s = storage volume needed to hold the first flush runoff (ft^3)
V_w = water quality volume (ft^3)
F = infiltration rate for sand (ft/hr) $\approx$1.18 ft/hr
T = filtering time (1 hour based on NRCS practice)
A_f = sand filter area (ft^2)

5. Calculate Submerged Storage Volume in Second Chamber

$$V_{2b} = A_f * d_f * n \tag{3.5}$$

Where:
- V_{2b} = submerged volume of filter chamber (ft^3)
- A_f = surface area of filter layer (second chamber) (ft^2)
- d_f = depth of filter layer (ft)
- n = composite of porosity for filter media

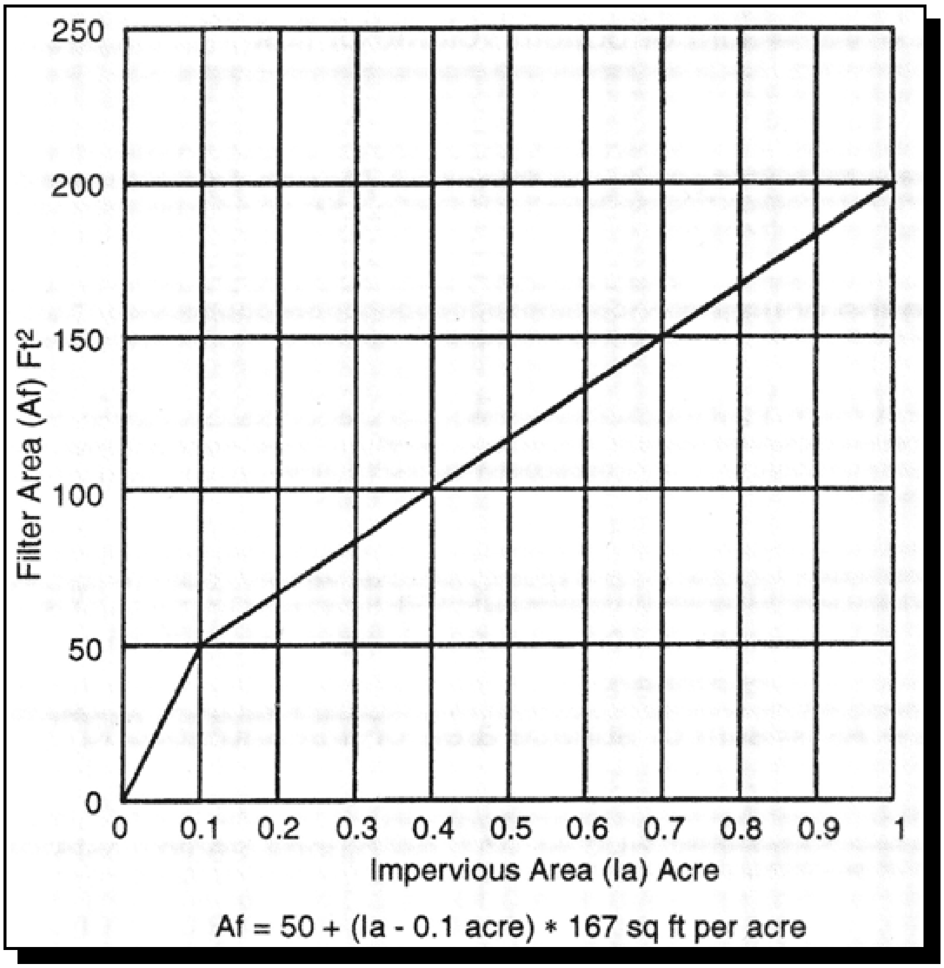


Figure 3.11 Determination of Filter Area

6. Calculate Submerged Storage Volume in First Chamber

$$V_{1b} = A_1 * d_f \tag{3.6}$$

Where: V_{1b} = submerged volume of first chamber (ft^3)
A_1 = surface area of first chamber (ft^2)
d_f = depth of filter layer (ft)

Note: $A_f/3 < A_1 < A_f/2$, for optimum design condition

7. Calculate Surface Storage Volume in First & Second Chambers

$$\left(V_{1t} + V_{2t}\right) = V_s - \left(V_{2b} + V_{1b}\right) \tag{3.7}$$

Where: $V_{1t} + V_{2t}$ = sum of surface volume of first & second chambers (ft^3)
V_s = storage volume needed to hold the first flush runoff (ft^3) (from eq. 3.4)
$V_{2b} + V_{1b}$ = sum of submerged volume of first & second chambers (ft^3)

8. Determine Maximum Storage Depth

$$D = H + d_f \tag{3.8}$$

Where: D = maximum storage depth (ft)
H = vertical distance between top of filter layer and bottom of bypass pipe outlet

$$= \frac{\left(V_{1t} + V_{2t}\right)}{\left(A_1 + A_f\right)}$$

$V_{1t} + V_{2t}$ = sum of surface volume of first & second chambers (ft^3)
$A_1 + A_f$ = sum of surface area of first chamber & filter layer (second chamber) (ft^2)
d_f = depth of filter layer (ft)

Note: D must be equal to or smaller than the difference between the invert in and invert out

9. Determine Size of Submerged Bypass Pipe

First, determine the submerged weir opening in first chamber, assuming orifice conditions:

$$Q_p = C * A_{w1} * \sqrt{2gh_{max}}$$

Therefore:

$$A_{w1} = \frac{Q_p}{C * (2gh_{max})^{0.5}} \tag{3.9}$$

Where:
- A_{w1} = area of weir opening in first chamber (ft^2) = $h_{w1} * l_{w1}$
- h_{w1} = weir height, minimum 1 ft
- l_{w1} = weir length (ft)
- Q_p = bypass peak flow (cfs) (from eq. 3.2)
- C = 0.6, orifice coefficient
- g = 32.2 ft/sec^2
- h_{max} = hydraulic head above the center line of weir (ft) from centroid of orifice?

Determine the capacity of the bypass pipe:

$$D_b = \left[\frac{2.16 * n * Q_p}{\sqrt{S}}\right]^{0.375} \tag{3.10}$$

Where:
- D_b = estimated bypass pipe diameter (ft)
- n = roughness coefficient (may vary from 0.011 to 0.021 depending on material)
- Q_p = bypass peak flow (cfs) (from eq. 3.2)
- S = pipe slope (use a slope value of 0.1% to 1%)

Note: PVC is normally preferred for the bypass pipe (n = 0.011)

In combined sewer areas, when the system is sized for both quantity and quality control, the following equation shall be used to size the quantity volume:

$$V_q = \left(Q_{p15} - Q_{p2}\right) * t_c * 1.25 \tag{3.11}$$

Where: V_q = quantity volume (ft^3)
Q_{p15} = 15-year peak flow (cfs)
Q_{p2} = 2-year peak flow (cfs)
t_c = time of concentration (minutes)

The water quality volume (V_w) shall be computed. Then the quantity control volume (V_q) shall also be computed using Equation 3.11.

The larger of the two volumes shall be used to size the sand filter structure. The last chamber should be divided into a third and fourth chamber. A rectangular weir and an orifice (at the bottom) shall be provided in the third chamber to control the excess volume from quantity control.

The overflow weir opening in the third chamber shall be designed to carry the 15-year design storm, while the orifice at the bottom shall be designed to handle the 2-year pre-development peak flow, using the submerged orifice formula (Equation 3.9).

<u>10. Determine Flow Through Filter and Detention Time After Storage Volume Fills Up</u>

Average flow through the filter:

$$q_f = k * A_f * i \tag{3.12}$$

Where: q_f = average flow through the filter (ft^3/hr)
k = sand permeability (ft/hr)
A_f = surface area of filter layer (ft^2)
i = hydraulic gradient (ft/ft) = $h_{max} / (2 * d_f)$

Estimate the detention time:

$$T_s = \frac{V_s}{q_f} \tag{3.13}$$

Where: T_s = average dewatering time for sand filter (hr)
V_s = storage volume needed (ft^3) (from eq. 3.4)
q_f = average flow through the filter (ft^3/hr) (from eq. 3.12)

11. Develop Inflow and Outflow Hydrographs

Figure 3.12 is a typical illustration of inflow/outflow hydrographs for a sand filter. For the inflow hydrograph, use the Modified Rational Method Hydrograph with:

$$t_p = t_c \quad \text{and}$$

$$t_R = 1.67\, t_c \tag{3.14}$$

Where: t_p = time to peak
t_c = time of concentration
t_R = recession period

For the outflow hydrograph use the following equations to determine when flow occurs.

When: $t_c * Q_p < 2V_s$ Then

$$t_p = 2t_c - \left(2t_c^2 - \frac{2V_s * t_c}{Q_p}\right)^{0.5} \tag{3.15}$$

When: $t_c * Q_p = 2V_s$ Then

$$t_p = 0.5t_c + \left(\frac{V_s}{Q_p}\right) \tag{3.16}$$

When: $t_c * Q_p > 2V_s$ Then

$$t_p = \left[\frac{2V_s * t_c}{Q_p}\right]^{0.5} \tag{3.17}$$

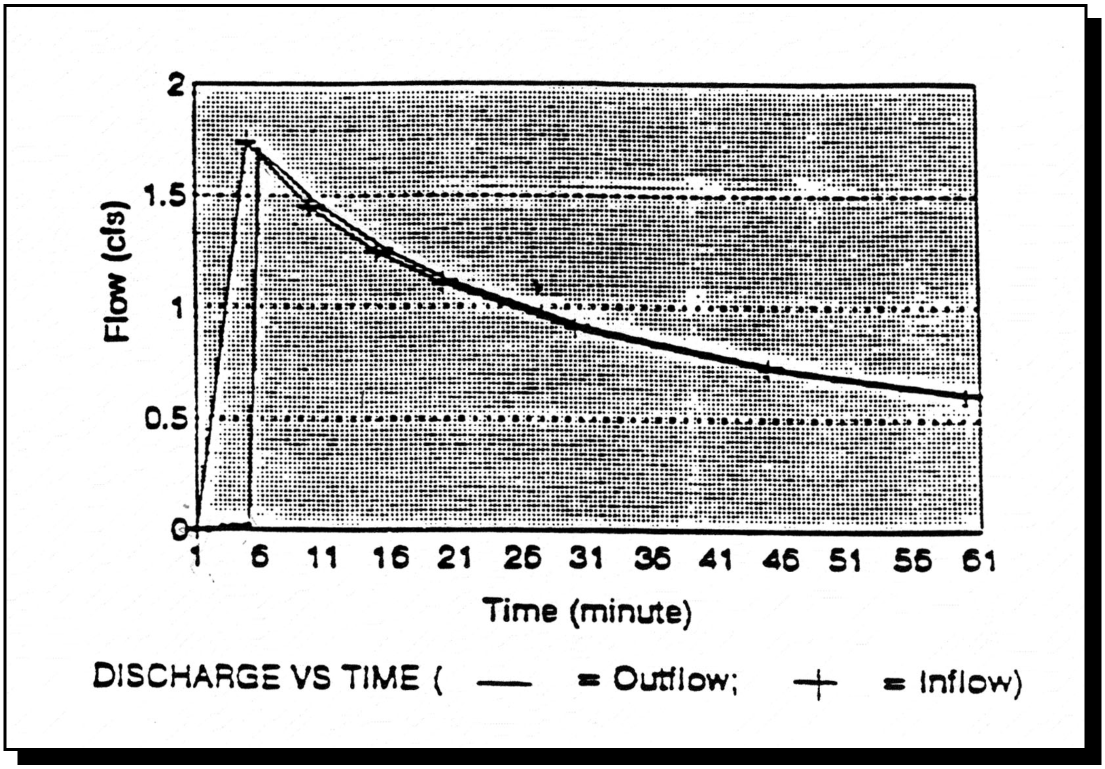


Figure 3.12   Typical Inflow and Outflow Hydrographs

Specific Design Methodology: One-Chamber Underground Sand Filter

The one-chamber underground sand filter is a gravity flow system. The water enters the system from a remote sump catch basin used as a primary settling device to remove trash, leaves, oil, grease and heavy sediment. It must be designed off-line with a flow splitter to maintain system efficiency.

1. Determine Design Invert Elevations
    (same as for three-chamber underground sand filter --- See Step 1)

2. Peak Discharge Calculation for Bypass Flow
    (same as for three-chamber underground sand filter --- See Step 2)

3. Determine Horizontal Cross Section Area (Filter Area)
    (same as for three-chamber underground sand filter --- See Step 3)

4. Determine Storage Volume Needed
    (same as for three-chamber underground sand filter --- See Step 4)

5. Determine Maximum Storage Depth

(same as for three-chamber underground sand filter --- See Step 8)

6. Determine Flow through Filter and Dewatering Time (T_d)

(same as for three-chamber underground sand filter --- See Step 10)

7. Develop Inflow and Outflow Hydrographs

(same as for three-chamber underground sand filter --- See Step 11)

Specific Design Methodology: Vertical Sand Filter

The vertical sand filter is a gravity-flow system consisting of three chambers. The first chamber is a pretreatment chamber that removes floatable organic material such as oil, grease, tree leaves and heavy sediment particles. It has a submerged orifice leading to a second chamber. The second chamber is the process chamber that has a 3 foot vertical filter with 4" diameter holes. The filter material is the same as the three-chamber sand filter. It has features that are similar to the three-chamber sand filter, such as a sub-surface drainage system, a bypass pipe, and a dewatering valve. The third chamber also receives the overflow from the second chamber or the first chamber depending on the initial design selection.

The design procedure is the same as the three-chamber underground sand filter except the following steps must be modified to meet the calculation requirements:

1. Determine Area of Sand Filter

$$A_f = 0.3\left[50 + \left(I_a - 0.1\,\text{ac}\right) * 167\ \text{ft}^2/\text{ac}\right] \tag{3.18}$$

Where:
- A_f = $A_{fv} + A_{fh}$ total filter area (ft^2)
- A_{fv} = vertical filter area -- wall area (ft^2)
- A_{fh} = horizontal filter area -- area on top of filter layer (ft^2)
- I_a = impervious area (ac)

2. Estimate the Detention Time

Draw down time from storage level 1 to storage level 2

$$q_{t1} = k * A_f * i$$

$$t_{d1} = \frac{V_w}{q_{t1}} \quad (3.19)$$

Draw down from storage level 2 to the bottom of structure

$$q_{t2} = k * A_{fv} * i$$

$$t_{d2} = \frac{V_w}{q_{t2}} \quad (3.20)$$

Total detention time:

$$T_d = t_{d1} + t_{d2} \quad (3.21)$$

Where:
- k = sand permeability (ft/hr)
- i = hydraulic gradient (ft/ft) = h_{max} / (2 * d_f)
- q_{t1} = discharge from maximum storage level to level 2 (cfs)
- q_{t2} = discharge from storage level 2 to bottom structure (cfs)
- t_{d1} = draw down time of level 1 (sec)
- t_{d2} = draw down time of level 2 (sec)
- T_d = total dewatering time from the system (sec)

Specific Design Methodology: Perimeter Sand Filter

The perimeter sand filter consists of two parallel trenches connected by a series of overflow weir notches at the top of the partitioning wall, which allows water to enter the second trench as sheet flow.

The first trench is a pretreatment chamber removing heavy sediment particles and debris. The second trench consists of at least 12" of sand. A subsurface drainage pipe must be installed in the gravel bed at the bottom of the second chamber to facilitate the filtering process and convey filter water into a receiving system.

The following procedures should be used in designing the perimeter sand filter:

1. Calculate the Required Surface Area of the Filter Bed

Use Figure 3.11 or equation 3.3:

$$A_f = 50 + (I_a - 0.1\,ac) * 167\ ft^2 / ac$$

2. Calculate the Total Surface Area of the Perimeter Sand Filter

$$A_t = 2A_f \tag{3.22}$$

Where: A_t = total surface area of the perimeter sand filter (ft^2)
A_f = sand filter area (ft^2)

3. Calculate the Storage Depth of the Perimeter Sand Filter

$$D_t = \frac{V_w}{A_t} + d_f \tag{3.23}$$

Where: D_t = storage depth of the trench (ft)
V_w = water quality volume (ft^3)
A_t = total surface area of the perimeter sand filter (ft^2)
d_f = depth of filter layer (ft)

4. Calculate the Overflow Weir Height

$$h = D_t = \frac{V_w}{A_t} + d_f \tag{3.24}$$

Where: h = overflow weir height (ft)
D_t = storage depth of the trench (ft)
V_w = water quality volume (ft^3)
A_t = total surface area of the perimeter sand filter (ft^2)
d_f = depth of filter layer (ft)

5. Calculate the Size of Overflow Weir

$$Q_p = C * L_w * h^{1.5} \tag{3.25}$$

Where: Q_p = bypass flow (cfs)
C = 3.33, weir coefficient
L_w = length of weir opening (ft)
h = overflow weir height (ft)

6. Determine the Final System Dimensions

$$L = \frac{A_t}{W} + 3 \tag{3.26}$$

Where: L = total length of the perimeter sand filter(ft)
A_t = total surface area of the perimeter sand filter (ft^2)
W = total width of the perimeter sand filter, use 4 ft to begin the first trial (ft)

Specific Design Methodology: Surface Sand Filter

The surface sand filter consists of a sedimentation basin followed by a filtration basin. The most common filter media is sand; however, a peat/sand mixture may be used to increase the removal efficiency of the system. Two possible configurations of the filtration systems are described below:

1. Full Sedimentation: In this system, the sedimentation basin should be designed to hold the entire water quality volume (V_w) and to release it to the filtration basin over an extended period of dewatering time (Figure 3.13).
2. Partial Sedimentation: In this system, a sedimentation basin should not be designed to hold the entire water quality volume (V_w) and should not incorporate an extended drawdown period (Figure 3.14).

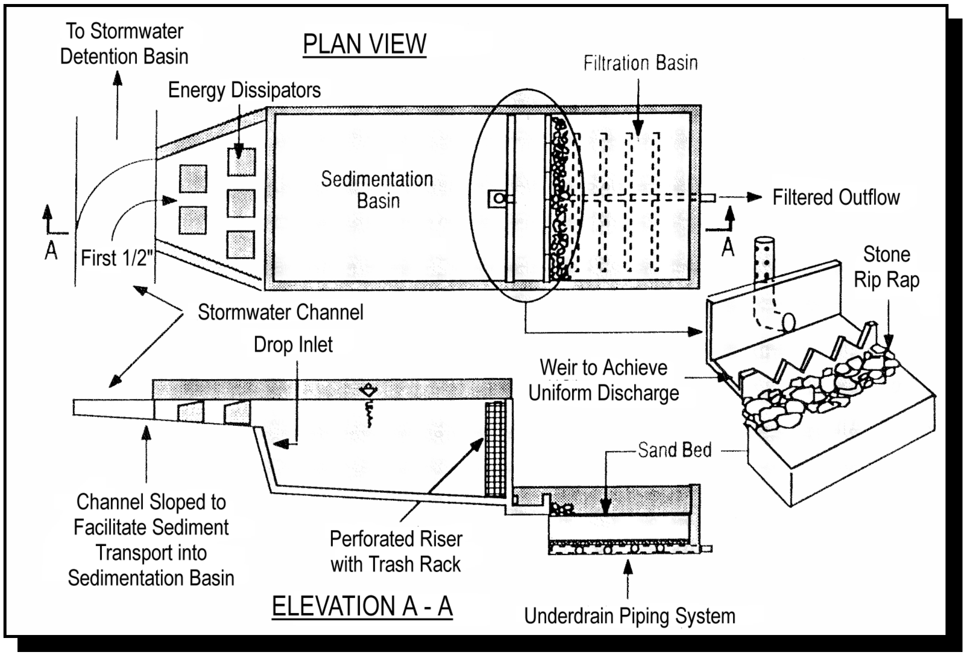


Figure 3.13 Conceptual Full Sedimentation Filtration System

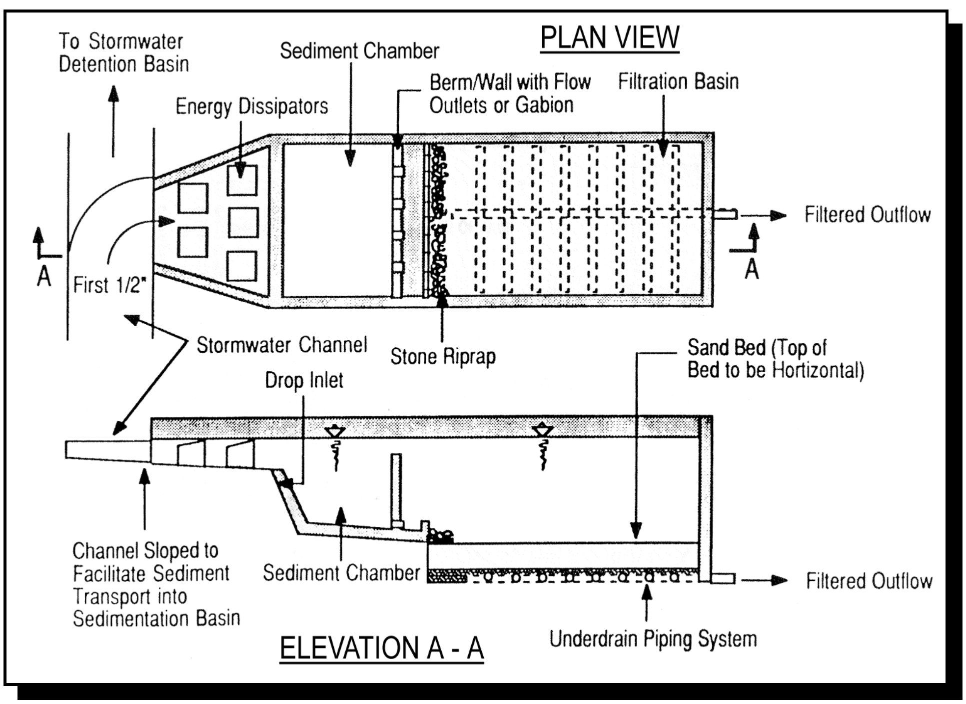


Figure 3.14 Conceptual Partial Sedimentation Filtration System

Design Methodology for Full Sedimentation with Filtration

1. Calculate the Basin Surface Areas

Filtration basin surface area:

$$A_f = \frac{V_w d_f}{k(h + d_f)t_f} \tag{3.27}$$

Where:
- A_f = surface area of filtration basin (ft^2)
- V_w = water quality volume to be treated (ft^3)
- d_f = sand bed depth (ft)
- k = coefficient of permeability for sand filter (ft/hr)
- h = average depth of water above surface of sand media (½ maximum depth) (ft)
- t_f = time required for runoff volume to pass through filter media (hr)

Sedimentation basin surface area:

$$A_s = \frac{I_a R}{10} \tag{3.28}$$

Where:
- A_s = surface area of sedimentation basin (ac)
- I_a = impervious drainage area (ac)
- R = runoff depth to be treated (Table 2.2) (ft)

2. Calculate the Basin Volumes

For the sedimentation basin, the storage volume (V_s) should be equal to or greater than the water quality volume (V_w).

For the filtration basin, the storage volume (V_f) should be equal to or greater than 20% of the water quality volume ($0.2V_w$).

Design Methodology for Partial Sedimentation with Filtration

1. Calculate the Basin Surface Areas and Volumes

Filtration basin surface area:

$$A_f = \frac{I_a R}{10} \tag{3.29}$$

Where: A_f = surface area of filtration basin (ac)
I_a = impervious drainage area (ac)
R = runoff depth to be treated (Table 2.2) (ft)

Sedimentation basin surface area:

$$A_s = V_w \left(\frac{1}{D_s} - \frac{1}{10} \right) \tag{3.30}$$

Where: A_s = surface area of sedimentation basin (ft^2)
V_w = water quality volume to be treated (ft^3)
D_s = sedimentation basin depth (ft)

Sedimentation basin and filtration basin volumes:

$$V_w = V_s + V_f \tag{3.31}$$

Where: V_s = sedimentation basin volume (ft^3) (equal to or greater than $0.2V_w$)
V_f = filtration basin volume (ft^3)
V_w = total water quality volume (ft^3)

Specific Design Methodology: Roof Downspout Filtration System

A Roof Downspout Filtration (RDF) system is a trench sand filter system. It is intended only for treating runoff from roofs. Roof gutters must be covered with rigid mesh screens to prevent leaves and other large debris from entering the system. The downspout or the inflow pipe must be connected through a sump catch basin to remove heavy sediment, debris, and floatable material before it discharges into the RDF system.

1. Compute the Area of the Filter Bed (A_f)

Use Figure 3.11 or equation 3.3:

$$A_f = 50 + \left(I_a - 0.1\,\text{ac}\right) * 167\ \text{ft}^2 / \text{ac}$$

2. Size the Stone Reservoir

$$V = V_w\left(1 + n\right) \tag{3.32}$$

$$A = \frac{V}{d} \tag{3.33}$$

$$V_f = A * \left(d_f + 0.25\right) * n \tag{3.34}$$

$$V_{sr} = V - V_f \tag{3.35}$$

$$d_{sr} = \frac{V_{sr}}{A} \tag{3.36}$$

Where:

Where:	V	=	total RDF volume (ft^3)
	V_w	=	water quality volume (ft^3)
	A	=	total RDF surface area (ft^2)
	d	=	total depth of system (less than or equal to 5 feet)
	V_f	=	volume of sand filter bed (ft^3)
	d_f	=	depth of sand filter bed (ft)
	0.25	=	depth of pea gravel layer (ft)
	n	=	porosity
	V_{sr}	=	volume of stone reservoir (ft^3)
	d_{sr}	=	depth of stone reservoir (ft)

3. Filter Bed Detail Standards and Specifications Should Be the Same as the Three-chamber Underground Sand Filter

3.1.5 Filtering Landscaping Criteria

A dense and vigorous vegetative cover shall be established over the contributing pervious drainage areas before runoff can be accepted into the facility.

Landscaping is critical to the performance and function of bioretention areas. Therefore, a landscaping plan shall be provided for bioretention areas.

Surface filters (e.g., surface sand and organic) can have a grass cover to aid in the pollutant adsorption. The grass should be capable of withstanding frequent periods of inundation and drought.

Planting recommendations for bioretention facilities are as follows:

- Native plant species should be specified over non-native species.
- Vegetation should be selected based on a specified zone of hydric tolerance.
- A selection of trees with an understory of shrubs and herbaceous materials should be provided.
- Woody vegetation should not be specified at inflow locations.
- Trees should be planted primarily along the perimeter of the facility.

3.1.6 Filtering Maintenance Criteria

Organic filters (F-6) or surface sand filters (F-1) that have a grass cover shall be mowed a minimum of 3 times per growing season to maintain maximum grass heights less than 12 inches.

A stone drop of at least six inches (pea gravel diaphragm) shall be provided at the inlet of bioretention facilities (F-7). Areas devoid of mulch shall be re-mulched on an annual basis. Dead or diseased plant material shall be replaced.

Direct maintenance access shall be provided to the pretreatment area and the filter bed.

The approved erosion and sediment control plans shall include specific measures to provide for the protection of the filter system before the final stabilization of the site.

No runoff shall be allowed to enter the sand filter system prior to completion of all construction activities, including revegetation and final site stabilization. Construction runoff shall be treated in separate sedimentation basins and routed to bypass the filter system. Should construction runoff enter the filter system prior to final site stabilization, all contaminated materials must be removed and replaced with new clean filter materials before a regulatory inspector approves its completion.

Three-Chamber Sand Filter, One-Chamber Sand Filter, Vertical Sand Filter:
The water level in the filter chamber shall be monitored by the owner on a quarterly basis and after every large storm for the first year after completion of construction. A log of the results shall be maintained, indicating the rate of dewatering after each storm and the water depth for each observation. Once the performance of the structure has been demonstrated, the monitoring schedule may be reduced to an annual basis.

The first chamber must be pumped out semi-annually. If the chamber contains an oil skim, it should be removed by a firm specializing in oil recovery and recycling. The remaining material may then be removed by a vacuum pump truck and disposed of in an approved landfill.

After approximately three to five years, the upper layer of the filter can be expected to become clogged with fine silt. When the draw down time for the filter exceeds 72 hours, the upper layer of gravel, geotextile fabric, and sand layer, depending on coloration, must be removed and replaced with new, clean materials conforming to the original specifications.

Perimeter Sand Filter:
During the first year of operation, the cover grates or precast lids on the chambers must be removed quarterly and a joint owner / District of Columbia storm water management inspection made to assure that the system is functioning. Once the District of Columbia inspectors are satisfied that the system is functioning properly, this inspection may be made on a semiannual basis.

When deposition of sediments in the filtration chamber indicates that the filter media is clogging and not performing properly, the top filter fabric and 2"-3" of sand layer must be removed and replaced to refacilitate the filtration process. The coloration of sand should determine how much sands needs to be removed and replaced.

Petroleum hydrocarbon contaminated sand or filter cloth must be disposed of according to District of Columbia solid waste disposal regulations.

Trash collected on the grates protecting the inlets shall be removed at least weekly to ensure the inflow capacity of the BMP is preserved.

Surface Sand Filter:
Major Maintenance Requirements for Sedimentation Basins

- *Removal of silt when accumulation exceeds 6".*
- *Removal of accumulated paper, trash, leaves and debris every 6 months or as required by routing District of Columbia inspector.*
- *Vegetation growing within the basin is not allowed to exceed 18" in height at any time.*
- *Corrective maintenance is required any time the sedimentation basin and sediment trap do not draw down completely after 48 hours (i.e., no standing water is allowed).*

Major Maintenance Requirements For Filtration Basins

- *Removal of silt or sediment when accumulation exceeds 0.5" and paper, trash, tree leaves and debris every 6 months or by order from routing District of Columbia inspector.*
- *Vegetation growing within the filter basin is not allowed to exceed 18" in height.*
- *Corrective measure is required any time drawdown does not occur within 36 hours after the sedimentation basin has emptied.*

Roof Downspout Filtration System:

- *Removal of oil and grease, silt, paper, trash and debris from the pretreatment sump every six months or as necessary.*
- *The system shall be inspected semi-annually by representatives of the owner and the city storm water management inspector to assure continued proper functioning.*
- *When the water will no longer draw down within the required 48 hours period, the subsurface drainage system shall be backwashed with high pressure water to refacilitate the filtration process or total removal and replacement of filter bed and gravel if necessary.*

See Appendix F for additional design and construction specifications for sand filter systems.

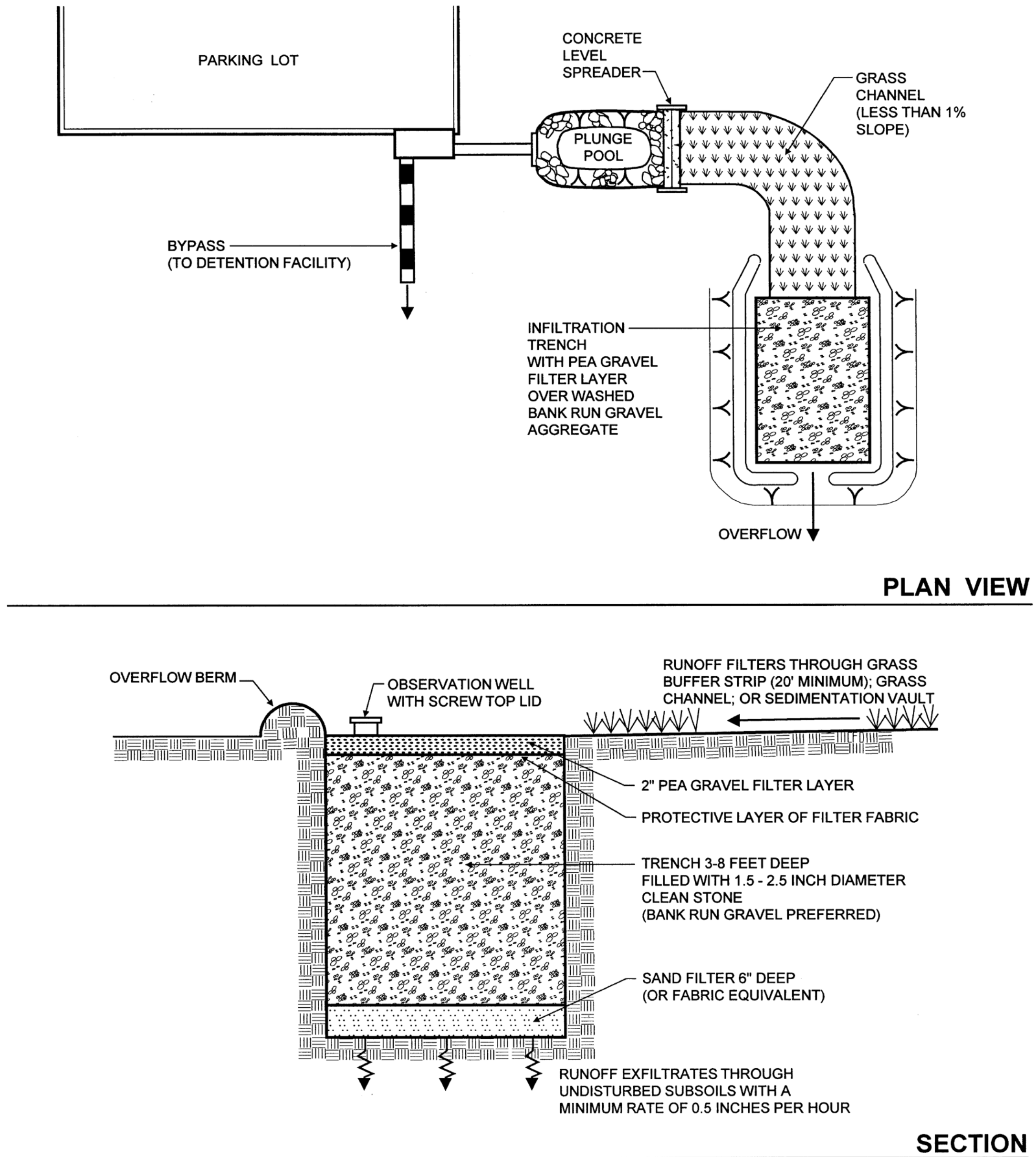
Section 3.2 Storm Water Infiltration

Definition: Practices that capture and temporarily store the V_w before allowing it to infiltrate into the soil over a two day period. Design variants include:

- I-1 infiltration trench
- I-2 infiltration basin

Infiltration practices may also provide storm water detention storage in certain limited cases. Extraordinary care shall be taken to assure that long-term infiltration rates are achieved through the use of performance bonds, post construction inspection and long-term maintenance.

