CHAPTER 2

UNCONVENTIONAL BMP DESIGN CRITERIA

Chapter 2
Unconventional BMP Facility Design Criteria
(Ultra-Urban BMPs)

I. INTRODUCTION

The three standard BMP facilities for stormwater quality management (dry ponds, wet ponds and infiltration devices) are not suitable for large areas of Alexandria because of space constraints or underlying soil conditions in our **ultra-urban environment.** On sites where standard BMPs are not feasible, the developer/engineer should consider the use of unconventional or innovative BMPs to provide the required non-point source pollution control. If the Director determines conventional BMPs to be infeasible, alternative facilities may be accepted if certain performance monitoring and other requirements are met. This chapter contains criteria for the design of several innovative BMP approaches which are being utilized in other jurisdictions outside Northern Virginia and a new **mandatory** approach for use in the City's combined sewer watersheds.

The City will allow the use of the innovative BMPs discussed in this chapter and recognize the phosphorous removal efficiencies postulated in Chapter 1 for each BMP on the condition that the applicant/developer agrees to the maintenance/monitoring requirements set forth in Chapter 3 of this manual (including outfitting of the BMP for monitoring as outlined in Appendix 2-8 and granting access rights for monitoring).

Other innovative or experimental approaches may be considered if the developer/engineer can demonstrate authoritative engineering references to support their use and the applicant agrees to monitor and demonstrate the actual pollutant removal performance of the BMP at his expense.

II. CONCEPT OF ULTRA-URBAN BMPs

Stormwater quality management in the ultra-urban environment involves the collection, pretreatment, storage and treatment to remove pollutants of a specific quantity of the most polluted stormwater. **Figure 2-1** illustrates this off-line ultra-urban BMP concept. In Virginia, the minimum quantity of stormwater to be treated is the **first one-half inch of runoff from the impervious areas on the site -- the Water Quality Volume (WQV).**

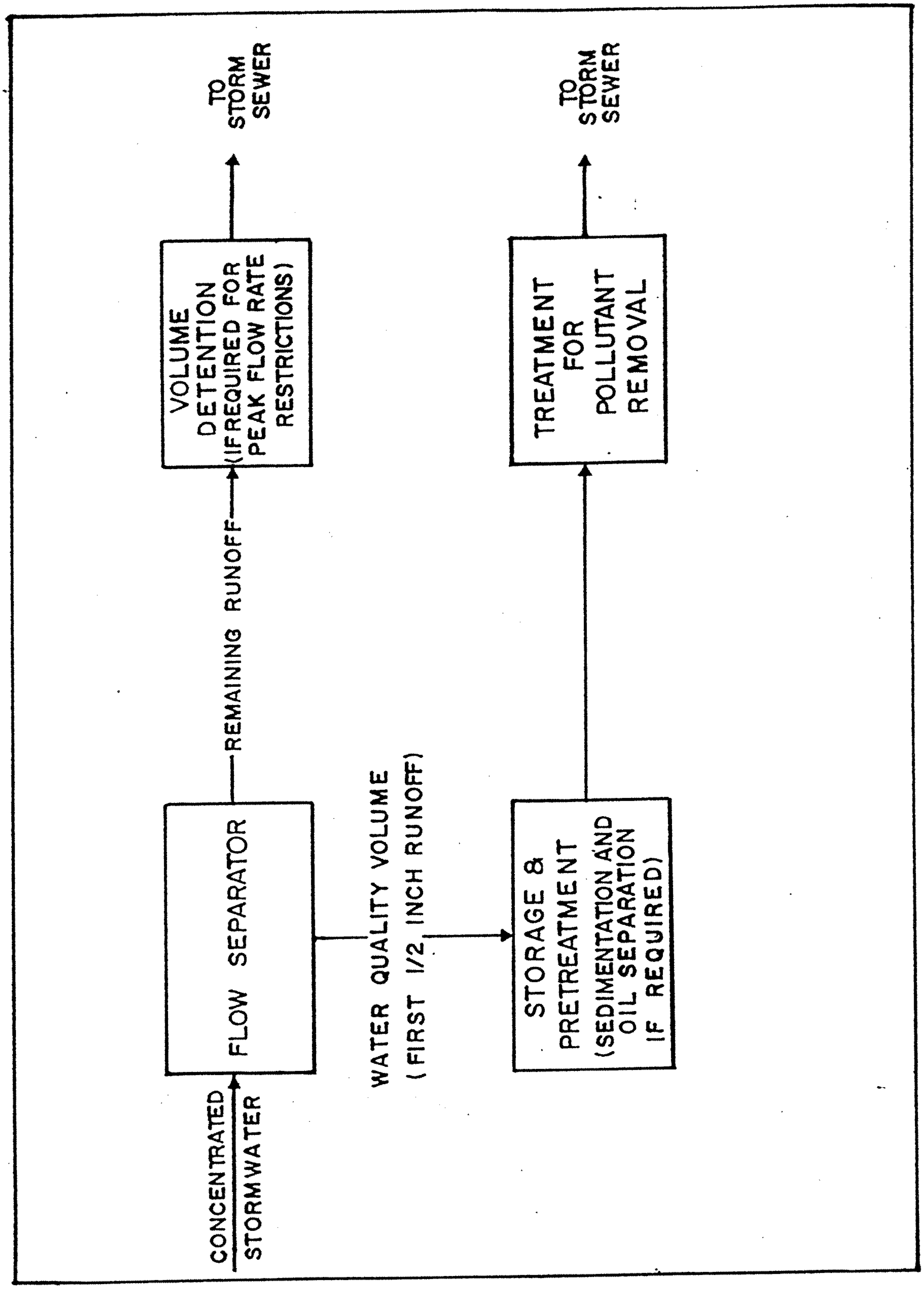


FIGURE 2-1 GENERAL CONFIGURATION OF AN OFF-LINE ULTRA-URBAN BMP.

A) Capturing the Water Quality Volume

The WQV may be computed using data from Calculations Worksheets A or B from Chapter 1 as follows:

$$WQV = I_a \times 43{,}560 \times 0.0417 \qquad (2\text{-}1)$$

where:

WQV is the Water Quality Volume in cubic feet,

I_a is the Area of impervious surface on the contributing watershed in acres,

43,560 is the number of square feet in an acre, and

0.0417 feet is the first half-inch of runoff.

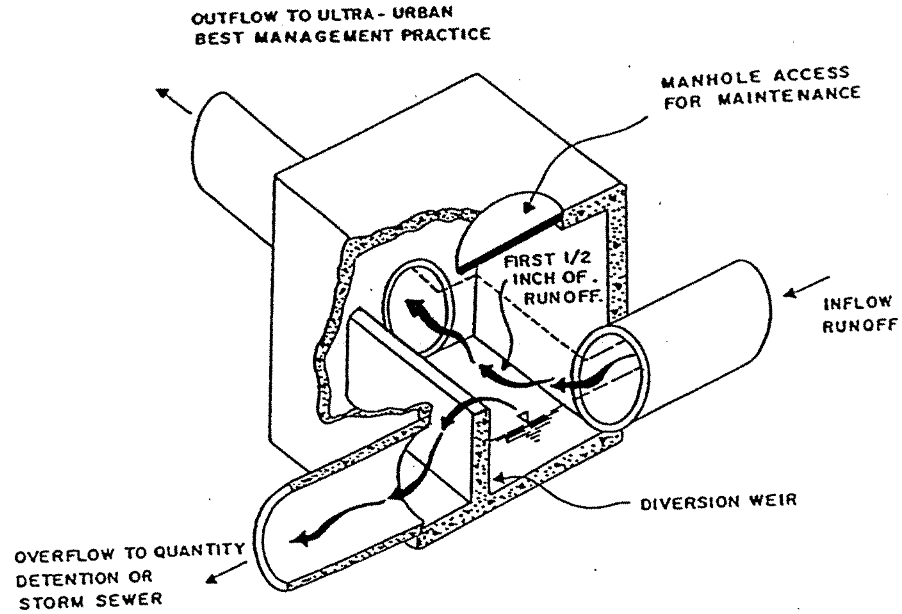
Reducing the constants yields:

$$WQV = 1816\ I_a \qquad (2\text{-}2)$$

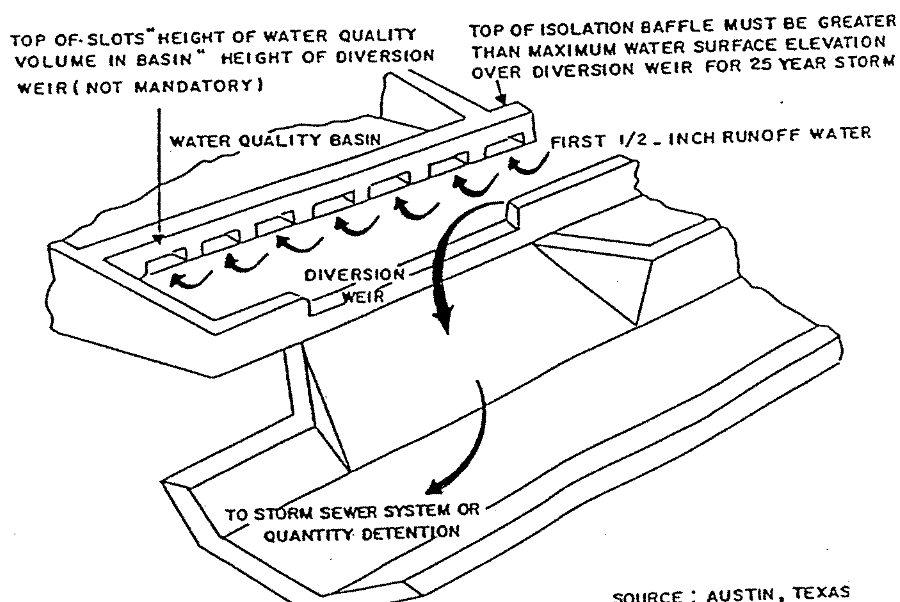
Capture and isolation of the WQV is typically achieved by isolation and diversion baffles and weirs. A typical approach for achieving isolation of the WQV is to construct an isolation/diversion weir in the stormwater channel or pipe such that the height of the weir equals the height of the water in the BMP when the entire WQV is being held. When additional runoff greater than the WQV enters the stormwater channel or pipe, it will spill over the isolation/diversion weir, and mixing with water stored in the BMP will be minimal. **Figure 2-2** shows two examples of these structures (Source: City of Austin, Texas).[1]

In many instances, it may be more efficient to build a flow splitter/bypass facility into the structure of the BMP itself by providing an overflow weir or orifice or a bypass pipe which conveys overflow from a collection/sedimentation chamber to a clearwell and thence to the storm sewer. Where retention of hydrocarbons is a concern, provision of a hooded (inverted elbow) pipe or orifice or the use of a commercial catch basin trap is usually required. Inverted elbows or catchbasin traps should penetrate the pool surface by 1/3 of the pool depth but at least one foot.

In Alexandria, bypass weirs, orifices, or pipes shall be designed to pass the peak flow rate of the 10-year storm (7 in./hr., 10 min. TOC).



PIPE INTERCEPTOR ISOLATION/DIVERSION STRUCTURE



SURFACE CHANNEL INTERCEPTOR ISOLATION/DIVERSION STRUCTURE

FIGURE 2-2 -- EXAMPLES OF ISOLATION/DIVERSION STRUCTURES

When designing overflow weirs, size the weir using the formula:[2]

$$Q_{10} = C_d(2/3)(2g)^{0.5}H^{0.667} \qquad (2\text{-}3)$$

where,

Q_{10} = peak flow rate for the 10-year storm in cfs
C_d = coefficient of discharge
g = acceleration of gravity (32.2 ft./sec.^{2}
H = the depth of ponded water above the crest of the weir in ft. (a minimum of 2" in Alexandria)
L = length of the weir in ft.

With the commonly used C_d of 0.62, this formula reduces to:

$$Q_{10} = 3.33LH^{0.667} \qquad (2\text{-}4)$$

When the length of the crest of the weir is less than the width of the channel or capture/sedimentation chamber, substitute the value (L - 0.1nH) for L in the above formulae (n = the number of end contractions, normally 2).

When a hooded overflow orifice is employed, use the orifice formula to size the overflow:[2]

$$Q_{10} = C_dA\ (2gh_{10})^{0.5} \qquad (2\text{-}5)$$

where Q_{10}, g, and C_d are as for the weir formula above, and,

A = area of the orifice in ft.^{2}
H_{10} = depth of ponded water above the flow line of the orifice

When a bypass pipe is employed, use Manning's equation to size the overflow pipe:[2]

$$V = \frac{1.49}{n} \times (R_h{}^{0.667})(S^{0.5}) \qquad (2\text{-}6)$$

where,

V = velocity of flow in fps
n = roughness coefficient (use 0.013 for concrete, 0.015 for PVC pipe, and 0.021 for corrugated metal)
S = slope of the pipe (energy gradient)(minimum 0.005)
R_h = the hydraulic radius in ft. = cross-sectional area of the pipe in ft.^{2} divided by the inside circumference of the pipe (wetted perimeter) in ft.

B) Pretreatment Requirements

Several conventional BMPs, such as buried infiltration devices, and most unconventional BMPs require some type of pretreatment system to remove excessive sediments which would result in premature clogging failure of the BMP and, in some cases, petroleum hydrocarbons. Pretreatment devices may be installed either at the point of collection or after separation of the WQV. They may be either separate devices or an integral part of the BMP itself.

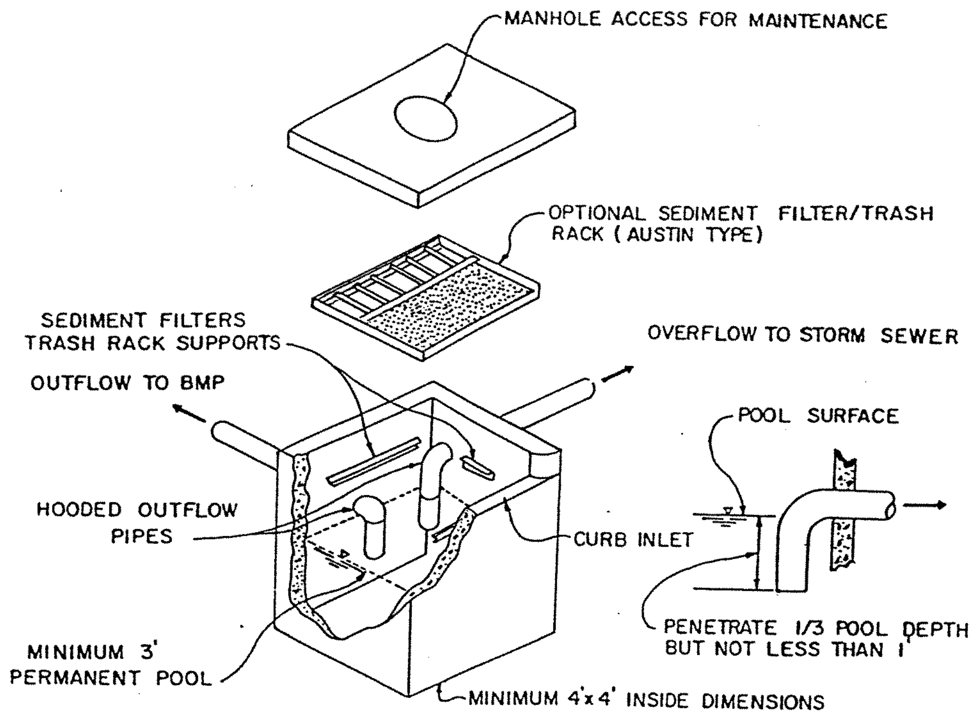
1) Water Quality Inlets (WQIs)

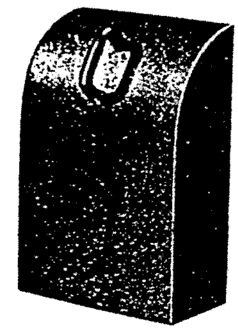
Water quality inlets (WQIs), or oil/grit separators (OGSs), were developed for the removal of sediment and hydrocarbon loadings which are frequently concentrated in parking lots and other areas where there is a great amount of vehicular traffic. Pages 7-9 through 7-12 of the NVPDC Northern Virginia BMP Handbook (NVBMPHB)[3] set forth information and design criteria for WQIs. For the reasons enumerated on page 7-10 of the NVBMPHB and recent research by the Washington Council of Governments,[50] WQIs **shall be assumed to have no pollutant removal efficiency** for applications within Alexandria.