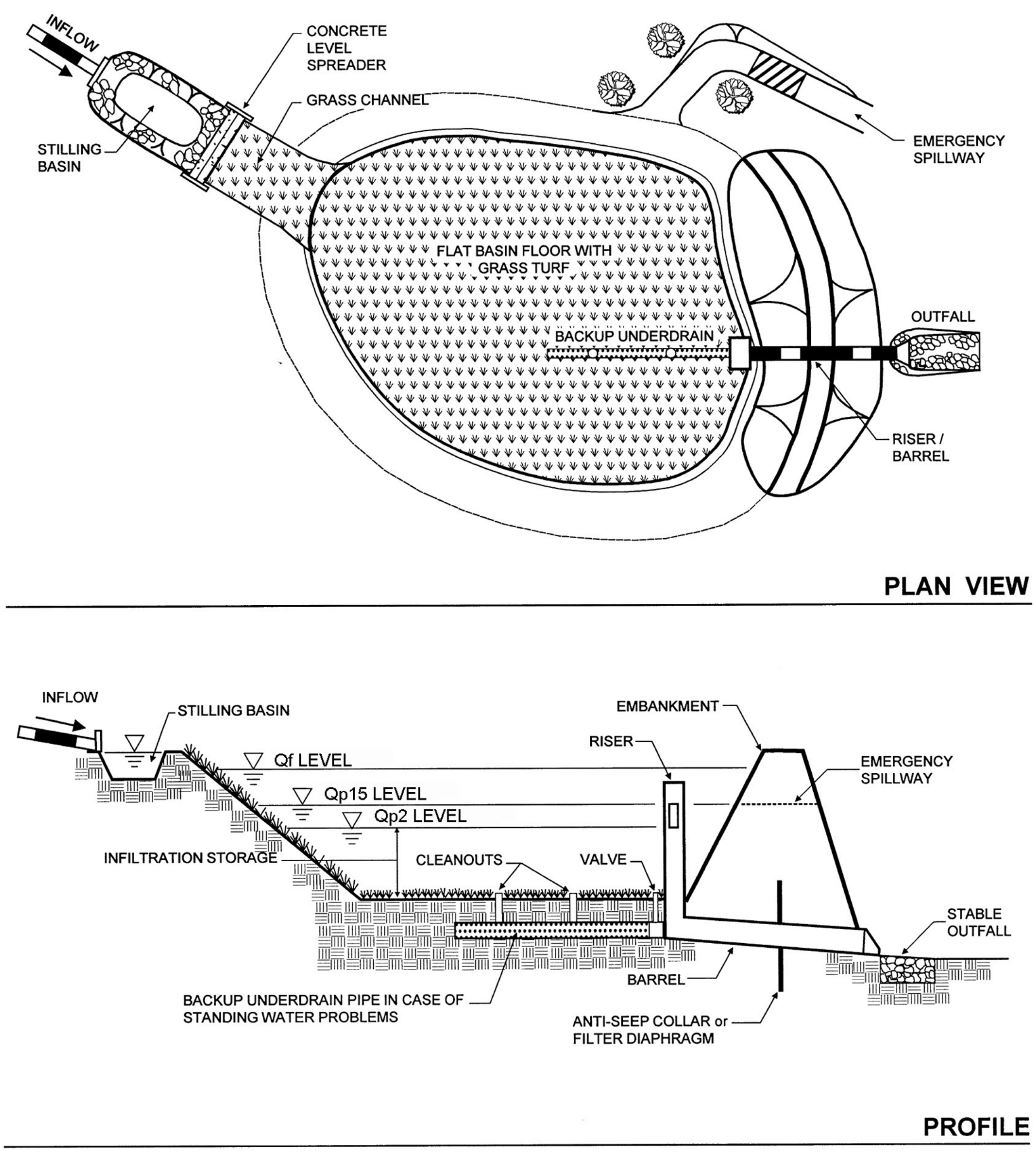
I-1	**Figure 3.15** Example of Infiltration Trench	I-1



Source: Adapted from MDE, 2000

The V_w is retained in an infiltration trench, where it percolates through the floor of the trench in a two day period.

I-2	**Figure 3.16** Example of Infiltration Basin	I-2



Source: Adapted from MDE, 2000

The V_w is retained in an infiltration basin, where it percolates through the floor of the basin in a two day period.

3.2.1 Infiltration Feasibility Criteria

The maximum contributing area to an individual infiltration practice should generally be less than 5 acres for trenches and less than 10 acres for basins.

Infiltration design depends primarily on soil percolation rates. The soils at the bottom and in the surrounding area of the retention structures must be tested to determine the final infiltration rates and to locate the water table depth. Soil boring locations should correspond to the location of the proposed infiltration device, and should have a minimum of one boring for every 50' length of infiltration device.

To be suitable for infiltration, underlying soils shall have an infiltration rate (f_c) of 0.52 inches per hour or greater, as initially determined from NRCS soil textural classification, and subsequently confirmed by field geotechnical tests. Geotechnical testing requirements are outlined in Appendix E.

Soils shall have a clay content of less than 20% and a silt/clay content of less than 40%.

Infiltration shall not be located on slopes greater than 6% or within fill soils.

The soil borings must indicate the depth to the seasonally high water table and bedrock, if any. The minimum distance acceptable between the bottom of the trench and the seasonally high water table is 4 feet. The minimum distance acceptable between the bottom of the trench and bedrock is 2 feet.

Infiltration systems designed to handle runoff from commercial or industrial impervious parking areas shall be a minimum of 100 feet from any water supply well.

A minimum of 5 feet horizontal distance shall be maintained between a utility line and infiltration trench. No utility line shall be placed over, under or within an infiltration trench.

Infiltration practices shall not be placed in locations that cause water problems to downgrade properties. Infiltration systems greater than 3 feet deep shall be setback at least 15 feet down-gradient from building structures.

Soil boring location stakes shall be left in the field for inspection purposes.

Soils investigation shall be performed by a licensed soils or geotechnical engineer.

3.2.2 Infiltration Conveyance Criteria

The overland flow path of surface runoff exceeding the capacity of the infiltration system shall be evaluated to preclude erosive concentrated flow during large storm events. If computed flow velocities exceed the non-erosive threshold, an overflow channel shall be provided to a stabilized water course. The non-erosive threshold is typically 4 feet per second for the 2-year event and 6 feet per second for the 15-year event.

All infiltration systems shall be designed to fully de-water the entire V_w *within 72 hours after the storm event.*

If runoff is delivered by a storm drain pipe or along the main conveyance system, the infiltration practice shall be designed as an off-line practice. Pretreatment shall be provided for storm drain pipes systems discharging directly to infiltration systems.

3.2.3 Infiltration Pretreatment Criteria

Pretreatment Volume

A minimum pretreatment volume of at least 25% of the V_w *shall be provided prior to entry to an infiltration facility, and can be provided in the form of a sedimentation basin, sump pit, grass channel, grass filter strip, plunge pool or other measure.*

Exit velocities from the pretreatment chamber shall not be erosive (above 6 fps) during the 15-year design storm.

If the f_c *for the underlying soils is greater than 2.0 inches per hour, 50% of the* V_w *shall be treated by another method prior to entry into an infiltration facility.*

Pretreatment Techniques to Prevent Clogging

Each infiltration system shall have redundant methods to protect the long term integrity of the infiltration rate. Three or more of the following techniques must be installed in every facility:

- *grass channel*
- *grass filter strip (minimum 20 feet and only if sheet flow is established and maintained)*
- *bottom sand layer*
- *upper filter fabric layer*
- *use of washed bank run gravel as aggregate*

3.2.4 Infiltration Treatment Criteria

Filter cloth or an equivalent material (Woven Monofilament Geotextile 104F) shall be installed on top and sides of trench. The sides of infiltration practices shall be lined with an acceptable filter fabric that prevents soil piping but has greater permeability than the parent soil.

A 6" layer of clean, washed sand may be used on the bottom of the trench, this area will not be counted in the trench volume calculation. Filter cloth or an equivalent material (Woven Monofilament Geotextile 104F) must be used to separate the sand layer from the stone aggregate.

The minimum allowable volume of voids is 0.35, and the trench shall be filled with 1.5" to 3" washed bank-run gravel.

The following formulas shall be used to determine the infiltration trench volume:

$$V = \frac{R * A}{12 * V_v} \tag{3.37}$$

Where
V = trench volume (ft^3)
R = runoff depth (in), see Table 2.2
A = infiltration trench surface area (ft^2)
V_v = volume of voids (0.35)
12 = conversion factor

The maximum trench depth shall be computed based on maximum detention time (T_d) of 72 hours. The head required for an inflow volume of R into the trench shall be determined using the following formula from TR-55:

$$Q_{pk} = R * I_a * q_u \tag{3.38}$$

Where:
Q_{pk} = peak discharge (cfs)
R = runoff depth to be treated (Table 2.2) (in)
I_a = impervious area (mi^2)
q_u = unit peak discharge (csm/in) (from TR-55)

$$h = \frac{1}{2g} * \left(\frac{Q_{pk}}{CA}\right)^2 \tag{3.39}$$

Where:
h = head loss (ft)

Q_{pk} = peak discharge (cfs)
C = 0.6 = entrance loss coefficient
A = infiltration trench surface area (ft^2)
g = 32.2ft/sec^2 (gravitational acceleration)

Infiltration practices shall be designed to exfiltrate the entire V_w through the floor of each practice.

Infiltration practices are best used in conjunction with other BMPs, and often downstream detention is still needed to meet the water quantity sizing criteria.

3.2.5 Infiltration Landscaping Criteria

A dense and vigorous vegetative cover shall be established over the contributing pervious drainage areas before runoff can be accepted into the facility.

3.2.6 Infiltration Maintenance Criteria

Infiltration systems may not receive runoff until the entire contributing drainage area has been completely stabilized.

An observation well shall be installed in every infiltration trench, consisting of an anchored 6" diameter perforated PVC pipe with a lockable cap installed flush with the ground surface.

Direct access shall be provided to all infiltration practices for maintenance and rehabilitation. If a stone reservoir or perforated pipe is used to temporarily store runoff prior to infiltration, the practice shall not be covered by an impermeable surface.

OSHA trench safety standards should be consulted if the infiltration trench will be excavated more than five feet.

A percolation test shall be required for infiltration trenches except for designs which drain through sand into a perforated pipe.

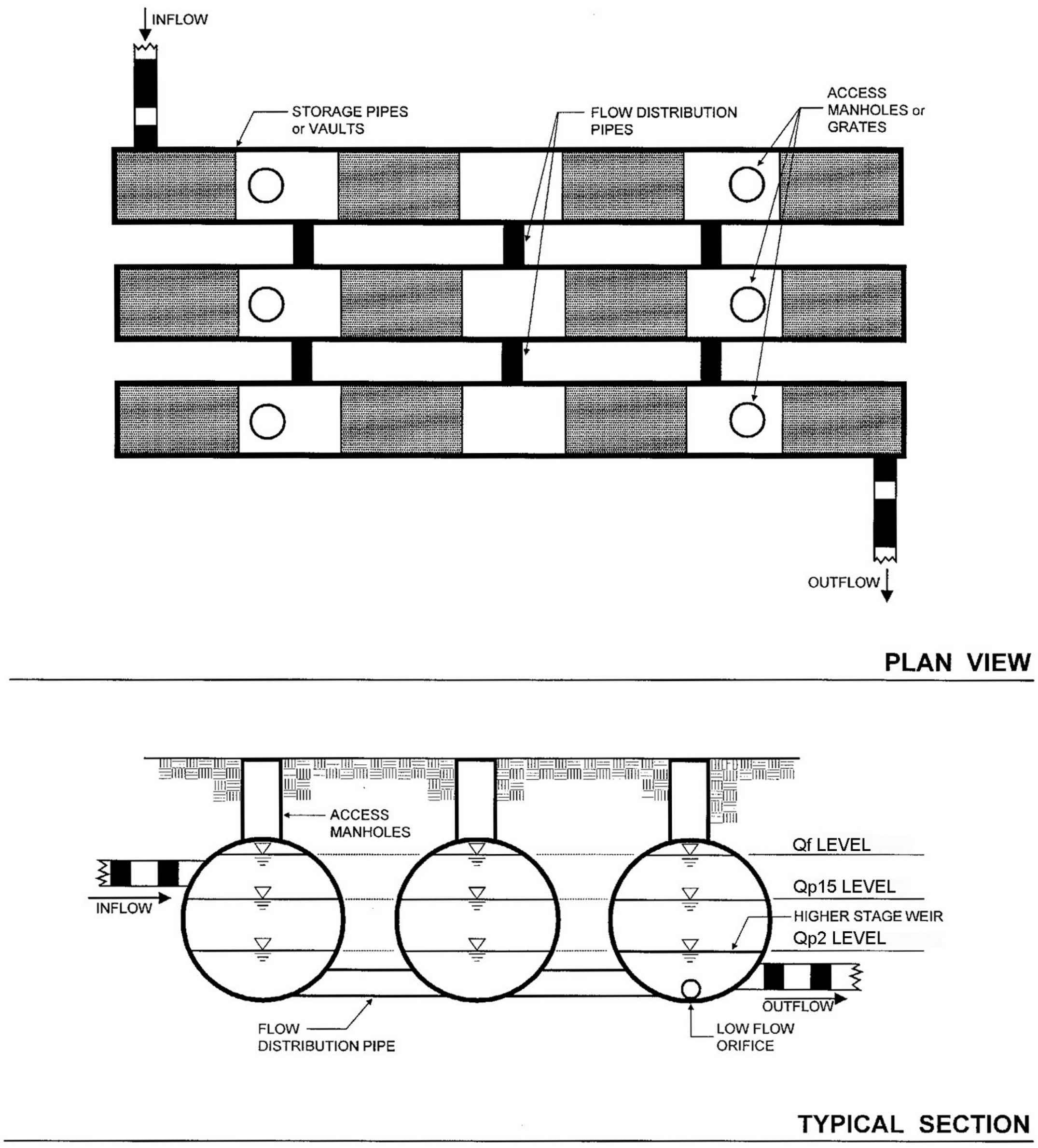
Section 3.3 Storage Practices

Definition: Storage practices are explicitly designed to provide storm water detention (Qp_2, Qp_{15}, and / or Q_f). Design variants include:

- S-1 underground vault
- S-2 dry pond

Storage practices are not considered an acceptable practice to meet the water quality volume requirement (V_w). Storage practices must generally be combined with a separate facility to meet these requirements.

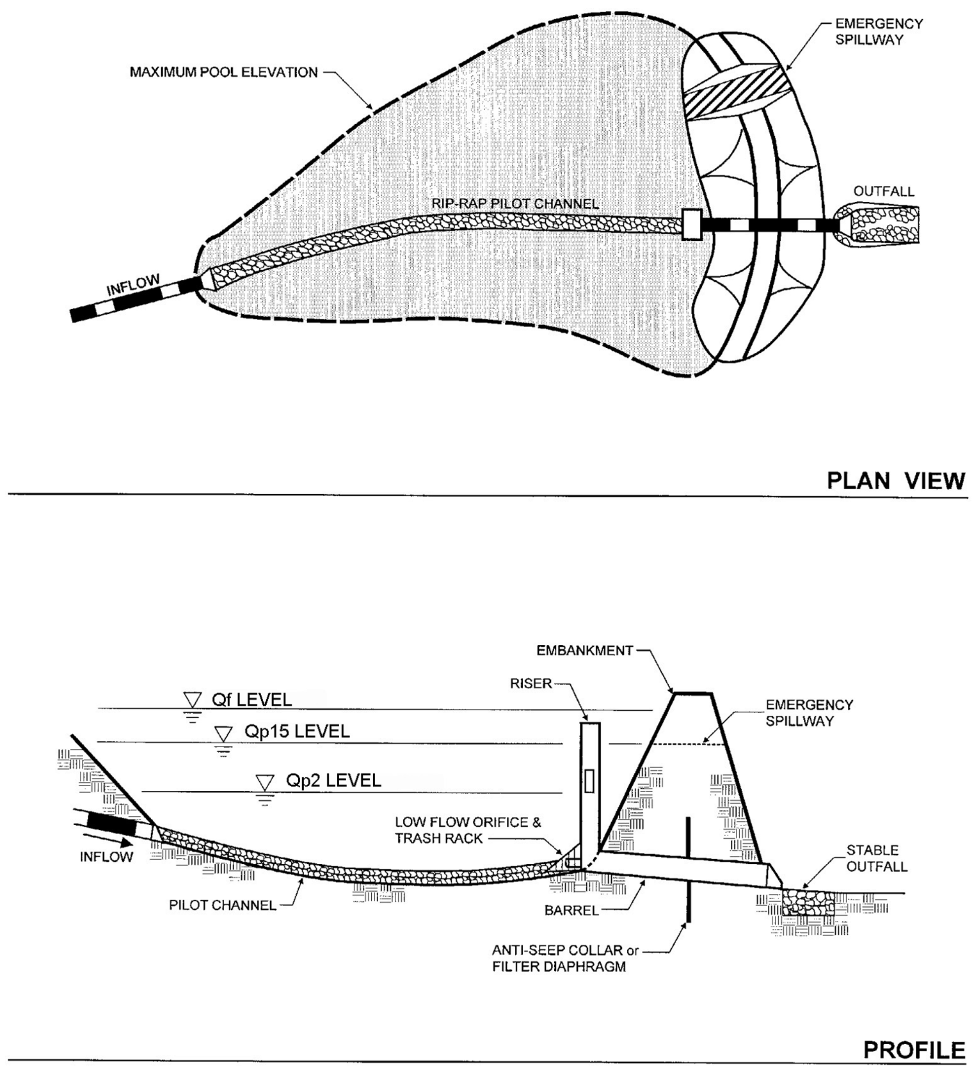
S-1	**Figure 3.17** Example of Underground Vault	S-1



Source: Adapted from MDE, 2000

An underground pipe system or vaults may be used to provide water quantity control (Q_p) but not water quality control (V_w).

S-2	**Figure 3.18** Example of Dry Pond	S-2



Source: Adapted from MDE, 2000

A typical dry pond provides water quantity control (Q_p) but not water quality control (V_w).

3.3.1 Storage Feasibility Criteria

For a dry pond system, no utility lines shall be permitted to cross any part of the embankment where the design water depth is greater than 2 feet.

All utilities must have a minimum 5' horizontal clearance from the facility.

Delineate the drainage area to show the downstream storm sewer system, surface water body, or water course and the extent of the underground facility.

A geotechnical report is required for all underground BMPs, including storage practices. Geotechnical testing requirements are outlined in Appendix E.

3.3.2 Storage Conveyance Criteria

The Department may require the use of the Natural Resources Conservation Service storage indication method or an equivalent acceptable method to route the design storms through the detention structure.

To prevent scouring of the pond bottom, stone pilot channels are required in all dry ponds. In no case shall a pond have a bottom slope less than 1% in the pilot channel and a 0.5% slope towards the outlet or pilot.

Velocity dissipation devices shall be placed at the outfall of all detention structures and along the length of any outfall channel as necessary to provide a non-erosive velocity of flow from the structure to a water course. An outfall analysis shall be included in the storm water management plan showing discharge velocities down to the nearest downstream water course. Where indicated, the developer / contractor must secure an off-site drainage easement for any improvements to the downstream channel.

Dry ponds shall have an earthen emergency spillway cut in natural ground unless waived by the Department. Emergency spillways cut in fill must be lined with filter cloth beneath PVC-coated gabion baskets.

The final release rate of the facility shall be modified if any increase in flooding or stream channel erosion would result at a downstream structure, highway, or natural point of restricted streamflow (see section 2.4 Additional Storm Water Management Requirements).

Show 100-year ponding and/or safe overflow pathways.

3.3.3 Storage Treatment Criteria

Provide structural details of the underground detention system.

Provide profile of entire system with inverts, pipe size, pipe type, slopes, and hydraulic grade line (HGL) through the facility.

Provide cross section(s) and plan view.

Water tight joints shall be provided at all pipe connections.

Underground detention structures shall be composed of reinforced concrete. Other materials may be used for storm water management detention when the Department has approved their application.

All structural information for non-standard structures or modified structures along with H-20 loading information must be provided for approvals.

Anti-flotation analysis is required to check for buoyancy problems in the high water table areas.

Anchors shall be designed to counter the pipe and structure buoyancy by at least a 1.2 factor of safety.

3.3.4 Storage Maintenance Criteria

All storage practices shall be designed so as to be accessible to annual maintenance.

Unless waived by the Department, a 5:1 slope and 15 foot wide entrance ramp is required for maintenance access to dry ponds

Trash racks shall be provided for low-flow pipes and for risers not having anti-vortex devices.

Section 3.4 Storm Water Ponds

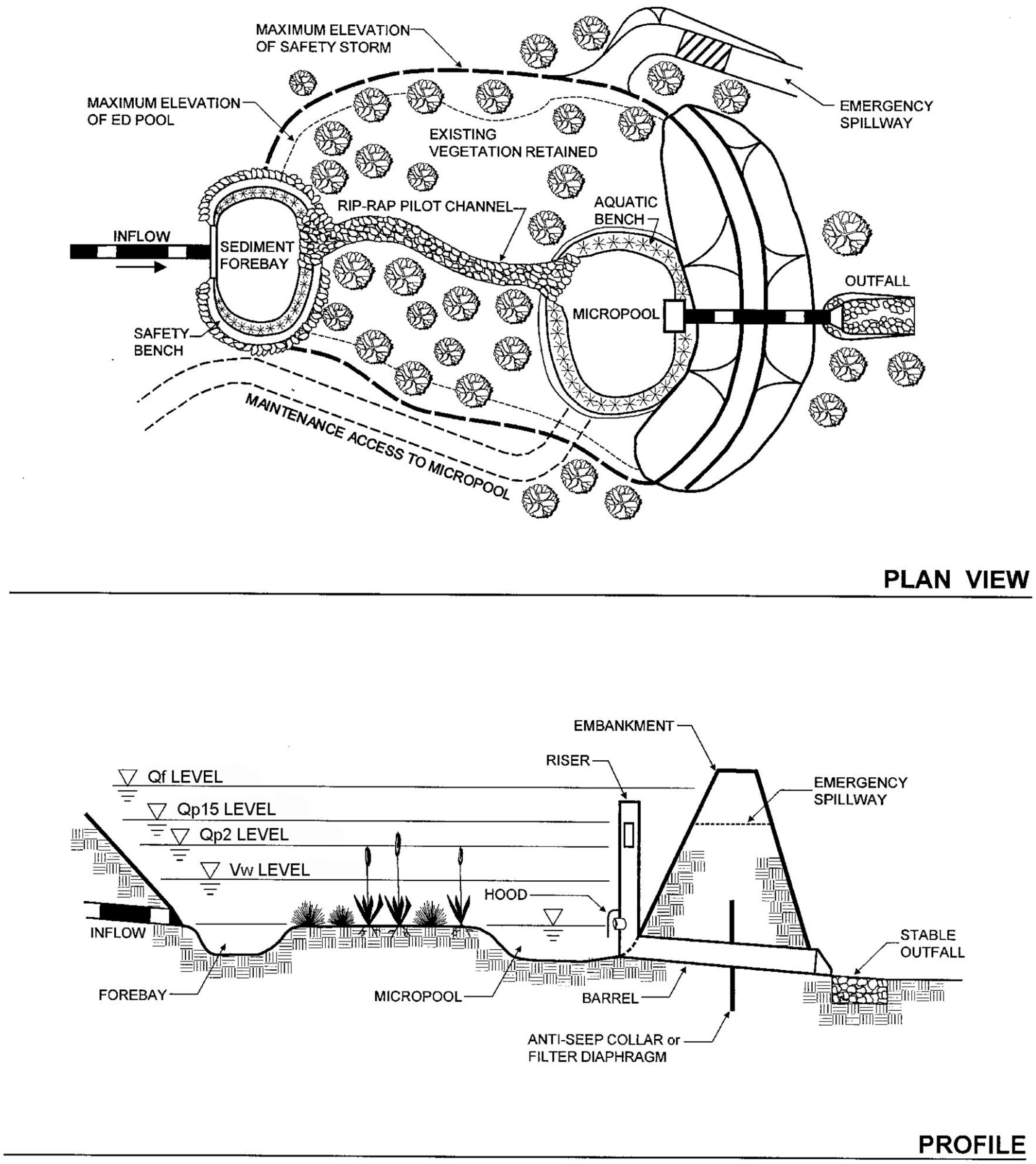
Definition: Practices that have a combination of a permanent pool, extended detention or shallow marsh that provide storage equivalent to the entire V_w. Design variants include:

- P-1 micropool extended detention pond
- P-2 wet pond
- P-3 wet extended detention pond
- P-4 pocket pond

Storm water ponds may also provide storm water detention storage (Qp_2, Qp_{15}, and / or Q_f) above the V_w storage.

The term "pocket" refers to a pond or wetland that has such a small contributing drainage area that little or no baseflow is available to sustain water elevations during dry weather. Instead, water elevations are heavily influenced and, in some cases, maintained by a locally shallow water table.

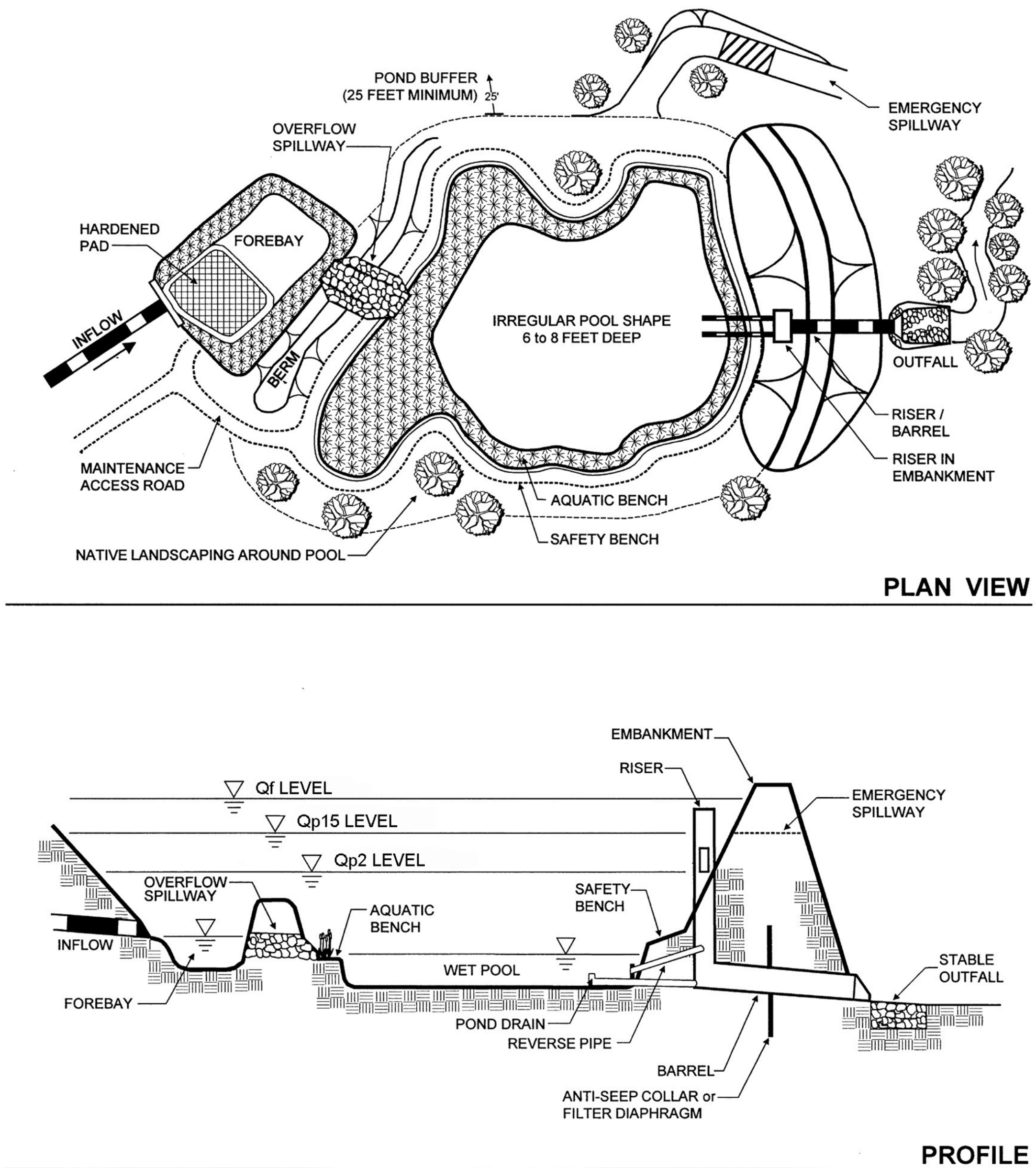
P-1	**Figure 3.19** Example of Micropool Extended Detention Pond	P-1



Source: Adapted from MDE, 2000

A micropool is provided in an extended detention pond to prevent resuspension of previously settled sediments and prevent clogging of the low flow orifice.

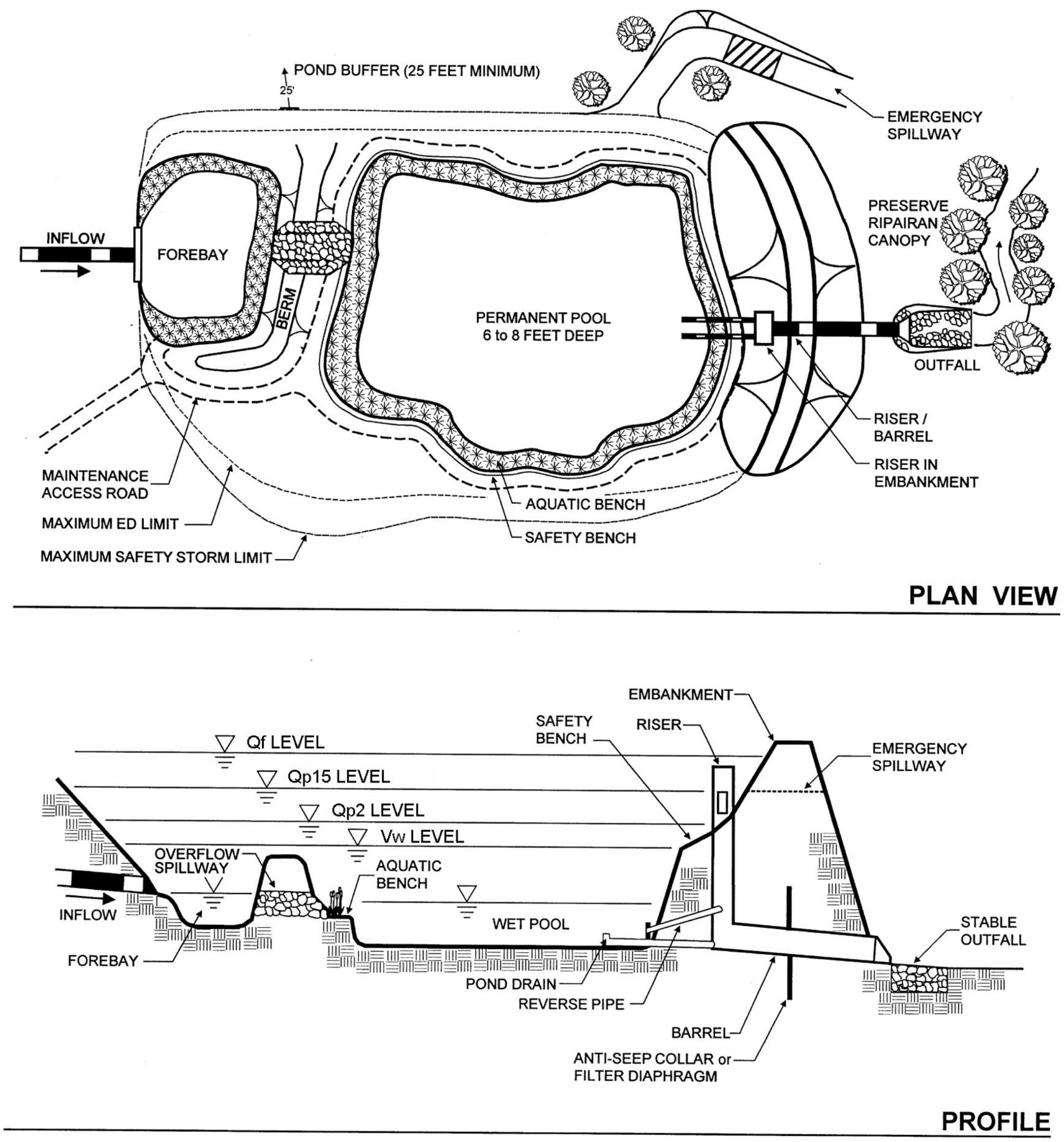
P-2	**Figure 3.20** Example of Wet Pond	P-2



Source: Adapted from MDE, 2000

A wet pond provides all of water quality volume storage in a permanent pool.