Water quality inlets are usually used as a pre-treatment device before stormwater is conveyed into a storm sewer or an infiltration device. In Alexandria, they will be used with certain of the **off-line** unconventional BMPs described below. Commercial OGSs such as "STORMCEPTOR" or those based on swirl concentrator principles are also acceptable for such purposes.

Three-chamber WQIs designed for use in Montgomery County, Maryland and the District of Columbia are shown on pages 7-9 and 7-10 of the NVBMPHB. The District of Columbia uses a single chamber WQI, or sump catchbasin, for use as pretreatment for infiltration devices. Alexandria uses similar sump catchbasins with metal gas traps in its combined sewer watersheds. **Figure 2-3A** illustrates a single chamber WQI developed for use in Alexandria for pretreatment with small ultra-urban BMPs (the hooded large storm overflow pipe may be replaced with a commercial catch basin trap).

The chambers of a WQI must be cleaned out **at least every three months** to prevent the pollutants therein from reaching concentrations which would constitute a hazardous waste. This can be done by vacuum pumping or siphoning of the permanent pool. Accordingly, the WQI





Commercial
Catch Basin
Trap

FIGURE 2-3A -- **SINGLE CHAMBER WATER QUALITY INLET (SUMP CATCHBASIN)**

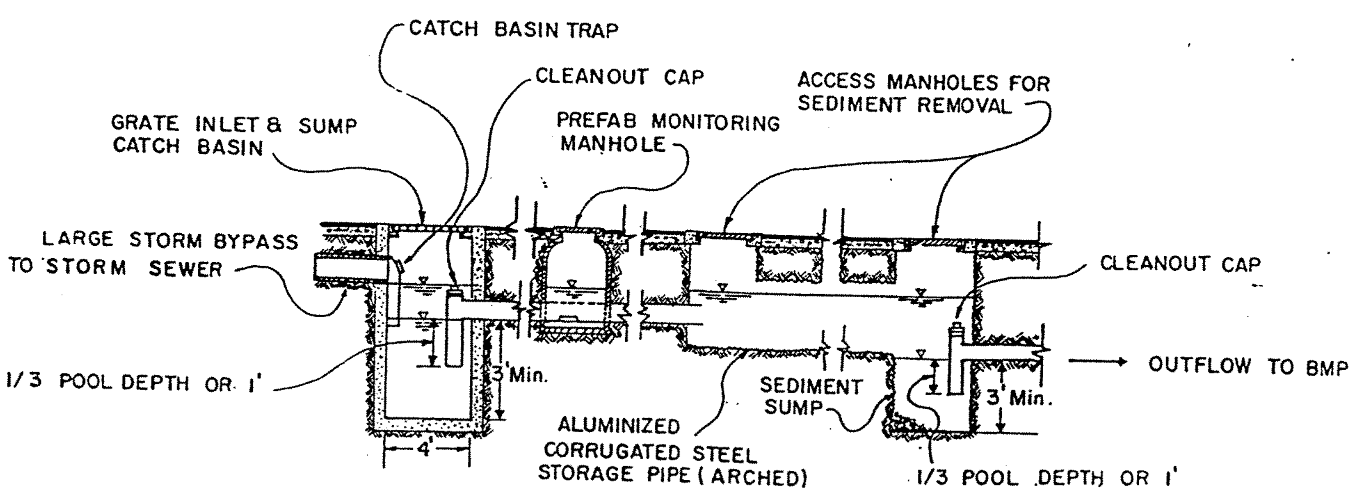


FIGURE 2-3B -- **UNDERGROUND STORAGE PIPE WITH GRATED SUMP INLET**

should be positioned where truck-mounted vacuum equipment will have ready access to the manhole openings in the top. After pumping, the structure must be refilled to a four (4) foot depth with clean water to restore the water seal for containing hydrocarbons.

2) Presettling Basins or Chambers

Sedimentation basins have traditionally been the first step in water or wastewater treatment. Where site conditions allow, presettling basins may provide a low cost approach to removal of sediments which can cause premature failure from sediment clogging of BMPs such as infiltration devices or filter systems. In situations where space is not a problem, presettling basins may be built directly into the ground. In the ultra-urban environment, where space utilization is an important economic consideration, underground presettling chambers may provide a more feasible solution.

The **minimum** area for presettling basins may be sized using the following formula derived by the Washington State Department of Ecology from the Camp-Hazen equation:[4]

$$A_s = -(Q_o/w) \times Ln(1-E) \quad \text{where} \qquad (2-7)$$

E = Trap efficiency = the fraction of suspended solids to remove; set equal to 0.8 to .95 (= 80% to 95% removal efficiency)
w = Settling velocity of target particle; silt is recommended using a settling velocity of 0.0004 ft/sec.
A_s = Surface area of presettling basin in sq. ft.
Q_o = Average release rate from the presettling basin
Ln is the natural logarithm

The average release rate, Q_o can be calculated by dividing the runoff treatment volume (usually WQV = 1816 I_a) by the detention time, t_d:

$$Q_o = \frac{WQV}{t_d} \qquad (2-8)$$

the Camp-Hazen equation then reduces to:

$$A_s = \frac{(-1816\ I_a) \times \ln(1-E)}{(t_d \times 3600\ \text{sec/hr}) \times 0.0004\ \text{ft/sec}} \qquad (2-9)$$

$$A_s = \frac{-1260 I_a \times \ln(1-E)}{t_d} \quad \text{where } t_d \text{ is in hrs. } I_a \text{ is in acres} \qquad (2-10)$$

Note that t_d must be set to allow total emptying of the BMP system within a maximum of 40 hours. Further sizing and design considerations for presettling or sediment basins are discussed on pages 2-28 through 2-36 below and in Appendix 2-5.

3) Grassed Filter Strips or Swales

Grassed swales or filter strips are commonly used to pretreat stormwater that will be processed in infiltration systems.[2] In selected circumstances, they may also be used as pretreatment for filter systems.[4] Properly designed grassed swales with a 200 foot length have demonstrated an average removal efficiency of 83 percent for suspended solids and 29 percent for total phosphorous in tests in the State of Washington.[5] However, TSS removal dropped to 60 percent for a 100-foot grassed swale. Nitrogen removal was **negative** in both cases for eleven of twelve storms monitored. The recommendations of this report are contained in Appendix 2-6.

Methodologies for designing grassed swales and grassed filter strips based on the above study have been developed by the Washington State Department of Ecology.[3] When designing grassed swales or grassed filter strips for use in Alexandria, these Washington DOE criteria and procedures, which are contained in Appendix 2-7, will be utilized.

C) Storage of the WQV

Following isolation of the WQV and pretreatment to remove sediments and other pollutants, water must be stored until it can be processed in the primary treatment device (up to 40 hours in Alexandria). Creating up to 1816 cubic feet of water storage per impervious acre on the site is often the most costly item in the overall BMP system. In some cases, as with sedimentation basins, storage may be combined with pretreatment. In others, separate storage galleries of round or arched-section pipe may be required. **Figure 2-3B** depicts underground pipe storage fed by a grated sump inlet. Some ultra-urban BMPs combine pretreatment, storage and primary treatment in a single underground vault.

D. Treatment of the WQV

Most of the ultra-urban BMPs described in this chapter employ intermittent sand filters. Originally developed during the 1800's for treating both water supplies and wastewater, intermittent sand filters have recently enjoyed a renaissance in the treatment of small wastewater flows.[6] The pollutant removal processes at work in intermittent sand filters involve physical, chemical and biological mechanisms. In addition to straining and sedimentation of suspended particles between the sand grains, chemical sorption on the grain surfaces and biological transformations may also play significant roles.[6]

The State of Florida and the City of Austin, Texas, pioneered the use of sand filters in the treatment of stormwater runoff. Sand filter systems for use as BMPs in Alexandria shall be sized using the Austin Sand Filter Formula derived from Darcy's Law by the Austin Environmental and Conservation Services Department (see Technical Notes, page 2-A1-1, for derivation).[1]

$$A_f = \frac{I_a H d_f}{k(h+d_f)t_f} \qquad (2\text{-}11)$$

where,

A_f = surface area of sand bed (acres or sq. ft.)
I_a = Impervious drainage area contributing runoff to the basin (acres or sq. ft.)
H = runoff depth to be treated (ft.)
d_f = sand bed depth (ft.)
k = coefficient of permeability for sand filter (ft/hr)
h = average depth (ft.) of water above surface of sand media between full and empty basin conditions (1/2 max. depth)
t_f = time required for runoff volume to pass through filter media (hrs.)

In Alexandria, the following values shall be used when designing sand filter systems:

$I_a H$ = the Water Quality Volume (WQV in $ft.^3 = 1816\ I_a$). (I_a in acres)
t_f = 40 hours (maximum)
k_{fs} = 3.5 feet per day for systems will full sedimentation protection preceding the filter (at least 95% silt removal efficiency when computed by the Camp-Hazen equation or when treating roof water only).
k_{ps} = 2.0 feet per day for systems with less than full sedimentation protection preceding the filter.

These "k" values were computed by Austin engineers based on observations of the actual performance of that city's sand filtration basins (see Technical Notes, pages 2-A1-2 and 2-A1-3, for further discussion).

With the specified "k" values, the formula for sand filter systems with **full sedimentation protection** reduces to:

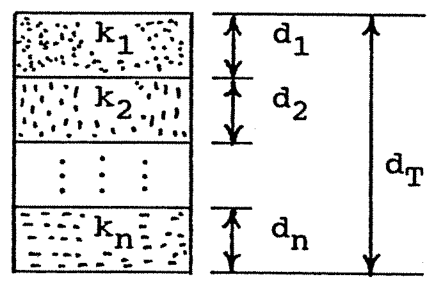
$$A_{f(FS)} = \frac{310 I_a d_f}{(h + d_f)} \quad \text{where } A_f \text{ is in ft.}^2 \text{ and } I_a \text{ is in acres} \qquad (2\text{-}12)$$

For sand filter systems with **partial sedimentation protection**, the formula reduces to:

$$A_{f(PS)} = \frac{545 I_a d_f}{(h + d_f)} \quad \text{where } A_f \text{ is in ft.}^2 \text{ and } I_a \text{ is in acres} \qquad (2\text{-}13)$$

When designing filters using a combination of sand and other filter media, compute a weighted average "k" value using the formula:[7]

$$k_T = \frac{d_T}{\frac{d_1}{k_1} + \frac{d_2}{k_2} + \ldots + \frac{d_n}{k_n}} \qquad (2\text{-}14)$$

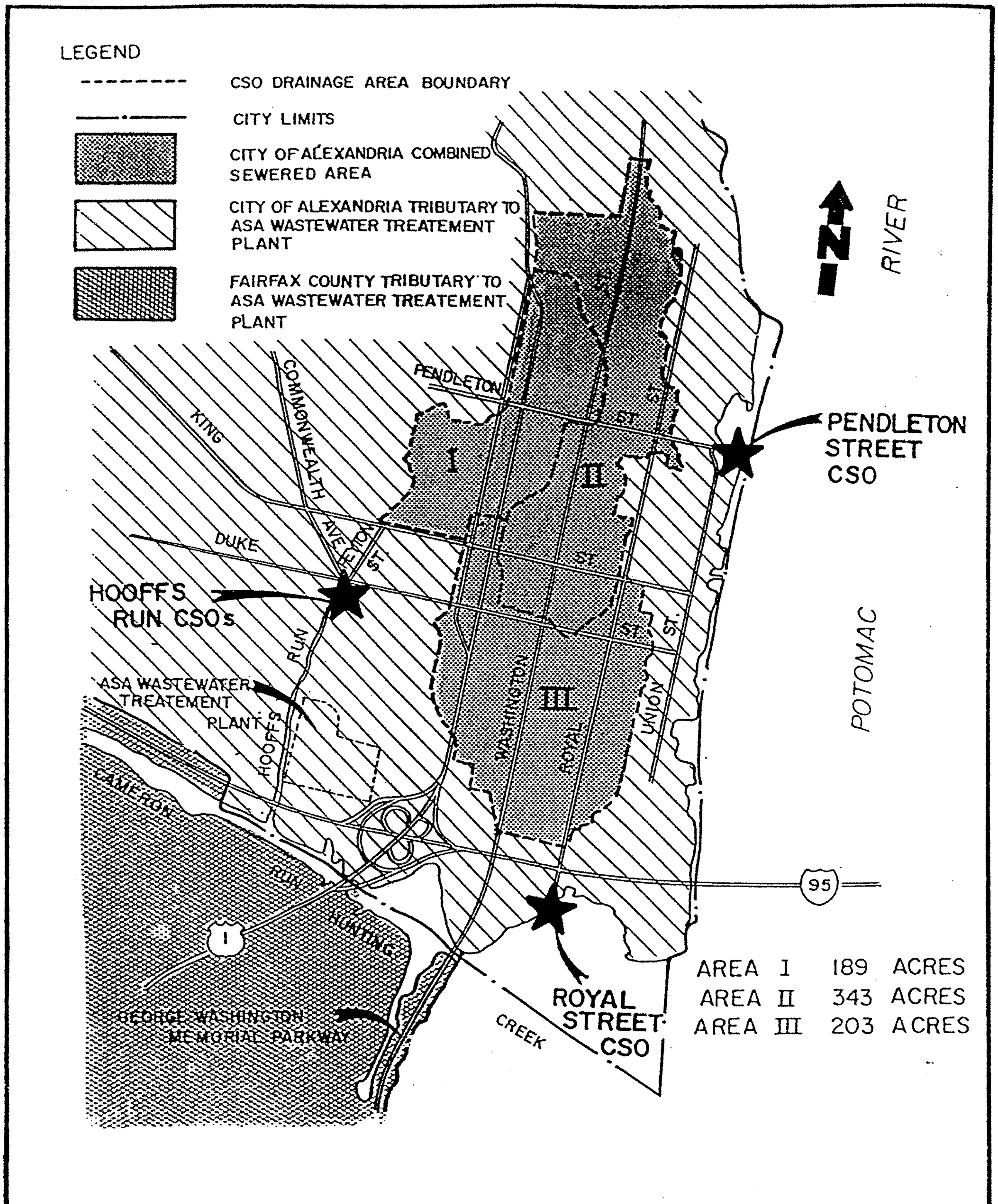


Pending completion of local monitoring studies to determine the removal efficiencies of intermittent sand filters with Virginia sands treating acid rains, Alexandria conservatively recognizes a phosphorous removal efficiency of 40 percent for sand filters designed with these formulae based on a long term Austin, Texas, monitoring program.[8]

III. UNIQUE REQUIREMENTS IN THE COMBINED SEWER WATERSHEDS

The older areas of Alexandria contain sections where storm and sanitary sewer services are combined into a single system of conduits. The three combined sewer watersheds, which total approximately six miles of combined sewer lines, are shown on **Map 2-1.**

During heavy or prolonged rainstorms, the three overflows (CSOs) may discharge into streams since existing interceptor systems (collector pipes) are not adequate to accommodate a significant volume of runoff in addition to the dry weather flow that needs to be directed to the wastewater treatment plant. Stormwater management in the combined sewer areas must address both the basic requirements of the Chesapeake Bay Preservation Ordinance and the necessity to assure that development does not increase the quantity or degrade the quality of combined sewer overflows. Unless otherwise approved by the Director, capture and storage of the WQV for later release into the combined sewers at a time when overflows will not occur will be **mandatory** in all cases where the City Chesapeake Bay Preservation Ordinance requires a BMP.



MAP 2-1 CITY OF ALEXANDRIA COMBINED SEWER AREAS

IV. WATER QUALITY VOLUME (WQV) STORAGE TANKS

A) Facility Description

This concept was originally developed for use in the City's combined sewer watersheds, but it may be applied in other situations where WQV runoff will not be routed into the storm sewer (such as landscaping irrigation systems or "gray water" toilet flushing systems). Stormwater runoff is channeled through an isolation/diverter chamber (see **Figure 2-2** on page 2-4), which directs the WQV into an underground or basement collection tank. The remaining runoff then overflows to enter a peak flow rate reducer or exit directly into the stormwater collection system. Water in the WQV storage tank is later released into the combined sewer system after the danger of overflows into waters of Virginia is past. The tank is emptied either by a pump or automatic valve controlled by a timer or telemetry to assure that no water is released while CSOs are occurring or in periods when inflow and infiltration are taxing the capacity of the wastewater treatment plant.