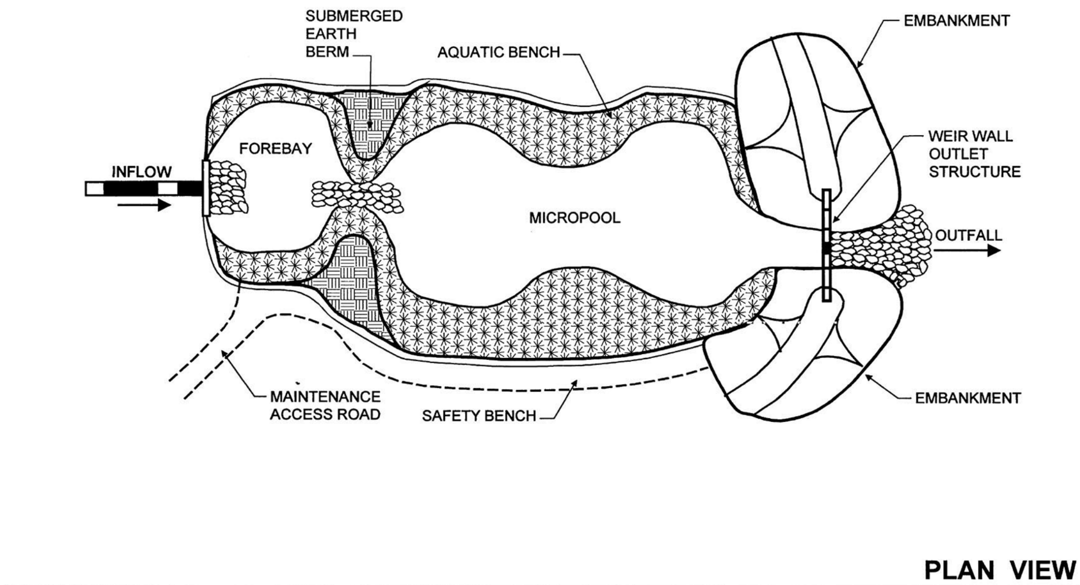
P-3	**Figure 3.21** Example of Wet Extended Detention Pond	P-3

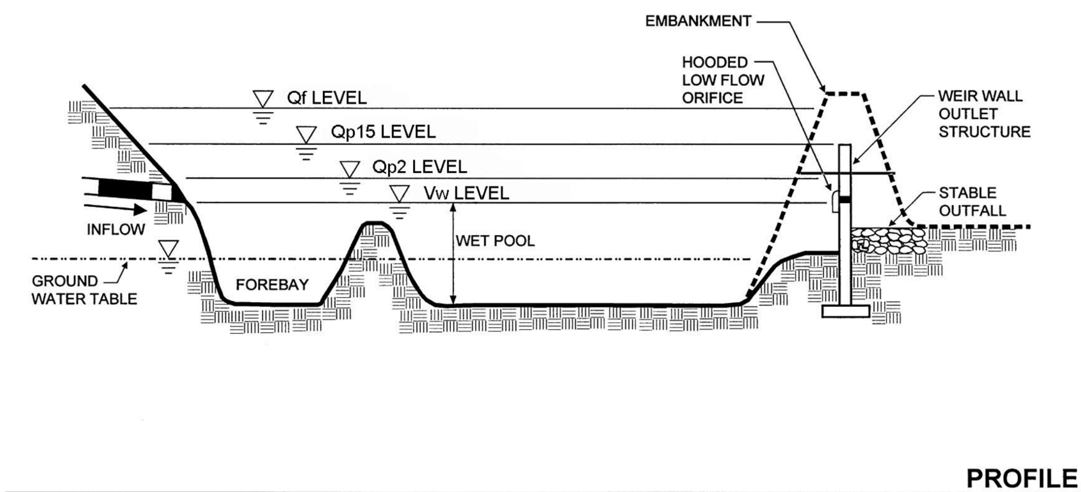


Source: Adapted from MDE, 2000

The wet ED pond provides water quality storage through a combination of permanent pool and extended detention storage.

P-4	**Figure 3.22** Example of Pocket Pond	P-4





Source: Adapted from MDE, 2000

A high water table or groundwater interception maintains permanent pool level in a pocket pond.

3.4.1 Pond Feasibility Criteria

Storm water ponds should have a minimum contributing drainage area of ten acres or more (25 or more are preferred), unless groundwater is confirmed as the primary water source (i.e., pocket pond).

Storm water ponds cannot be located within jurisdictional waters, including wetlands, without obtaining a Section 404 permit under the Clean Water Act.

For an open pond system, no utility lines shall be permitted to cross any part of the embankment where the design water depth is greater than 2 feet.

Sediment control ponds that are to be converted into permanent storm water management facilities must be designed according to all specifications stated in this book. Approval must be obtained before any such pond can be used for storm water management control.

3.4.2 Pond Conveyance Criteria

When reinforced concrete pipe is used for the principal spillway to increase its longevity, "O-ring" gaskets (ASTM C361) shall be used to create watertight joints.

To prevent scouring of the pond bottom, stone pilot channels are required in all ponds or portions of ponds above the permanent pool. In no case should a pond have a bottom slope less than 1% in the pilot channel and a 0.5% slope towards the outlet or pilot.

Inlet Protection

Inlet pipes to the pond can be partially submerged.

A forebay shall be provided at each inflow location, unless the inflow provides less than 10% of the total design storm inflow to the pond.

Adequate Outfall Protection

Velocity dissipation devices shall be placed at the outfall of all detention or retention structures and along the length of any outfall channel as necessary to provide a non-erosive velocity of flow from the structure to a water course. An outfall analysis should be included in the storm water management plan showing discharge velocities down to the nearest downstream water course. Where indicated, the developer / contractor must secure an off-site drainage easement for any improvements to the downstream channel.

Ponds must have an earthen emergency spillway cut in natural ground unless waived by the

Department. Emergency spillways cut in fill must be lined with filter cloth beneath PVC-coated gabion baskets.

The final release rate of the facility shall be modified if any increase in flooding or stream channel erosion would result at a downstream structure, highway, or natural point of restricted streamflow (see section 2.4 Additional Storm Water Management Requirements).

Flared pipe sections that discharge at or near the stream invert or into a step-pool arrangement should be used at the spillway outlet.

If a pond daylights to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. Excessive use of rip-rap should be avoided.

3.4.3 Pond Pretreatment Criteria

Sediment Forebay

Each pond shall have a sediment forebay or equivalent upstream pretreatment. The forebay shall consist of a separate cell, formed by an acceptable barrier (e.g. concrete, gabions, earthen embankment).

The forebay shall be sized to contain 0.1 inches per impervious acre of contributing drainage, and should be between 4 and 6 feet deep. The forebay storage volume counts toward the total V_w requirement. Exit velocities from the forebay shall be non-erosive. Non-erosive velocities are 4 feet per second for the two-year event, and 6 feet per second for the 15-year event.

Direct maintenance access for appropriate equipment shall be provided to the forebay.

The bottom of the forebay may be hardened to make sediment removal easier.

A fixed vertical sediment depth marker should be installed in the forebay to measure sediment deposition over time.

3.4.4 Pond Treatment Criteria

Minimum Water Quality Volume (V_w)

Provide water quality treatment storage to capture the computed V_w from the contributing drainage area through any combination of permanent pool, extended detention (V_w-ED) or marsh.

It is generally desirable to provide water quality treatment off-line when topography, head and space permit (i.e., apart from storm water quantity storage).

Water quality storage may be provided in multiple cells. Performance is enhanced when multiple treatment pathways are provided by using multiple cells, longer flowpaths, high surface area to volume ratios, complex microtopography, and/or redundant treatment methods (combinations of pool, ED, and marsh).

Minimum Pond Geometry

The minimum length to width ratio (i.e., length relative to width) for ponds is 1.5:1. Greater flowpaths and irregular shapes are recommended.

Maximum depth of the permanent pool should not generally exceed eight feet unless the pond is designed for multiple uses. Ponds should be wedge-shaped, narrowest at the inlet and widest at the outlet.

3.4.5 Pond Landscaping Criteria

Pond Benches

The perimeter of all deep pool areas (four feet or greater in depth) should be surrounded by two benches:

- A safety bench that extends 15 feet outward from the normal water edge to the toe of the pond side slope. The maximum slope of the safety bench shall be 6%.

- An aquatic bench that extends up to 15 feet inward from the normal shoreline and has a maximum depth of 18" below the normal pool water surface elevation.

Landscaping Plan

A landscaping plan for a storm water pond and its buffer shall be prepared to indicate how aquatic and terrestrial areas will be vegetatively stabilized and established.

Wherever possible, wetland plants should be encouraged in a pond design, either along the aquatic bench (fringe wetlands), the safety bench and side slopes (ED wetlands) or within shallow areas of the pool itself.

The best elevations for establishing wetland plants, either through transplantation or volunteer colonization, are within 6" (plus or minus) of the normal pool.

The soils of a pond buffer are often severely compacted during the construction process to ensure stability. The density of these compacted soils is so great that it effectively prevents root penetration, and therefore, may lead to premature mortality or loss of vigor. Consequently, it is advisable to excavate large and deep holes around the proposed planting sites, and backfill these with uncompacted topsoil.

As a rule of thumb, planting holes should be 3 times deeper and wider than the diameter of the rootball (for balled and burlap stock), and 5 times deeper and wider for container grown stock. This practice should enable the stock to develop unconfined root systems. Avoid species that require full shade, are susceptible to winterkill, or are prone to wind damage. Extra mulching around the base of the tree or shrub is strongly recommended as a means of conserving moisture and suppressing weeds.

Pond Buffers and Setbacks

A pond buffer should be provided that extends 25 feet outward from the maximum water surface elevation of the pond. The pond buffer should be contiguous with other buffer areas. An additional setback may be provided to permanent structures.

Existing trees should be preserved in the buffer area during construction. It is desirable to locate forest conservation areas adjacent to ponds. To discourage resident geese populations, the buffer can be planted with trees, shrubs and native ground covers.

Woody vegetation shall not be planted or allowed to grow within 15 feet of the toe of the embankment and 25 feet from the principal spillway structure.

Annual mowing of the pond buffer is only required along maintenance rights-of-way and the embankment. The remaining buffer can be managed as a meadow (mowing every other year) or forest.

3.4.6 Pond Maintenance Criteria

Maintenance Measures

Trash racks shall be provided for low-flow pipes and for riser openings not having anti-vortex devices.

Maintenance responsibility for a pond and its buffer shall be vested with a responsible authority by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval.

Sediment removal in the forebay should occur every 5 to 7 years or after 50% of total forebay capacity has been lost.

Maintenance Access

All ponds must be designed so as to be accessible to annual maintenance. Unless waived by the Department, a 5:1 and 15 foot wide entrance ramp shall be required for maintenance access.

A maintenance right of way or easement shall extend to a pond from a public or private road.

The maintenance access should extend to the forebay, safety bench, riser, and outlet and be designed to allow vehicles to turn around.

Non-clogging Low Flow Orifice

The low flow orifice shall have a minimum diameter of 3", and shall be adequately protected from clogging by an acceptable external trash rack. The low flow orifice diameter may be reduced to 1" if internal orifice protection is used (i.e., a perforated vertical stand pipe with holes or slots that are protected by wire-cloth and a stone filtering jacket).

The preferred method is a submerged reverse-slope pipe that extends downward from the riser to an inflow point one foot below the normal pool elevation.

Alternative methods are to employ a broad crested rectangular, V-notch, or proportional weir, protected by a half-round CMP that extends at least 12" below the normal pool.

The use of horizontal perforated pipe protected by geotextile and gravel is not recommended.

Riser in Embankment

The riser should be located within the embankment for maintenance access, safety and aesthetics.

Access to the riser is to be provided by lockable manhole covers, and manhole steps within easy reach of valves and other controls. The principal spillway opening can be "fenced" with pipe or rebar at 8" intervals for safety purposes.

Pond Drain

Each pond shall have a drain pipe that can completely or partially drain the pond. The drain pipe shall have an elbow within the pond to prevent sediment deposition, and a diameter capable of draining the pond within 24 hours.

Care should be exercised during pond drawdowns to prevent downstream discharge of sediments or anoxic water and rapid drawdown. The approving authority shall be notified before draining a pond.

Adjustable Gate Valve

Both the V_w outlet pipe and the pond drain should be equipped with an adjustable gate valve (typically a handwheel activated knife gate valve).

Both the V_w outlet pipe and the pond drain should be sized one pipe size greater than the calculated design diameter.

Valves should be located inside of the riser at a point where they (a) will not normally be inundated and (b) can be operated in a safe manner.

To prevent vandalism, the handwheel should be chained to a ringbolt, manhole step or other fixed object.

Safety Features

Fencing of ponds is not generally desirable, but may be required in some cases. A preferred method is to manage the contours of the pond to eliminate dropoffs and other safety hazards.

Side slopes to the pond shall not exceed 3:1 (h:v), and shall terminate on a 15 ft wide safety bench. Both the safety bench and the aquatic bench may be landscaped to prevent access to the pool. The bench requirement may be waived if slopes are 4:1 or gentler.

The principal spillway opening shall not permit access by small children, and endwalls above pipe outfalls greater than 48" in diameter shall be fenced to prevent a hazard.

Section 3.5 Storm Water Wetlands

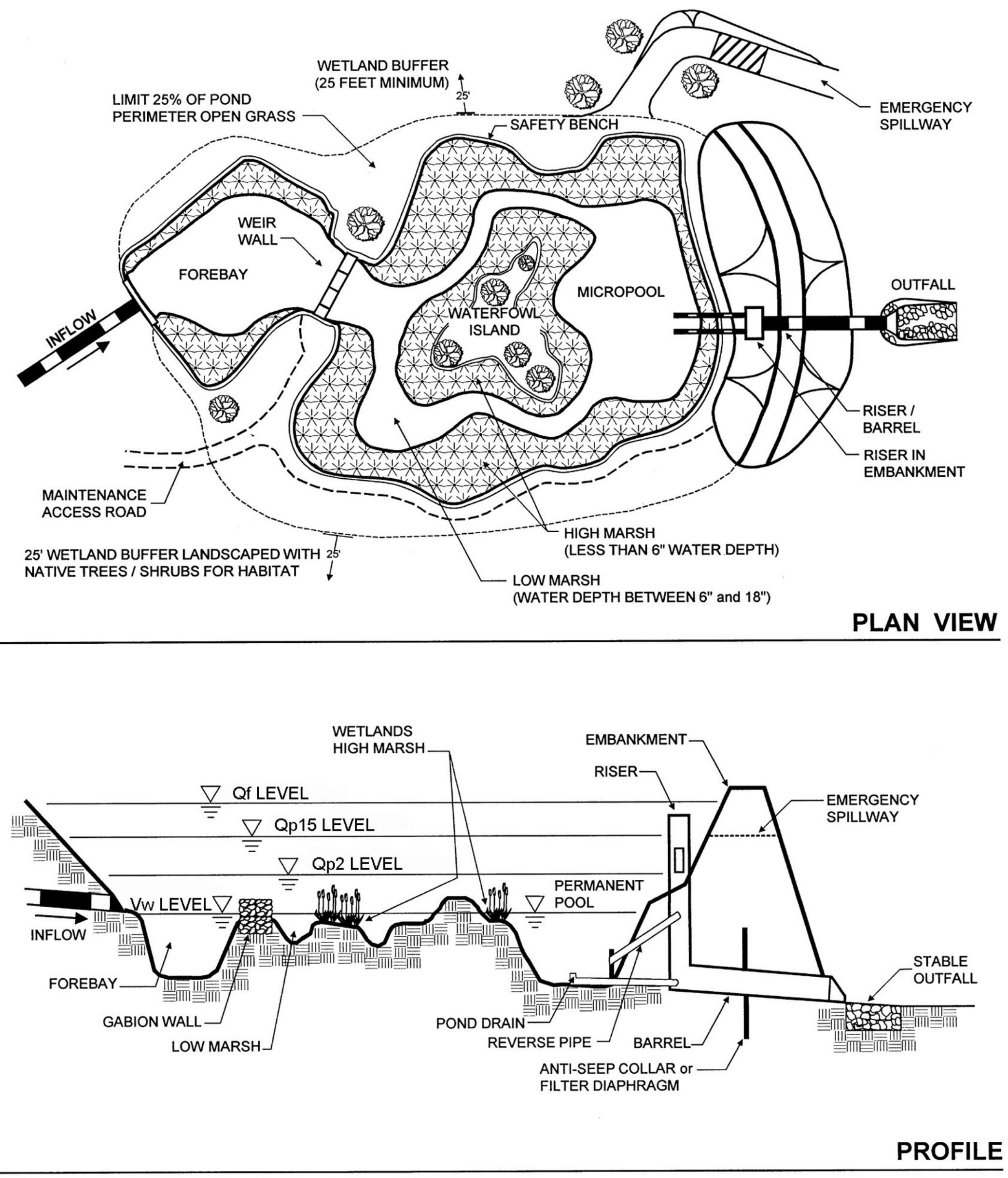
Definition: Practices that create shallow marsh areas to treat urban storm water which often incorporate small permanent pools and/or extended detention storage to achieve the full V_w. Design variants include:

- W-1 shallow wetland
- W-2 extended detention shallow wetland
- W-3 pocket wetland

Storm water wetlands may also provide storm water detention storage (Qp_2, Qp_{15}, and / or Q_f) above the V_w storage.

IMPORTANT NOTE: ALL OF THE POND PERFORMANCE CRITERIA PRESENTED IN SECTION 3.4 ALSO APPLY TO THE DESIGN OF STORM WATER WETLANDS. ADDITIONAL CRITERIA THAT GOVERN THE GEOMETRY AND ESTABLISHMENT OF CREATED WETLANDS ARE PRESENTED IN THIS SECTION.

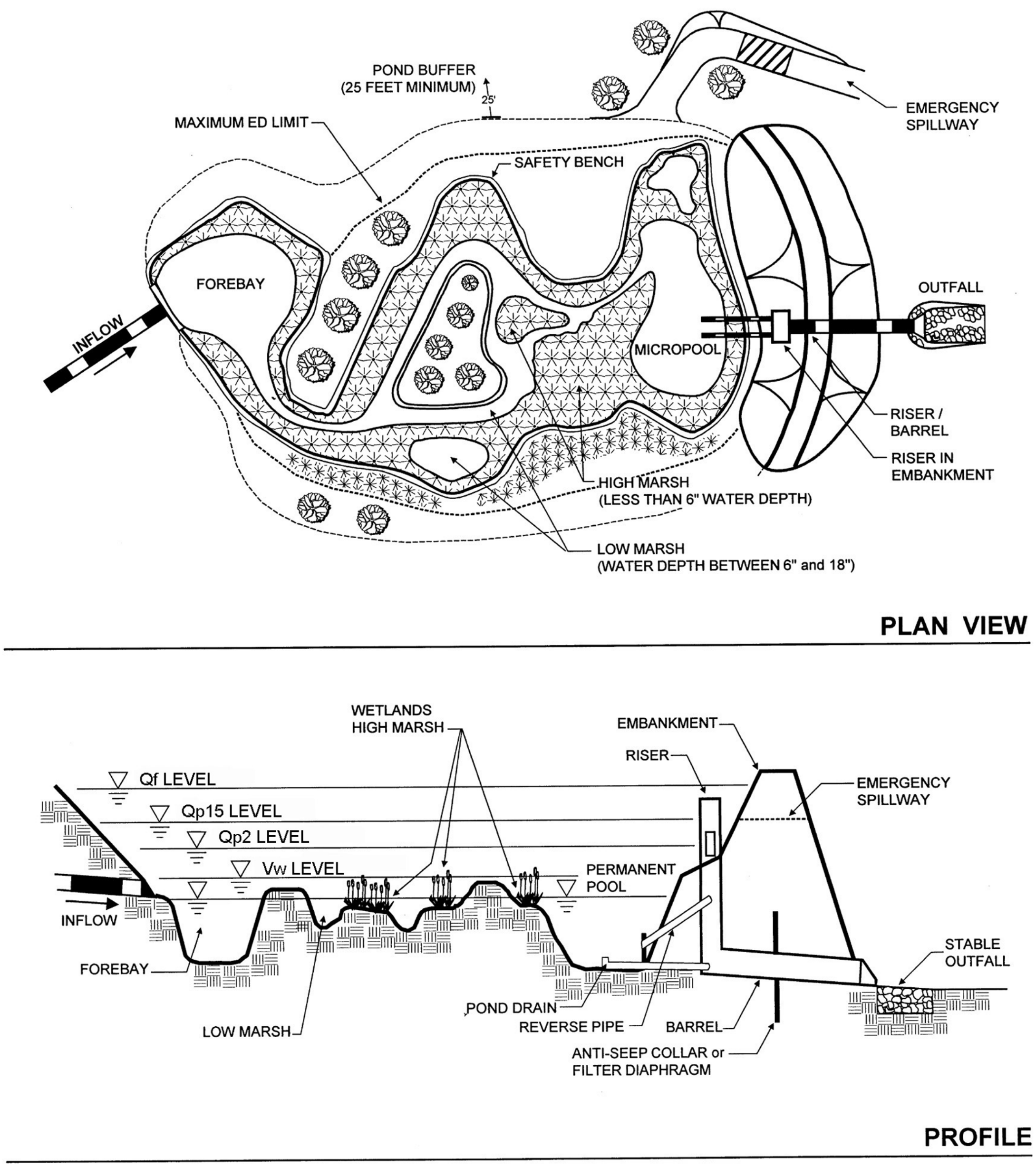
W-1	**Figure 3.23** Example of Shallow Wetland	W-1



Source: Adapted from MDE, 2000

Shallow wetlands provide V_w in a very shallow pool that has a large surface area.

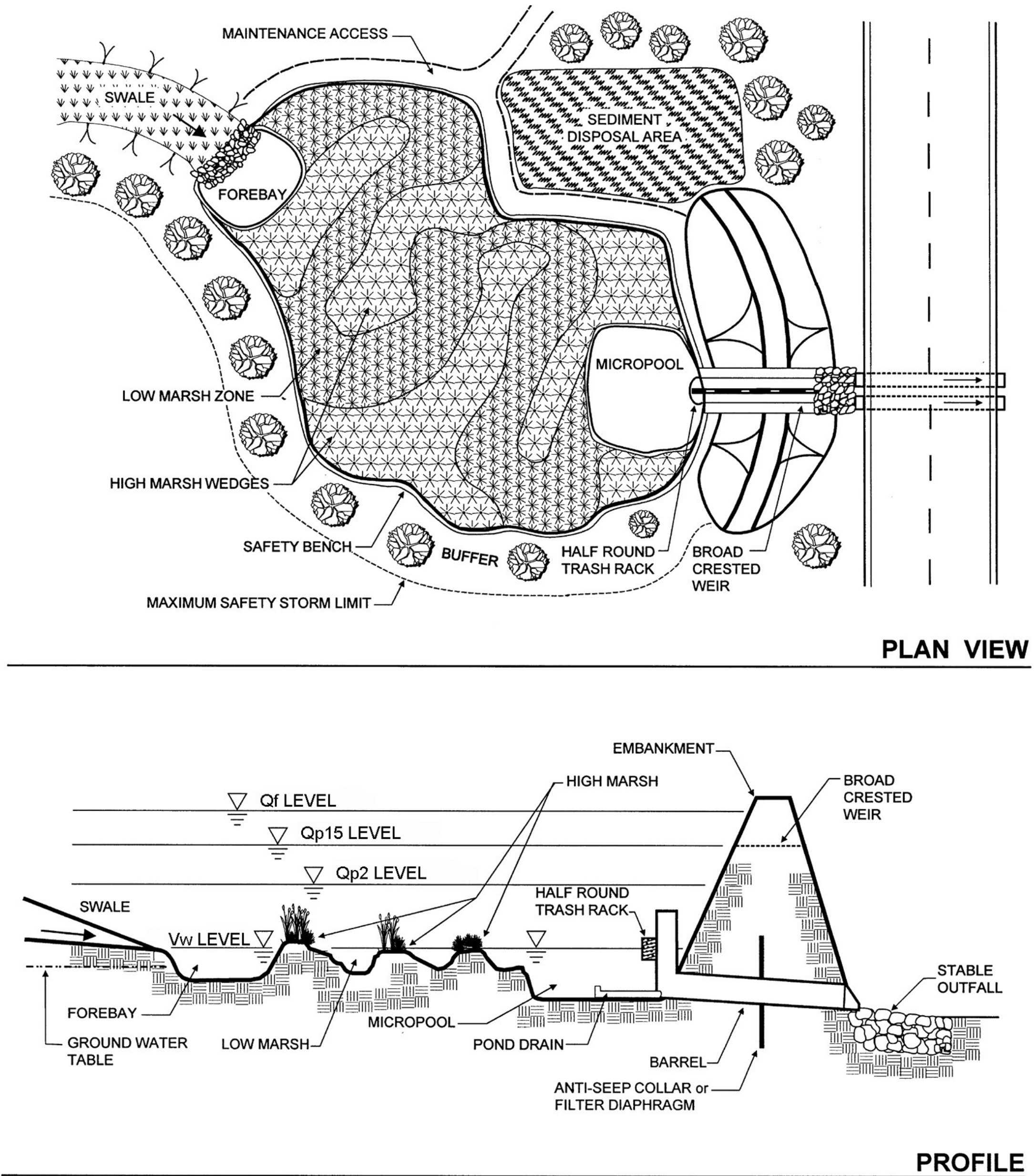
W-2	**Figure 3.24** Example of Extended Detention Shallow Wetland	W-2



Source: Adapted from MDE, 2000

ED shallow wetlands provide water quality storage by a combination of shallow marsh pool and extended detention storage.

W-3	**Figure 3.25** Example of Pocket Wetland	W-3



Source: Adapted from MDE, 2000

A high water table or groundwater interception helps maintain the shallow marsh pool in a pocket wetland.

3.5.1 Wetland Feasibility Criteria

A water balance should be performed to demonstrate that a storm water wetland can withstand a thirty day drought at summer evaporation rates without completely drawing down.

3.5.2 Wetland Conveyance Criteria

A minimum flow path of 2:1 shall be provided across the storm water wetland. This path may be achieved by constructing internal berms (e.g., high marsh wedges or rock filter cells). Microtopography is encouraged to enhance wetland diversity.

3.5.3 Wetland Pretreatment Criteria

Sediment regulation is critical to sustain storm water wetlands. *Consequently, a forebay shall be located at the inlet, and a micropool shall be located at the outlet.* Forebays are designed in the same manner as ponds (see Section 3.4.3). A micropool is a three to six foot deep pool used to protect the low flow pipe from clogging and to prevent sediment resuspension.

3.5.4 Wetland Treatment Criteria

The surface area of the entire storm water wetland shall be at least 1% of the contributing drainage area (1.5% for the shallow marsh design).

At least 25% of the total V_w should be in deepwater zones with a depth greater than four feet (the forebay and micropool are used to meet this criteria. In addition, the deepwater zones serve to keep mosquito populations in check by providing habitat for fish and other pond life that prey on mosquito larvae.

A minimum of 35% of the total surface area shall have a depth of 6" or less, and at least 65% of the total surface area shall be shallower than 18".

The bed of the wetland should be graded to create maximum internal geometry and microtopography.

If extended detention (ED) is utilized in a storm water wetland, the V_w-ED volume shall not comprise more than 50% of the total V_w, and its maximum water surface elevation shall not extend more than three feet above the normal pool. Quantity control storage can be provided above the maximum V_w elevation within the wetland.

To promote greater nitrogen removal, rock beds may be used as a medium for the growth of wetland plants. The rock should be 1" to 3" in diameter, placed up to the normal pool elevation.

3.5.5 Wetland Landscaping Criteria

A landscaping plan should be provided that indicates the methods used to establish and maintain wetland coverage. Minimum elements of a plan include: delineation of landscaping zones, selection of corresponding plant species, planting plan, sequence for preparing wetland bed (including soil amendments, if needed), and sources of plant material.

Structures such as fascines, coconut rolls, straw bales, or filter fence can be used to create shallow marsh cells in high velocity areas of the storm water wetland.

The landscaping plan should provide elements that promote greater wildlife and waterfowl use within the wetland and buffers.

A wetland buffer may extend 25 feet outward from the maximum water surface elevation, with an additional 15 foot setback to structures.

Wetland Establishment Guidance

The most common and reliable technique for establishing an emergent wetland community in a storm water wetland is to transplant nursery stock obtained from local aquatic plant nurseries. The following guidance is suggested when transplants are used to establish a wetland.

The transplanting window extends from early April to mid-June. Planting after these dates is quite risky, as the wetland plants need a full growing season to build the root reserves needed to get through the winter. If at all possible, the plants should be ordered at least three months in advance to ensure the availability of the desirable species.

The optimal depth requirements for several common species of emergent wetland plants are often 6" of water or less.

To add diversity to the wetland, 5 to 7 species of emergent wetland plants should be planted. Of these, at least three species should be selected from the "aggressive colonizer" group (e.g., bulrush, pickerelweed, arrow arum, three square and rice cutgrass).

No more than half the wetland surface area needs to be planted. If the appropriate planting depths are achieved, the entire wetland should be colonized within three years.

The wetland area should be sub-divided into separate planting zones of more or less constant depth.

One plant species should be planted within each flagged planting zone, based on their approximate depth requirements.

Individual plants should generally be planted 18" on center in clumps (Some species may be planted closer, while others can be planted at greater distances, a wetland specialist or qualified landscaping consultant may specify different planting strategies).

Post-nursery care of wetland plants is very important in the interval between delivery of the plants and their subsequent planting, as they are prone to desiccation. Stock should be frequently watered and shaded while on-site.

A wet hydroseed mix should be used to establish permanent vegetative cover in the buffer above of the permanent pool. For rapid germination, scarify the soil to 0.5" prior to hydroseeding. Alternatively, red fescue or annual rye can be used as a temporary cover for the wet species.

Because most storm water wetlands are excavated to deep sub-soils, they often lack the nutrients and organic matter needed to support vigorous growth of wetland plants. At these sites, 3" to 6" of topsoil or wetland mulch should be added to all depth zones in the wetland from one foot below the normal pool to 6" above. Wetland mulch is preferable to topsoil if it is available.

The storm water wetland should be staked at the onset of the planting season. Depths in the wetland should be measured to the nearest inch to confirm the original planting zones. At this time, it may be necessary to modify the landscape plan to reflect altered depths or the availability of wetland plant stock. Surveyed planting zones should be marked on an “as-built” or design plan, and located in the field using stakes or flags. The wetland drain should be fully opened at least three days prior to the planting date (which should coincide with the delivery date for the wetland plant stock).

Wetland mulch is another technique to establish a wetland plant community which utilizes the seedbank of wetland soils to provide the propagules for marsh development. The majority of the seedbank is contained within the upper 6" of the donor wetland soils. The mulch is best collected at the end of the growing season. Best results are obtained when the mulch is spread 3" to 6" deep over the high marsh and semi-wet zones of the wetland (-6" to +6" relative to the normal pool). Donor soils for wetland mulch shall not be removed from natural wetlands.

3.5.6 Wetland Maintenance Criteria

If a minimum coverage of 50% is not achieved in the planted wetland zones after the second growing season, a reinforcement planting will be required.