WQV storage reservoirs may be either prefabricated tanks or vaults fabricated on-site from such materials as Portland Cement concrete. Either single or multiple tanks may be employed. For instance, one tank might process roof runoff water while another processes runoff from other impervious areas such as a parking garage upper level. **Figure 2-5** illustrates several possible combinations.

B) Pollutant Removal Rates

When WQV water is discharged directly into a combined or the sanitary sewer or used in gray-water flushing systems, the pollutant removal efficiency of the system becomes that of the receiving wastewater treatment plant. The phosphorous removal capacity of such plants is typically in the 95-100 percent range. The plants are required to achieve receiving water quality standards in order to retain their National Pollution Discharge Elimination permits from state water control boards. The phosphorous removal efficiency of the Alexandria Sanitation Authority plant is 98.7%. When the WQV water is reused and retained on-site for landscape irrigation, pollutant removal may approach 100% **if the water is not allowed to escape from the site.**

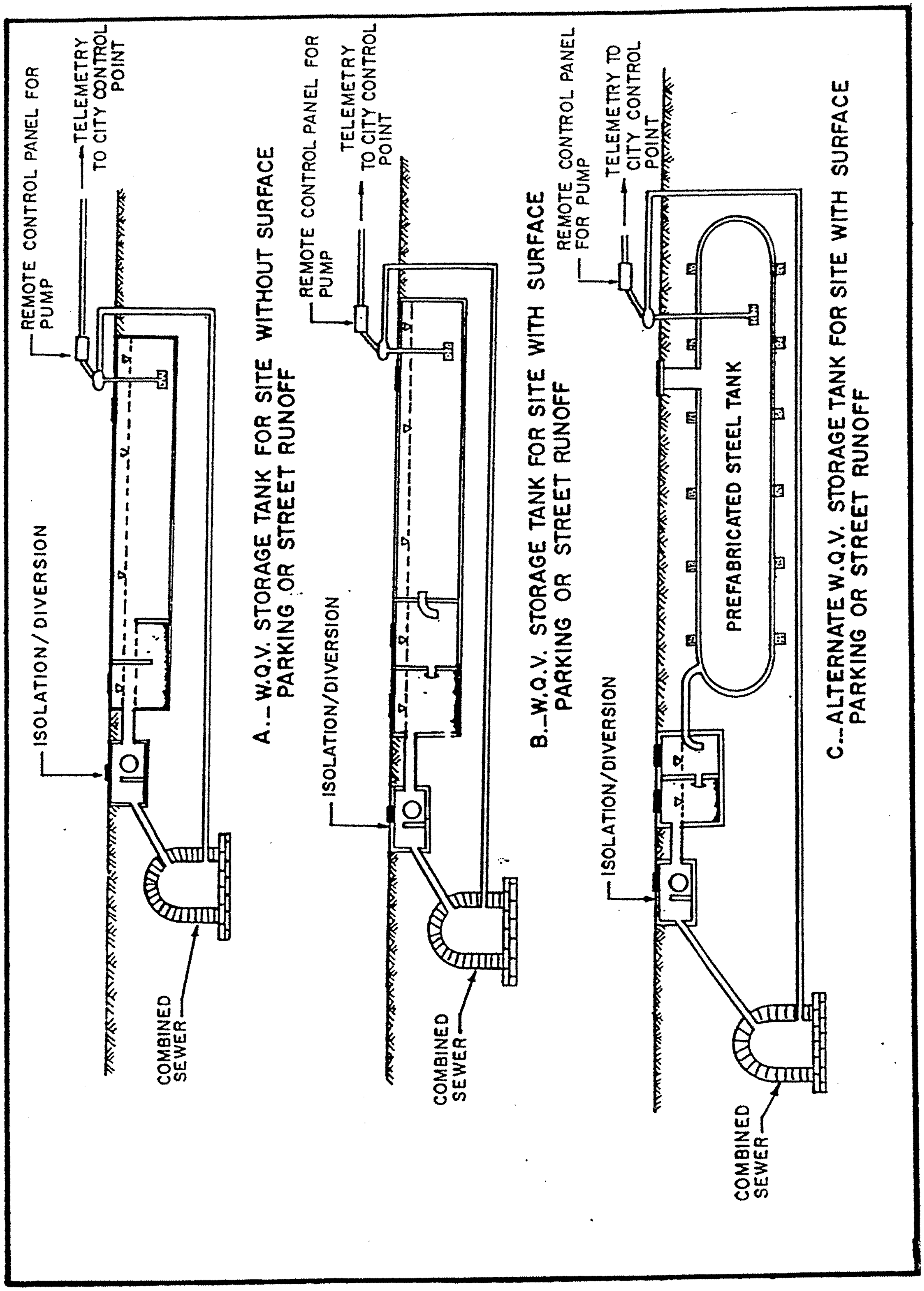


FIGURE 2-5 ALTERNATE CONFIGURATIONS FOR WATER QUALITY VOLUME STORAGE TANK.

C) Pretreatment Requirements

Some method must be provided to remove sediments before storage and treatment of the WQV. Sediment-trapping baffles at the inflow end of the WQV tank may be sufficient for systems dealing solely with roof runoff. In **all** cases where the WQV Storage Tank must process runoff from streets or vehicular parking spaces which are exposed directly to the weather, the runoff **must** be treated in a water quality inlet to provide a measure of petroleum hydrocarbon removal before the runoff is allowed to enter the storage tank.

D) Design Considerations

1) Applicability

Unless otherwise approved by the Director, WQV Storage Tanks will be employed in all cases in the combined sewer watersheds when a BMP is required by the provisions of the CBPO. WQV Storage Tanks may be used anywhere in the City when the nutrient-rich detained water will be transferred to another cistern and used **solely on-site** for such purposes as landscape watering. They may also be proposed for use in areas other than the combined sewer watersheds if the applicant can obtain a permit from the Alexandria Sanitation Authority to connect the WQV Storage Tank outfall to the sanitary sewer system (ASA will require that a report of chemical sampling of WQV water from a similar facility in the general area accompany the permit application).

2) Practicality

WQV Storage Tanks are especially practicable in heavily built up areas where land values are very high. They are also useful in combined sewer watersheds in minimizing the impact of new or redevelopment on existing CSOs.

3) Maximum Drainage Areas for WQV Storage Tanks

WQV Storage Tanks are not intended to be used as an alternative to traditional Best Management Practices on sites where sufficient room exists to install such devices as wet ponds or extended dry detention. Maximum allowable size for SQV Storage Tanks will also be a function of wastewater treatment plant capacity. Normally, WQV detention with wastewater treatment plant processing should be limited to two acres or less.

4) **Position in Stormwater Management System**

WQV Storage Tanks must always be placed off-line to the stormwater collection and disposal system. This requirement precludes both flushing of collected sediments and nutrient-rich water into the storm sewer and short-circuiting of the total stormwater flow into the sanitary sewer.

5) **Discharge Requirements and Telemetry**

As with other temporary detention Best Management Practices, WQV Storage Tanks must be emptied within 48 hours to be ready to accept subsequent storms. When discharge into the combined or sanitary sewers is involved, pumps should be sized to completely empty the tank within two hours. When the City or Sanitation Authority requires that plant operators control the initiation of discharge, the necessary remote controls and telemetry to the applicable control point must be provided. When centralized control is not required, a timing device must be installed to assure discharge at off-peak hours specified by the Director.

E) **WQV Storage Tank Design Procedures**

1) **Volume Requirement**

Determine the WQV using equation 2-2 from page 2-3:

$$WQV = 1816\ I_a \qquad (2\text{-}3)$$

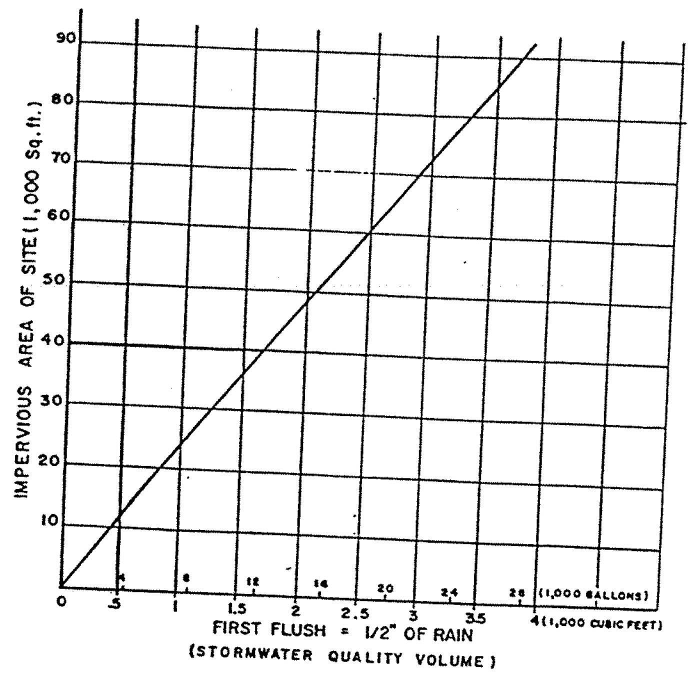
where WQV is in cubic feet, and

I_a = the impervious area on the contributing watershed in acres.

(Multiplying WQV by 7.481 converts the storage to gallons).

Figure 2-6 graphs these relationships to provide for ready computation of storage requirements. Note that a 5000-gallon service station tank would contain the water quality volume from 16,041 square feet of impervious area.

Figure 2-6: Graph of Stormwater Quality Volume Storage Requirements



2) Size the Water Quality Inlet

Use the procedures in the <u>NVBMPHB</u>, pages 7-9 through 7-12.

3) Determine the Dimensions of the Storage Tank

The relationships of the various dimensions has no particular affect on the functioning of the storage tank. Space restrictions of the site will likely dictate dimensional relationships. Overall dimensions must be adjusted to recognize that the permanent pool in the WQI and the depth beneath the pump intake are "dead storage" to be accommodated over and above the WQV. the total storage required is expressed by the formula:

$$V_T = WQV = V_{PP} + V_{BP} \qquad (2\text{-}14)$$

where, in similar units,

V_T = Total Storage Volume

V_{PP} = Volume of the Permanent Pool

V_{BP} = Volume below the Pump Intake (in the Storage Chamber)

Figure 2-7 illustrates a WQV Storage Tank for use with a parking garage in the combined sewer watershed area. Note the Water Quality Inlet at upstream end of the system and the access hatches for cleaning the facility. The WQV storage capacity of the system is the sum of the "dry" storage volume in the WQI plus the storage volume of the downstream tank. The permanent pool in the WQI and the volume beneath the pump intake are additional dead storage.

F) Construction and Maintenance Requirements

A Maintenance Agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with the Chapter 3 of this manual must be executed by the developer/owner **before** the Final Site Plan for the development will be released for construction.

Construction and maintenance requirements for Austin Sand Filter Systems are delineated in detail on pages 2-A3-1 and 2-A3-2. These requirements shall be reproduced <u>verbatim</u> on the Stormwater Management Plan sheets of the Final Site Plan.

G) Use in Integrated Stormwater Management

The Alexandria CBPO requires that the post-development peak flow rate of runoff from a regulated development site cannot exceed the pre-development peak flow rate for either a two-year storm event or a ten-year storm event. The erosion and sediment control ordinance contains similar peak-shaving requirements. Off-line WQV Storage Tanks may be utilized to assure stormwater quality followed by on-line peak-shaving devices such as oversized pipes with restricted release orifices.

H) Use of WQV Water for Toilet Flushing (the Gray-Water Concept)

Separate gray-water systems for toilet flushing are coming into use in many areas where potable water supplies are scarce. The City will entertain proposals of this nature. Permission of the ASA is also required. Collected water would be pumped into a gray-water tank or cistern to free the WQV Storage Tank to accept the next storm. Creating an economical source of gray-water would likely require the further collection of a portion or all of the additional stormwater runoff. A cross-connection with the potable water system protected in such a way as to preclude any possibility of back flow would be required for periods when the gray-water cistern was empty due to drought..PA

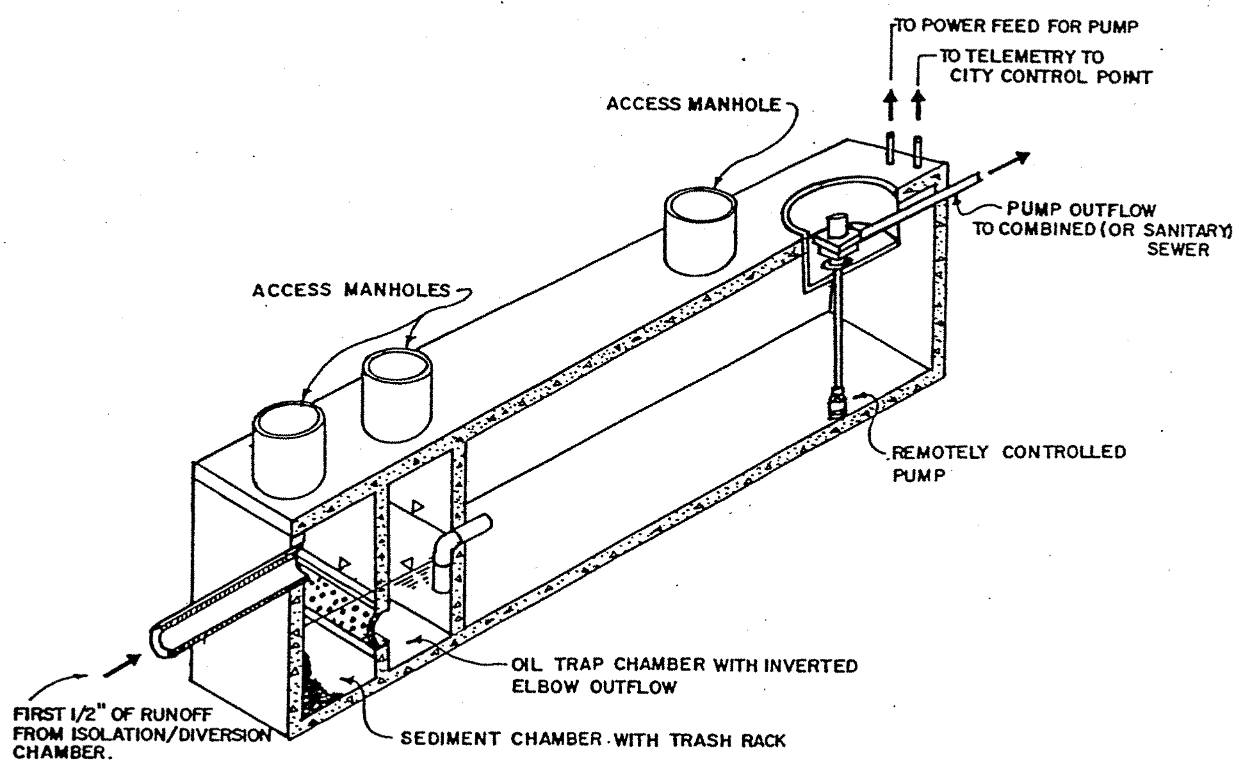


FIGURE 2 — 7 CENTERLINE CUTAWAY OF WQV STORAGE TANK FOR PARKING GARAGE

V. AUSTIN SAND FILTRATION SYSTEMS

A) Facility Description

The City of Austin, Texas, and the State of Florida have used similar basin sand filtration systems as stormwater BMPs for a number of years. Basin sand filters (BSFs) may be constructed inside a concrete shell, or, where conditions allow, be built directly into the terrain over a waterproof geomembrane. Alexandria will allow the use of BSFs conditioned on the developer's agreement to outfit the BMP for monitoring as outlined in Appendix 2-8 and allowing the City unlimited access for inspection and monitoring purposes (a maintenance/monitoring agreement must be executed prior to release of the final site plan). The following information was provided by the City of Austin Environmental and Conservation Services Department. The End Notes listed in this section are the references used in the Austin Environmental Criteria Manual[1].

In order to ensure the long-term effectiveness of sand filtration systems, it is necessary to protect the filter media from excessive sediment loading. The WQV runoff must therefore be routed through a sedimentation basin before treatment in the filtration basin; subsequent additional runoff is diverted to a stormwater detention basin if required to comply with the peak flow runoff restrictions of Article XIII of the Alexandria Zoning Ordinance (see Chapter 4). **Figure 2-8** illustrates this general configuration. Austin specifies two possible configurations of stormwater sand filtration systems:

2) Configuration 1 (Full Sedimentation)

In this configuration, sedimentation occurs in a presettling basin designed to hold the entire WQV and release it to the filtration basin over an extended draw-down period. **Figures 2-9A and 2-10** illustrate this system constructed in a structural concrete shell.

2) Configuration 2 (Partial Sedimentation)

In this configuration, the sedimentation chamber holds a minimum of 20 percent of the WQV and does not incorporate an extended draw-down period. This removes the heavier sediment and trash litter only and requires more intensive mainenance than the full sedimentation system. In order to compensate for the more rapid clogging of the filter media, a larger filter area is also required. **Figure 2-9B** depicts this configuration built directly into the terrain over a geomembrane.

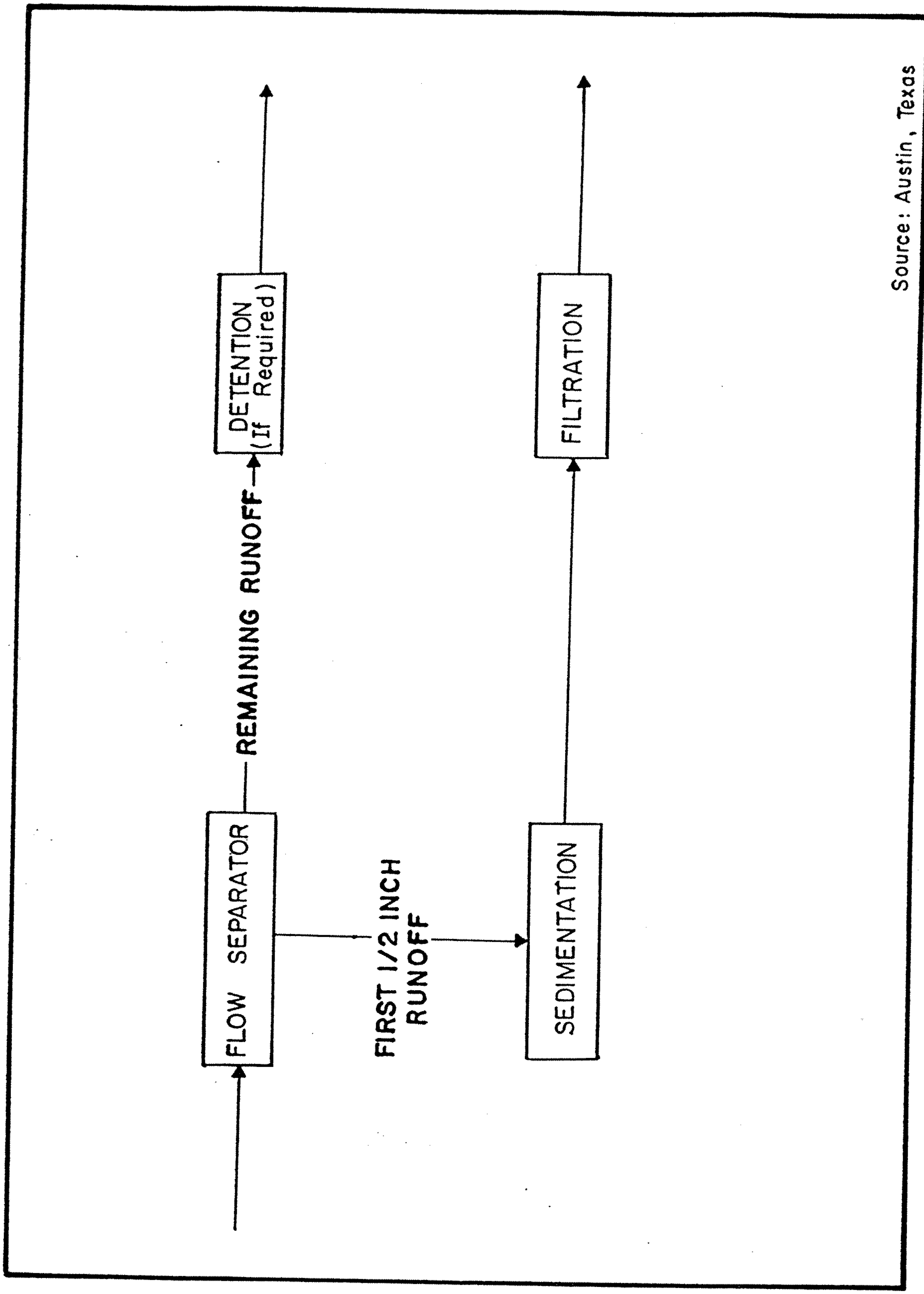
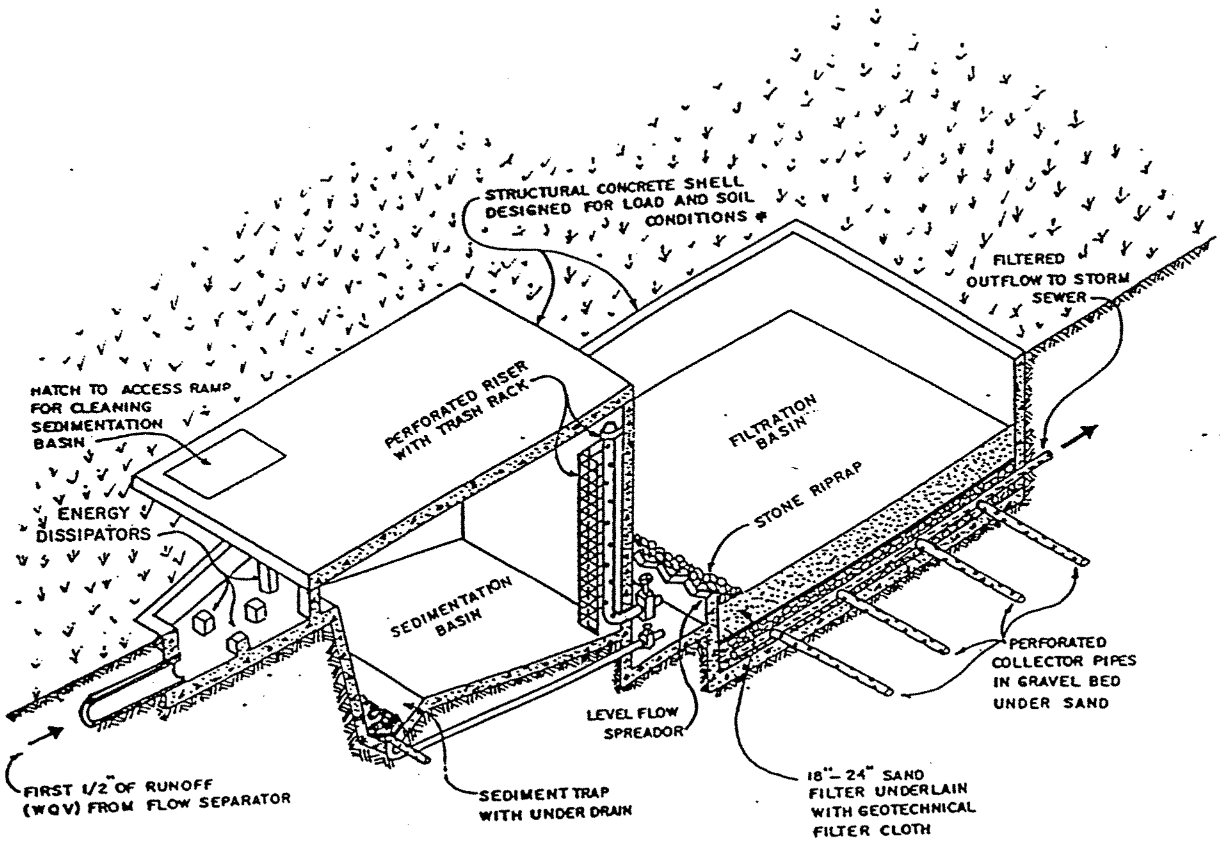


FIGURE 2-8 FILTRATION SYSTEM GENERAL CONFIGURATION



*NOTE: SEDIMENTATION POND MAY BE USED IN LIEU OF CONCRETE BASIN. FILTRATION BASIN MAY BE BUILT DIRECTLY INTO GROUND OVER AN IMPERVIOUS GEOMEMBRANE IF SOIL CONDITION ALLOW.

FIGURE 2-9A -- AUSTIN SAND FILTER WITH FULL SEDIMENTATION PROTECTION

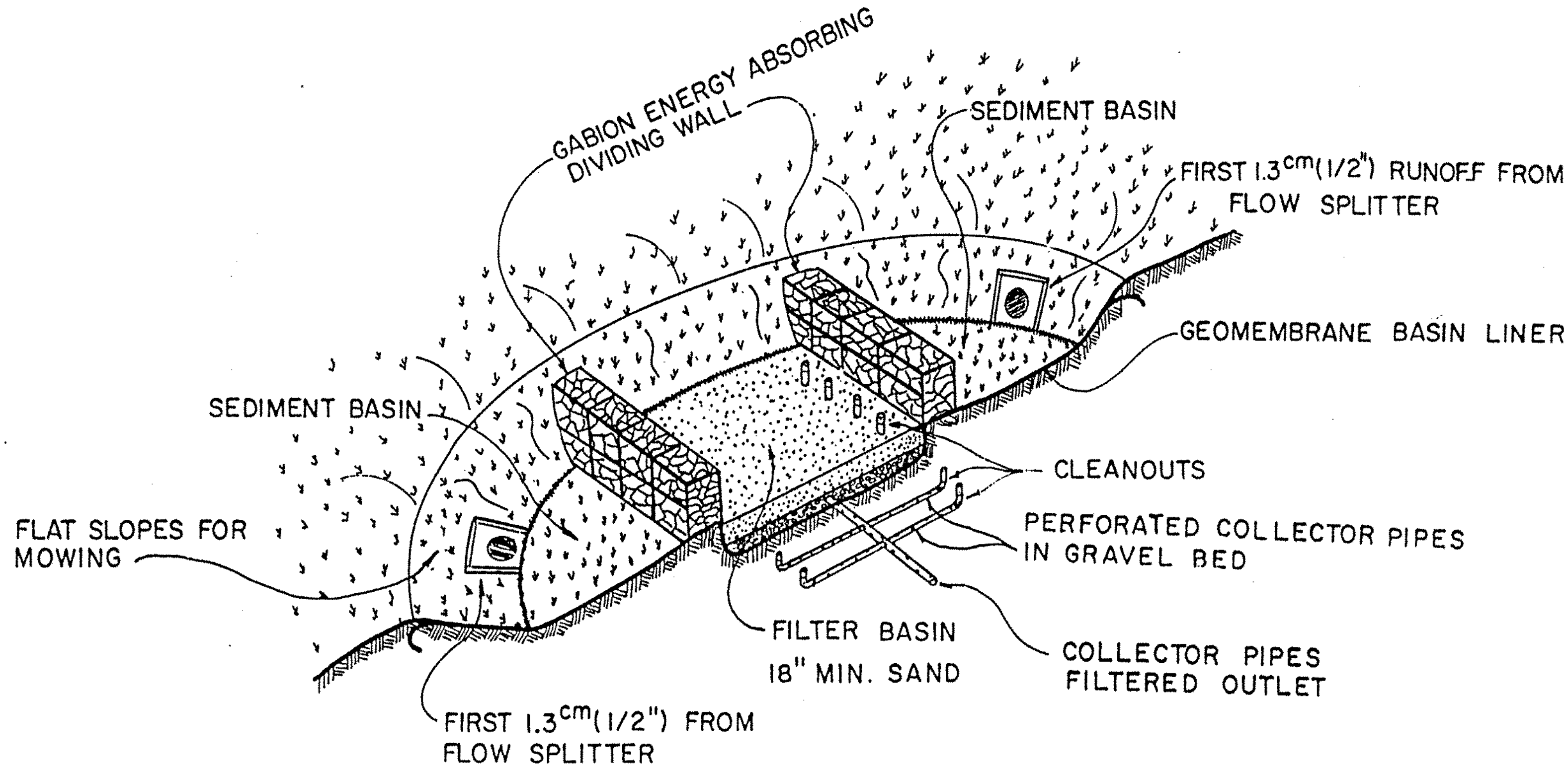


FIGURE 2-9B -- AUSTIN SAND FILTER WITH PARTIAL SEDIMENTATION PROTECTION

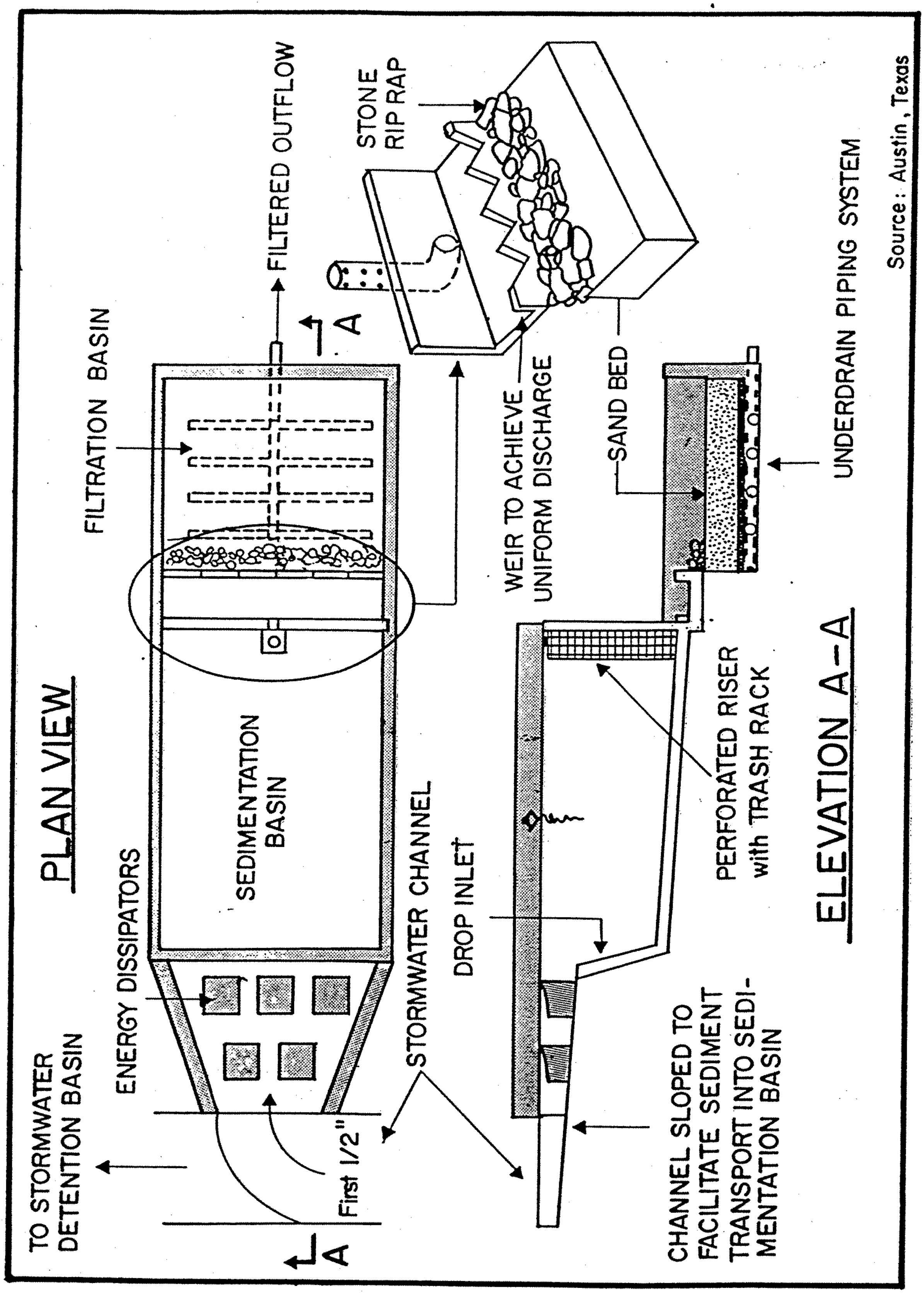


FIGURE 2-10 CONCEPTUAL FULL SEDIMENTATION — FILTRATION SYSTEM

B) **Pollutant Removal Rates.**

For filtration systems designed in accordance with the guidelines in this section, the following pollutant removal efficiencies are currently being recognized by the City of Austin based on their long-term monitoring program:[8]

Removal Efficiency	Fecal Coliform	TSS	TN	TKN	NO_3-N	TP04	BOD	TOC	Fe	Pb	Zn
(%)	37	87	32	62	na	61	52	62	86	81	80

If the developer agrees to equip the system for monitoring and grant the City unlimited access to the BMP for inspection and monitoring purposes, **Alexandria will recognize a 60 percent total phosphorous removal efficiency** for the Austin Sand Filtration System.