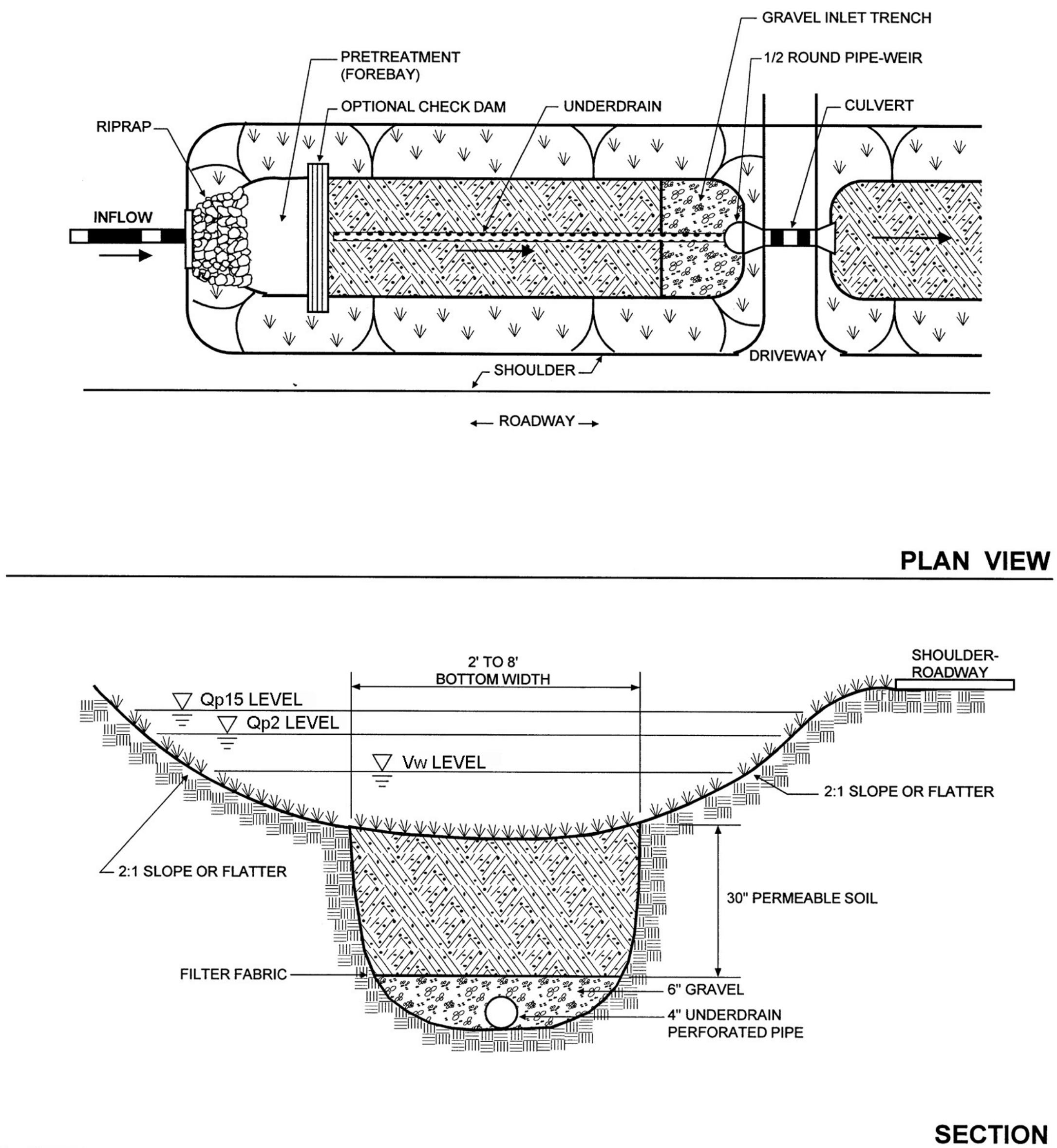
Section 3.6 Open Channel Systems

Definition: Vegetated open channels that are explicitly designed to capture and treat the full V_w within dry or wet cells formed by checkdams or other means. Design variants include:

- O-1 dry swale
- O-2 wet swale

Open channel systems shall not be designed to provide storm water detention except under extremely unusual conditions. Open channel systems must generally be combined with a separate facility to meet these requirements.

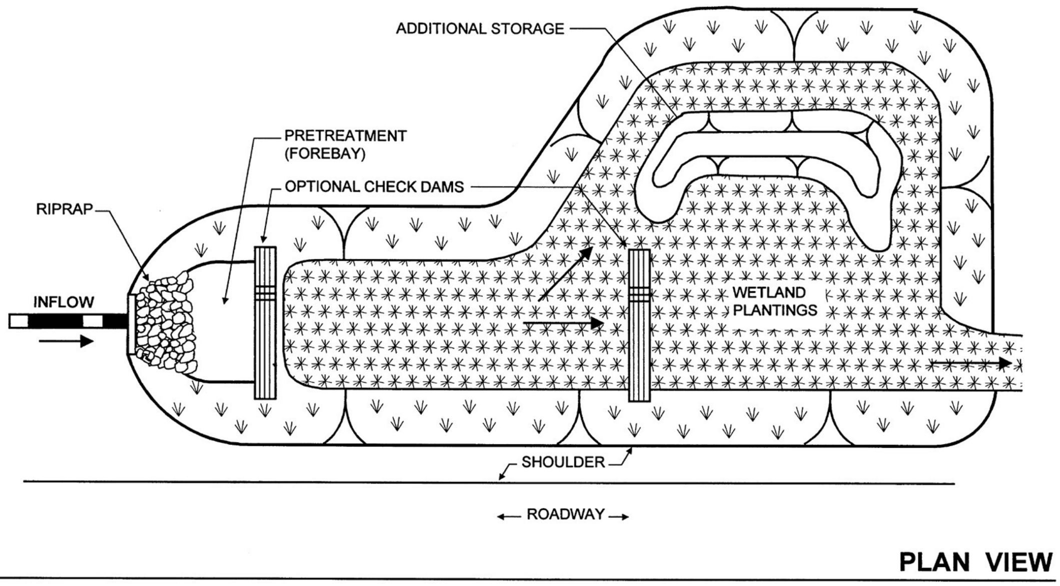
O-1	**Figure 3.26** Example of Dry Swale	O-1

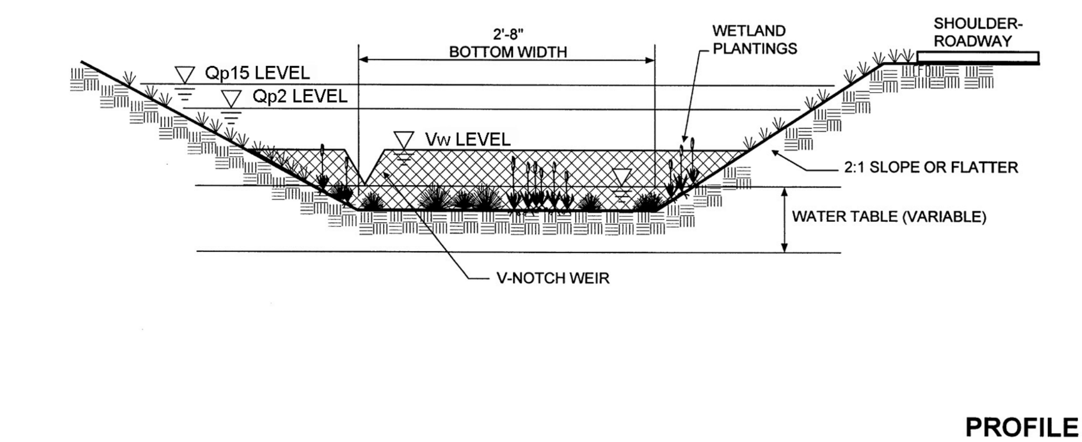


Source: Adapted from MDE, 2000

Dry swales may be used at low density residential projects or for very small impervious areas.

O-2	**Figure 3.27** Example of Wet Swale	O-2





Source: Adapted from MDE, 2000

Wet swales are ideal for treating highway runoff in low lying or flat terrain areas.

3.6.1 Open Channel Feasibility Criteria

Dry swales and wet swales shall have longitudinal slopes less than 4.0% to qualify for V_w treatment.

Open channel systems, designed for V_w treatment, are primarily applicable for land uses such as roads, highways, and residential development.

3.6.2 Open Channel Conveyance Criteria

The peak velocity for the 15-year storm shall be non-erosive (generally less than 6 fps) for the soil and vegetative cover provided.

The final designed channel shall provide 1 foot minimum freeboard above the designated water surface profile of the channel.

Channels should be designed with moderate side slopes (flatter than 3:1) for most conditions. In no event, can side slopes be steeper than 2:1.

Open channel systems which directly receive runoff from impervious surfaces may have a 6" drop onto a protected shelf (pea gravel diaphragm) to minimize the clogging potential of the inlet.

An underdrain system shall be provided for the dry swale to ensure a maximum ponding time of 48 hours.

3.6.3 Open Channel Pretreatment Criteria

Pretreatment of 0.1" of runoff per impervious acre storage shall be provided. This storage is usually obtained by providing checkdams at pipe inlets and/or driveway crossings.

A pea gravel diaphragm and gentle side slopes should be provided along the top of channels to provide pretreatment for lateral sheet flows.

3.6.4 Open Channel Treatment Criteria

Channel invert and tops of banks are to be shown in plan and profile views. A cross sectional view of each configuration should be shown for proposed channels. Completed limits of grading should be shown for proposed channels. For proposed channels, the transition at the entrance and outfall is to be clearly shown on plan and profile views.

Dry and wet swales should be designed to temporarily store the V_w within the facility to be released over a maximum 48 hour duration.

Dry swales and wet swales should have a bottom width no wider than 8 feet to avoid potential gullying and channel braiding.

Dry and wet swales should maintain a maximum ponding depth of one foot at the "mid-point" of the channel, and a maximum depth of 18" at the end point of the channel (for storage of the V_w).

3.6.5 Open Channel Landscaping Criteria

Wet swales are not recommended for residential developments as they can create potential nuisance or ponding conditions.

Landscape design shall specify proper grass species and wetland plants based on specific site, soils and hydric conditions present along the channel.

3.6.6 Open Channel Maintenance Criteria

Open channel systems should be mowed as required during the growing season to maintain grass heights in the 4" to 6" range. Wet swales, employing wetland vegetation, do not require frequent mowing of the channel.

Sediment build-up within the bottom of the channel or filter strip should be removed when 25% of the original V_w volume has been exceeded.

Chapter

3

Performance Criteria for BMP Groups

3.0 Performance Criteria for BMP Groups

This chapter outlines performance criteria for the six groups of structural storm water BMPs that include filtering systems, infiltration practices, storage practices, ponds, wetlands, and open channels.

Each set of BMP performance criteria, in turn, is based on six factors:

- General Feasibility
- Conveyance
- Pretreatment
- Treatment/Geometry
- Environmental/Landscaping
- Maintenance

The criteria represent a set of conditions which ensures an effective and long-lived BMP. Mandatory performance criteria are distinguished from suggested design criteria. In the text, mandatary performance criteria are distinguished by italics, whereas suggested design criteria are shown in normal typeface.

Chapter

4

Selecting and Locating the Most Effective BMP System

4.0 Selecting the Best BMP at a Site

This chapter outlines a process for selecting the best BMP or group of BMPs for a development site, and provides guidance on factors to consider on where to put them. The process is used to screen which BMPs can meet the pollutant removal targets for the V_w, and guides the designer through four steps that progressively screen:

- Storm Water Management Suitability
- Physical Feasibility Factors
- Community and Environmental Factors
- Checklist: Location and Permitting Considerations

More detail on the step-wise screening process is provided below:

Step ❶ Storm Water Management Suitability

Can the BMP meet all storm water sizing criteria at the site or are a combination of BMPs needed? In this step, designers can screen the BMP list using Matrix No. 1 to determine if a particular BMP can meet the V_w, Q_p, and/or Q_f storage requirements. In addition, the first matrix provides comparative indices on land consumption and safety risk that may preclude a BMP. At the end of this step, the designer can screen the BMP options down to a manageable number and determine if a single BMP or a group of BMPs are needed to meet storm water sizing criteria at the site.

Step ❷ Physical Feasibility Factors

Are there any physical constraints at the project site that may restrict or preclude the use of a particular BMP? In this step, the designer screens the BMP list using Matrix No. 2 to determine if the soils, water table, drainage area, slope or head conditions present at a particular development site might limit the use of a BMP. In addition, the matrix indicates which BMP options work well in highly urbanized areas.

Step ❸ Community and Environmental Factors

Do the remaining BMPs have any important community or environmental benefits or drawbacks that might influence the selection process? In this step, a matrix is used to compare the BMP options with regard to maintenance, habitat, community acceptance, cost and other environmental factors.

Step ❹ Location and Permitting Considerations

What environmental features must be avoided or considered when locating the BMP system at a site *to fully comply with local and federal regulations?* In this step, the designer follows an environmental features checklist that asks whether any of the following are present at the site: wetlands, waters of the United States, floodplains, and development infrastructure. Brief guidance is then provided on how to locate BMPs to avoid impacts to sensitive resources. If a BMP must be located within a sensitive environmental area, a brief summary of applicable permit requirements is provided.

Section 4.1 Storm Water Management Suitability

The first matrix (Table 4.1) examines the capability of each BMP option to meet the storm water management sizing criteria outlined in Chapter 2. Thus, it shows whether a BMP has the:

Ability to Meet the Water Quality Volume Requirement (V_w). It should be noted that not all practices are capable of meeting the V_w requirement. Thus, if a BMP cannot meet the V_w requirement, the matrix can help identify supplemental practices that can.

Ability to Provide Quantity Control (Q_{p2} and/or Q_{p15}). The matrix shows whether a BMP can typically meet the peak discharge requirement for the site. Again, the finding that a particular BMP cannot meet the requirement does not necessarily mean that it should be eliminated from consideration, but rather, is a reminder that more than one practice may be needed at a site (e.g., a bioretention area and a downstream storm water detention pond or detention structure).

Safety Index. A comparative index that expresses the potential safety risk of a BMP when designed according to the performance criteria outlined in Chapter 3. The safety factor is included at this stage of the screening process because liability and safety are of paramount concern in many residential settings.

Space Consumption Index. This comparative index expresses how much space a BMP typically consumes at a site. Again, this factor is included in this early screening stage because many BMPs are severely constrained by land consumption.

Table 4.1 BMP Selection Matrix No. 1 - Storm Water Management Suitability

Code	BMP List	V_w Ability	Q_p Control	Q_f Control	Safe	Space
F-1	Surface SF	●	○	○	safe	low
F-2	1-Chamber Underground SF	●	○	○	depends	low
F-3	3-Chamber Underground SF	●	◗	○	depends	low
F-4	Perimeter SF	●	○	○	safe	low
F-5	Vertical SF	●	○	○	safe	low
F-6	Organic Filter	●	○	○	safe	low
F-7	Bioretention	●	○	○	safe	medium
F-8	Roof Downspout System	●	○	○	safe	low
I-1	Infiltration Trench	●	◗	◗	safe	low
I-2	Infiltration Basin	●	◗	◗	safe	medium
S-1	Underground Vault	○	●	●	safe	low
S-2	Dry Pond	○	●	●	safe	medium
P-1	Micropool ED Pond	●	●	●	safe	low
P-2	Wet Pond	●	●	●	depends	medium
P-3	Wet ED Pond	●	●	●	depends	low
P-4	Pocket Pond	●	●	●	depends	low
W-1	Shallow Wetland	●	●	●	safe	high
W-2	ED Shallow Wetland	●	●	●	depends	medium
W-3	Pocket Wetland	●	●	●	safe	medium
O-1	Dry Swale	●	○	○	safe	medium
O-2	Wet Swale	●	○	○	safe	medium

V_w, Q_p, Q_f:
● = Yes
○ = No
◗ = Depends

Safety:
safe = low risk
unsafe = potential safety risks
depends = depends on site-specific conditions

Space:
low = BMP consumes relatively small amount of land
high = BMP consumes relatively high fraction of land
medium = depends on design

Section 4.2 Physical Feasibility Factors

At this point, the designer has narrowed down the BMP list to a manageable size and can evaluate the remaining options given the actual physical conditions at a site. This matrix (Table 4.2) will ultimately cross-reference the testing protocols needed to confirm physical conditions at the site. The six primary factors are:

Soils. The key evaluation factors are based on an initial investigation of the NRCS hydrologic soils groups at the site. Note that more detailed geotechnical tests are usually required for infiltration feasibility and during design to confirm permeability and other factors.

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

Drainage Area. This column indicates the minimum or maximum drainage area that is considered suitable for the practice. If the drainage area present at a site is slightly greater than the maximum allowable drainage area for a practice, some leeway is permitted. The minimum drainage areas indicated for ponds and wetlands should not be considered inflexible limits, and may be increased or decreased depending on water availability (baseflow or groundwater) or the mechanisms employed to prevent clogging or ensure an impermeable pond bottom.

Slope. This column evaluates the effect of slope on the practice. Specifically, the slope restrictions refer to how flat the area where the practice is installed must be.

Head. This column provides an estimate of the elevation difference needed at a site (from the inflow to the outflow) to allow for gravity operation within the practice.

Ultra-Urban Sites. This column identifies BMPs that work well in the ultra-urban environment, where space is limited and original soils have been disturbed. These BMPs are frequently used at redevelopment sites.

Table 4.2 BMP Selection Matrix No. 2 - Physical Feasibility Factors

Code	Bmp List	Soils	Water Table	Drainage Area	Site Slope	Head	Ultra Urban
F-1	Surface SF	OK	4 ft	10 ac max[1]	6% max	5 ft	depends
F-2	1-Chamber Underground SF			10,000 sq ft max		5 to 7ft	OK
F-3	3-Chamber Underground SF			2 ac max[1]			
F-4	Perimeter SF					2 to 3 ft	
F-5	Vertical SF					4 to 5 ft	
F-6	Organic Filter			5 ac max[1]		2 to 4 ft	
F-7	Bioretention	Made Soil				5 ft	
F-8	Roof Downspout System	OK		20,000 sq ft		N/A	
I-1	Infiltration Trench	f_c > 0.52 inch/hr	4 ft	5 ac max	6% max	1 ft	depends
I-2	Infiltration Basin			10 ac max		3 ft	not practical
S-1	Underground Vault	OK	4 ft	no limit[2]	no limit[4]	2 to 3 ft	OK
S-2	Dry Pond	"A" soils may require pond liner "B" soils may require testing	no restrictions		15% max		not practical
P-1	Micropool ED Pond	"A" soils may require pond liner "B" soils may require testing	4 ft if hotspot or aquifer	10 ac min[3]	15% max	6 to 8 ft	not practical
P-2	Wet Pond			25 ac min[3]			
P-3	Wet ED Pond						
P-4	Pocket Pond	OK	below WT[4]	5 ac max[1]		4 ft	OK
W-1	Shallow Wetland	A soils may require liner	4 ft if hotspot or aquifer	25 ac min	8% max	3 to 5 ft	not practical
W-2	ED Shallow Wetland						
W-3	Pocket Wetland	OK	below WT[4]	5 ac max		2 to 3 ft	depends
O-1	Dry Swale	Made Soil	4 ft	5 ac max	4% max	3 to 5 ft	not practical
O-2	Wet Swale	OK	below WT[4]			1 ft	

Notes: OK= not restricted; f_c= infiltration rate or permeability
WT= water table; N/A= not applicable
PT = pretreatment

1. drainage area can be larger in some instances.
2. no limit but practical drainage area limitations may exist due to minimum orifice size (e.g., 1" diameter with internal orifice)
3. unless adequate water balance and anti-clogging device installed
4. may not be used to accept runoff from storm water hotspot areas

Section 4.3 Community and Environmental Factors

The third step assesses community and environmental factors involved in BMP selection. This matrix (Table 4.3) employs a comparative index approach. The table indicates whether a BMP has a high, medium, or low benefit in each of four categories. A fifth category includes miscellaneous factors to consider.

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

Community Acceptance. This column assesses community acceptance, as measured by three factors: market and preference surveys, reported nuisance problems, and visual orientation (i.e., is it prominently located, or is it in a discrete underground location). It should be noted that a low rank can often be improved by a better landscaping plan.

Affordability. The BMPs are ranked according to their relative construction cost per impervious acre treated as determined from cost surveys and local experience.

Habitat. BMPs are evaluated on their ability to provide wildlife or wetland habitat, assuming that an effort is made to landscape them appropriately. Objective criteria include size, water features, wetland features and vegetative cover of the BMP and its buffer.

Other Factors. This column indicates other considerations in BMP selection.

Table 4.3 BMP Selection Matrix No. 3 - Community and Environmental Factors

Code	Bmp List	Ease of Maintenance	Community Acceptance	Affordability	Habitat	Other Factors
F-1	Surface SF	Medium	Medium	Low	Low	Minimize concrete
F-2	1-Chamber Underground SF	Low	High	Low	Low	Out of sight
F-3	3-Chamber Underground SF	Low	High	Low	Low	Out of sight
F-4	Perimeter SF	Low	High	Low	Low	Traffic bearing
F-5	Vertical SF	Low	High	Low	Low	
F-6	Organic SF	Medium	High	Low	Low	Change compost
F-7	Bioretention	Low	Medium	Medium	Low	Landscaping
F-8	Roof Downspout System	Low	High	Medium	Low	Frequent inspection
I-1	Infiltration Trench	Low	High	Medium	Low	Avoid large stone
I-2	Infiltration Basin	Low	Low	Medium	Low	Frequent pooling
S-1	Underground Vault	Medium	High	Low	Low	
S-2	Dry Pond	Medium	Medium	High	Low	
P-1	Micropool ED Pond	Medium	Medium	High	Medium	Trash/debris
P-2	Wet Pond	High	High	High	High	High pond premium
P-3	Wet ED Pond	High	High	High	High	
P-4	Pocket Pond	Low	Medium	High	Low	Drawdowns
W-1	Shallow Wetland	Medium	High	Medium	High	
W-2	ED Shallow Wetland	Medium	Medium	Medium	High	Limit ED depth
W-3	Pocket Wetland	Low	Low	High	Medium	Drawdowns
O-1	Dry Swale	High	High	Medium	Low	
O-2	Wet Swale	High	Medium	High	Low	Possible mosquitos

High = High Benefit
Medium = Medium Benefit
Low = Low Benefit

Section 4.4 Checklist: Location and Permitting Considerations

In the last step, a designer assesses the physical and environmental features at the site to determine the optimal location for the selected BMP or group of BMPs (Table 4.4). The checklist below provides a condensed summary on current BMP restrictions as they relate to common site features that may be regulated under local or federal law. These restrictions fall into one of three general categories:

1. Locating a BMP within an area that is expressly *prohibited* by law.

2. Locating a BMP within an area that is *strongly discouraged*, and is only allowed on a case by case basis. Local and/or federal permits shall be obtained, and the applicant will need to supply additional documentation to justify locating the BMP within the regulated area.

3. BMPs must be *setback* a fixed distance from the site feature.

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

Table 4.4 Location and Permitting Considerations

Site Feature	Location And Permitting Guidance
Jurisdictional Wetland U.S. Army Corps of Engineers Section 404 Permit	■ Wetlands should be delineated prior to siting storm water BMPs. ■ Use of natural wetlands for storm water treatment is *strongly discouraged*. ■ BMPs are also *restricted* in the 25 to 100 foot required wetland buffer. ■ Buffers may be utilized as a non-structural filter strip (i.e., accept sheetflow). ■ Must justify that no practical upland treatment alternatives exist. ■ Storm water must be treated prior to discharge into a wetland. ■ Where practical, excess storm water flows should be conveyed away from jurisdictional wetlands.
Stream Channel (Waters of the U.S.) U.S. Army Corps of Engineers Section 404 Permit	■ Stream channels should be delineated prior to design. ■ In-stream ponds (should be located near the origin of first order streams) and require review and permit. ■ Must justify that no practical upland treatment alternatives exist. ■ Temporary runoff storage (Cp_v) preferred over permanent pools (V_w). ■ Implement measures that reduce downstream warming.
100 Year Floodplain District of Columbia Emergency Management Agency District of Columbia Department of Health	■ Grading and fill for BMP construction is *strongly discouraged* within the ultimate 100 year floodplain, as delineated by FEMA flood insurance rate maps, FEMA flood boundary and floodway maps, or more stringent local floodplain maps. ■ Floodplain fill cannot raise the floodplain water surface elevation by more than a tenth of a foot.
Utilities District of Columbia Water and Sewer Authority	■ Locate existing utilities prior to design. ■ Note the location of proposed utilities to serve development. ■ BMPs are *discouraged* within utility easement or right of way for public or private utilities.

Table 4.4 Location and Permitting Considerations

Site Feature	Location And Permitting Guidance
Roads District of Columbia Department of Public Works	■ Consult DPW for any *setback* requirement from local roads. ■ Approval must also be obtained for any storm water discharges to a District-owned conveyance channel.
Structures District of Columbia Department of Public Works District of Columbia Water and Sewer Authority Department of Consumer and Regulatory Affairs	■ Consult local review authority for BMP *setbacks* from structures. ■ Recommended *setbacks* for each BMP group are provided in the performance criteria in Chapter 3 of this manual.
Water Wells District of Columbia Bureau of Water Quality	■ 100 foot *setback* for storm water infiltration. ■ 50 foot *setback* for all other BMPs. ■ Water appropriation permit needed if well water used for water supply to a BMP.
Combined Sewer Watersheds District of Columbia Water and Sewer Authority	■ Provide full quantity and quality control.

Chapter

5.0

Storm Water Implementation

5.1 Storm Water Management Plans

For all new and redevelopment projects, the applicant is responsible for submitting a storm water management plan which meets the design requirements provided by the District of Columbia Storm Water Management Regulations (District of Columbia Municipal Regulations (DCMR) Title 21, Chapter 5), and the detailed requirements of this guidebook. Each plan submitted should be signed by a professional engineer, licensed in the District of Columbia. A flow chart depicting the permitting process for the Watershed Protection Division is provided in Appendix G.

5.1.1 Submitting, Review and Approval of Storm Water Management Plans

- The plan should contain supporting computations, drawings, and sufficient information to evaluate the environmental characteristics of the effected areas, the potential impacts of the proposed development on water resources, the effectiveness and acceptability of storm water control facilities for managing storm water runoff, and maintenance and construction schedules. The applicant/contractor should certify on the drawings that all clearing, grading, drainage construction, and development is accomplished in strict accordance with the approved plan.

- The applicant should submit the storm water management plan, including all documentation, to the "One Stop Permit and Business Center." Projects that require immediate approval will be handled here. All other projects will be submitted to the Sediment and Storm Water Technical Services Branch of the Department for review. For each project, four sets of project plans shall be submitted for distribution to various review agencies. The Sediment and Storm Water Technical Services Branch will review the plan to determine compliance with the requirements of 21 DCMR, Chapter 5.

- Within 10 to 30 working days of the submission date of a plan, the Technical Review Staff of the Department shall review the plan and make a determination to approve or disapprove the plan.

- If it is determined that more information is needed or that a significant number of changes must be made before the plan can be approved, the applicant may withdraw the plan, make the necessary changes, and re-submit the plan. All re-submissions shall contain a list of the changes made. A new 10 to 30 day review period begins on the date of submission.

- If plan approval is denied, the reasons for the action shall be communicated to the applicant in writing.

The minimum information submitted for support of a storm water management plan or application for a waiver shall include:

- Site Development Submittal Information Sheet
- Site Plan
- Pre/Post-Development Hydrologic Computations
- Hydraulic Computations

Site Development Submittal Information Sheet

Applicants shall complete a copy of the Site Development Submittal Information Sheet as part of their submission package. A copy of the Site Development Submittal Information Sheet is provided in Appendix G and may be obtained from Department of Health technical staff at the "One Stop Permit and Business Center."

Site Plan

The following information shall be submitted on a site drawing of existing and proposed conditions:

(a) A plan showing property boundaries and the complete address of the property.
(b) Lot number, square number or parcel number designation (if applicable).
(c) North arrow, scale, date.
(d) Property lines (include longitude and latitude).
(e) Location of easements (if applicable).
(f) Existing and proposed structures, utilities, roads and other paved areas.
(g) Existing and proposed topographic contours.
(h) Soil information for design purposes.
(i) Area(s) of soil disturbance.
(j) Location of existing stream(s), wetlands, or other natural features within the project area.
(k) All plans and profiles must be drawn at a scale of 1" = 10', 1" = 20', 1" = 30', 1" = 40', 1" = 50', or 1" = 80', although 1" = 10', 1" = 20' and 1" = 30', are the most commonly used scales. Vertical scale for profiles shall be 1" = 2', 1" = 4', 1" = 5', or 1" = 10'.
(l) Location and size of existing utility lines including gas lines, sanitary lines, telephone lines or poles, and water mains.
(m) A legend identifying all symbols used on the plan.
(n) Applicable flood boundaries for sites lying wholly or partially within the 100-year floodplain.
(o) Information regarding the mitigation of any off-site impacts anticipated as a result of the proposed development.
(p) Construction specifications.
(q) Design and "As-Built" Certification.
    i. Certification by a Professional Engineer registered in the District of Columbia that the design of the storm water management facility conforms to engineering principles applicable to the treatment and disposal of storm water pollutants. The As-Built Storm Water Management Plan Guidelines are provided in Appendix G.

  ii. Certification and submission of the As-Built Certification by Professional Engineer form (provided in Appendix G) and one set of the “As-Built” plans within 21 days after completion of construction of the storm water management facility.

(r) Maintenance of Storm Water Management Facilities
  i. A maintenance agreement and a maintenance schedule must be submitted as part of the storm water management plan.

A covenant stating the property owner’s specific maintenance responsibilities must be recorded with the owner’s deed, at the Record of Deeds. The Declaration of Covenants for a Storm Water Management Facility is provided in Appendix G.

Pre/Post-Development Hydrologic Computations

The pre/post-runoff analysis shall include:

(a) A summary of soil conditions and field data.
(b) Pre/post-project curve number computation.
(c) Time of concentration calculation.
(d) Travel time calculation.
(e) Peak discharge computation for each subwatershed for the 24-hour storms of 2-year and 15-year frequencies. All hydrologic computations shall be included on the plan.

Hydraulic Computations

Hydraulic computations for the final design of water quality and quantity control structures may be accomplished by hand or through the use of software using equations/formulae generally accepted in the water resources industry. The summary of collection or management systems should include the following:

(a) Existing and proposed drainage area must be delineated on separate plans with the flow paths used for calculation of the times of concentration.
(b) Hydraulic capacity and flow velocity for drainage conveyance, including ditch, swales, pipes, inlets, and gutter. Plan profiles for all open conveyance and pipelines, with energy and hydraulic gradients shown thereon.
(c) The proposed development layout including:
  i. Storm water lines and inlets.
  ii. Location and design of BMP on site.
  iii. A list of design assumptions (e.g. design basis, 15-year return period, etc.).
  iv. The boundary of the contributing drainage area to the BMP.
  v. Schedule of structures (a listing of the structures, details, elevations including inverts, etc.).

vi. Manhole to manhole listing of pipe size, pipe type, slope, computed velocity, and computed flow rate (i.e., a storm drain pipe schedule).

5.2 Permits

5.2.1 Permit Requirements

A nonpoint source (storm water discharge) permit shall not be issued for any project unless a storm water management exemption or waiver is granted, or a storm water management plan meeting the requirements of 21 DCMR, Chapter 5 has been approved by the Department. Where applicable, a storm water management permit will not be issued until it is certified that the following have been properly executed:

- Recorded easements for the storm water management facility
- Easements to provide adequate access for inspection and maintenance to a public right-of-way

5.2.2 Permit Fee

A non-refundable permit fee will be collected at the time the permit is issued. The permit fee will provide for the cost of storm water management plan review, administration, management of the storm water permitting process, and inspection of all projects subject to the requirements of Section 526 through 535.

Waivers will be subjected to the permit and waiver fees.

5.2.3 Permit Suspension And Revocation

Any nonpoint source permit issued may be suspended or revoked after written notice is given to the permittee for any of the following reasons:

- Violation(s) of the conditions of the storm water management plan approval
- Changes in site runoff characteristics upon which a waiver was granted
- Construction which is not in accordance with the approved plans
- Noncompliance with correction notices(s) or stop work order(s)
- The existence of an immediate danger in a downstream area or adjacent properties in the opinion of the Department.

5.3 Inspection Requirements

5.3.1 Inspection Schedule and Reports

Prior to the approval of a storm water management plan, the applicant/contractor will submit a proposed construction and inspection control schedule. The proposed construction and inspection schedule should be included in the storm water management plan. The Department of Health will conduct inspections at the construction stages specified in the provisions, and file reports of inspections during construction of storm water management systems to ensure compliance with the approved plans.

No scheduled storm water management work will proceed until the Department's authorized representative, accompanied by the professional engineer responsible for certifying the "As-Built" plans, inspects and approves the work previously completed and the Department furnishes the applicant with results of the inspection soon after completion of each required inspection.

After receiving written notice from the Department, the applicant shall promptly correct any portion of the work which does not comply with the approved plans. The notice will set forth the nature of corrections required and the time frame within which corrections shall be made.

5.3.2 Inspection Requirements During Construction

- The Department, through its authorized representative, shall conduct on-site inspections at stages of construction as determined by the Department. Inspection report forms for sand filters and for infiltration devices are provided in Appendix G.

- All specifications for inspections at the various stages of construction should be incorporated into the storm water management plans.

- The developer shall notify the Department 24 hours prior to beginning the construction of any on-site or off-site storm water management facility subject to these regulations.

- The professional engineer for the project shall accompany the Department representative on all on-site inspections.

- A final inspection shall be conducted by the Department upon completion of the storm water management facility to determine if the completed work is constructed in accordance with approved plans.