D) **Design Considerations**

1) **Applicability**

Austin Sand Filtration Systems should be considered for use on redevelopment sites where topography, space constraints, safety considerations, or high property values militate against the use of traditional BMPs. The relatively moderate phosphorous removal efficiency may not present a problem in redevelopment if the pre-development percentage of impervious cover was relatively high, such as in shopping center redevelopment. Sites of several acres where space is available for the surface filter system would be suitable. In a new development situation, however, total allowable post-development impervious cover if the filtration system served 100 percent of the site would be 72 percent.

2) **Practicality**

Sand filtration systems have a demonstrated record of success in Austin. Construction is fairly simple, and the sediment basin and filter surfaces are accessible for maintenance. Gaining sufficient gradient to allow gravity flow through the system may present a problem in the more flat areas of the City. In such cases, it may be necessary to use a pump to transfer the water from the sedimentation basin to the surface of the filter. The City of Austin provided the following cost data on sedimentation/filtration systems:

RESERVED FOR FUTURE USE

Table 2-1

COST OF AUSTIN SAND FILTRATION SYSTEMS
June 20, 1990

DRAINAGE AREA (ac)	WATER QUALITY VOLUME (c.f.)	COST/ACRE ($/ac.)	COST/CU. FT. ($/c.f.)	TOTAL COST ($)
1.0	1,815	13,613* 19,058#	7.50* 10.50#	13,613* 19,058#
2.0	3,630	8,440* 9,801#	4.65* 5.40#	16,880* 19,602#
5.0	9,075	5,136	2.83	25,682
10.0	18,150	3,812	2.10	38,115
15.0	27,225	3,086	1.70	46,283
20.0	36,300	2,723	1.50	54,450
30.0	54,450	2,360	1.30	70,785

Footnotes:

*Calculated from data provided by Murfee Engineers

#Calculated from data provided by Stormwater Management

All other values derived from combined data

3) Topography Considerations

The systems discussed in this section are all designed to function by gravity flow between the components. On sites which do not provide enough vertical relief to operate the ASF by gravity flow, the system must be augmented with a clear well and pumps to lift the stormwater from the sedimentation basin and sediment trap to the filter basin.

4) Accessibility for monitoring Equipment

The isolation/diversion structure, sediment basin, and filtered water outflow pipe, must be readily accessible for the installation of automatic monitoring equipment to measure both the chemical composition of the stormwater. Installation at the owner's expense of prefabricated monitoring manholes with integral flow measuring flumes and flow meter piping will be required unless other accessibility measures are approved by the Director, T&ES. See Appendix 2-8 for details. The maintenance/monitoring agreement required by Chapter 3 of this Supplement shall grant the T&ES and its contractors unlimited access to these facilities for the purposes of monitoring actual BMP performance.

E) Design Procedures for Full Sedimentation with Filtration

In this configuration, the sedimentation basin receives the WQV and detains it for a minimum draw-down time (time required to empty the basin from a full WQV condition) of 24 hours. The effluent from the sedimentation basin is discharged into the filtration basin.

1) Basin Surface Areas

Austin conducted a literature review of sedimentation basins (End Notes 9-25) and slow rate filters (End Notes 13 and 25) to establish design criteria.

For sedimentation basins, the removal of discrete particles by gravity settling is primarily a function of surface loading, "Q_o/A_s ", where "Q_o " is the rate of outflow from the basin and "A_s" is the basin surface area. Basin depth is of secondary importance as settling is inhibited only when basin depths are too shallow (particle resuspension and turbulence effects). For sedimentation, surface area is the primary design parameter for a fixed minimum draw-down time, t_d, of 24 hours. Removal efficiency, E, is also a function of particle size distribution. For design purposes, the particles selected for complete removal in the sedimentation basin are those which are greater than or equal in size to silt with the following characteristics: particle diameter 0.00007 foot (20 microns) and specific gravity of 2.65. These are typical values for urban runoff (see End Notes 10 and 22).

For filtration basins, surface area is the primary design parameter. The required surface area is a function of sand permeability, bed depth, hydraulic head and sediment loading. A filtration rate of 0.0545 gallons per minute per square foot has been selected for design criteria (10.5 feet per day or 3.4 million gallons per acre per day). This filtration rate is based on a Darcy's Law coefficient of permeability k =

3.5 feet per day, an average hydraulic head (h) of three (3) feet and a sand bed depth (d_f) of 18 inches, and a filter drawdown time, t_f of 40 hours. See Appendix 2-1 for an explanation of how the filtration rate and coefficient of permeability were determined. End Notes 26-29 provide additional information.

Substituting these values in the basic Austin Filter Formula (equation 2-11) yields:

$$A_f = I_a H/18 \qquad (2\text{-}15)$$

where A_f " is the minimum surface area of the filtration media in acres, "I_a" is the contributing impervious runoff area in acres and "H" is the runoff depth in feet (0.5 inch = 0.0417 feet when treating the WQV).

For the sedimentation basin, Austin uses a standard maximum pooling depth of ten feet and computes the required surface area, A_s in acres, with the formula:

$$A_s = I_a H/10 \qquad (2\text{-}16)$$

This area yields a silt removal efficiency of greater than a 95% when computed with the Camp-Hazen equation.

When treating the water quality volume, these formulae reduce to:

$$A_s = 0.0042 I_a \qquad (2\text{-}17)$$

$$A_f = 0.0023 I_a \qquad (2\text{-}18)$$

When designing for parameter values (h, d_f, runoff volumes, t_d, etc.) differing from those assumed by Austin, revert to the basic Austin Filter Formulae (equations 2-11, 2-12, and 2-13) shown on pages 2-9 and 2-11 and the Camp-Hazen Equation (2-10--page 2-8).

Greater sedimentation pond surface areas are acceptable as long as the depth of the sedimentation basin is a minimum of three (3) feet. In cases where topography limits sedimentation basin ponding depth to less than ten (10) feet, the required surface area of the basin will be greater than "A_s " in order to accommodate the water quality volume.

If sedimentation ponding depth is greater than ten (100) feet, then the surface area must be equal to $0.0042 I_a$. The Area/Depth Curves shown in **Figure 2-11** graphically solves these equations.

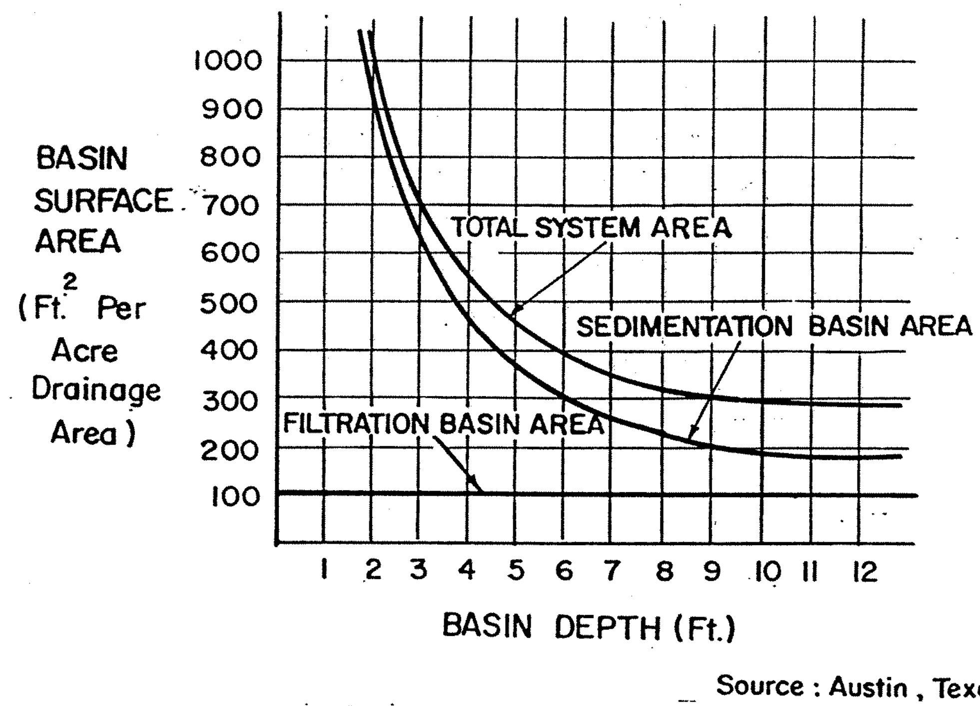


Figure 2-11
Sedimentation/Filtration Basin Surface Area Graphs
Full Sedimentation

2) **Basin Volumes**

The storage capacity of the sedimentation basin shall be greater than or equal to the water quality volume. It is recommended that a minimum 0.5 foot of freeboard above the maximum water surface elevation be provided. If desired, a sediment trap may be included at the bottom of the basin and may be credited with up to five (5) percent of the water quality volume.

The storage capacity of the filtration basin, above the surface of the filter media, should be greater than or equal to 20 percent of the water quality volume. This capacity is necessary in order to account for backwater effects resulting from partially clogged filter media.

3) Sedimentation Basin Details

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 maintenance requirements.

- o Inlet Structure - The inlet structure design must be adequate for isolating the water quality volume from the 25 year design storm and to convey the peak flow for the 25 year design storm past the basin. The water quality volume should be discharged uniformly and at low velocity into the sedimentation basin in order to maintain near quiescent conditions which are necessary for effective treatment. It is desirable for the heavier suspended material to drop out near the front of the basin; thus a drop inlet structure is recommended in order to facilitate sediment removal and maintenance. **Figure 2-10** presents an example of this type of system. Energy dissipation devices may be necessary in order to reduce inlet velocities which exceed three (3) feet per second.

- o Outlet Structure - The outlet structure conveys the water quality volume from the sedimentation basin to the filtration basin. The outlet structure shall be designed to provide for a minimum draw-down time of 24 hours. A perforated pipe or equivalent is the recommended outlet structure. The 24 hour draw-down time should be achieved by installing a throttle plate or other flow control device at the end of the riser pipe (the discharges through the perforations should not be used for draw-down time design purposes). The perforated riser pipe should be selected from the following table:

TA8LE 2-2

PERFORATED RISER PIPES

Riser Pipe Nominal Dia. (inches)	Vertical Spacing Between Rows (Center to Center in 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

This information is based on commercially available pipe. Equivalent designs are acceptable.

A trash rack shall 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. The bottom rows of perforations of the riser pipe should be protected from clogging. To prevent clogging of the bottom perforations it is recommended that geotextile fabric be wrapped over the pipe's bottom rows and that a cone of one (1) to three (3) inch diameter gravel be placed around the pipe (see Reference 75). If a geotextile fabric wrap is not used then the gravel cone must not include any gravel small enough to enter the riser pipe perforations. **Figure 2-12** illustrates these considerations.

o Basin liner

Impermeable liners may be either clay, concrete or geomembrane. If geomembrane is used, suitable geotextile fabric shall be placed below and on the top of the membrane for puncture protection. Clay liners shall meet the following specifications:

TABLE 2-3

CLAY LINER SPECIFICATIONS

Property	Test Method	Unit	Specification
Permeability	ASTM D-2434	Cm/Sec	1×10^{-6}
Plasticity Index of Clay	ASTM D-423 & D-424	%	Not less than 15
Liquid Limit of Clay	ASTM D-2216	%	Not less than 30
Clay Particles Passing	ASTM D-422	%	Not less than 30
Clay Compaction	ASTM D-2216	%	95% of Standard Proctor Density

Source: City of Austin

The clay liner shall have a minimum thickness of 12 inches.

If a geomembrane liner is used it shall have a minimum thickness of 30 mils and be ultraviolet resistant.

The geotextile fabric (for protection of geomembrane) shall meet the following specifications:

Property	Test Method	Unit	Specification	
Material	Nonwoven geotextile fabric			
Unit Weight		Oz/Sq.Yd.	8	(min.)
Filtration Rate		In/Sec	0.08	(min.)
Puncture Strength	ASTM D-751(Modified)	Lb.	125	(min.)
Mullen Burst Strength	ASTM D-751	Psi	400	(min.)
Tensile Strength	ASTM D-1682	Lb.	300	(min.)
Equiv. Opening Size	US Standard Sieve	No.	80	(min.)

Equivalent methods for protection of the geomembrane liner will be considered by the Department of Transportation and Environmental Services on a case by case basis. Equivalency will be judged on the basis of ability to protect the geomembrane from puncture, tearing and abrasion.

Concrete liners may be used for sedimentation chambers and for sedimentation and filtration basins. Concrete shall be at least five (5) inch thick Class A3 defined in the Virginia Department of Transportation Road and Bridge Specifications

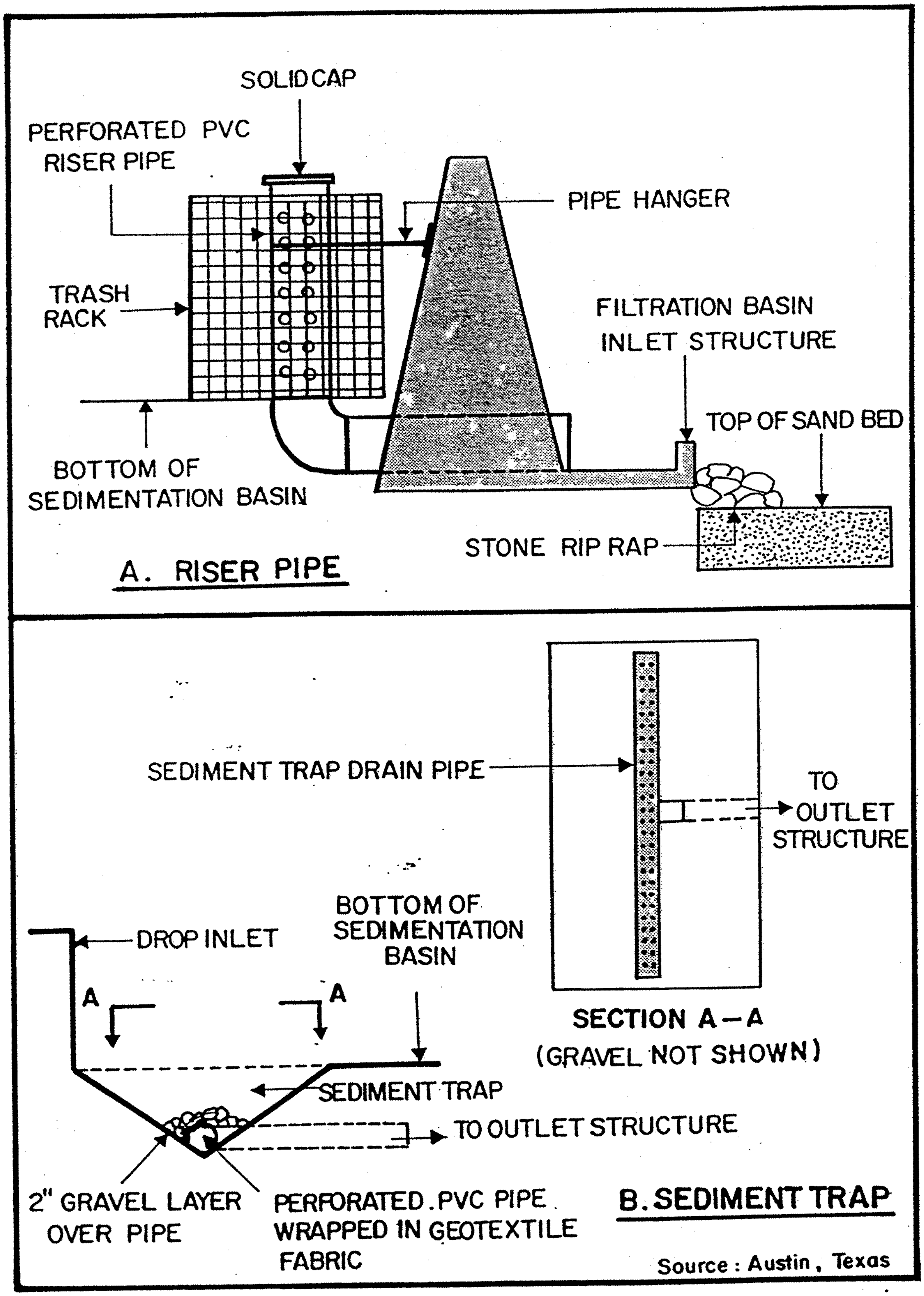


FIGURE 2 - 12 EXAMPLE RISER PIPE AND SEDIMENT TRAP DETAILS

(January 1987)[30] and shall be appropriately reinforced. An ordinary surface finish is required. When the underlying soil is clay or has an unconfined compressive strength of 0.25 ton per square foot or less, the concrete shall have a minimum six (6) inch compacted aggregate base consisting of coarse sand and river stone, crushed stone or equivalent with diameter of 0.75 to one (1) inch. Where visible, the concrete shall be inspected annually and all cracks shall be sealed.

- 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 by-passed by the flow regime and are, therefore, ineffective in the settling process) should be minimized. Baffles may be used to mitigate short circuiting and/or dead storage problems. End Notes 9 and 31 provide additional information on these conditions. **Figure 2-13** illustrates basin geometry considerations. **Figure 2-14** illustrates the use of baffles to improve sedimentation basin performance.

- 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 storage. Sediment traps may reduce maintenance requirements by reducing the frequency of sediment removal. It is recommended that the sediment trap volume be equal to ten (10) percent of the sedimentation basin volume.

Water collected in the sediment trap shall be conveyed to the filtration basin in order to prevent standing water conditions from occurring. All water collected in the sediment trap shall 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 1/4 inch per foot (two (2) percent slope). Access for cleaning the sediment trap drain system is necessary. **Figure 2-12** illustrates sediment trap details.

- o Maintenance Access Ramp

  Provision must be made to allow equipment access for removing accumulated sediments. An equipment

  access ramp should be provided along one wall of .the sediment basin to allow the use of at least compact front-end loaders such as "Bobcats."

4) Sand Filtration Basin Details

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

- o Inlet Structure

  The inlet structure should spread the flow uniformly across the surface of the filter media. Flow spreaders, weirs or multiple orifice openings are recommended. **Figure 2-10** illustrates inlet structure designs.

- o Sand Bed

  The sand bed may be a choice of one of the two configurations given below.

Note: **Sand bed depths are final, compacted depths. Consolidation effects must be taken into account.**

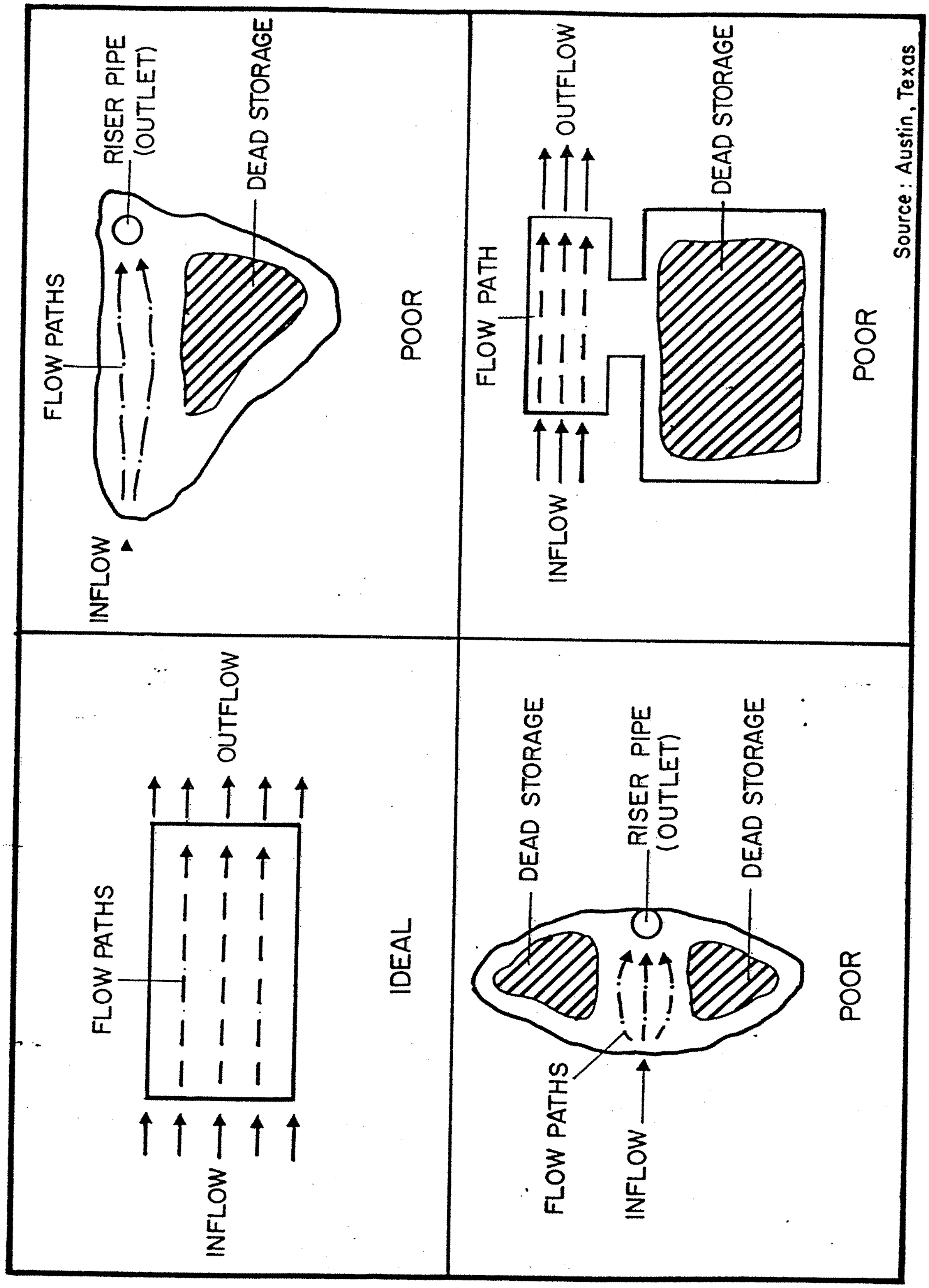


FIGURE 2-13 SEDIMENT BASIN CONFIGURATIONS

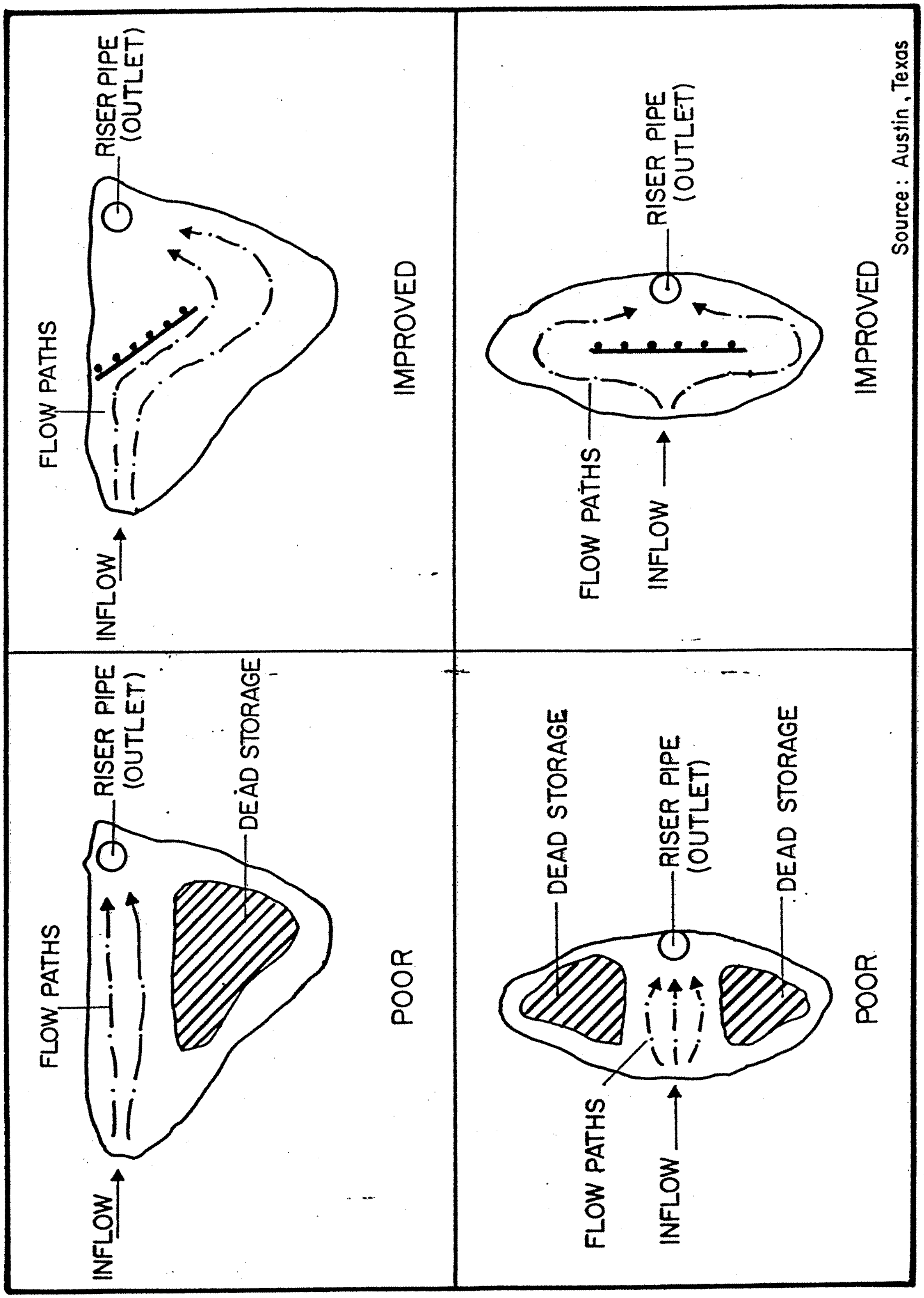


FIGURE 2-14 SEDIMENTATION BASIN BAFFLES

- Sand Bed with Gravel Layer (**Figure 2-15**)

  The top layer is to be a minimum of 18 inches of **ASTM C-33 Concrete Sand** (smaller sand size is acceptable). Under the sand shall be a layer of 1/2 to two (2) inch diameter gravel which provides a minimum of two (2) inches of cover over the top of the underdrain lateral pipes. No gravel is required under the lateral pipes. The sand and gravel must be separated by a layer of geotextile fabric meeting the specifications listed above under "Basin Liner."

- Sand Bed - Trench Design (**Figure 2-15**)

  The top layer shall be 12-18 inches of **ASTM C-33 Concrete Sand.** Laterals shall be placed in trenches with a covering of 1/2 to two (2) inch gravel and geotextile fabric. The laterals shall 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. The geotextile fabric specifications are listed above under "Basin Liner." The drainage matting specifications are listed below.

TABLE 2-4

DRAINAGE MATTING SPECIFICATIONS

Property	Test Method	Unit	Specification
Material	Nonwoven geotextile fabric		
Unit Weight		Oz/Sq.Yd.	20
Flow Rate (fabric)		GPM/Ft^2	180 (min.)
Permeability	ASTM D-2434	Cm/Sec	12.4 X 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 (min.)
Mullen Burst Strength	ASTM D-1117	Psi	140 (min.)
Equiv. Opening Size	US Standard Sieve	No.	100 (70-120)
Flow Rate (drainage core)	Drexel Univ. Test Method	GPM/ft.width	14

Source: City of Austin

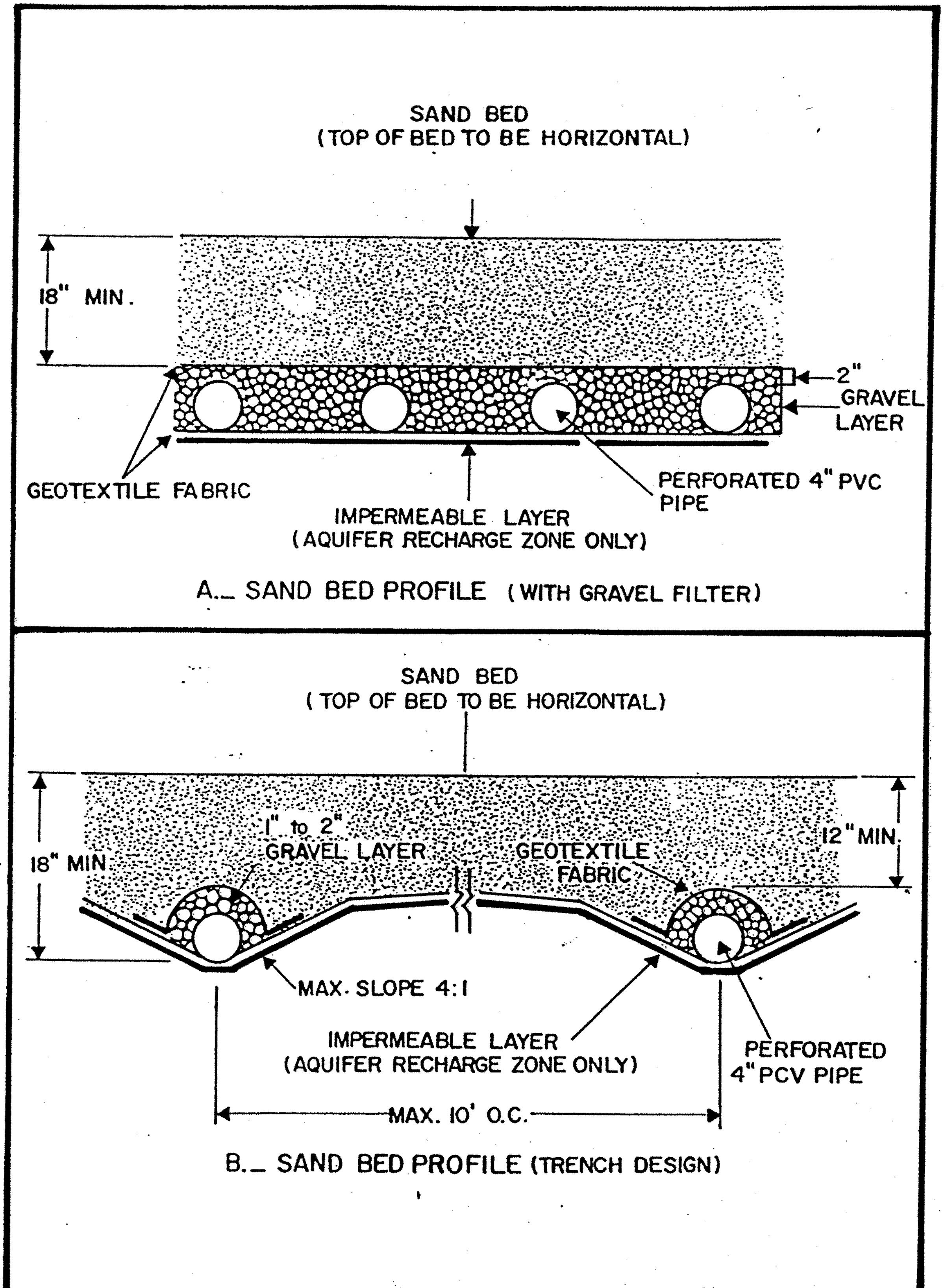


FIGURE 2 - 15 SAND BED FILTRATION CONFIGURATIONS

- o 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. Internal diameters of lateral branch pipes should be four (4) inches or greater and perforations should be 3/8 inch. Each row of perforations shall contain at least four (4) holes and the maximum spacing between rows of perforations shall be six (6) inches. All piping is to be schedule 40 polyvinyl chloride or greater strength. A maximum spacing of ten (10) feet between laterals is recommended. Lesser spacings are acceptable.