After starting initial site operations, regular inspections will be made at the following specified states of construction:

- Infiltration systems shall be constructed at the following stages so as to ensure proper placement and allow for infiltration into the subgrade:

  (a) During on-site/off-site percolation/infiltration test
  (b) Upon completion of stripping, stockpiling, construction of temporary sediment control and drainage facilities
  (c) Upon completion of excavation to subgrade
  (d) Throughout the placement of perforated PVC/HDPE standpipes (for observation wells) including bypass pipes (where applicable), geotextile materials, gravel, or crushed stone course and backfill
  (e) Upon completion of final grading and establishment of permanent stabilization

- Flow attenuation devices, such as open vegetated swales upon completion of construction

- Retention and detention structures, at the following stages:

  (a) Upon completion of excavation to sub-foundation and where required, installation of structural supports or reinforcement for structures, including but not limited to the following.
    - Core trenches for structural embankments
    - Inlet-outlet structures and anti-seep structures
    - Watertight connectors on pipes
    - Trenches for enclosed storm water drainage facilities

  (b) During testing of the structure watertightness
  (c) During placement of structural fill, concrete and installation of piping and catch basins
  (d) During backfill of foundations and trenches
  (e) During embankment construction
  (f) Upon completion of final grading and establishment of permanent stabilization

- Storm water filtering systems, at the following stages:

  (a) Upon completion of excavation to sub-foundation and installation of structural supports or reinforcement for the structure;
  (b) During testing of the structure watertightness;
  (c) During placement of concrete and installation of piping and catch basins;
  (d) During backfill around the structure;
  (e) During pre-fabrication of structure at manufacturing plant;
  (f) During pouring of floors, walls and top slab;
  (g) During installation of manholes/trap doors, steps, orifices/weirs, bypass pipes, and

sump pit (when applicable);
(h) During placement of filter bed; and
(i) Upon completion of final grading and establishment of permanent stabilization.

5.3.3 Final Inspection Reports

A final inspection will be conducted by the Department to determine if the completed work is constructed in accordance with approved plans and the intent of 21 DCMR, Chapter 5, a registered professional engineer licensed in the District of Columbia is required to certify "As-Built" plans that the storm water management facility has been constructed in accordance with the approved plans and specifications (the As-Built Certification by Professional Engineer form is provided in Appendix G. The "As-Built" certification shall be on the original storm water management plan. Upon completion, these plans will be submitted to the Department for processing. The estimated time for processing will be two weeks (ten working days), after which the plans will be returned to the engineer. The applicant shall receive written notification of the final inspection results. The Department will maintain a permanent file of inspection reports.

5.3.4 Inspection for Preventive Maintenance

Preventive maintenance will be ensured through inspection of all infiltration systems, swales, retention, or detention structures by the Department. The inspection will occur twice every year during the first five years of operation and at least once every two years thereafter. Maintenance inspection forms are provided in Appendix G.

Preventive maintenance inspection reports will be maintained by the Department on all storm water management structures. The reports shall conform to the detailed requirement of the Department.

If, after an inspection by the Department, the condition of a storm water management facility presents an immediate danger to the public safety or health because of an unsafe condition or improper maintenance, the Department will take such action as may be necessary to protect the public and make the facility safe. Any costs incurred by the Department will be assessed against the owner(s).

5.4 Maintenance

5.4.1 Maintenance Responsibility

The owner of the property on which work has been done pursuant to 21 DCMR, Chapter 5 for private storm water management facilities, or any other persons or agent in control of such property, shall maintain in good condition and promptly repair and restore all grade surfaces, walls, drains, structures, vegetation, erosion and sediment control measures, and other protective devices. Such repairs or restorations will be in accordance with approved plans.

A maintenance agreement and a maintenance schedule must be submitted as part of the storm water management plan. A covenant stating the property owner's specific maintenance responsibilities must be recorded with the owner's deed, at the Record of Deeds. A maintenance schedule for any storm water management facility will be developed for the life of the project and shall state the maintenance to be completed, the time for completion, and who will perform the maintenance including provisions for normal and abnormal maintenance. The maintenance schedule will be printed on the storm water management plan.

5.4.2 Maintenance Agreement

The Department will not issue any nonpoint source permit for which storm water management is required until the Department certifies that the applicant or owner has executed an inspection and maintenance agreement binding on all subsequent owners of land served by the private storm water management facility. Such agreement should provide for access to the facility at reasonable times, and for regular inspection by the Department or its authorized representative, and for regular or special assessments of property owners to ensure that the facility is maintained in proper working condition. The Declaration of Covenants for a Storm Water Management Facility is provided in Appendix G.

The Agreement should be recorded in the land records of the District of Columbia by the applicant and/or owner. The agreement should also provide that, if after written notice by the Department to correct a violation requiring maintenance work, satisfactory corrections are not made by the owner(s) of the land served by the facility within a reasonable period of time, not to exceed 45-60 days unless extended for good cause shown, the Department may perform all necessary work to place the facility in proper working condition. The owner(s) of property served by the facility will be assessed the cost of the work and any penalties and there will be a lien on any property served by the facility, which may be placed on the tax bill and collected as ordinary taxes by the District of Columbia.

5.5 Penalties

Any person convicted of violating the storm water provisions of 21 DCMR, Chapter 5 will be guilty of a misdemeanor, and upon conviction thereof, will be subject to a fine of at least two thousand five hundred dollars ($2,500) and no more than twenty-five thousand dollars ($25,000) or imprisonment not exceed to exceed one year or both. Each day that a violation continues will be deemed a separate offense. In addition penalties for failure to comply with a final compliance order, a final cease and desist order or a final suspension, revocation or denial order shall be in accordance with Section 17 of the Water Pollution Control Act of 1984, as amended.

In any instance where a civil fine, penalty or fee has been established pursuant to the Civil

Infractions Act and the Civil Infractions Regulations found in 21 DCMR, Chapter 32, the civil fine, penalty or fee may be imposed as an alternative sanction to the penalties set forth in the Water Pollution Control Act.

Enforcement procedures for the storm water management regulations are outlined in 21 DCMR, Chapter 22.

Any court of competent jurisdiction will have the right to issue restraining orders, temporary or permanent injunctions, or mandamuses or other appropriate forms of remedy or relief.

5.6 Appeals

Any person aggrieved by the action of any official charged with the enforcement of the storm water management provisions of 21 DCMR, Chapter 5 as a result of the disapproval of an (properly filed) application for a permit, issuance of a written notice of violation, or an alleged failure to properly enforce 21 DCMR, Chapter 5 in regard to a specific application, will have the right to appeal the action to the Director of the Department.

The appeal should be filed in writing 15 days of the date from the official transmittal of the final decision, or determination of the applicant, should state clearly the grounds on which the appeal is based, and should be processed in the manner prescribed for hearing administrative appeals under the Civil Infraction Act of 1985, as amended.

In addition, any person adversely affected or aggrieved by a final compliance order, cease and desist order or other administrative order issued pursuant to the provisions of 21 DCMR, Chapter 22, may appeal the action by filing a petition for review in the District of Columbia Court of Appeals within thirty (30) days of the date of service of the final order upon the party making the appeal.

5.7 Exemptions

The following development activities shall be exempt from the provisions of the storm water management requirements of 21 DCMR, Chapter 5:

- Minor land disturbing activities such as home gardening and individual home landscaping repairs and maintenance work

- Single family dwelling utility service connections and construction or utility construction where the excavated material is removed from the job site

- Tilling, planting or harvesting of agricultural or horticultural crops

- Installation of fence and sign posts or poles

- Emergency work to protect a life, list or property, and emergency repairs, provided that if the land disturbing activity would have required an approved erosion and sedimentation control plan if the activity were not an emergency, then the land disturbed shall be shaped and stabilized in accordance with the requirements of the Department

- Additions or modifications to existing single family residential structures, detached garages, sheds, swimming pools or similar improvement

- Construction or grading operations, or both, that do not disturb more than five thousand square feet of land area, unless such construction or grading operations shall be part of an approved subdivision plan which contains provisions for storm water management; or

- Residential development consisting of single family dwellings each of which shall be situated on lots of two or more acres.

5.8 Waivers and Variances

The Director, or his or her designee, may waive the storm water management requirements for individual developments, provided, that the applicant first submits to the Department, a written request containing descriptions, drawings, and any other information that shall be necessary to evaluate the proposed development. Separate written requests for waivers shall be submitted for each addition, extension or modification to a development.

In order to be eligible for a waiver, an applicant shall demonstrate that storm water runoff from the subject property will not adversely impact the receiving wetlands, water course or waterway because:

- The proposed development will not generate more than a ten percent (10%) increase in the two-year pre-development peak discharge rate.

- The site is surrounded by developed areas which are served by an existing network of public storm drainage systems of adequate capacity to accommodate the runoff from the proposed development, except for the following:

    1) shopping centers
    2) industrial or commercial developments
    3) subdivision
    4) roads
    5) parking lots

- Provisions that control the direct outfall to tidewater, when the first one-half inch (0.5") is treated in a water quality structure meeting standards and specifications.

The Director may grant a variance from any of the provisions in 526 through 535 of this chapter if there are exceptional circumstances applicable to the site, and where such strict adherence to these provisions will result in unnecessary hardship or practical difficulty.

A written request for variance shall be submitted to the Branch Chief, stating the specific variance sought and the reason.

Appendix

A

Acceptable Hydrologic Methods and Models

A.1 Acceptable Hydrologic Methods and Models

The following are the acceptable methodologies and computer models for estimating runoff hydrographs before and after development. These methods are used to predict the runoff response from given rainfall information and site surface characteristic conditions. The design storm frequencies used in all of the hydrologic engineering calculations will be based on design storms required in this guidebook unless circumstances make consideration of another storm intensity criteria appropriate.

- Rational Method & Modified Rational Method
- Natural Resource Conservation Service TR-55
- TR-20, HEC-1, and SWMM computer models

These methods are given as valid in principle, and are applicable to most storm water management design situations in the District of Columbia. Other methods may be used when the District reviewing authority approves their application.

A.2 Rational and Modified Rational Methods

These methods will be permitted for use in a development of five acres or less. When applying these methods, the following steps must be taken in the design consideration:

- In the case of more than one sub-drainage area, the longest time of concentration shall be selected.

- Individual sub-drainage flows shall not be summed to get the total flow for the watershed.

- The runoff coefficient, C, shall be a composite of the future site development conditions for all contributing areas to the discharge point. Runoff coefficient factors for typical District of Columbia land uses are provided in Table A.1.

- The flow time in storm sewers shall be taken into account in computing the watershed time of concentration.

- The storm duration shall be dependent upon the watershed time of concentration.

- The storm intensity can be selected from the selected storm duration.

Table A.1 Runoff Coefficient Factors for Typical District of Columbia Land Uses

Zone	Predominant Use	Minimum Lot Dimensions		Runoff Coefficient C
		Width (feet)	Area (sq ft)	
R-1-A	One-family detached dwelling	75	7,500	0.60
R-1-B	One-family detached dwelling	50	5,000	0.65
R-2	One-family semi-detached dwelling	30	3,000	0.65
R-3	Row dwelling	20	2,000	0.70
R-4	Row dwelling	18	1,800	0.75
R-5-A	Low density apartment	--	--	0.70
R-5-B	Medium density apartment house	--	--	0.75
R-5-C	Medium high density apartment house	--	--	0.80
R-5-D	High density building	--	--	0.80
C	Commercial	--	--	0.85 - 0.95
M	General Industry	--	--	0.80 - 0.90
Park		--	--	0.35

A.3 Natural Resource Conservation Service TR-55 (Desktop Model)

Application of the TR-55 Model to determine peak discharge and the volume of detention storage in a project drainage area shall be dependent upon the following conditions:

- Use Type II and 24-hour rainfall distributions as developed by the Natural Resource Conservation Service (Figure A.1 and Table A.2).

- Average antecedent moisture conditions are defined as a total of 1.4" to 2.1" of rainfall during the five-day period immediately preceding the design rainfall. Adjustments shall be made to simulate dry or wet antecedent moisture conditions.

- All flow shall be assumed to be sheet flow, shallow concentrated flow, or open channel flow.

- Drainage areas exceeding 25 acres that are heterogeneous with respect to land use, Runoff Curve Number (RCN) or Time of Concentration (T_c) shall require a separate hydrological analysis for each sub-area including T_c, RCN, soils and land use. Hydrographs shall be combined in a way described by Table 5-2 of the TR-55 publication.

A.4 HEC-1, TR-55, Quick TR-55, TR-20, and SWMM Computer Models

If the application of the above high-speed computer models is needed, the complete input data file printout and diskette will be submitted with the storm water management plans at the 85% submittal stage. Submission of storm water management plans shall include the following computer model documentation:

- For the TR-20 method, supporting computation for T_c and a drainage area map indicating all hydrologic sub-areas shall be submitted.

- For the TR-55 & TR-20 methods, sheet flow length shall be less than 100 feet; use a 24-hour rainfall for each design storm to compute the travel time for sheet flow.

- For all computer models, supporting computation prepared for the data input file, such as hard copy and diskette, shall be submitted with the storm water management plans.

In general, designs shall be based on the following criteria:

- Inflow-outflow hydrographs shall be computed for each design storm presented graphically, and submitted for all plans.

- Pre-development runoff conditions shall be computed for the existing land use of the property, assuming good hydrologic conditions and land with grass cover.

- Post-development shall be computed using a NRCS runoff curve number for future land use assuming good hydrologic and appropriate land use conditions.

- Drainage areas exceeding 25 acres that are heterogenous with respect to land-use, RCN, runoff coefficient and T_c shall require a separate hydrological analysis for each sub-area, including T_c, RCN/C, soil type and land use.

- Pre-development time of concentration shall be based on the sum total of computed or estimated overland flow time and travel in natural swales, streams, creeks and rivers, but never less than six minutes.

- Post-development time of concentration shall be based on the sum total of the inlet time and travel time in improved channels or storm drains, but shall not be less than six minutes.

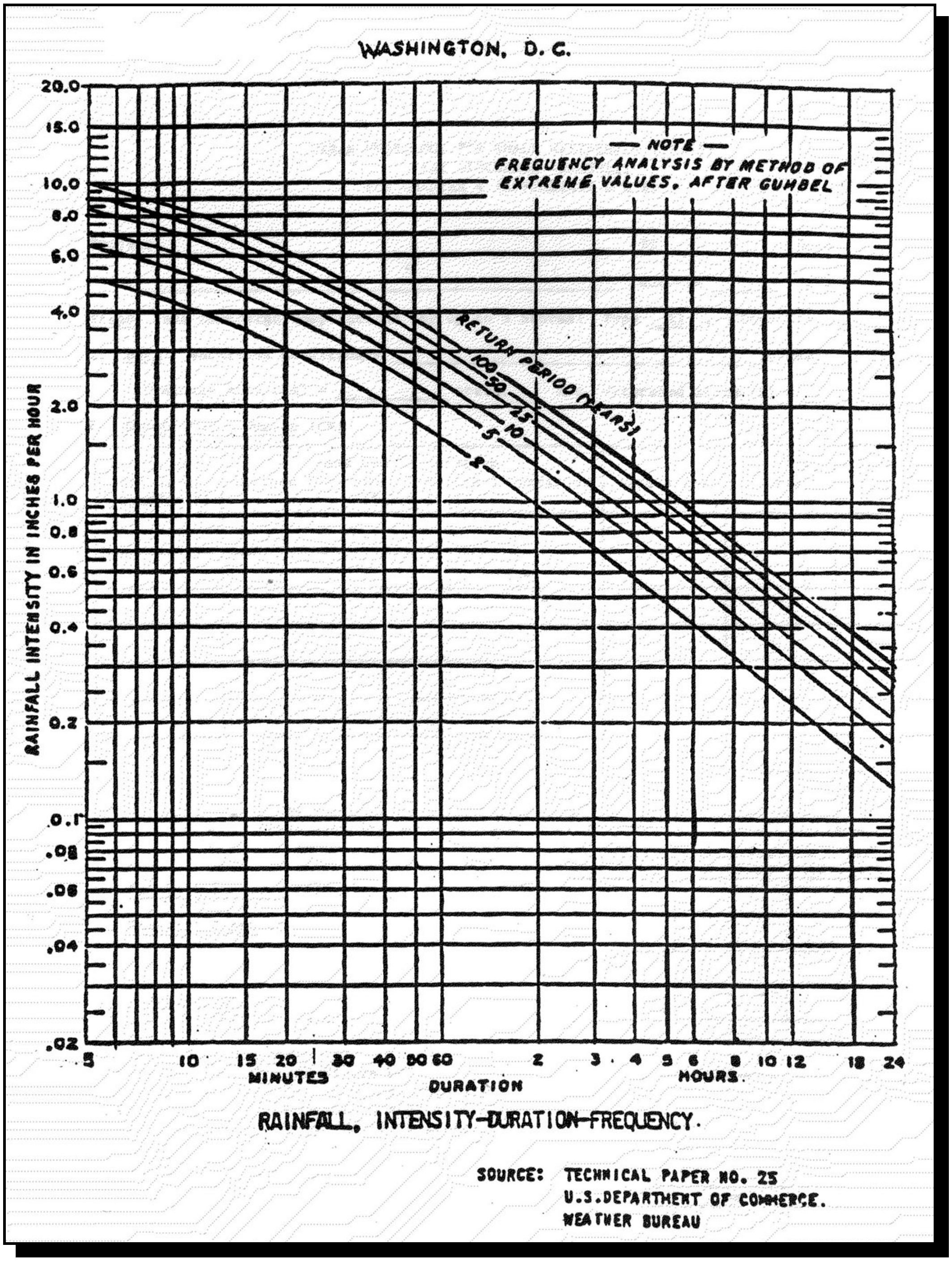


Figure A.1 District of Columbia Rainfall Intensity - Duration - Frequency Curve

Table A.2 Depth-Duration-Intensity-Frequency Rainfall Values for the District of Columbia

	1 Year		2 Year		5 Year		10 Year		15 Year		20 Year		25 Year		50 Year		100 Year	
Time (min)	**d**	**i**	**d**	**i**	**d**	**i**	**d**	**i**	**d**	**i**	**d**	**i**	**d**	**i**	**d**	**i**	**d**	**i**
5	0.38	4.60	0.44	5.28	0.54	6.42	0.61	7.34	0.63	7.56	0.64	7.63	0.66	7.93	0.72	8.61	0.74	8.89
10	0.65	3.88	0.74	4.44	0.89	5.34	1.02	6.11	1.05	6.30	1.06	6.36	1.10	6.64	1.20	7.20	1.25	7.90
15	0.83	3.32	0.96	3.83	1.15	4.59	1.32	5.27	1.36	5.44	1.38	5.50	1.43	5.72	1.54	6.14	1.61	6.42
20	0.97	2.91	1.12	3.36	1.35	4.04	1.55	4.65	1.61	4.82	1.63	4.88	1.69	5.08	1.78	5.35	1.86	5.59
30	1.13	2.26	1.35	2.70	1.64	3.27	1.90	3.79	1.97	3.95	2.02	4.03	2.10	4.19	2.24	4.49	2.32	4.65
45	1.29	1.72	1.57	2.09	1.93	2.57	2.25	3.01	2.37	3.16	2.44	3.25	2.54	3.38	2.83	3.77	2.96	3.94
60	1.38	1.38	1.70	1.70	2.13	2.13	2.51	2.51	2.66	2.66	2.76	2.76	2.87	2.87	3.22	3.22	3.39	3.39
80	1.49	1.12	1.81	1.36	2.33	1.75	2.77	2.08	2.96	2.22	3.10	2.32	3.22	2.42				
100	1.55	0.93	1.89	1.13	2.48	1.49	2.98	1.79	3.20	1.92	3.37	2.02	3.51	2.10				
120	1.58	0.79	1.94	0.97	2.60	1.30	3.14	1.57	3.40	1.70	3.61	1.80	3.75	1.87				
150	1.65	0.66	2.00	0.80	2.75	1.10	3.34	1.34	3.65	1.46	3.90	1.56	4.06	1.62				
180	1.68	0.56	2.04	0.68	2.86	0.95	3.50	1.17	3.87	1.29	4.16	1.39	4.32	1.44				
360			2.13	0.35	3.29	0.55	4.15	0.69	4.68	0.78	5.23	0.87	5.43	0.91				
720			2.16	0.18	3.73	0.31	4.84	0.40	5.64	0.47	6.49	0.54	6.75	0.56				
1440			2.17	0.09	4.18	0.17	5.62	0.23	6.79	0.28	8.04	0.34	8.35	0.35				

Notes: t = time in minutes
d = rainfall depth in inches
i = rainfall intensity in inches per hour
In general, i = (60 * d) / t

- Hydrologic Soil Groups approved for use in the District of Columbia are contained in the *Soil Survey of the District of Columbia Handbook*.

- On sites where substantial grading has occurred or will occur, or on fill sites, adjustments (see Table A.3) shall be made to the hydrologic soil group classifications.

- Schematic diagrams must be provided for all TR-20 and HEC-1 routings.

Table A.3 Soil Group Adjustment

Existing Soil	Adjusting Soil
A	B
B	C
C	D
D	D

Appendix

B

Design of Storm Water Conveyance Systems

B.1 Design of Storm Water Conveyance Systems

The Chezy-Manning formula is to be used to compute the system's transport capacities:

$$Q = \frac{1.486}{n} * A * R^{2/3} * S^{1/2}$$

Where: Q = channel flow (cfs)
n = Manning's roughness coefficient (Table B.1)
A = cross-sectional area of flow (ft^2)
R = hydraulic radius (ft)
S = channel slope (ft/ft)

Table B.1 Manning's Roughness Coefficient (n) Values for Various Channel Materials

Channel Materials	Roughness Coefficient
Concrete pipe and precast culverts	0.013
Monolithic concrete in boxes, channels	0.015
PVC pipes 24" to 36" 42" and larger	0.011 0.019 0.021
Sodded channel with water depth < 1.5'	0.050
Sodded channel with water depth <1.5'	0.035
Smooth earth channel or bottom of wide channels with sodded slopes	0.025
Rip-rap channels	0.035

Note: Where drainage systems are composed of more than one of the above channel materials, a composite roughness coefficient must be computed in proportion to the wetted perimeter of the different materials.

Also, the computation for the flow velocity of the channel shall use the continuity equation as follows:

$$Q = A * V$$

Where: V = velocity (ft/sec)
A = cross-sectional area of the flow (ft^2)

1) For Gutters:

With uniform cross slope and composite gutter section use the following equation:

$$Q = \frac{0.50}{n} * S_x^{1.67} * S^{0.5} * T^{2.67}$$

Where: Q = flow rate (cfs)
n = Manning's roughness coefficient (Table B.1)
S_x = cross slope (ft/ft)
S = longitudinal slope (ft/ft)
T = width of flow (spread) (ft)

2) For Inlets:

All inlets shall be sized to intercept a minimum of 70% of incoming flow.

3) Street Capacity (Spread):

Water shall not cross the centerline of the street or exceed the width or depth permitted by the District of Columbia Department of Public Works, Bureau of Transportation Construction Services, Design and Engineering Division.

4) Manhole and Inlet Energy Losses:

The following formulas shall be used to calculate headloss:

$$HL = \frac{V_{(outlet)}^2 - V_r^2}{2g} + SL$$

$$V_r = \frac{Q\,(V\cos\frac{a}{2})\ (\text{inlet } 1) + Q\,(V\cos\frac{a}{2})\ (\text{inlet } 2) + \ldots}{Q(\text{outlet})}$$

Where: HL = headloss in the structure

V_r = resultant velocity
$g = 32.2 ft/sec^2$ (gravitational acceleration)
SL = minimum structure loss
$a = (180^0$ - angle between the inlet & outlet pipes)

Table B.2 provides the minimum structure loss for inlets, manholes, and other inlet structures for use in the headloss calculation.

Table B.2 Minimum Structure Loss to Use in HGL Calculation

Velocity (ft/sec)*	Structure Loss (SL)
2	0.00
3	0.05
4	0.10
5	0.15
6	0.20
6	0.25
* Velocities leaving the structure.	

Headloss at the field connection is to be calculated like those structures eliminating the structure loss. For the angular loss coefficient, cos a/2 is assumed to be 1.

5) Open Channels:

- Calculations shall be provided for all channels, streams, ditches, swales and etc., including a typical section of each reach and a plan view with reach locations. In the case of existing natural streams/swales, a field survey of the stream (swale) cross sections may be required prior to the final approval.

- The final designed channel shall provide 6" minimum freeboard above the designated water surface profile of the channel.

- If the base flow exists for a long period of time or velocities are more than five feet per second in earth and sodded channel linings, gabion or rip-rap protection shall be provided at the intersection of the inverts and side slopes of the channels unless it can be demonstrated that the final bank and vegetation are sufficiently erosion-resistant to withstand the designed flows, and the channel will stay within the floodplain easement throughout the project life.

- Channel inverts and tops of bank are to be shown in plan and profile views.

- For a designed channel, a cross section view of each configuration shall be shown.

- For proposed channels, a final grading plan shall be provided.

- The limits of a recorded 100-year floodplain easement or surface water easement sufficient to convey the 100 year flow shall be shown.

- The minimum 25' horizontal clearance between a residential structure and 100 year floodplain shall be indicated in the plan.

- For designed channels, transition at the entrance and outfall is to be clearly shown on the site plan and profile views.

6) Pipe Systems:

- Individual storm water traps shall be installed on the storm drain branch serving each storm water management facility, or a single trap shall be installed in the main storm drain after it leaves the storm water management facility and before it connects with the city's combined sewer. Such traps shall be provided with an accessible cleanout. The traps shall not be required for storm drains which are connected to a separate storm sewer system.

- All pipes are to be made of reinforced concrete pipe (RCP) unless otherwise specified and approved by the District reviewing authority(s).

- The minimum pipe size to be used for any part of the public storm drainage system shall be 15" in diameter. The minimum pipe size to be used for any part of a private storm drainage system shall follow the current requirements of the District of Columbia Plumbing Code.

- The material and installation of the storm drain for any part of public storm sewer shall follow District of Columbia DPW Standard & Specifications, Section 02730.

- The minimum pipe size and material to be used for any part of private storm drain shall follow the current District of Columbia Plumbing Code.

- An alternative overflow path for the 100-year storm is to be shown on the plan view if the path is not directly over the pipe. Where applicable, proposed grading shall ensure that overflow will be into attenuation facilities designed to control the 100-year storm.

- A pipe schedule tabulating pipe lengths by diameter and class is to be included on the drawings. Public and private systems are to be separated.

- Profiles of the proposed storm drains shall indicate size, type, and class of pipe, percent grade, existing ground and proposed ground over the proposed system, and invert elevations at both ends of each pipe run. Pipe elevations and grades shall be set to avoid hydrostatic surcharge during design conditions. Where hydrostatic surcharge greater than one foot of head cannot be avoided, a rubber gasket pipe is to be specified.

7) Culverts:

- Culverts shall be built at the lowest point to pass the water across embankment of pond or highway. Inlet structure shall be designed to resist long term erosion and increased hydraulic capacities of culverts. Outlet structures shall be designed to protect outlets from future scouring. The following formulas are to be used in computing the culvert:

If the outlet is submerged then the culvert discharge is controlled by the tail water elevation:

$$h = h_e + h_f + h_v$$

Where: h = head required to pass given quantity of water through culvert flowing in outlet control with barrel flowing full throughout its length
h_e = entrance loss
h_f = friction loss
h_v = velocity head

And

$$h = k_e\left(\frac{V^2}{2g}\right) + \frac{n^2V^2L}{2.21R^{4/3}} + \frac{V^2}{2g}$$

$$h = \left[k_e + \left(\frac{n^2L}{2.21R^{4/3}}\right) * 2g + 1\right] * \left(\frac{V^2}{2g}\right)$$

$$h = \left[k_e + \left(\frac{n^2L}{2.21R^{4/3}}\right) * 2g + 1\right] * \left(\frac{8Q^2}{9.87gD^4}\right)$$

Where: k_e = entrance loss coefficient = 0.5 for a square edged entrance
k_e = entrance loss coefficient = 0.1 for a well rounded entrance
V = mean or average velocity in the culvert barrel (ft/sec)
g = 32.2ft/sec^2 (gravitational acceleration)
n = Manning's roughness coefficient = 0.012 for concrete pipe
L = length of culvert barrel (ft)
R = 0.25D = hydraulic radius (ft)
Q = flow (cfs)
D = diameter (ft)

- If the normal depth of the culvert is larger than the barrel height, the culvert will flow into a full or partially full pipe. The culvert discharge is controlled by the entrance conditions or entrance control.

$$Q = C_d A \left(2gh\right)^{0.5}$$

Where: Q = discharge (cfs)
C_d = discharge coefficient = 0.62 for square-edged entrance
C_d = discharge coefficient = 0.1 for well-rounded entrance
A = cross sectional area (ft^2)
g = 32.2ft/sec^2 (gravitational acceleration)
h = hydrostatic head above the center of the orifice (ft)

- If the hydrostatic head is less than 1.2D, the culvert will flow under no pressure as an open channel system.

- If the flows are submerged at both ends of the culvert, use Figure B.1.

8) Hydraulic Gradient:

- A hydraulic gradient shall be drawn in color on the system profiles. This gradient shall take into consideration pipe and channel friction losses, computing structures losses, tail water conditions and entrance losses. All pipe systems shall be designed so that they will operate without building up a surcharged hydrostatic head under design flow conditions. The HGL should be no more than 1 foot above the pipe crown. If pipes have a HGL more than 1 foot above the pipe crown, rubber gaskets shall be required.

- If the storm water management facility discharges into a storm sewer or a combined sewer system, a detailed hydraulic gradient analysis of the system including the receiving system must be submitted with the final storm water management plans for the 15 and 100-year flow frequencies. If the time characteristics of the hydraulic gradient are unknown, the designed storm water management facility shall be functional under expected minimum and maximum gradients.

9) Manholes and Inlets:

- District of Columbia DPW structures shall be used. All structures are to be numbered and listed in the structure schedule and shall include type, standard detail number, size, top elevation, slot elevation and locations, and modification notes.

- Access structures shall be spaced as follows:

15"-24" drain	400' max.
27"-42" drain	600' max.
Large than 42"	controlled by site conditions.

- A minimum drop of 0.1 foot shall be provided through the structure invert.

- Drainage boundary and contours are to be shown around each inlet to ensure that positive drainage to the proposed inlet is provided.

- Invert elevations of the pipes entering and leaving the structures are to be shown in the profile view.

- Yard or grate inlets shall show the 15-year and 100-year ponding limits (if applicable). A depth of not more than two feet is allowed from the throat or grate to the 100-year storm elevation.

- Public street inlets shall follow District of Columbia DPW criteria.

- Additional structures may be required on steep slopes to reduce excessive pipe depths and/or to provide deliberate drops in the main line to facilitate safe conveyance to a proper outfall discharge point. In order to provide an outfall at a suitable slope (i.e., less than 5% slope), drop structures may need to be used to reduce the velocity before discharging on a rip-rap area.

- Curb inlets located on private cul-de-sacs shall have a maximum 10 linear feet opening.

- Where two or more pipes enter a structure, a minimum of two feet horizontal clearance must be maintained between the pipes connected to the structure at the same elevation.