  The minimum grade of piping shall be 1/8 inch per foot (one (1) percent slope). Access for cleaning all underdrain piping is needed.

  Note: No draw-down time is to be associated with sand filtration basins, only with sedimentation basins. Thus, it is not necessary to have a specially designed orifice for the filtration outlet structure.

- o Basin Liner

  If an impermeable liner is required it shall meet the specifications given on page 2-32 and 2-33 under "Basin Liner." If an impermeable liner is not required then a geotextile fabric liner shall be installed which meets the specifications listed above under "Basin Liner" unless the pond has been excavated to bedrock.

G) Design Considerations for Partial Sedimentation with Filtration

In this system a sediment chamber is located in front of the filtration basin. The purpose of the settling chamber is to remove larger suspended material (e.g., sand and trash litter), thus it only serves as a partial sedimentation basin. The sediment chamber is not required to hold the entire water quality volume and will not incorporate an extended draw-down period. The sediment chamber is typically separated from the filtration basin by a berm or wall with flow spreading outlets installed, or by a gabion. **Figure 2-16** illustrates this system.

1) Basin Surface Areas and Volume

A filtration rate of .0312 gallons per minute per square foot has been selected for design criteria (six (6) feet per day or two (2) million gallons per acre per day). This filtration rate is based on a Darcy's Law coefficient of permeability of two (2) feet per day, an average hydraulic head of three (3) feet and a sand bed depth of 18 inches, and a filter drawdown time, t_f of 40 hours. This filtration rate is less than that assumed for the filtration basin in the full sedimentation-filtration system due to higher sediment loading and consequent clogging of the filter media. Appendix 2-1 contains an explanation of how the filtration rate and coefficient of permeability were determined. End Notes 22, 26 and 27 provide additional information.

The following equation gives the minimum surface area required for the filtration basin:

$$A_f = I_a H/10 \qquad (2\text{-}19)$$

where "A_f" is the required surface area of the media in acres, "I_a" is the impervious area in the drainage area, in acres, contributing runoff to the filtration basin, and "H" is the runoff depth in feet (0.5 inch = 0.0417 feet when treating the water quality volume).

When treating the WQV, this reduces to:

$$A_f = 0.0042 I_a \qquad (2\text{-}20)$$

When designing for parameter values differing from those assumed by Austin, use the partial sedimentation (PS) Austin Filter Formula on page 2-11.

The combined volume of the sediment chamber and filtration basin must be equal to the water quality volume, i.e., $V_s + V_f$ = WQV when "V_s" is the settling chamber volume and "V_f" is the filtration basin volume.

The surface area for the sediment chamber, "A_s", is found by dividing the volume of the chamber, "V_s", by its depth "D_s". D_s can be assumed to equal D_f where "D_f" is the depth of the filtration basin.

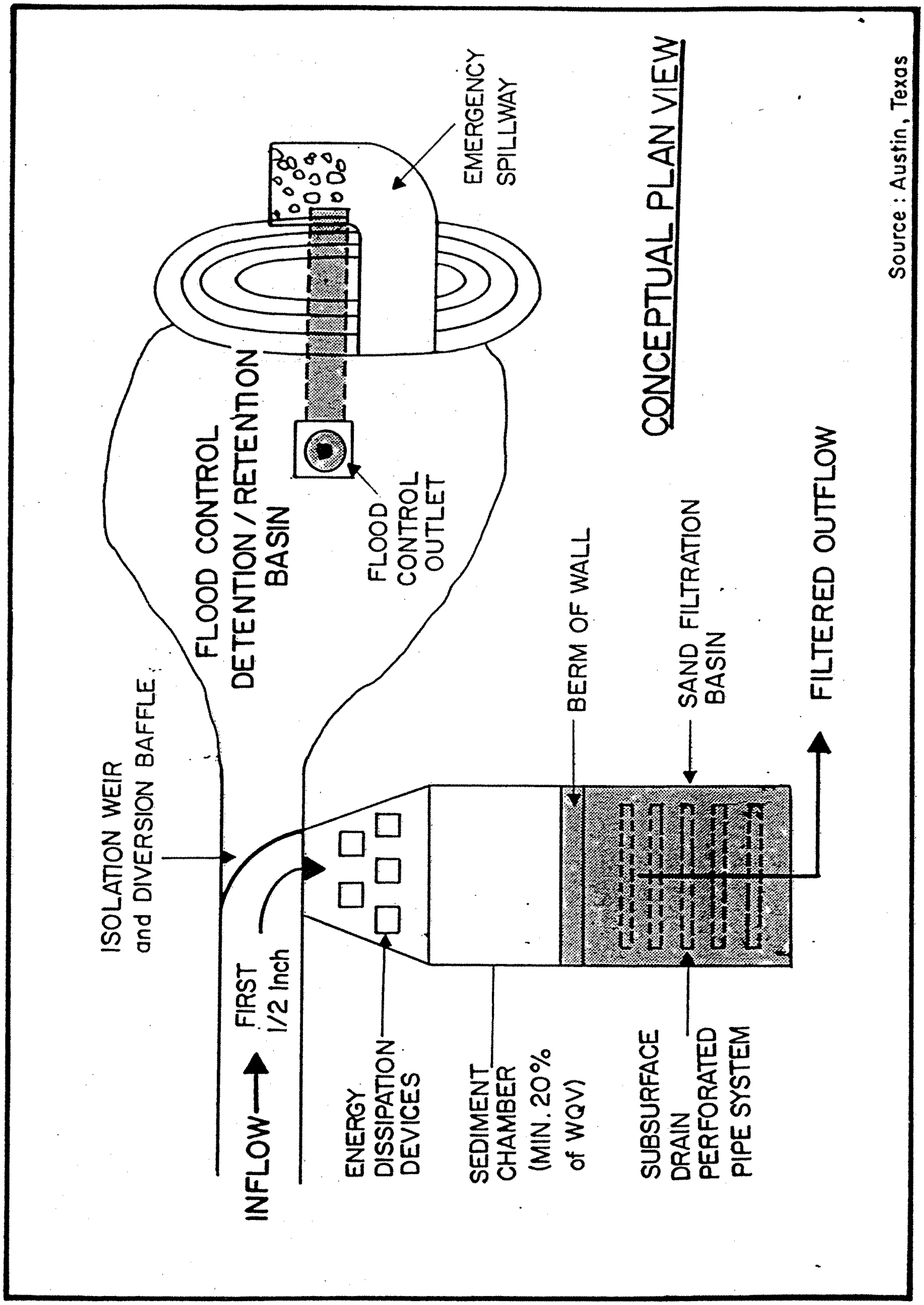


FIGURE 2-16 PARTIAL SEDIMENTATION-FILTRATION (PLAN VIEW)

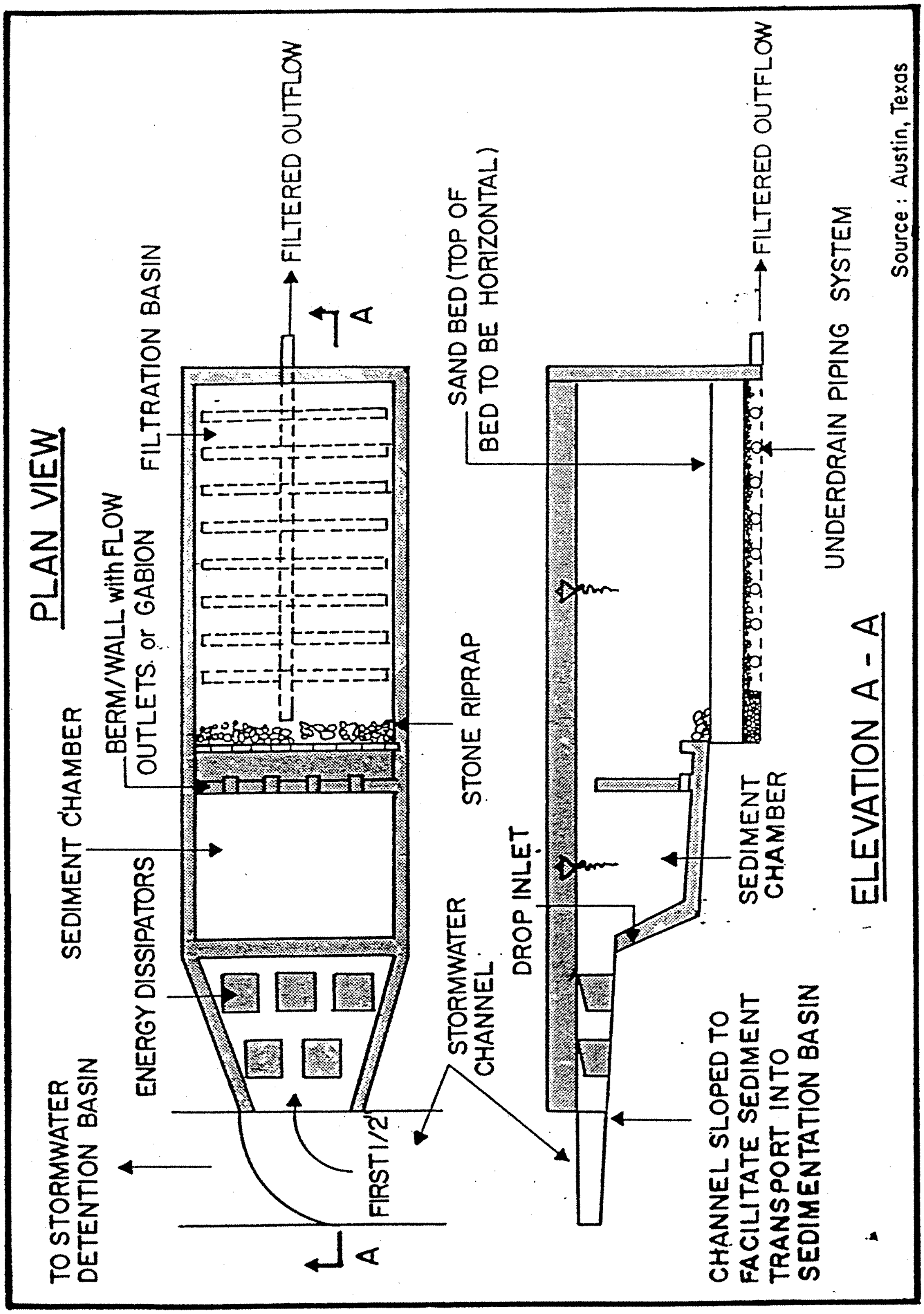


FIGURE 2 - 17 CONCEPTUAL PARTIAL SEDIMENTATION — FILTRATION SYSTEM

The following equation can thus be derived to give the sediment chamber average surface area:

$$A_s = I_a H\left[\frac{1}{D_s} - \frac{1}{10}\right] \qquad (2\text{-}21)$$

where "A_s" is the sediment chamber surface area in acres, "I_a" is the contributing impervious drainage area in acres, "H" is the runoff depth in feet (0.5 inch = 0.0417 feet) and "D_s" is the sediment chamber basin depth (= D_f, the filtration basin depth).

The volume of the sediment chamber, "V_s", shall be a minimum of 20 percent of the water quality volume. the design shall ensure that under no circumstances does the sediment chamber allow water to return to the isolation/diversion structure, i.e., isolation of the water quality volume must be ensured. **Figure 2-18** provides graphic solutions to sizing the basin.

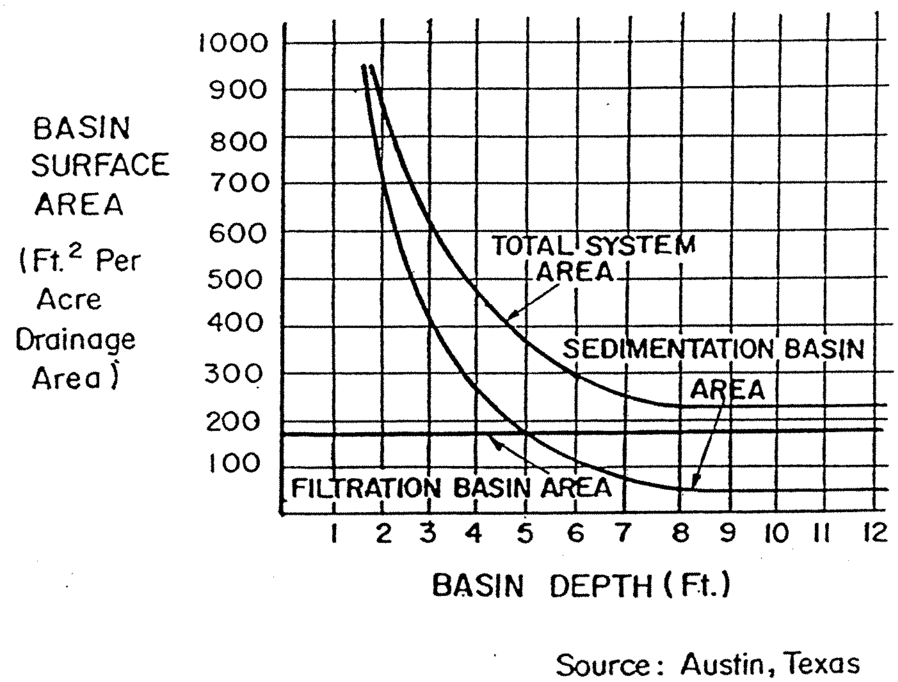


Figure 2-18
Sedimentation/Filtration Basin Surface Areas
Partial Sedimentation

2) **Sediment Chamber Details**

The sediment chamber consists of an inlet structure, outlet structure and basin liner.

- o Inlet Structure [see Inlet Structures under Full Sedimentation above]

- o Outlet Structure - The outlet structure should be a berm or wall with multiple outlet ports or a gabion so as to discharge the flow evenly to the filtration basin. Rock gabions should be constructed using 6-8 inch diameter rocks. The berm/wall/gabion height should not exceed six (6) feet and high flows 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 gouging of the sand media (e.g., concrete splash pad or riprap). **Figure 2-17** illustrates these design considerations.