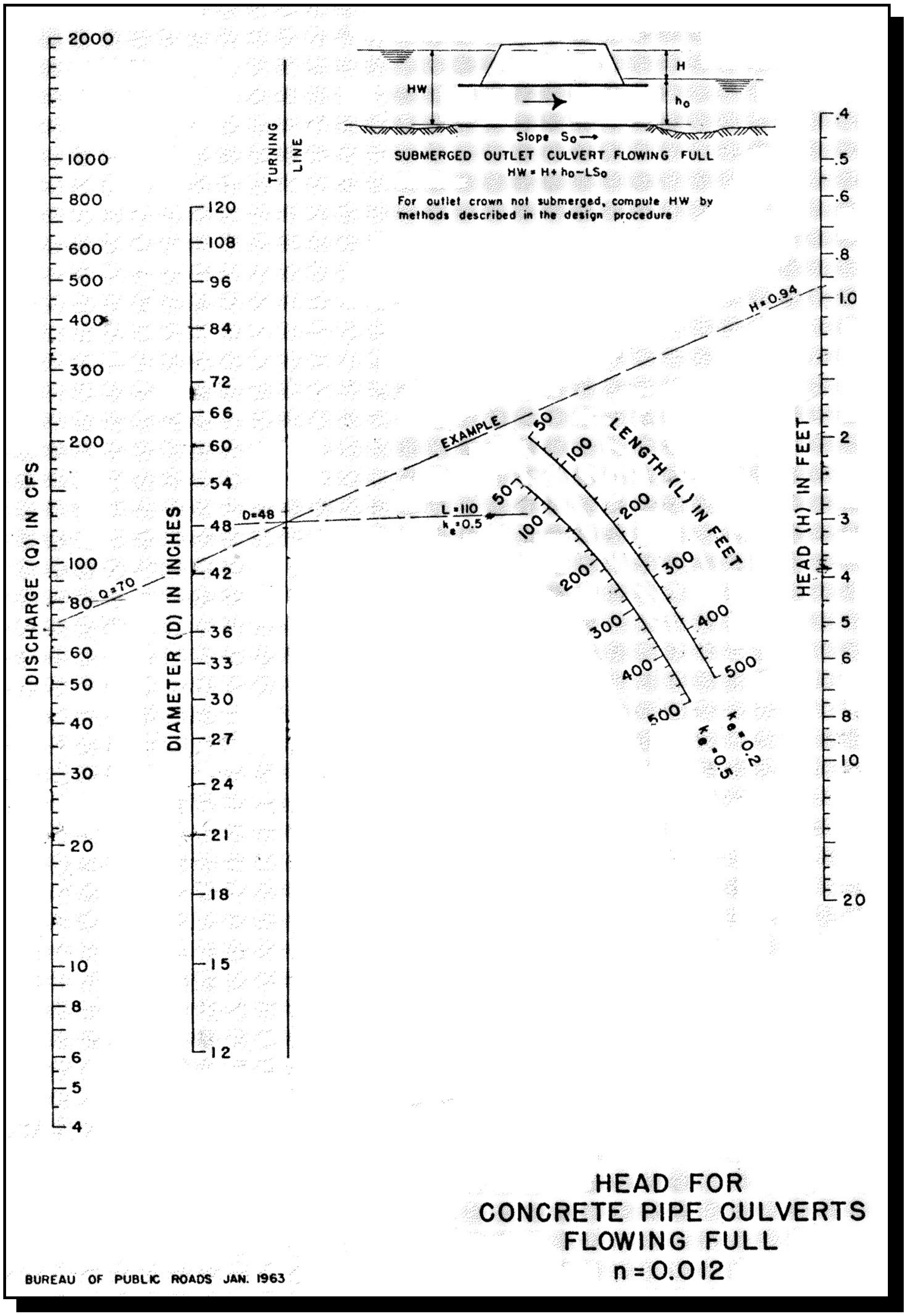


Figure B.1 Typical Nomograph for Culverts Under Outlet Control

- For commercial/industrial areas, inlets should be kept at least five feet away from the driveway aprons.

- The determination of the minimum width of a structure based on incoming pipes is based on the following formula:

$$W = \frac{D}{\sin\theta} + \frac{T}{\tan\theta}$$

Where:
D = pipe diameter (outside)
T = inlet wall thickness
W = minimum structure width (inside)
θ = angle of pipe entering structure

10) Clearance With Other Utilities:

- All proposed and existing utilities crossing or parallel to designed storm sewer systems shall be shown on the plan and profile.

- Storm drain and utility crossings shall not have be less than a 45-degree angle between them.

- A minimum vertical clearance of one foot and a minimum horizontal clearance of five feet, wall to wall, shall be provided between storm drainage lines and other utilities. Exceptions may be granted on a case-by-case basis when justified.

Appendix C

Design of Flow Control Structures

C.1 Design of Flow Control Structures

Flow control devices are orifices and weirs. The following formulas shall be used in computing maximum release rates from the designed storm water management facility

1) Circular Orifices:

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

Where: Q = orifice discharge (cfs)
C = discharge coefficient = 0.6
A = orifice cross-sectional area = $3.1416(D^2/4)$ (ft^2)
g = $32.2 ft/sec^2$ (gravitational acceleration)
h = hydraulic head above the center of the orifice (ft)

When h < D, the orifice shall be treated as a weir:

$$Q = CLH^{3/2}$$

Where: Q = flow through the weir (cfs)
C = 3
L = diameter of orifice (ft)
H = hydraulic head above bottom of weir opening (ft)

2) Flow Under Gates:
Flow under a vertical gate can be treated as a square orifice. For submerged conditions:

When outflow is not influenced by downstream water level:

$$Q = b * a * C * \left[2g * \left(\frac{H_o}{H_o + H_i} \right) \right]^{0.5}$$

Where: Q = flow through the gate (cfs)
b = width of gate (ft)
a = gate opening height (ft)
C = discharge coefficient
g = $32.2 ft/sec^2$ (gravitational acceleration)

When outflow is influenced by downstream water level:

$$Q' = KQ$$

Where K = coefficient found in Figure C.1

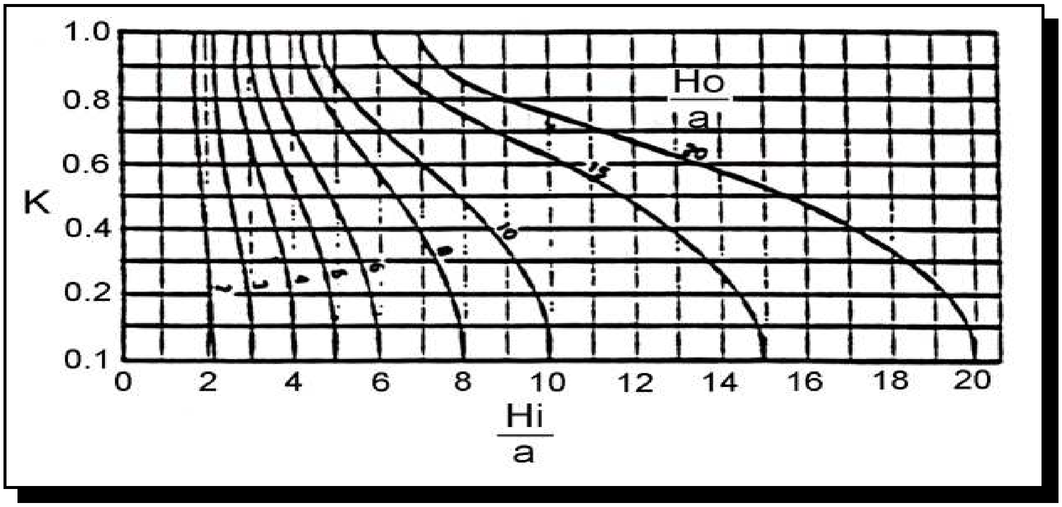


Figure C.1 Absolute Downstream Control of Flow Under Gate

3) Weirs:

Rectangular: $Q = 3.33H^{1.5}(L - 0.2H)$

60º V-notch: $Q = 1.43H^{2.5}$

90º V-notch $Q = 2.49H^{2.48}$

Where: Q = flow through the weir (cfs)
H = hydraulic head above the bottom of the weir (ft)
L = length of the weir crest (ft)

Appendix

D

Pollutant Load Calculations

D.1 Pollutant Load Calculations

For all development sites, the following calculations must be performed and certified by a professional engineer (civil or environmental engineer) licensed to practice in the District of Columbia.

1. Estimate the post-development pollutant export of total nitrogen (TN), total phosphorus (TP), and total suspended solids (TSS)
2. Estimate the annual TN, TP, and TSS loads which should be removed by the application of approved BMP(s).

All new development is required to provide these calculations by using the methods outlined below. The loading calculation sheets should be submitted at the 85% project design completion stage to be reviewed by the District of Columbia storm water management reviewers before the submission for final approval.

D.2 Estimating Post-Development Pollutant Export

The Simple Method is used for estimating pollutant export from new construction sites (Schueler, 1987). This method shall be used on development sites less than one square mile in area. For larger developments, the engineer is required to provide a more detailed analysis, based on the latest water quality models. The pollutant load is evaluated by the following equation:

$$L = P * P_j * R_v * C * A * 0.226$$

Where:
L = annual pollutant load from site (lbs/year)
P = average rainfall depth (use 40 inches/year)
P_j = factor that corrects P for storms that produce no runoff (use 0.9)
C = flow weighted mean concentration of pollutant, see Tables D.1 & D.2 (mg/l)
A = area of development sites (acres)
R_v = runoff coefficient, which expresses the fraction of rainfall converted into runoff
= 0.05 + 0.009 * (percent of site imperviousness)
e.g. For 20% imperviousness, use 20, not 0.20
0.226 = unit conversion factor.

Table D.1 Concentration Values (C) for Selected Levels of Impervious Cover for Use in Estimating Pollutant Loads from New or Redevelopment Sites in the District of Columbia

Land Use	Site Imperviousness (%)	TP (mg/l)	TN (mg/l)	BOD (mg/l)	Pb (mg/l)	Zn (mg/l)
Rural Residential	0	0.11	0.8	2.1	0.02	0.01
	5	0.20	1.6	4.0	0.03	0.01
	10	0.30	2.3	5.8	0.04	0.02
	15	0.39	3.0	7.7	0.06	0.03
	20	0.49	3.8	9.6	0.07	0.04
	25	0.58	4.5	11.4	0.08	0.05
	30	0.68	5.2	13.3	0.10	0.05
	35	0.77	6.0	15.2	0.11	0.06
Townhouse, Garden Apartment	40	0.87	6.7	17.1	0.12	0.07
	45	0.97	7.4	18.9	0.14	0.07
	50	1.06	8.2	20.8	0.15	0.08
	55	1.16	8.4	22.7	0.16	0.09
	60	1.25	9.6	24.6	0.18	0.09
High Rise, Light Commercial, Industrial	65	1.35	10.4	26.4	0.19	0.10
	70	1.44	11.1	28.3	0.21	0.10
	75	1.54	11.8	30.2	0.22	0.11
	80	1.63	12.6	32.0	0.23	0.11
Heavy Commercial, Downtown Shopping Center	85	1.73	13.3	33.9	0.25	0.12
	90	1.82	14.0	35.8	0.26	0.13
	95	1.92	14.8	37.7	0.27	0.13
	100	2.00	15.4	39.2	0.28	0.14

Table D.2 Concentration Values (C) of Sediment for Selected Levels of Impervious Cover for Use in Estimating Pollutant Loads from New or Redevelopment Sites in the District of Columbia

Land Use	Site Imperviousness (%)	Clay Loam (mg/l)	Silt Loam (mg/l)	Loam (mg/l)	Sandy Loam (mg/l)
Residential	20%	0.16	0.16	0.15	0.12
	25%	0.19	0.18	0.17	0.15
	35%	0.23	0.23	0.22	0.22
	40%	0.29	0.29	0.27	0.21
	50%	0.33	0.33	0.31	0.28
	60%	0.18	0.18	0.18	0.16
	75%	0.29	0.29	0.29	0.28
Central Business District	95%	0.25	0.25	0.25	0.25
	90%	0.24	0.24	0.24	0.24
Industrial	60%	0.18	0.18	0.18	0.18
	80%	0.22	0.22	0.22	0.22
Idle Land	1%	0.07	0.06	0.05	0.01

D.3 Estimating Annual Pollutant Removal Based on BMP Efficiency

This section provides a standard method of estimating annual pollutant loads which should be removed by the application of approved BMP(s). This procedure should only apply to the calculation of a post-development condition. This technique only provides for the general planning estimate of likely BMP(s) installed at sites less than 50 acres. More sophisticated methods may be needed to analyze larger and more complex developments.

To estimate the annual TN, TP, and TSS loads which should be removed by the application of approved BMP(s) use the following equation:

$$T_r = L * \%BMP_{RE}$$

Where: T_r = total annual pollutant removal (lbs/yr)
L = annual pollutant load from site (lbs/year) (see previous section)
$\%BMP_{RE}$ = BMP removal efficiency (see Table D.3)

Table D.3 Post-Construction BMP Effectiveness Summary

Best Management Practice (BMP)	Median % Removal					Main Removal Efficiency Factors
	TSS	TP	TN	Cu	Zn	
Filtering Systems						
Surface Sand Filter	87	59	32	49	80	■ Treatment volume ■ Filter media ■ Sediment storage volume ■ Depth of filter
One-Chamber Underground Sand Filter	ND	ND	ND	ND	ND	
Three-Chamber Underground Sand Filter	ND	ND	ND	ND	ND	
Perimeter Sand Filter[1]	79	41	47	25	69	
Vertical Sand Filter[1]	58	45	5	32	56	
Organic Filter	88	61	41[1]	66[1]	89	
Bioretention[1]	ND	65	49	97	95	
Roof Downspout System	ND	ND	ND	ND	ND	
Infiltration Practices						
Infiltration Trench[1]	ND	100	42	ND	ND	■ Percolation ■ Basin surface area ■ Storage volume
Infiltration Basin	ND	ND	ND	ND	ND	■ Percolation ■ Basin surface area ■ Storage volume
Storm Water Ponds						
Micropool Extended Detention Pond	ND	ND	ND	ND	ND	■ Pool volume ■ Pond shape ■ Detention time
Wet Pond	79	49	32	58	65	
Wet Extended Detention Pond	80	55	35	44	69	
Pocket Pond[2]	87	78	28[1]	55	65	

Table D.3 Post-Construction BMP Effectiveness Summary

Best Management Practice (BMP)	Median % Removal					Main Removal Efficiency Factors
	TSS	TP	TN	Cu	Zn	
Storm Water Wetlands						
Shallow Wetland	83	43	26	33	42	■ Storage volume ■ Detention time ■ Pool shape ■ Wetland biota ■ Seasonal variation
Extended Detention Shallow Wetland[1]	69	39	56	ND	-74	
Pocket Wetland[2]	57[1]	57[1]	44[1]	25[1]	52[1]	
Open Channels						
Dry Swale[1]	93	83	92	70	86	■ Runoff volume ■ Infiltration rates ■ Slope, length ■ Roughness ■ Geometry
Wet Swale[1]	74	28	40	11	33	
Grass Channel[1]	68	29	ND	42	45	

1. Data based on fewer than five data points
2. Drainage area < 10 acres

NOTES:
- ND indicates that the data is not available.
- Micropool ED Ponds are presumed to have removal rates similar to the Wet ED Pond. While this practice has not been monitored the pollutant removal mechanisms are similar.
- Infiltration practices are difficult to monitor, but are presumed to have high removal rates based on filtration processes of the soil and pollutant land application studies.
- One-Chamber Underground Sand Filter, Three-Chamber Underground Sand Filter, Roof Downspout System are presumed to have similar removal to other filtering practices.
- TSS = Total Suspended Solids; TP = Total Phosphorus; TN = Total Nitrogen; Cu = Copper; Zn = Zinc

Source: Winer, 2000

Appendix

E

Geotechnical Information Requirements for Underground BMPs

E.1 General Notes Pertinent to All Geotechnical Testing

A geotechnical report is required for all underground BMPs, including infiltration practices, filtering systems, and storage practices. The following must be taken into account when producing this report:

- Soil boring information is to be obtained from at least one boring at the center of the proposed structure location.
- Minimum boring depth is to equal to the depth of the structure plus four feet.
- Laboratory testing should include permeability, grain size, liquid limit, plastic limit, nature moisture, standard penetration test, consolidation and shear tests.
- The geotechnical report should include seepage, uplift and settlement analysis based on consolidation tests of saturated foundation soils.
- Soil boring should be in the Unified Soil Classification System. If an underground water table is encountered, it should be indicated in the boring logs.
- For infiltration trench (I-1) and basin (I-2) practices, a minimum field infiltration rate (f_c) of 0.52 inches per hour is required; lower rates preclude the use of these practices. For surface sand filter (F-1) and bioretention (F-7) practices, no minimum infiltration rate is required if these facilities are designed with a “day-lighting” underdrain system; otherwise these facilities require a 0.52 inch per hour rate.
- Number of required borings is based on the size of the proposed facility. Testing is done in two phases, (1) Initial Feasibility, and (2) Concept Design.
- Testing is to be conducted by a qualified professional. This professional shall either be a registered professional engineer, soils scientist or geologist and must be licensed in the District of Columbia.

Specific requirements for infiltration practices and filtering systems are discussed below.

E.2 Initial Feasibility Testing

Feasibility testing is conducted to determine whether full-scale testing is necessary, screen unsuitable sites, and reduce testing costs. A soil boring is not required at this stage. However, a designer or landowner may opt to engage Concept Design Borings per Table E.1 at his or her discretion, without feasibility testing.

Initial testing involves either one field test per facility, regardless of type or size, or previous testing data, such as the following:

- on-site septic percolation testing, within 200 feet of the proposed BMP location, and on the same contour which can establish initial rate, water table and/or depth to bedrock,
- geotechnical report on the site prepared by a qualified geotechnical consultant, or
- Natural Resources Conservation Service (NRCS) Soil Mapping showing an unsuitable soil group such as a hydrologic group "D" soil in a low-lying area or a Marlboro Clay.

If the results of initial feasibility testing as determined by a qualified professional show that an infiltration rate of greater than 0.52 inches per hour is probable, then the number of concept design test pits shall be per Table E.1. An encased soil boring may be substituted for a test pit, if desired.

Table E.1 Infiltration Testing Summary Table

Type of Facility	Initial Feasibility Testing	Conceptual Design Testing (initial testing yields a rate greater than 0.52"/hr)	Conceptual Design Testing (initial testing yields a rate lower than 0.52"/hr)
Infiltration Trench	1 field percolation test, test pit not required	1 infiltration test and 1 test pit per 50 ft of trench	not acceptable practice
Infiltration Basin	1 field percolation test, test pit not required	1 infiltration test and 1 test pit per 200 ft^2 of basin area	not acceptable practice
Surface Sand Filter	1 field percolation test, test pit not required	1 infiltration test and 1 test pit per 200 ft^2 of filter area (no underdrains required*)	underdrains required
Bioretention	1 field percolation test, test pit not required	1 infiltration test and 1 test pit per 200 ft^2 of filter area (no underdrains required*)	underdrains required

* underdrain installation is still strongly recommended

E.3 Documentation

Infiltration testing data shall be documented, and include a description of the infiltration testing method. This is to ensure that the tester understands the procedure.

E.4 Test Pit/Boring Requirements

a. Excavate a test pit or dig a standard soil boring to a depth of 4 feet below the proposed facility bottom;

b. Determine depth to groundwater table (if within 4 feet of proposed bottom) upon initial digging or drilling, and again 24 hours later;

c. Conduct Standard Penetration Testing (SPT) every 2 feet to a depth of 4 feet below the facility bottom;

d. Determine Unified Soil Classification (USC) System textures at the proposed bottom and 4 feet below the bottom of the best management practice (BMP);

e. Determine depth to bedrock (if within 4 feet of proposed bottom);

f. The soil description should include all soil horizons; and

g. The location of the test pit or boring shall correspond to the BMP location; test pit/soil boring stakes are to be left in the field for inspection purposes and shall be clearly labeled as such.

E.5 Infiltration Testing Requirements (field testing required)

a. Install casing (solid 5" diameter, 30" length) to 24" below proposed BMP bottom (see Figure E.1).

b. Remove any smeared soiled surfaces and provide a natural soil interface into which water may percolate. Remove all loose material from the casing. Upon the tester's discretion, a 2" layer of coarse sand or fine gravel may be placed to protect the bottom from scouring and sediment. Fill casing with clean water to a depth of 24" and allow to pre-soak for twenty-four hours.

c. Twenty-four hours later, refill casing with another 24" of clean water and monitor water level (measured drop from the top of the casing) for 1 hour. Repeat this procedure (filling the casing each time) three additional times, for a total of four observations. Upon the tester's discretion, the final field rate may either be the average of the four observations, or the value of the last observation. The final rate shall be reported in inches per hour.

d. May be done through a boring or open excavation.

e. The location of the test shall correspond to the BMP location.

f. Upon completion of the testing, the casings shall be immediately pulled, and the test pit shall be back-filled.

E.6 Laboratory Testing

Use grain-size sieve analysis and hydrometer tests (where appropriate) to determine USDA soils classification and textural analysis. Visual field inspection by a qualified professional may also be used, provided it is documented. The use of lab testing to establish infiltration rates is prohibited.

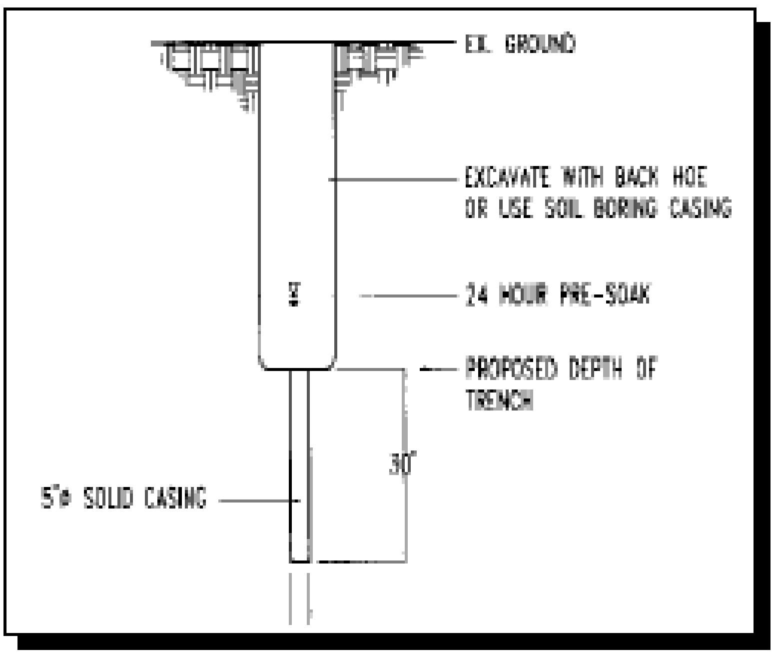


Figure E.1 Infiltration Testing Requirements

E.7 Additional Infiltration Practice Requirements:

- Soils shall have a clay content of less than 20% and a silt/clay content of less than 40%.

- Infiltration shall not be located on slopes greater than 6% or within fill soils.

- The soil borings must indicate the depth to the seasonally high water table and bedrock, if any. The minimum distance acceptable between the bottom of the trench and the seasonally high water table and bedrock is 2 feet.

- Infiltration systems designed to handle runoff from commercial or industrial impervious parking areas shall be a minimum of 100 feet from any water supply well.

- Soil boring location stakes shall be left in the field for inspection purposes.

E.8 Additional Filtering System Requirements:

- The seasonally high groundwater table and bedrock shall be located at least 2 below the footing of the structure.

E.9 Additional Bioretention Requirements:

All areas tested for application of F-7 facilities shall be back-filled with a suitable sandy loam planting media. The borrow source of this media, which may be the same or different from the bioretention area location itself, must be tested as follows.

If the borrow area is undisturbed soil one test is required per 200 square feet of borrow area. The test consists of "grab" samples at one foot depth intervals to the bottom of the borrow area. All samples at the testing location are then mixed, and the resulting sample is then lab-tested to meet the following criteria:

a. USDA minimum textural analysis requirements: A textural analysis is required from the site stockpiled topsoil. If topsoil is imported, then a texture analysis shall be performed for each location where the topsoil was excavated.

   Minimum requirements:
      sand 35 - 60%
      silt 30 - 55%
      clay 10 - 25%

b. The soil shall be a uniform mix, free of stones, stumps, roots or other similar objects larger than 1".

Appendix

F

Additional Design and Construction Requirements for Sand Filter Systems

F.1 Design Requirements

In addition to the requirements detailed in Chapter 3, the following information should be provided for underground sand filter design:

- Delineate the drainage area to show the downstream storm sewer system or stream and the extent of the underground facility.
- Show 100-year ponding and/or safe overflow pathways.
- Provide structural details of the underground detention system.
- Provide a profile of entire system with inverts, pipe size, pipe type, slopes, and hydraulic grade line (HGL) through the facility.
- Provide cross section(s) and a plan view.
- Provide water tight joints at all pipe connections.
- The underground detention structure should be composed of reinforced concrete. Other materials may be used for storm water management detention when the reviewing authority has approved their application.
- All structural information for non-standard structures or modified structures along with H-20 loading information must be provided for approvals.
- Anti-flotation analysis is required to check for buoyancy problems in high water table areas.
- Anchors should be designed to counter the pipe and structure buoyancy by at least a 1.2 factor of safety.

F.2 Design Details and Specifications

This section discusses design details and specifications for:

- vertical sand filters
- surface sand filters - sedimentation basin
- surface sand filters - sand filtration basins

Vertical Sand Filter - Filter Layer

Figure F.1 is a typical cross-section of the filter chamber.

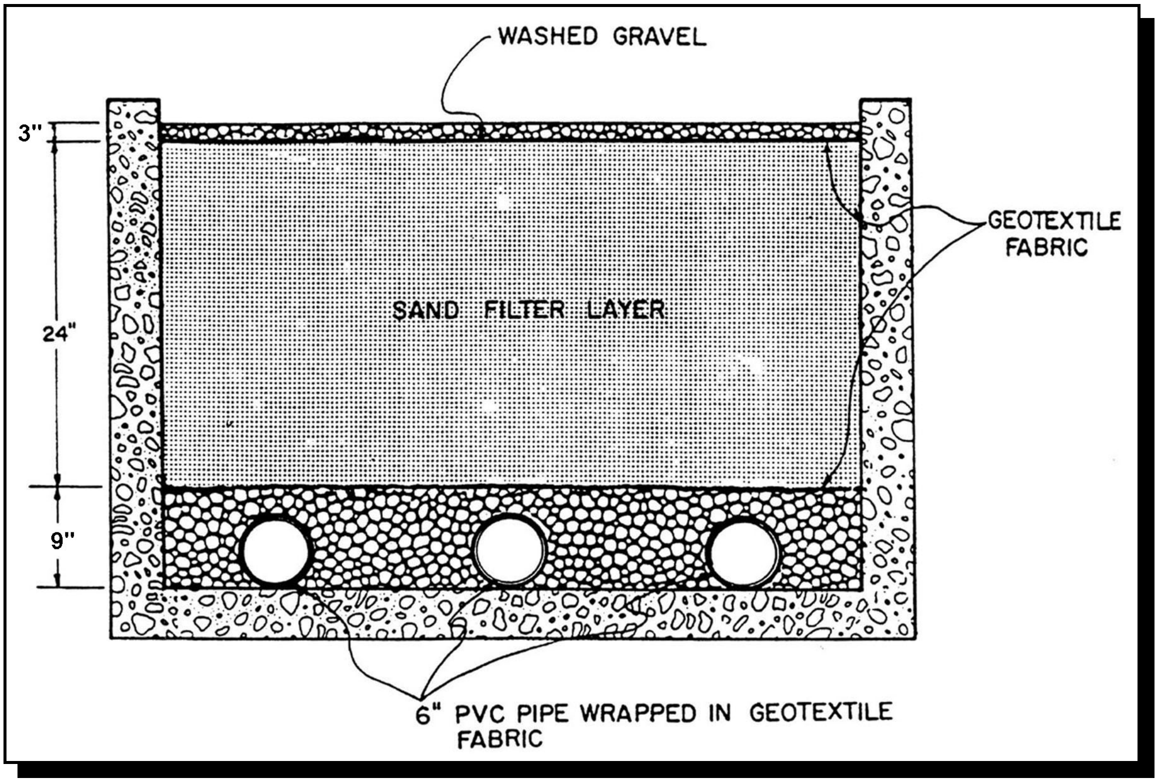


Figure F.1 Typical Cross-Section of the Filter Chamber (Source: City of Austin, 1988)

Upper Filter Layer
The washed gravel or aggregate layer at the top of the filter may be 1" to 3" thick and should be D.C. #57 gravel.

Geotextile Fabric
The geotextile fabric below the top gravel layer and below the sand should be Woven Monofilament Geotextile 104F with the specifications listed in Table F.1.

Table F.1 Specifications for the Woven Monofilament Geotextile 104F

Property	Test Method	Units	Value	104F
Mechanical				
Grab Tensile MD/XD	ASTM D-4632	lbs	Typical	400/275
			MARV	370/250
Grab Elongation MD/XD	ASTM D-4632	%	Typical	26/26
			MARV	24/24
Puncture Strength	ASTM D-4833	lbs	Typical	150
			MARV	120
Mullen Burst	ASTM D-3786	psi	Typical	520
			MARV	480
Trapezoidal Tear MD/XD	ASTM D-4533	lbs	Typical	120/85
			MARV	100/70
Hydraulic				
Percent Open Area (POA)	$\frac{\text{Opening Area}}{\text{Total Area}} * 100$	%	Typical	5
			MARV	4
Apparent Opening Size (AOS)	ASTM D-4751	US Sieve	Typical	70-100
			MARV	70
Permittivity	ASTM D-4491	sec	Typical	0.40
			MARV	0.28
Permeability	ASTM D-4491	cm/sec	Typical	0.015
			MARV	0.010
Water Flow Rate	ASTM D-4491	gpm/ft^2	Typical	26
			MARV	18
Physical				
Weight	ASTM D-5261	oz/yd^2	Typical	6.3
			MARV	5.9
Thickness	ASTM D-5199	mils	Typical	15
			MARV	13
Endurance				
UV Resistance	ASTM D-4355	% Retained @ 500 hours	MARV	90
Packaging				
Roll Width	Measured	in	Typical	72/144
Roll Length	Measured	in	Typical	300
Roll Weight	Calculated	lbs	Typical	108/192
Area	Calculated	yd^2	Typical	200/400

Notes:
- "MARV" indicated minimum average roll value calculated as the typical minus two standard deviations. Statistically, it yields a 97.7% degree of confidence that any sample taken during quality assurance testing will exceed the value reported.
- "MD" indicates the machine/warp/roll direction.
- "XD" indicates the cross-machine/fill/across the roll direction.

The fabric roll should be cut with sufficient dimensions to cover the entire wetted perimeter of the filter area with a 6" minimum overlap.

Sand Filter Layer
The sand filter layer should be 18" to 24" deep. ASTM C33 concrete sand is recommended, but sand with similar specifications may be used.

Bottom Gravel Layer
The bottom gravel layer surrounding the collector (perforated) pipes should be 0.5" to 2" diameter gravel and provide at least 3" of cover over the top of the drainage pipes. No gravel is required under the pipes. The gravel and the sand layer above must be separated by a layer of geotextile fabric that meets the specifications listed in Table F.1.

Underdrain Piping
The underdrain piping consists of three 6" perforated pipes, an appropriate number of 4" pipes, and should be reinforced to withstand the load of the overburden. Perforations should be 3/8". All piping should be to schedule 40 polyvinyl chloride or greater strength.

The minimum grade of piping should be 1/8 inch per foot or 1% of the slope. Access should be provided for cleaning all underdrain piping. Clean-outs for each pipe shall extend to the maximum surface elevation of the structure and be flush with the top slab of the structure.

Each pipe should be carefully wrapped with geotextile fabric that meets the stipulated specifications before placement in the filter.

Surface Sand Filter - Sedimentation Basin

The sedimentation basin consists of an inlet structure, outlet structure, and basin liner. The sedimentation basin design should maximize the distance from where the heavier sediment is deposited near the inlet to where the outlet structure is located. This will improve basin performance and reduce future maintenance requirements.

Inlet Structure
The inlet structure should be designed to capture the first flush and convey the peak flow of a 15-year storm through the basin. The water quality shall be discharged uniformly as sheet flow into the sedimentation basin in order to achieve the relatively quiescent state. Sediments having higher specific gravity will tend to settle down near the inlet structure. For this reason, the drop inlet is recommended in order to facilitate future sediment removal and maintenance (Figure F.2).

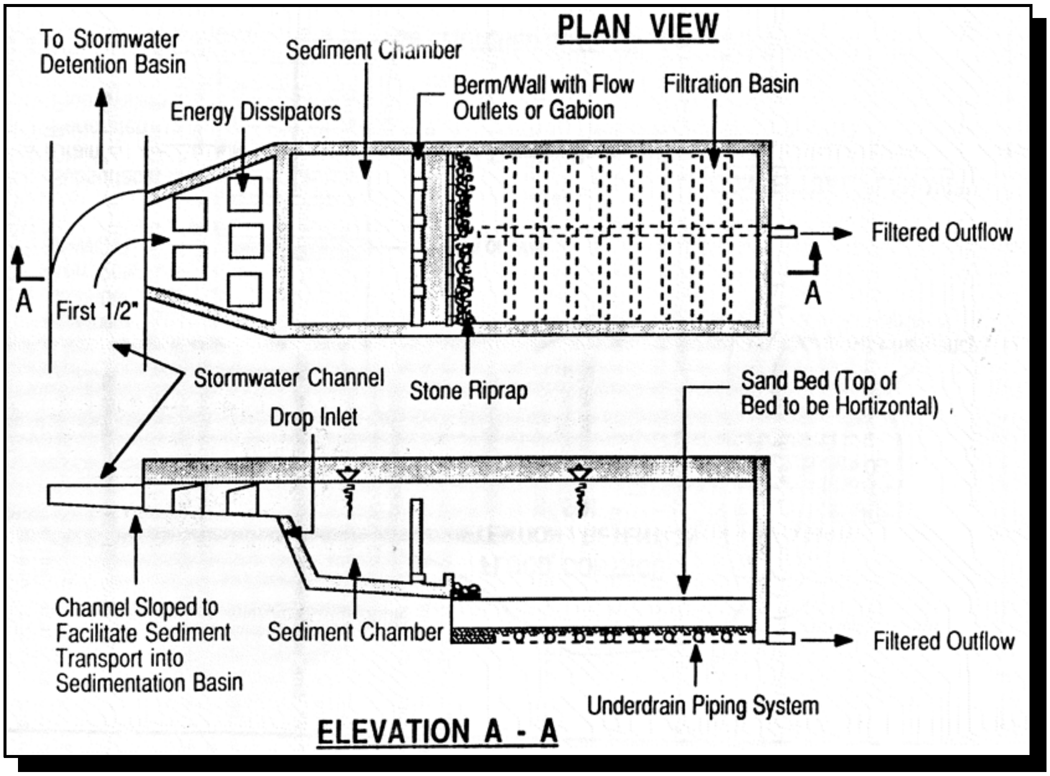


Figure F.2 Conceptual Partial Sedimentation Filtration System

<u>Outlet Structure</u>

The outlet structure conveys the water from the sedimentation basin to the filtration basin. The outlet structure should be designed to provide for a minimum dewatering time of 24 hours for full-sediment basin design or discharge flow evenly to the filtration basin for partial sediment basin design. A perforated pipe or equivalent is the recommended outlet structure. The 24 hours dewatering time should be achieved by installing a throttle plate or other flow control device at the end of the riser pipe. The perforated riser pipe should be selected from Table F.2.

Table F.2 Selection of Perforated Riser Pipes

Riser Pipe Nominal Diameter (inches)	Vertical Spacing Between Rows (inches)	Number of Perforations Per Row	Diameter of Perforations (inches)
6	2.5	9	1
8	2.5	12	1
10	2.5	16	1

Source: City of Austin

For the partial sedimentation basin design, the outlet structure should be a berm or wall with multiple outlet ports or gabion so as to discharge the flow evenly to the filtration basin. Rock gabions should be constructed using 6" to 12 " diameter rocks. The berm/wall/gabion height should not exceed 6 feet and high flow should be allowed to overtop the structure (weir flow). Outlet ports should not be located along the vertical center axis of the berm/wall so as to induce flow-spreading. The outflow side should incorporate features to prevent scouring of the sand bed (e.g., concrete, splash pad or rip rap) (Figure F.3).

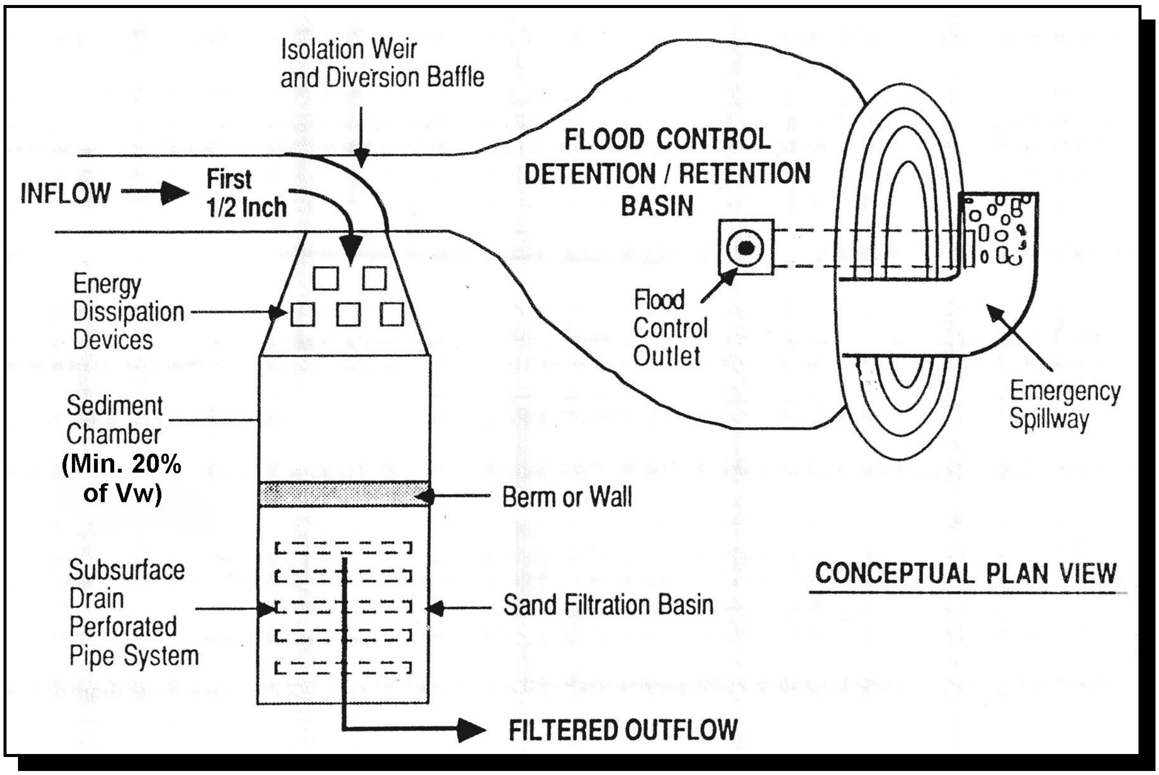


Figure F.3 Conceptual Partial Sedimentation with Filtration System
Source: City of Austin, 1988

A trash rack should be provided for the outlet. Openings in the rack should not exceed 1/3 the diameter of the vertical riser pipe. The rack should be made of durable material, resistant to rust and ultraviolet rays. For the bottom rows of perforations, it is recommended that geotextile fabric be wrapped over the pipe's bottom rows and that a layer of 1" to 2" of gravel be placed around the pipe.

Basin Liner

Impermeable liners may be either clay, concrete or geomembrane. If geomembrane is used, suitable geotextile fabric should be placed below and on the top of the geomembrane for puncture protection. Clay liners should meet the specifications outlined in Table F.3.

Table F.3 Clay Liner Specifications

Property	Test Method	Unit	Specification
Permeability	ASTM D-2434	cm/sec	1 x 10^{-6}
Plasticity Index of Clay	ASTM D-423 & D-424	cm/sec	Not < 15
Liquid Limit of Clay	ASTM D-2216	cm/sec	Not < 30
Clay Part. Passing	ASTM D-422	cm/sec	Not < 30
Clay Compaction	ASTM D-2216	cm/sec	95% of Standard Proctor Density

Source: City of Austin

The clay liner should have a minimum thickness of 12".

If a geomembrane liner is used it should have a minimum thickness of 30 mils and be ultraviolet resistant. The geotextile fabric (for protection of geomembrane) should meet specifications outlined in Table F.1.

Basin Geometry

The shape of the sedimentation basin and the flow regime within this basin will influence how effectively the basin volume is utilized in the sedimentation process. 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 in order to maximize particle settling opportunities.

Short circuiting (i.e., flow reaching the outlet structure before it passes through the sedimentation basin volume) flow should be avoided. Dead storage areas (areas within the basin which are bypassed by the flow regime and are, therefore, ineffective in the settling process) should be minimized (Figure F.4). Baffles may be used to mitigate short circuiting and/or dead storage problems (Figures F.5).

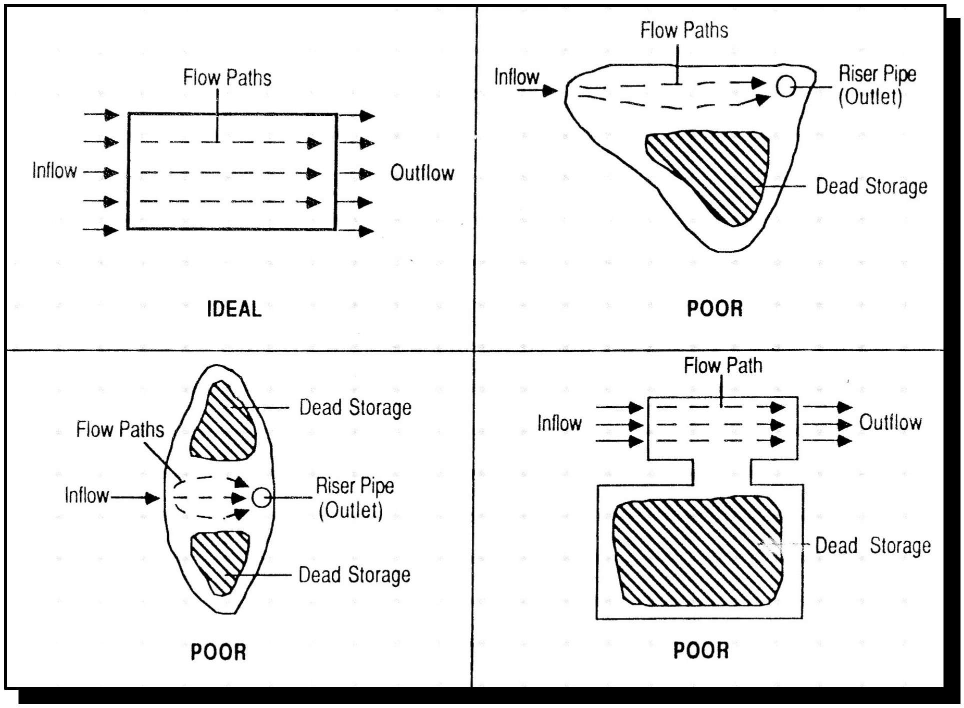


Figure F.4 Sedimentation Basin Configurations
Source: City of Austin, 1988

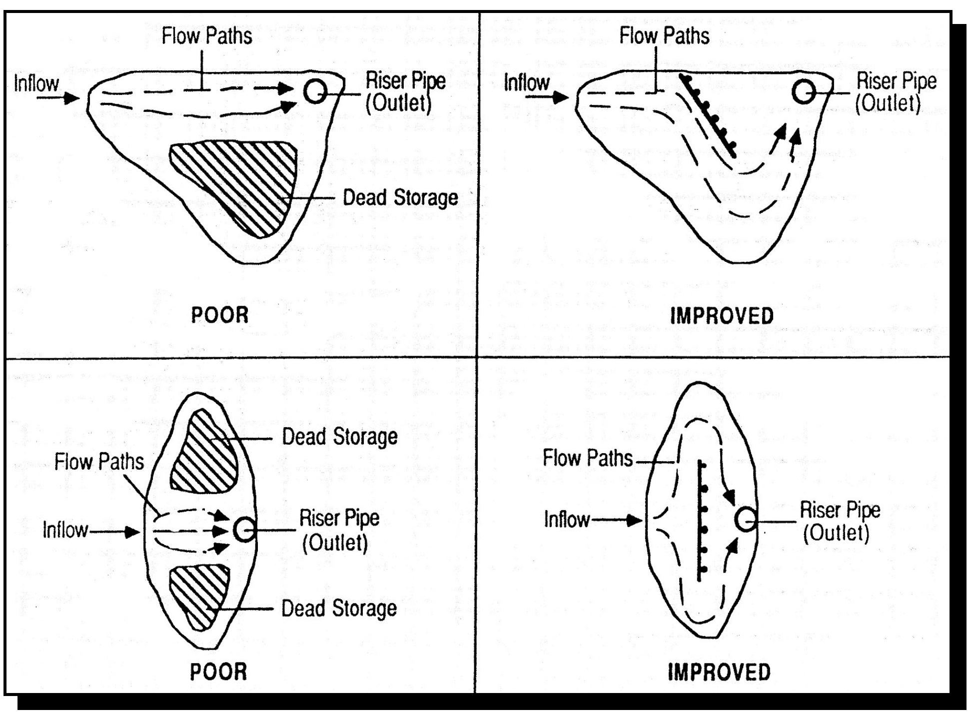


Figure F.5 Sedimentation Basin Baffles
Source: City of Austin, 1988

Sediment Trap (Optional)
A sediment trap is a storage area which captures sediment and removes it from the basin flow regime. In so doing the sediment trap inhibits resuspension of solids during subsequent runoff events, improving long-term removal efficiency. The trap also maintains adequate volume to hold the water quality volume which would otherwise be partially lost due to sediment build-up. Sediment traps may reduce maintenance requirements by reducing the frequency of sediment removal. It is recommended that the sediment trap volume be equal to 10% of the sedimentation basin volume.

Water collected in the sediment trap should be conveyed to the filtration basin in order to prevent standing water conditions from occurring. All water collected in the sediment trap should drain out within 60 hours. The invert of the drain pipe should be above the surface of the sand bed filtration basin. The minimum grading of the piping to the filtration basin should be 0.25 inch per foot (2% slope). Access for clearing the sediment trap drain system is necessary. Figure F.6 illustrates the sediment trap details.

Surface Sand Filters – Sand Filtration Basin

The sand bed filtration basin consists of the inlet structure, sand bed, underdrain piping and basin liner.

Inlet Structure
The inlet structure should spread the flow uniformly across the surface of the filter media. Flow spreaders, weir or multiple orifice openings are recommended (Figure F.7).

Sand Bed
The sand bed may be a choice of one of the following four configurations:

1. Sand Bed with Full Bottom Gravel Layer (Figure F.8) - The top layer is to be a minimum of 18" of ASTM C-33 Concrete Sand; smaller sand size is acceptable. Under the sand should be at least 9" of 0.5" to 2" diameter gravel and a subsurface drainage system. The sand and gravel must be separated by a layer of geotextile fabric that meets the specification list in Table F.1.

2. Sand Bed - Trench Design (Figure F.8) - The top layer should be 12" to 18" of ASTM C-33 Concrete Sand. Laterals should be placed in trenches with a covering of 0.5" to 2" gravel and geotextile fabric. The laterals should be underlain by a layer of drainage matting. The geotextile fabric is needed to prevent the filter media from infiltrating into the lateral piping. The drainage matting is needed to provide for adequate vertical and horizontal hydraulic conductivity to the laterals, and should meet the specifications in Table F.4. The geotextile fabric should meet the specifications in Table F.1.

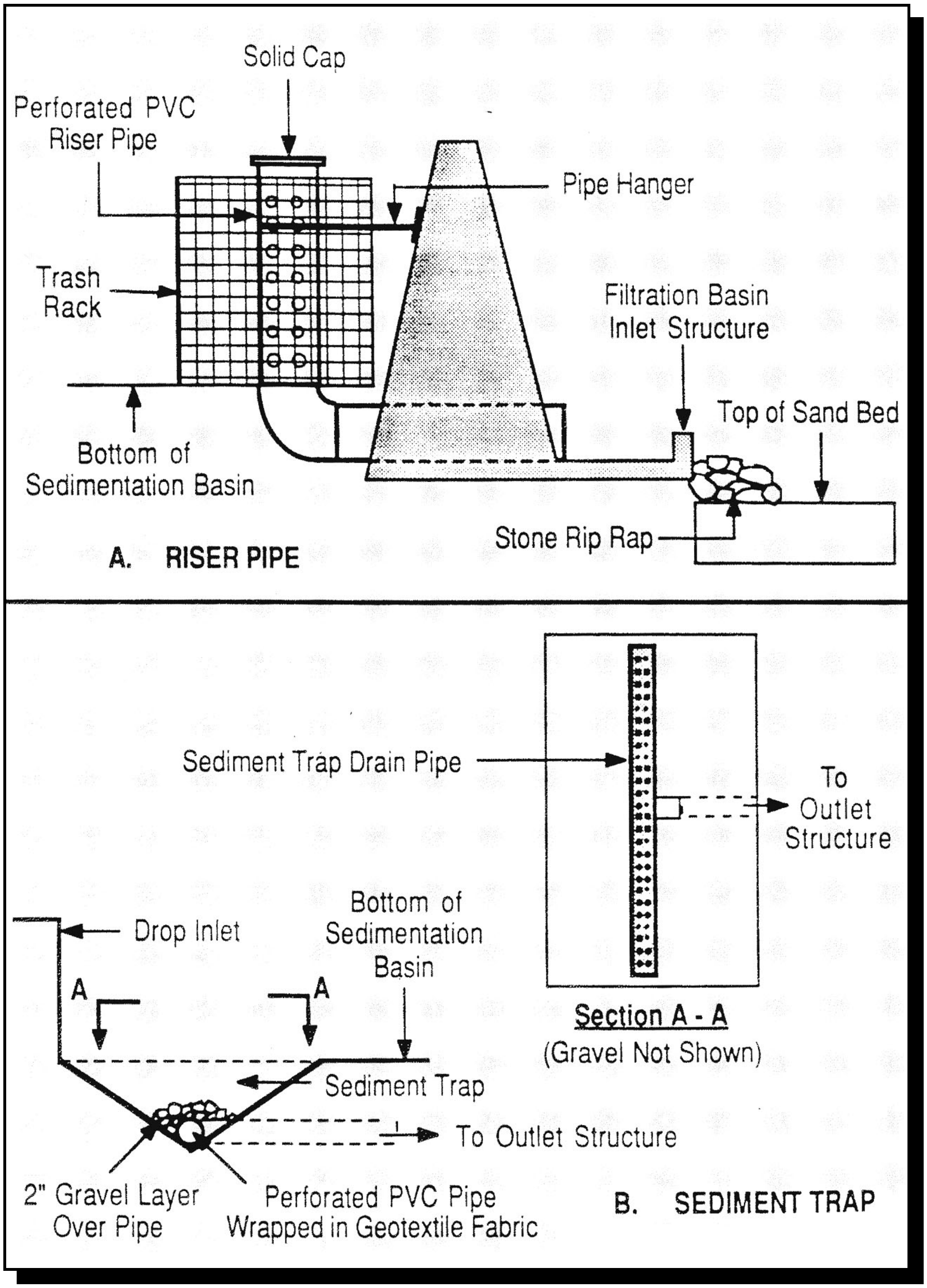


Figure F.6 Example Riser Pipe and Sediment Trap Details
Source: City of Austin, 1988

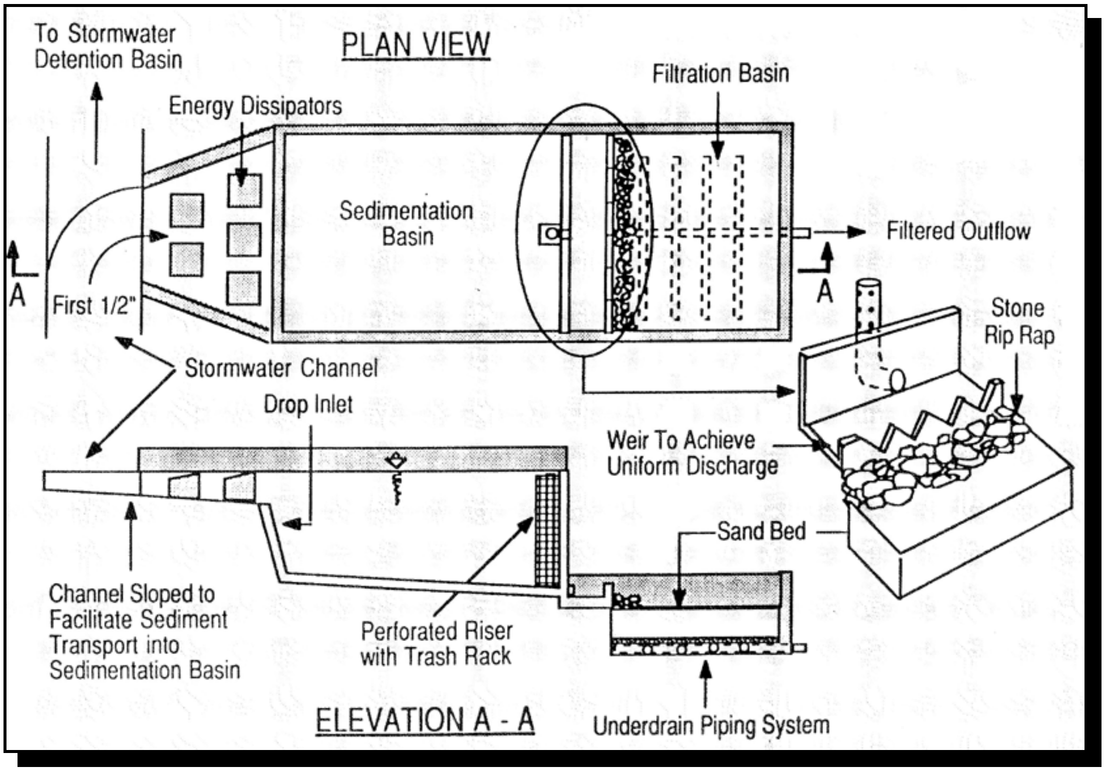


Figure F.7 Conceptual Full Sedimentation Filtration System

3. Sand Bed with Full Top and Bottom Gravel Layers (Figure F.9) - The top gravel layer is to be a minimum of 1" of D.C. #57 gravel. Under the top gravel layer should be minimum 12" of ASTM C-33 Concrete Sand. Beneath the sand shall be 8" of D.C. #57 gravel with a subsurface drainage system. The sand and gravel must be separated by a layer of geotextile fabric meeting the specifications list in Table F.1.

4. Sand Bed with Full Top Peat and Bottom Gravel Layers (Figure F.10) - The upper peat layer must be a minimum of 12" thick. In order to eliminate the possibility of saturating the peat bed, a 1.0 inch per hour infiltration rate is required to limit the maximum surface ponding by using a custombend hemic/fibric mixture or one that is recommended by a soil scientist or engineer. Approximately 1.5" of Ag-Lime calcitic limestone must be mixed into the top 4 to 6" of peat. The minimum 4" layer of 50%-50% peat and sand mixture must be placed immediately under the peat layer. This is to allow for uniform flow of water through the bed. A geotextile fabric should be placed under the 24" vertical thickness sand layer to separate sand from the bottom gravel layer. Other design elements should be the same as for the surface sand filter.

Table F.4 Drainage Matting Specifications

Property	Test Method	Unit	Specification
Material	Non-woven geotextile fabric		
Unit Weight		oz/yd^2	20
Flow Rate (fabric)		gpm/ft^2	180 min.
Permeability	ASTM D-2434	cm/sec	12.4×10^{-2}
Grab Strength (fabric)	ASTM D-1682	lb	Dry Lg.90 Dry Wd: 7 Wet Lg.95 Wet Wd:7
Puncture Strength (fabric)	COE CV-02215	lb	42 minimum
Mullen Burst Strength	ASTM D-1117	psi	140 minimum
Equivalent Opening Size	U.S. Standard Sieve	No.	100 (70-120)
Flow Rate (drainage core)	Drexel University	gpm / ft width	14

Source: City of Austin

Underdrain Piping

The underdrain piping consists of the main collector pipe(s) and perforated lateral branch pipes. The piping should be reinforced to withstand the weight of the overburden. The internal diameters of the lateral branch pipes should be 4" or greater and perforations should be 3/8". All PVC pipe is to be schedule 40 or greater strength. A maximum spacing of ten feet between laterals is recommended. Lesser spacings are acceptable.

The minimum pipe slope should be 1/8 inch per foot or 1% slope. All pipes must provide clean-out caps for future maintenance.

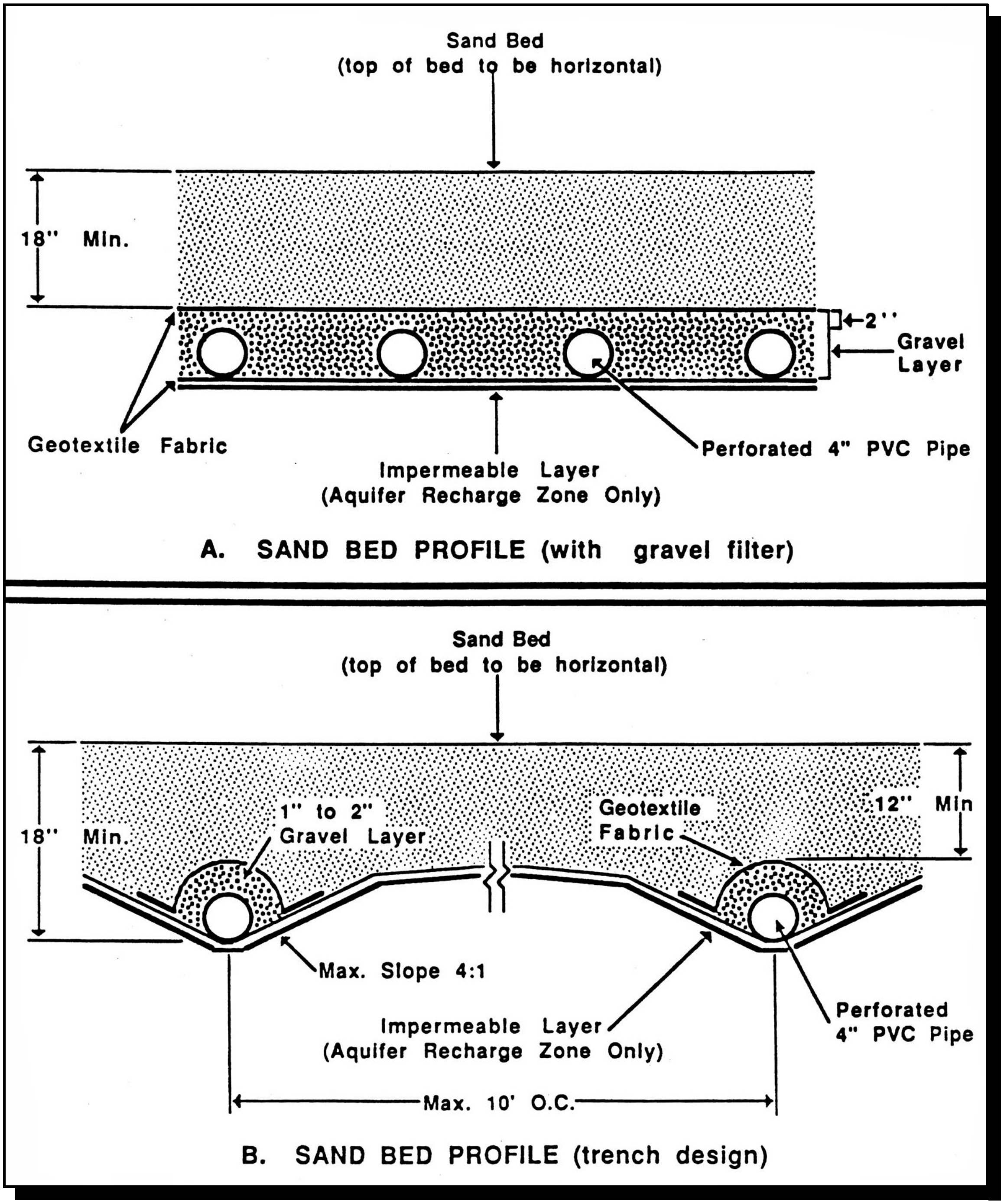


Figure F.8 Sand Bed Filtration Configurations
Source: City of Austin, 1988

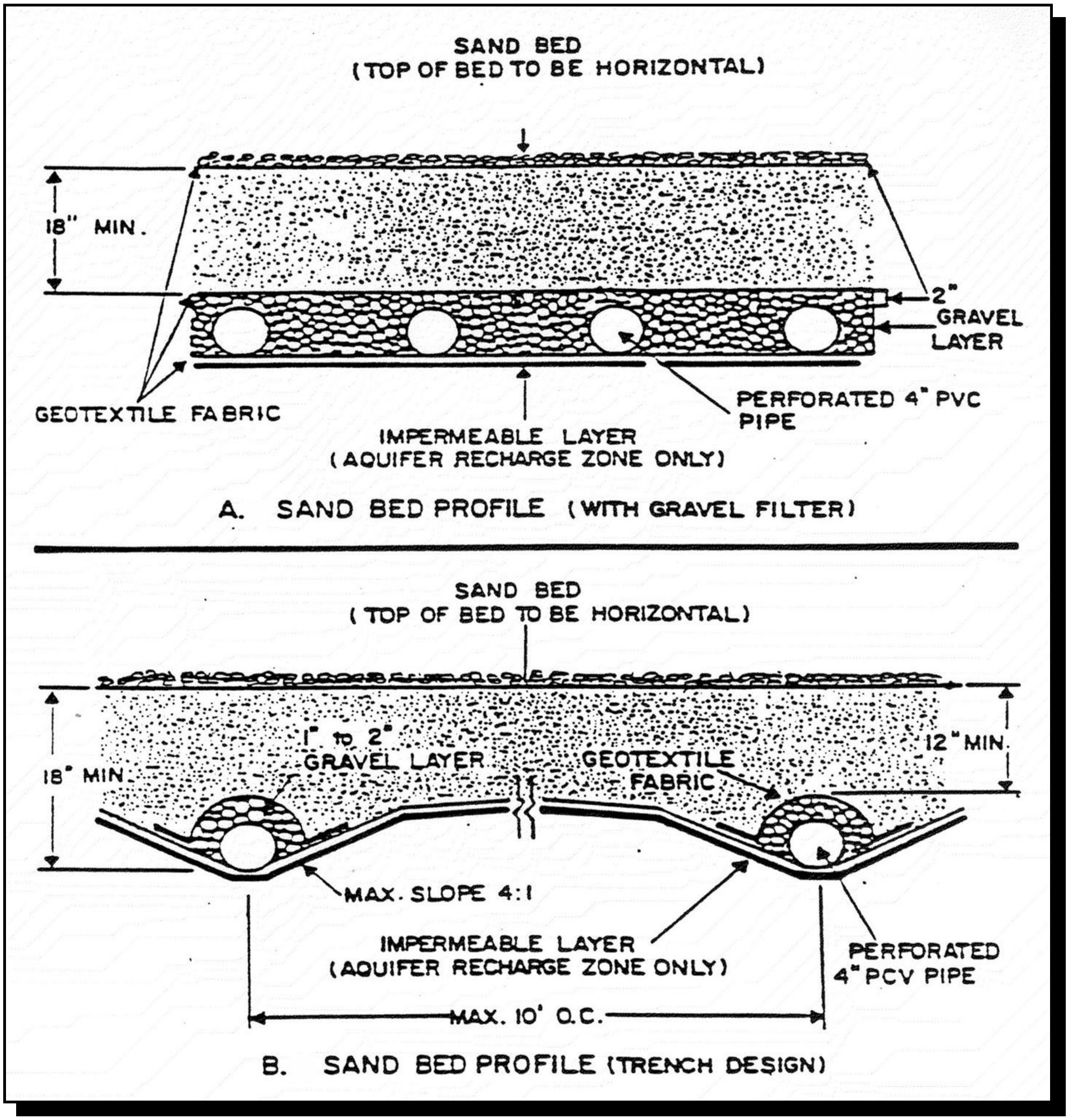


Figure F.9 Sand Bed Filtration Configurations with Top Gravel Layer
Source: District of Columbia

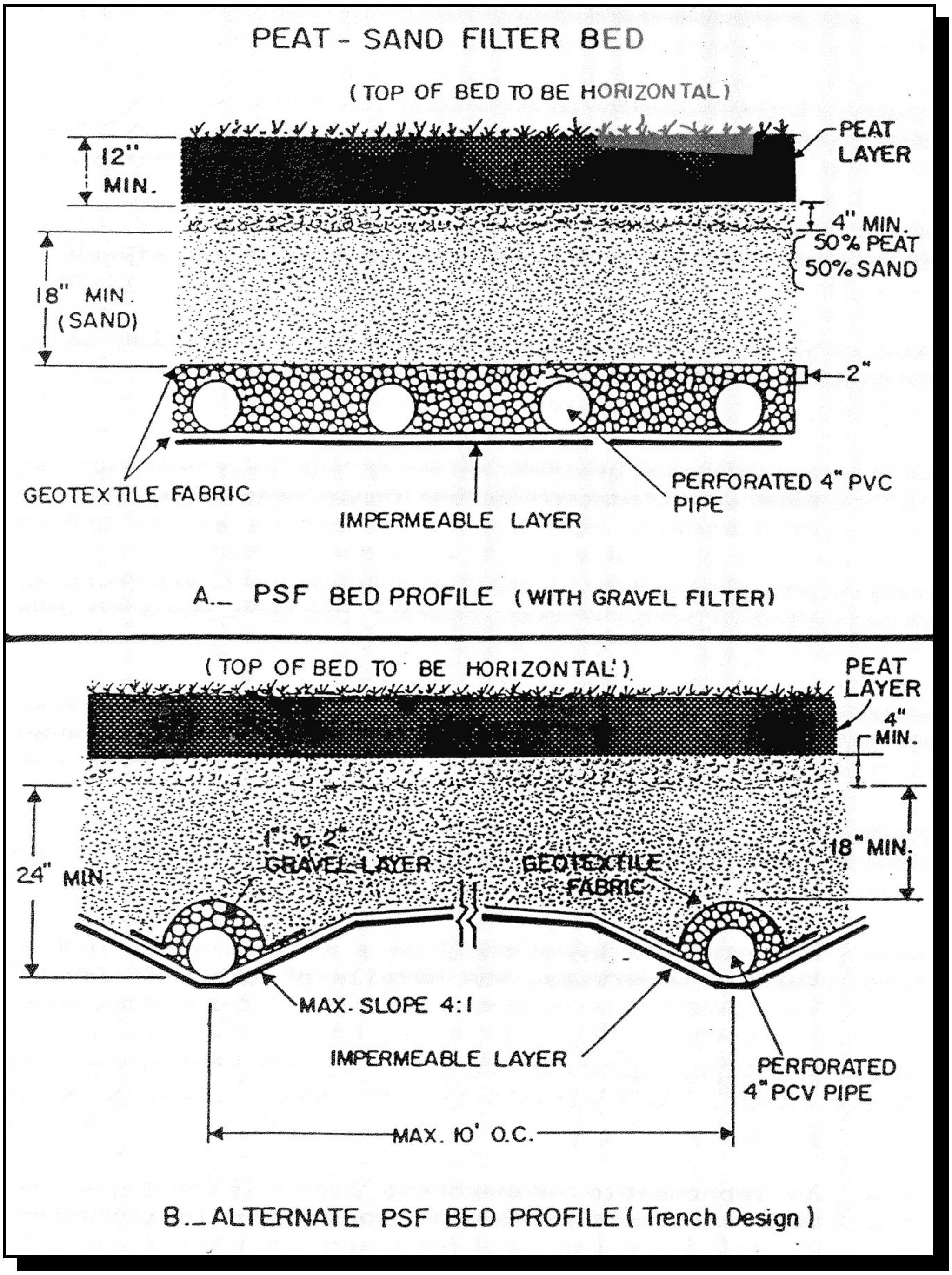


Figure F.10 Peat - Sand Filter Bed Configuration
Source: City of Alexandria, 1992

F.3 Construction Requirements

This section discusses construction requirements for:

- One-Chamber Underground Sand Filters
- Three-Chamber Underground Sand Filters
- Vertical Sand Filters
- Perimeter Sand Filters
- Surface Sand Filters
- Roof Downspout Filtration Systems

1-Chamber Underground SF, 3-Chamber Underground SF, Vertical SF

- The sand filter may be either cast-in-place or precast. In the District of Columbia, precast structures require advance approval. Cast-in-place should be as per Section 02720.01 to 02726.06 of the District of Columbia Public Works Water and Sewer Specification and Detailed Drawings in Office Manual.

- The approved erosion and sediment control plans should include specific measures to provide for the protection of the filter system before the final stabilization of the site.

- Excavation for the sand filter and connecting pipes should include removal of all materials and objects encountered in the excavation; disposal of excavated material as specified in the approved erosion and sediment control plans, maintenance and subsequent removal of any sheeting, shoring and bracing; dewatering and precautions, and work necessary to prevent damage to adjacent properties resulting from this excavation.

- Access manholes and steps to the filtration system should conform to District of Columbia DPW standards.

- After completion of the sand filter shell, a leak test should be performed to verify water tightness before the filter layers are installed.

- All filter materials in the second chamber should be placed according to construction and materials standards and specifications, as specified on an approved construction plan.

- No runoff should be allowed to enter the sand filter system prior to completion of all construction activities, including revegetation and final site stabilization. Construction runoff should be treated in separate sedimentation basins and routed to bypass the filter system. Should construction runoff enter the filter system prior to final site stabilization, all contaminated materials must be removed and replaced with new clean filter materials before a regulatory inspector approves it's completion.

- The water level in the filter chamber should be monitored by the design engineer after the first storm event, before the project is certified as been completed. If the dewatering time of the filter chamber takes longer than 24 hours, the top gravel layer and filter fabric underneath, must be replaced with a more rapid draining fabric and clean gravel. The structure should then be checked again to ensure a detention time that is less than 24 hours.

Perimeter Sand Filters

- Erosion and sediment control measures must be established to prevent any inflow of storm water into the perimeter sand filter system until construction on site is completed and the entire drainage area have been stabilized with vegetated cover.

- The inverts of the notches, multiple orifices or weirs separating the sedimentation chamber from the filter chamber must be constructed completely level to achieve uniform sheet flow to the filter chamber.

- Inflow grates or slotted curbs may conform to the grade of the completed pavement as long as the filters, notches, multiple orifices, and weirs connecting the sedimentation chamber area are completely level .

- The minimum slope of the underdrain pipe should be 0.5%.

- If precast concrete lids are used, lifting rings or threaded sockets must be provided to allow for easy removal with lifting equipment.

- The facility must not be placed in service until all soil surfaces in the drainage area have been finally stabilized with vegetation cover.

Surface Sand Filters

- Provisions must be made for access to the basin for maintenance purposes. A maintenance vehicle access ramp is necessary. The slope of the ramp should not exceed 4:1.

- The design should minimize susceptibility to vandalism by use of strong materials for exposed piping and accessories.

- Side slopes for earthen embankments structures should not exceed 3:1 to facilitate mowing.

- The temporary erosion and sedimentation control plan must be configured to permit construction of the pond while maintaining erosion and sedimentation control.

- No runoff is to enter the sand filtration basin prior to completion of construction and site stabilization.

Roof Downspout Filtration Systems

- Erosion and sediment control measures must be configured to prevent any flow of storm water into the RDF system until construction on site is complete and all soil surfaces on the drainage watershed have been stabilized with vegetation.

- During excavation of the trench to design dimensions, the excavated materials must be placed away from the excavation in a downstream area to prevent redeposition during subsequent runoff events. Large tree roots should be trimmed flush with the sides to protect the filter fabric and geomembrane during its installation.

- There should be no voids between the filter fabric geomembrane. If boulders or similar obstacles are removed from the excavation sides, the void should be filled with natural soils before the filter layers are installed.

- The collector gravel, sand and crushed stone aggregate should be placed in the trench using a backhoe or front-end loader with a drop height near the bottom on the RDF system. Aggregate should not be dumped into the trench by a truck.

- Before the sand is placed, the RDF System must be lined with filter fabric. The fabric must be wrapped around the sand layer with at least 6" of overlap. After the aggregate is placed on the top of the sand layer, the filter fabric should be wrapped around the top with at least 6" overlap.

- The reservoir stone should be clean, washed crushed aggregate and should be placed in loose lifts of about 12" and light compacted with plate compactors. Compaction assure fabric conformity to the sides and should reduce the potential for clogging and settlement problems.

- There should be no mixing of clean aggregate with natural or fill soils. All contaminated aggregate should be removed and replaced with clean aggregate.

- The RDF should not be placed in service until all soil surfaces in the drainage watershed have been stabilized with vegetated cover.

- An Inspection well shall be installed up to the final grade with a cap.

Appendix

G

Additional Storm Water Implementation Information

This Appendix includes the reports and forms necessary for storm water implementation in the District of Columbia (see Chapter 5). This includes:

- Flow Chart of the Permit Process for the Watershed Protection Division
- Site Development Submittal Information
- Information Requirement for New Storm Water Management Discharge Facility Application
- Application for Construction Permits on Private Property
- Filed Job Notification Form
- Infiltration Device Inspection Report
- Sand Filter Inspection Report
- As-Built Storm Water Management Plan Guidelines
- As-Built Certification by Professional Engineer / Statement by Person Responsible for Maintenance
- Declaration of Covenants for a Storm Water Management Facility
- Storm Water Management Facilities Maintenance Inspection Form
- Oil Grit Separators Maintenance Inspection Form
- Sand Filter Maintenance Inspection Form
- Roof Top / Underground Detention Structures Maintenance Inspection Form
- Maintenance Service Completion Inspection Form

Appendix H

Rooftop Storage Guidance and Criteria

H.1 Roof Top Storage Design Guidance and Criteria

1. Roof top storage shall be designed to detain the 15-year, 80-minute storm, and emergency overflow provisions must be adequate to discharge the 100-year, 45-minute storm.

Frequency (years)	Duration, t (minutes)	Depth, d (inches)	Intensity, i (inches/hour)
15	80	2.96	2.22
100	45	2.96	3.94

2. If a proper design is submitted for the 15-year storm, sufficient storage will normally be provided for the two-year storm, and separate calculations need not be made.

3. Rainfall from this design storm results in an accumulated storage depth of 2.96 inches (approximately 3 inches).

   A. Based on a snow load of 30 pounds per square foot or 5.8 inches of water, properly designed roofs are structurally capable of holding three inches of detained storm water with a reasonable factor of safety.

   B. Roofs calculated to store depths greater than three inches shall be required to show structural adequacy of the roof design.

4. No less than two roof drains shall be installed in roof areas of 10,000 square feet or less, and at least four drains in roof areas over 10,000 square feet in area. Roof areas exceeding 40,000 square feet shall have one drain for each 10,000 square foot area.

5. Emergency overflow measures adequate to discharge the 100-year, 45-minute storm must be provided.

   A. If parapet walls exceed 3 inches in height, the designer shall provide openings (scuppers) in the parapet wall sufficient to discharge the design storm flow at a water level not exceeding 5 inches.

   B. One scupper shall be provided for every 20,000 square feet of roof area, and the invert of the scupper shall not be more than 3.5 inches above the roof level. (If such openings are not practical, then detention rings shall be sized accordingly).

6. Detention rings shall be placed around all roof drains that do not have controlled flow.

A. The number of holes or size of openings in the rings shall be computed based on the area of roof drained and run-off criteria.

B. The minimum spacing of sets of holes is 2 inches center-to-center.

C. The height of the ring is determined by the roof slope and shall be 3 inches maximum.

D. The diameter of the rings shall be sized to accommodate the required openings and, if scuppers are not provided, to allow the 100-year design storm to overtop the ring (overflow design is based on weir computations with the weir length equal to the circumference of the detention ring).

E. Conductors and leaders shall also be sized to pass the expected flow from the 100-year design storm.

7. The maximum time of drawdown on the roof shall not exceed 17 hours.

8. Josam Manufacturing Company and Zurn Industries, Inc. market “controlled-flow” roof drains. These products, or their equivalent, are acceptable.

9. Computations required on plans:

a) Roof area in square feet.
b) Storage provided at three-inch depth.
c) Maximum allowable discharge rate.
d) Inflow-outflow hydrograph analysis or acceptable charts (for Josam Manufacturing Company and Zurn Industries, Inc. standard drains, the peak discharge rates as given in their charts are acceptable for drainage calculation purposes without requiring full inflow-outflow hydrograph analysis).
e) Number of drains required.
f) Sizing of openings required in detention rings.
g) Sizing of ring to accept openings and to pass 100-year design storm.

H.2 Roof Top Storage Design Example

Given:

- A flat-roofed building with the following dimensions: 200 ft by 50 ft
- Predevelopment runoff coefficient, C = 0.40
- Predevelopment time of concentration, T_c = 10 minutes

Computations:

a) Roof area in square feet:

Roof area = (200 feet) (50 feet) = $\underline{10{,}000\ ft^2}$

b) Storage provided at three-inch depth:

Storage volume = (10,000 ft^2) (3 in) (1 ft /12 in) = $\underline{2{,}500\ ft^3}$

c) Maximum allowable discharge rate:

Maximum allowable discharge rate = predevelopment rate of runoff

Q = CIA = (0.40) (5.92) (10,000 ft^2 / 43,560) = 0.54 cfs

d) Inflow-outflow hydrograph analysis:

From Figure H.1, one set of holes with 3 inches of water will produce runoff or discharge of 6 gpm (0.0134 cfs). See Figure H.2 for a diagram of a typical ponding ring.

e) Number of drains required:

Number of drains required for 10,000 square feet of roof area = 2

f) Sizing of openings required in detention rings:

Number of hole = allowable discharge (Q) divided by 0.0134 cfs per l set of holes.

(0.54 cfs) / (2 drains) = 0.27 cfs per drain

Number of holes = (0.27 cfs) / (0.0134 cfs / set of holes) = 20.1 sets

20.1 sets of holes per drain (Use 20 sets of holes)

g) Sizing of ring to accept openings and to pass 100-year design storm:

Hole sets are spaced 2 inches on center

Circumference = π * diameter

(20 sets) (2 inches / set) = π * diameter

Diameter, D = 12.73 inches

Use 15 inches (see below if separate emergency overflow is not provided)

h) If detention rings are to act as emergency overflow measures:

Q_{100} = CIA

Where: t_c = 5 minutes
C = 1.0
A = 10,000 square feet / 43,560

Q_{100} = (1.0) (9.84) (10,000 ft^2 / 43,560) = 2.26 cfs

Weir formula: $Q = CLH^{3/2}$

Where: C = 3.33
L = circumference = π D
D = diameter
H = 2 inches = 0.17 feet

Assume all hole sets are clogged and the maximum allowable water depth on the roof is 5 inches, or 2 inches above the 3-inch high ring.

Q per drain = (2.26 cfs) / (2 drains) = (3.33) (π D) $(0.17 \text{ feet})^{3/2}$

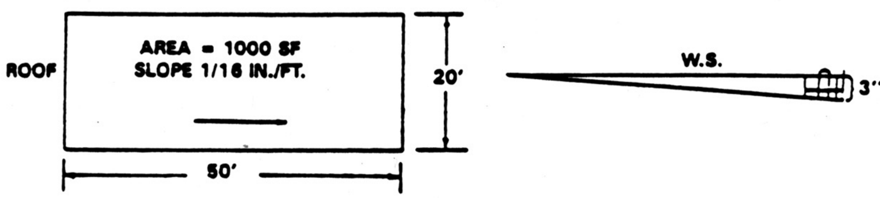
D = 1.54 feet = 18.5 inches

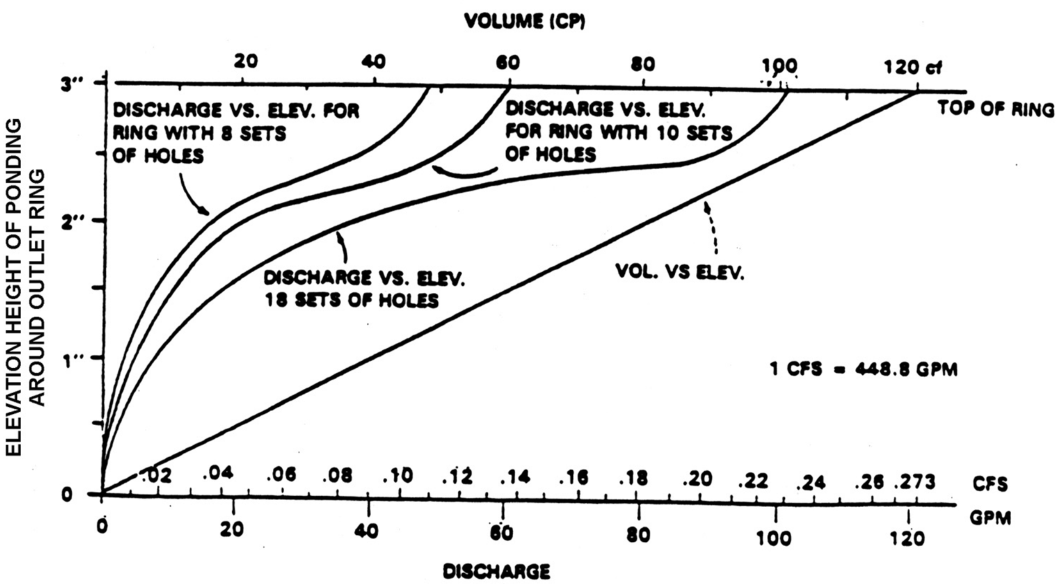
Use diameter of 20 inches

HYDRAULIC CHARACTERISTICS OF ROOF DRAIN RINGS

WATER DEPTH AT INLET (INCHES)	CAPACITY THROUGH HOLES DISCHARGE (GPM)			
	1 SET HOLES	8 SETS	10 SETS	18 SETS
1.5	1	8	10	18
2.0	2	16	20	36
2.5	5	40	50	90
3.0	6	48	60	108

ROOFTOP DETENTION PERFORMANCE CURVE





	ROOFTOP STORM WATER DETENTION	REVISION NO.	DATE	PAGE NO

Figure H.1 Rooftop Storm Water Detention

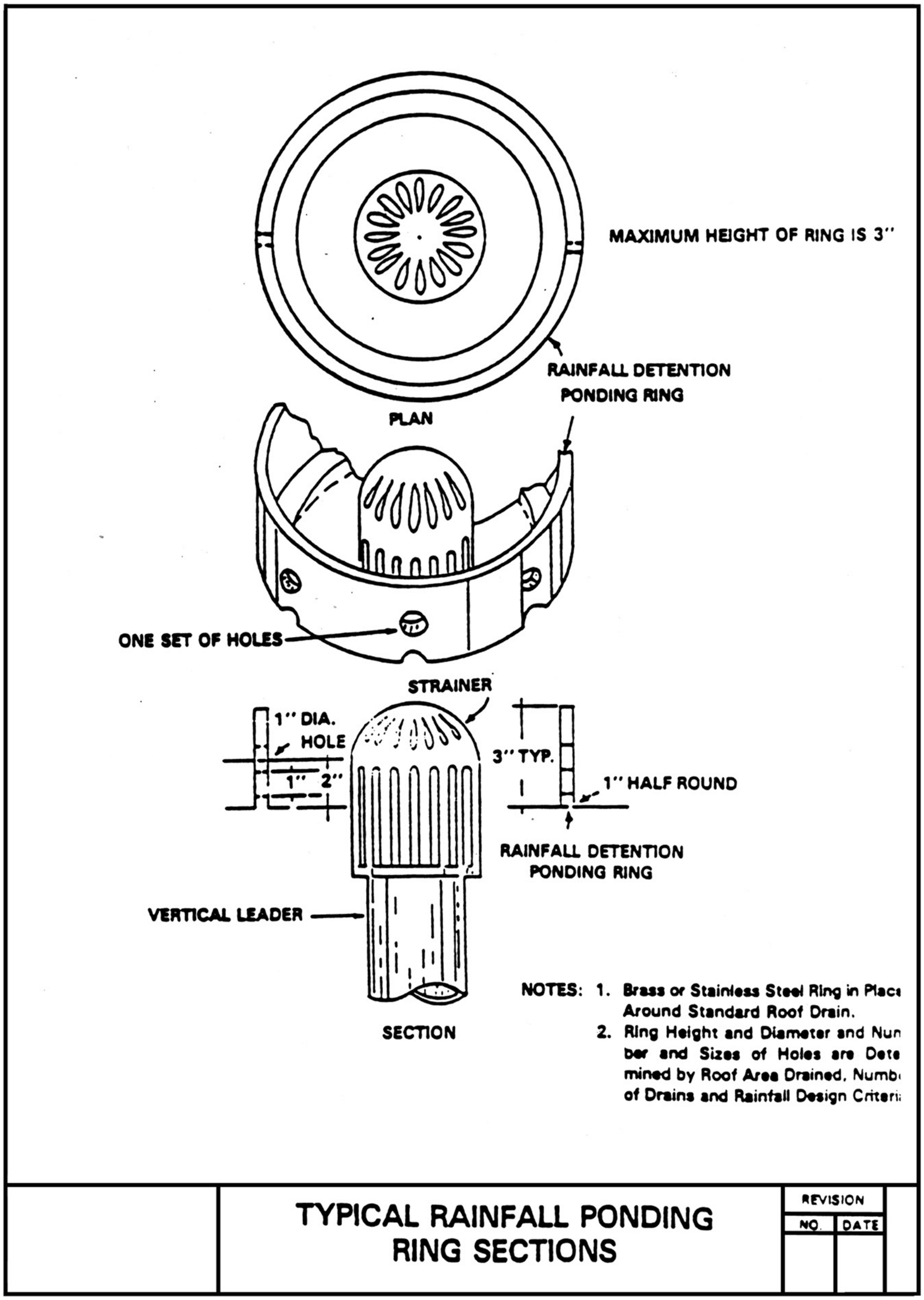


Figure H.2 Typical Rainfall Ponding Ring Sections

Appendix I

Sand Filter Design Example

I.1 Sand Filter Design Example

Data Required:

- Storm frequency = 15 years
- Storm duration = 24 hours
- Time of concentration = T_c = 5 minutes
- From District of Columbia rainfall intensity - duration - frequency curve:
  intensity = i = 7.56 inches/hour
  rainfall depth = d = 0.63 inches

Assumptions:

- The site is a parking lot with area = A = 10,000 ft^2
- The sand filter will outfall to a storm sewer.
- The distance is 50 feet from the sand filter's invert-out to the storm sewer pipe connection
- The invert of the city storm sewer = 92 feet at the proposed connection point (note: allow a 2% slope in the pipe connecting the invert-out of the sand filter to the city storm sewer invert)
- The runoff coefficient = c = 1.0 (assuming 100% impervious surface)
- The first flush of runoff, or runoff depth to be treated = R = 0.5 inches

1. Determine Design Invert Elevations

Determine the final surface elevation, invert in, invert out and bottom invert elevation of the structure.

- Invert in = 98 ft
- Invert out = 93 ft (beginning approximation)

  Note: Actual head difference available within the filter between invert-in and invert-out is 98 ft - 93 ft = 5 ft

- Assume inflow pipe diameter = 1 ft
- Assume over the crown pipe cover = 2 ft
- Final surface elevation above sand filter = 101 ft = 98 ft + 2 ft + 1 ft
- Depth of filter layer: $d_{f\text{-}max}$ = 3 feet
  $d_{f\text{-}min}$ = 1.8 feet

2. Peak Discharge Calculation for Bypass Flow

The District of Columbia uses the 15-year storm (with t_c = 5 minutes) for post-development runoff.

Using the Rational Method:

$$Q_{p15} = CIA$$

Where:
Q_{p15} = bypass peak flow (cfs)
C = runoff coefficient = 1.0
I = rainfall intensity = 7.56 in/hr
A = drainage area = 0.23 acres
T_c = time of concentration (used in selecting rainfall intensity)
= 5 min

Q_{p15} = (1.0) (7.56 in/hr) (0.23 ac) = <u>1.74 cfs</u>

3. Determine Sand Filter Area A_f

Use the following equation:

$$A_f = 50 + (I_a - 0.1ac) * (167 ft^2 / ac)$$

Where:
A_f = surface area of filter layer (second chamber) (ft^2)
I_a = impervious area = 0.23 ac

A_f = 50 + (0.23 ac - 0.1 ac) (1.67 ft^2/ac) = 71.2 ft^2

Use A_f = <u>80 ft^2</u>

4. Determine Storage Volume Needed

First, determine the water quality volume that must be treated in the sand filter using the following equation:

$$V_w = \frac{R * I_a}{12}$$

Where: V_w = water quality volume (ft^3)
R = runoff depth = 0.5 in for parking lots
I_a = impervious area (ft^2) = 10,000 ft^2
12 = conversion factor

$$V_w = \frac{0.5\ \text{in} * 10{,}000\ \text{ft}^2}{12} = \underline{416.7\ \text{ft}^3}$$

Calculate the volume of storage (V_s) needed to accommodate the first flush of runoff from the parking lot using the following equation:

$$V_s = V_w - (F * T * A_f)$$

Where: V_s = storage volume needed to hold the first flush runoff (ft^3)
V_w = water quality volume = 416.7 ft^3
F = infiltration rate for sand ≈1.18 ft/hr
T = filtering time = 1 hour based on NRCS practice
A_f = surface area of filter layer (second chamber) = 80 ft^2

$$V_s = 416.7\ \text{ft}^3 - (1.18\ \text{ft/hr})(1\ \text{hr})(80\ \text{ft}^2) = \underline{322.3\ \text{ft}^3}$$

5. Calculate Submerged Storage Volume in Second Chamber

$$V_{2b} = A_f * d_f * n$$

Where:
- V_{2b} = submerged volume of filter chamber (ft^3)
- A_f = surface area of filter layer (second chamber) = 80 ft^2
- d_f = depth of filter layer = 3 ft
- n = composite of porosity for filter media, for sand + gravel + perforated pipe = 0.6

V_{2b} = (80 ft^2) (3 ft) (0.6) = 144 ft^3

6. Calculate Submerged Storage Volume in First Chamber

$$V_{1b} = A_1 * d_f$$

Where:
- V_{1b} = submerged volume of first chamber (ft^3)
- A_1 = surface area of first chamber (ft^2)
- d_f = depth of filter layer (ft)

V_{1b} = (25 ft^2) (3 ft) = 75 ft^3

7. Calculate Surface Storage Volume in First & Second Chambers

$$\left(V_{1t} + V_{2t}\right) = V_s - \left(V_{2b} + V_{1b}\right)$$

Where:
- $V_{1t} + V_{2t}$ = sum of surface volume of first & second chambers (ft^3)
- V_s = storage volume needed to hold the first flush runoff (ft^3)
- V_{1b} = submerged volume of first chamber (ft^3)
- V_{2b} = submerged volume of filter chamber (ft^3)

($V_{1t} + V_{2t}$) = 322.3 ft^3 - (144 ft^3 + 75 ft^3) = 103.3 ft^3

8. Determine H, Height Difference Available Between Top of Filter Layer and Bypass Pipe Outlet Invert

$$V_{1t} + V_{2t} = (A_1 * H) + (A_f * H)$$

Where: $V_{1t} + V_{2t}$ = sum of surface volume of first & second chambers (ft^3)
A_1 = surface area of first chamber (ft^2)
A_f = surface area of filter layer (second chamber) = 80 ft^2
H = vertical distance between top of filter layer and bypass pipe outlet invert

Therefore:

$$H = \frac{(V_{1t} + V_{2t})}{(A_1 + A_f)} = \frac{103.3\ ft^3}{(25\ ft^2 + 80\ ft^2)} = 0.98\ ft$$

Use H = <u>1 ft</u>

8. Determine Maximum Storage Depth

$$D = H + d$$

Where: D = maximum storage depth (ft)
H = vertical distance between top of filter layer and bypass pipe outlet invert = 1 ft
d = depth of filter layer (ft) = 3 ft

Note: D must be equal to or smaller than the difference between the invert in and invert out (5 ft)

D = 1 ft + 3 ft = 4 ft $\leq$ 5 ft (good)

Use D = <u>5 ft</u>

8.5 Determine Design Invert Out:

design invert out = invert in - D

Where: invert in = 93 ft
D = maximum storage depth = 5 ft

design invert out = 98 ft - 5 ft

design invert out = <u>93 ft</u> $\geq$93 ft (good, invert out approximated in Step 1 = 93 ft)

Note: As first approximated in step 1, invert of outlet pipe is at 93 ft elevation. This elevation is higher than the public storm sewer (at 92 ft, see Step 1) at connection point and provides 1 ft (93 ft - 92 ft = 1 ft) of vertical elevation to accommodate the 2% slope requirement.

In this example, the connection to the storm sewer is 50 ft away from the invert-out of the sand filter. Therefore: 1 ft / 50 ft = 0.02 or 2% slope.

9. Determine Size of Bypass Pipe

Determine the capacity of the bypass pipe:

$$D = \left[\frac{2.16 * n * Q_{p15}}{\sqrt{S}}\right]^{0.375}$$

Where: D = estimated bypass pipe diameter (ft)
n = Manning's roughness coefficient = 0.011
Q_{p15} = bypass peak flow (cfs) = 1.74 cfs
S = pipe slope = assume 0.5% = 0.005

$$D = \left[\frac{2.16 * 0.011 * 1.74}{\sqrt{0.005}}\right]^{0.375} = 0.82 \text{ ft}$$

Use D = <u>1 ft (12")</u>

10. Determine submerged weir opening in first chamber

Since the weir opening in the first chamber is submerged, the orifice equation is used to calculate the dimensions of the weir opening:

$$Q_{p15} = C * A_{w1} * \sqrt{2gh_{max}}$$

Therefore:

$$A_{w1} = \frac{Q_{p15}}{C * (2gh_{max})^{0.5}}$$

Where:
- A_{w1} = area of weir opening in first chamber (ft^2) = h_{w1} * l_{w1}
- h_{w1} = weir height, assume 1 ft
- l_{w1} = weir length (ft)
- Q_{p15} = bypass peak flow (cfs) = 1.74 cfs
- C = 0.6
- g = 32.2 ft/sec^2
- h_{max} = hydraulic head above the center line of weir (ft)
  = [(invert in - invert out) - (h/2)]
  = [(98 ft - 94 ft) - ½] = 3.5 ft

$$A_{w1} = \frac{1.74 \text{ cfs}}{0.6 * (2 * 32.2 \text{ ft/sec}^2 * 3.5 \text{ ft})^{0.5}} = 0.193 \text{ ft}^2 = h_{w1} * l_{w1}$$

Therefore:

$$l_{w1} = \frac{A_{w1}}{h_{w1}} = \frac{0.193 \text{ ft}^2}{1 \text{ ft}} = 0.193 \text{ ft} \quad \text{(minimum requirement)}$$

Note: Assume 50% of the weir opening is clogged; therefore, use the following

$$l_{w1} = 2\left(\frac{A_{w1}}{h_{w1}}\right) = 2\left(\frac{0.193 \text{ ft}^2}{1 \text{ ft}}\right) = 0.386 \text{ ft}$$

Use l_{w1} = <u>1 ft</u>

11. Structure dimensions (internal only):

Second Chamber:

$$A_f = L_2 * W$$

Where: A_f = surface area of filter layer (second chamber) = 80 ft^2
L_2 = length of filter layer (second chamber) (ft)
W = width of chamber, use 6 ft

Therefore:

$$L_2 = \frac{A_f}{W} = \frac{80\ ft^2}{6\ ft} = 13.3\ ft$$

Use L_2 = 13.5 ft

First Chamber:

$$A_1 = L_1 * W$$

Where: A_1 = surface area of first chamber = 25 ft^2
L_1 = length of first chamber (ft)
W = width of chamber, use 6 ft

Therefore:

$$L_1 = \frac{A_1}{W} = \frac{25\ ft^2}{6\ ft} = 4.16\ ft$$

Use L_1 = 4.5 ft

Third Chamber:

Select L_3 = length of third chamber = 3 ft

Therefore:

Length of structure, $L = L_1 + L_2 + L_3$ = 4.5 ft +13.5 ft + 3 ft = 21 ft

11.5 Note: To prevent "short circuiting" in the filter chamber,

$$L_1 + L_2 \geq 3 * W$$

$$4.5 \text{ ft} + 13.5 \text{ ft} \geq 3 \text{ ft} * 6 \text{ ft}$$

$$18 \text{ ft} \geq 18 \text{ ft (good)}$$

Total inner height inside sand filter:

$$D_t = D + \text{inflow pipe diameter} + \text{free board}$$

$$D_t = 5 \text{ ft} + 1 \text{ ft} + 2 \text{ ft} = \underline{8 \text{ ft}}$$

12. Determine Flow Through Filter and Detention Time After Storage Volume Fills Up

To determine the average flow through the filter, use:

$$q_f = k * A_f * i = k * A_f * \frac{h_{max}}{2 * d_f}$$

Where:

q_f	=	average flow through the filter (ft^3/hr)
k	=	sand permeability (ft/hr) = 0.6 ft/hr for mixed sand
A_f	=	surface area of filter layer (ft^2) = 80 ft^2
i	=	hydraulic gradient (ft/ft)
d_f	=	thickness of the sand layer = 18 inches
h_{max}	=	[(d + H) - (perforated pipe diameter / 2)]
	=	[(3 ft + 1 ft) - (6 in / 2) (1 ft / 12 in)] = 3.75 ft

Note: For accurate hydraulic computation, assume there are 6" diameter PVC perforated pipes at the bottom of the sand filter.

q_f = (0.6 ft/hr) * (80 ft^2) * [3.75 ft / (2 * 1.5 ft)] = $\underline{60.0 \text{ ft}^3/\text{hr}}$ = 0.016 cfs ≈ $\underline{0.02 \text{ cfs}}$

To estimate the detention time, use:

$$T_s = \frac{V_s}{q_f}$$

Where: T_s = average dewatering time for sand filter (hr)
V_s = storage volume needed (ft^3) = 322.3 ft^3
q_f = average flow through the filter (ft^3/hr) = 60.0 ft^3/hr

T_s = (322.3 ft^3) / (60.0 ft^3/hr) = <u>5.4 hrs</u> $\leq$ 72 hrs (good)

11. Develop Inflow and Outflow Hydrographs

A. Inflow hydrograph data:

- area = 10,000 ft^2 = 0.23 ac
- frequency = 15 yr
- duration = 24 hr
- time of concentration, T_c = 5 min
- runoff coefficient, C = 1.0

Rational Method: Q = CIA = 1.0 * I * 0.23 ac

T (min)	I (in / hr)	Q (cfs)
0	0	0
5	7.56	1.74
10	6.30	1.45
15	5.44	1.25
20	4.82	1.11
30	3.95	0.91
45	3.16	0.73
60	2.66	0.61

B. Time at which outflow hydrograph begins (choose 1):

When: $t_c * Q_p < 2V_s$ Then

$$T = 2t_c - \left(2t_c^2 - \frac{2V_s * t_c}{Q_p}\right)^{0.5}$$

When: $t_c * Q_p = 2V_s$ Then

$$T = 0.5t_c + \left(\frac{V_s}{Q_p}\right)$$

When: $t_c * Q_p > 2V_s$ Then

$$T = \left[\frac{2V_s * t_c}{Q_p}\right]^{0.5}$$

Note: Tc * Qp __?__ 2 * Vw

(5 min) * (1.74 cfs) __?__ 2 * (322.3 ft^3)

522 ft^3 < 645 ft^3

Note: Since the inequality fits #1 above, use the appropriate equation to solve for T.

When: $t_c * Q_p < 2V_s$ Then

$$t_p = 2t_c - \left(2t_c^2 - \frac{2V_s * t_c}{Q_p}\right)^{0.5}$$

$$t_p = 2 * 5\,\text{min} - \left(2\,(5\,\text{min})^2 - \frac{2\,(322.3\,\text{ft}^3)\,(5\,\text{min})}{1.74\,\text{cfs}}\right)^{0.5} = \underline{5.6\,\text{min}}$$

Note: At a filling time, T = 5.6 min, the flow or Q ≈ 0.02 cfs

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