- o Basin Liner [same as for Full Sedimentation]

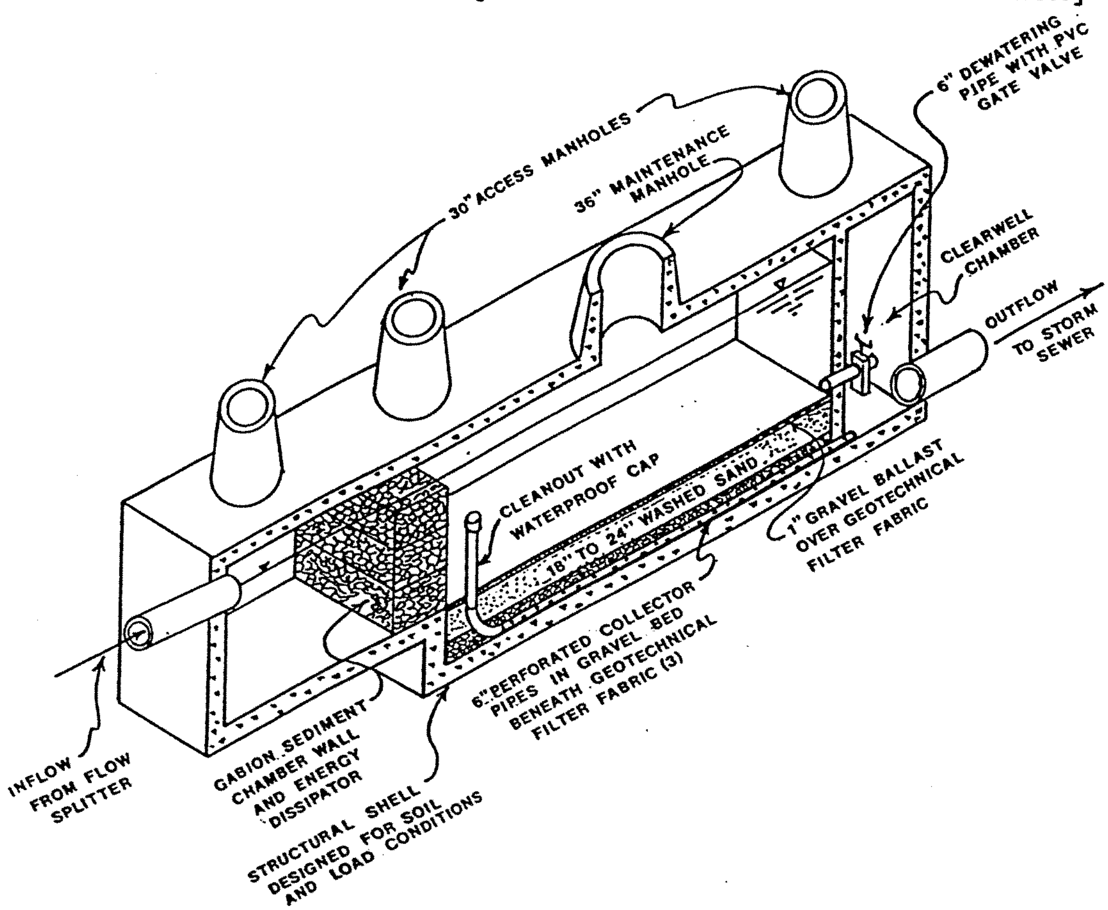


FIGURE 2-18A -- DRY VAULT UNDERGROUND SAND FILTER SYSTEM

3) **Sand Filtration Basin Details (same as for Full Sedimentation)**

H) Dry Vault Underground Sand Filter System

The Austin partial sedimentation sand filter may also be placed in underground vaults. **Figure 2-18AS** shows a modified vault design developed by Alexandria from both Austin and D.C. methodologies. The partial sedimentation chamber is sized to contain 20 percent of the WQV. A gabion wall is used to separate the partial sedimentation chamber from the filter area and absorb energy. Heavy sediments are deposited in this first chamber, which dries out between storms. The filter is augmented with features developed by the District of Columbia Environmental Regulation Administration with the following specifications:

Figure 2-22 on page 2-57 shows the filter cross-section.

1) Upper Aggregate Layer

The washed aggregate or gravel layer at the top of the filter shall be one inch thick and meet ASTM standard specifications (1 inch maximum diameter.)

2) Geotechnical Fabrics

The filter fabric beneath the one-inch layer of gravel on top of the filter shall be Enkadrain 9120 filter fabric or equivalent with the following specifications:.

Property	Test Method	Unit	Specification	
Material	Nonwoven geotextile fabric			
Unit Weight	ASTM D-1777	Oz/sq.yd.	4.3	(min.)
Flow Rate	Falling Head Test	gpm/sq.ft	120	(min.)
Puncture Strength	ASTM D-751(Modified)	Lb.	60	(min.)
Thickness		in.	0.8	(min.)

The geotechnical filter fabric beneath the sand layer shall be the same as for other Austin Sand Filters and shall be cut with sufficient dimensions to cover the entire wetted perimeter of the filtering area with a six-inch overlap up the chamber walls.

3) Sand Filter Layer and Gravel Bed Around Collector Pipes

Same as for other Austin Sand Bed with Gravel Layer (see p.2-39 - 2-40).

4) Underdrain Piping

Same as for other Austin Sand Filters except that three 6-inch perforated pipes shall be used. The piping should be reinforced to withstand the weight of the overburden. Perforations should be 3/8 inch, and each row of perforations shall contain at least six (6) holes. Maximum spacing between rows of perforations shall be six (6) inches. All piping is to be schedule 40 polyvinyl chloride or greater strength.

5) Weepholes

In addition to the underdrain pipes, weepholes should be installed between the filter chamber and the clearwell to provide relief in case of pipe clogging. The weepholes shall be three (3) inches in diameter. Minimum spacing shall be nine (9) inches center to center. The openings on the filter side of the dividing wall shall be covered to the width of the trench with 12 inch high plastic hardware cloth of 1/4 inch mesh or galvanized steel wire, minimum wire diameter 0.03-inch, number 4 mesh hardware cloth anchored firmly to the dividing wall structure and folded 6 inches back under the bottom stone.

Worksheet G on page 2-A3-7 is provided to assist with designing Dry Vault Sand Filters.

I) Austin Sand Filter Construction and Maintenance Requirements

A Maintenance Agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with the Chapter 3 of this manual must be executed by the developer/owner **before** the Final Site Plan for the development will be released for construction.

Construction and maintenance requirements for Austin Sand Filter Systems are delineated in detail on pages 2-A3-3 and 2-A3-4. Construction and maintenance requirements for Dry Vault Sand Filters are delineated on pages 2-A3-15 and 2-A3-16. These requirements shall be reproduced <u>verbatim</u> on the Stormwater Management Plan sheets of the Final Site Plan.

VI. DISTRICT OF COLUMBIA (D.C.) UNDERGROUND SAND FILTERS

A) Facility Description

Mr. Hung V. Truong of the D.C. Environmental Regulation Administration has developed an underground stormwater sand filter contained in a structural shell with three chambers. The shell may be either precast or cast-in-place concrete, corrugated metal pipe, or fiberglass tanks. Over seventy of the structures have been installed since 1987. **Figures 2-19A and 2-19B** depict two versions of Mr. Truong's system. When used in Alexandria, D.C. Sand Filters (DCSFs) will normally be placed off-line and be sized to treat the WQV.

The three feet deep plunge pool in the first chamber and the throat of the second chamber, which are hydraulically connected by an underwater rectangular opening, absorbs energy and provides pretreatment, trapping grit and floating organic material such as oil, grease, and tree leaves.

The second chamber also contains a typical intermittent sand filter. When a filter section thinner than the depth of the plunge pool is used, a second, energy absorbing pool at the level of the filter top is positioned ahead of the filter. **Figure 2-19B** illustrates this configuration. As with the Austin system, the filter material consists of gravel, sand, and filter fabric. At the bottom is a subsurface drainage system of pierced PVC pipe in a gravel bed. The primary filter media is 18-24 inches of sand. A layer of plastic reinforced geotechnical filter cloth secured by gravel ballast is placed on top of the sand. The top filter cloth is a pre-planned failure plane which can readily be replaced when the filter surface becomes clogged. A dewatering drain controlled by a gate valve must be installed to facilitate maintenance.

The third chamber, or clearwell, collects the flow from the underdrain pipes and directs it to the storm sewer.

Since D.C. often places these structures in-line, additional overflow weirs or bypass pipes are usually provided. **Figure 2-19B** illustrates the currently recommended method, a hooded large storm bypass pipe directly connecting the first chamber with the clearwell. When used off-line to treat the WQV, an overflow or bypass is neither necessary or desired.

B) Pollutant Removal Rates

The D.C. Environmental Regulation Administration is conducting a program of monitoring to establish the actual removal

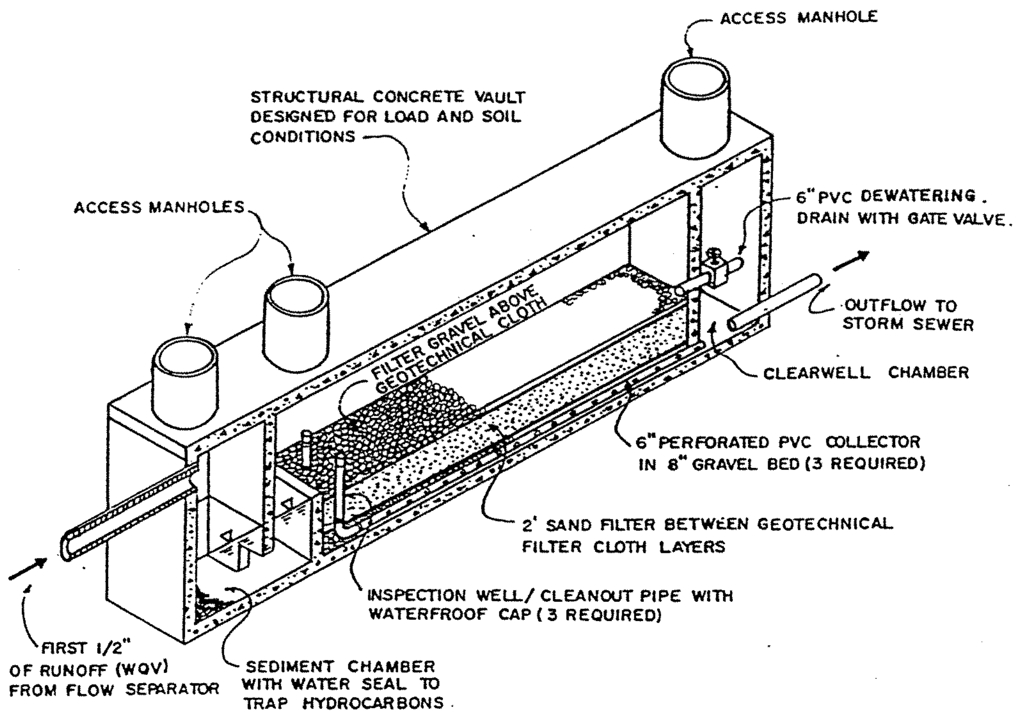


FIGURE 2-19A -- ORIGINAL D.C. SAND FILTER (DCSF) SYSTEM

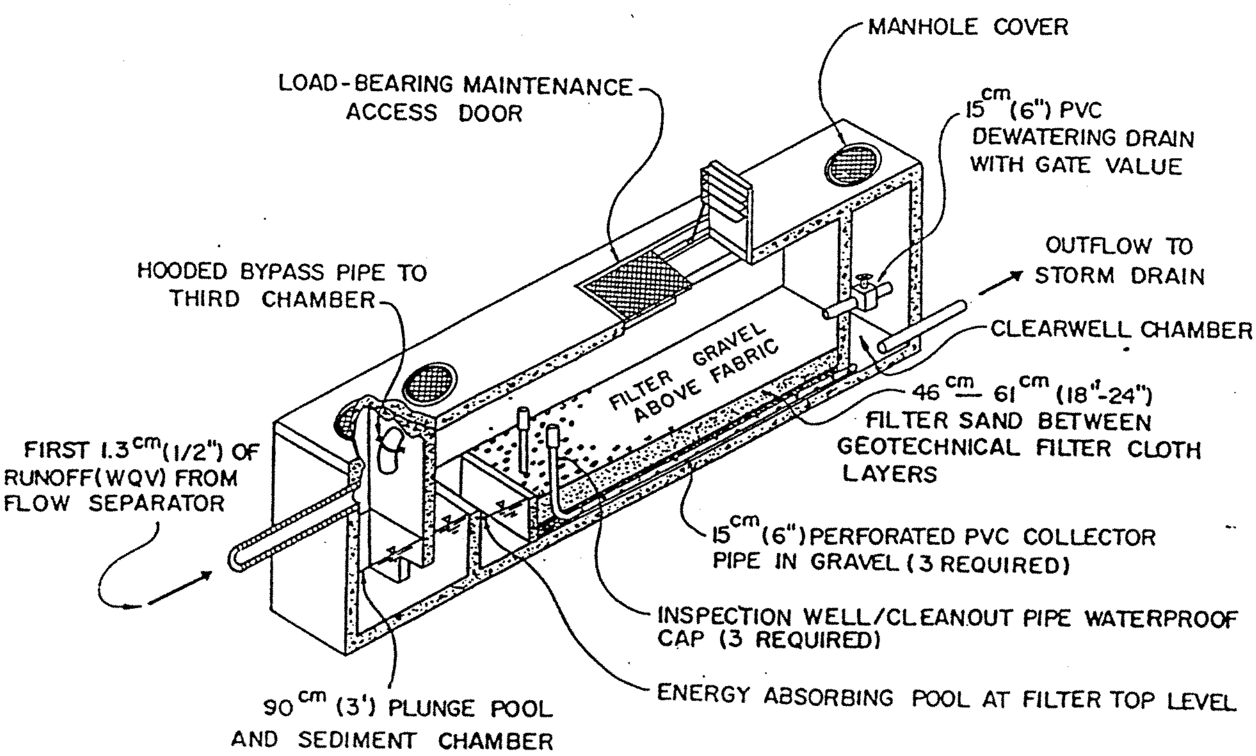


FIGURE 2-19B -- THIN FILTER DCSF WITH ENERGY DISSIPATION POOL AND LARGE STORM BYPASS PIPE

rates of this system. As of this writing, no data is available. Based on the results of the Austin, Texas, monitoring sand filter program (see page 2-25), Alexandria will recognize a total phosphorous removal efficiency of 60% for DCSFs if the applicant agrees to outfit the system for monitoring and grant unlimited access to the City and its contractors for monitoring purposes.

C) Design Considerations

1) Applicability

A major advantage of the D.C. sand filter is that it does not take up any space on the surface. It can be placed under on-site roadways (e.g, not public rights of way), parking lots, or sidewalks; and under planting spaces adjacent to buildings. Mr. Troung advises that the system works best for watersheds of approximately one acre of impervious surface. For larger watersheds, use two or more DCSFs.

In Alexandria, these systems will be utilized only for off-line applications to treat the WQV. If a flow splitter is not installed ahead of the DCSF, an integral large storm bypass pipe from the sediment chamber to the clearwell must be provided. The bypass pipe must be located to one side to avoid blocking the access manholes or maintenance access doors. Quantity detention must be provided in a separate facility.

2) Practicability

Several years of success with this system in D.C. have demonstrated its practicality for use in the Middle Atlantic States area. Costs vary with the size of the structure and the character of the site. When first introduced in 1987, systems constructed in D.C. cost approximately $35,000 per impervious acre treated. Use of precasting has reduced costs to approximately $12,000 to $16,000 per impervious acre at present.[49]

3) Groundwater and Bedrock

The seasonally-high groundwater table and bedrock should be located at least two (2) to (4) feet below the footing of the filter structure.

4) Drawdown Time

As with WQV Storage Tanks, drawdown time should not exceed 40 hours so that the BMP will be free to process follow-on storms.

5) **Structural Requirements**

The load-carrying capacity of the filter structure must be considered when it is located under parking lots, driveways, roadways, and, certain sidewalks (such as those adjacent to State highways). Traffic intensity may also be a factor. The structure must be designed by a licensed structural engineer and the plans require City approval.

6) **Design Storm**

The inlet design or integral large storm bypass must be adequate for isolating the WQV from the 10 year storm (7 in./hr., 10 min. TOC) and for conveying the peak flow of that storm past the filter system. Since DCSFs will be used only as off-line facilities in Alexandria, the interior hydraulics of the filter are not as critical as when used as an on-line facility. The system should draw down in approximately 40 hours.

7) **Infrastructure Elevations**

For cost considerations, it is preferable that the DCSF work by gravity flow. This requires sufficient vertical clearance between the invert of the prospective inflow storm piping and the invert of the storm sewer which will receive the outflow. In cases where gravity flow is not possible, a clearwell sump and pump are required to discharge the effluent into storm sewer.

8) **Accessibility and Headroom for Maintenance**

All three DCSF chambers must have personnel access manholes and built-in access ladders .The DCFS must also be accessible to vacuum trucks for removing accumulated sediments and hydrocarbons at least every six months. Approximately every 3-5 years, the filter can be expected to clog to the point that replacement of the top layer of washed gravel and the top layer of filter cloth will be required. **A minimum headspace of 60 inches above the filter will be required if the ceiling to the chamber is a fixed structure.** A 36-inch diameter maintenance manhole or a rectangular load bearing access door (minimum 4 ft. x 4 ft.) should be positioned directly over the center of the filter. When site conditions will not allow 60 inches of headspace, City staff will consider allowing reduced clearance if load bearing access doors or removable covers, such are sometimes employed over underground utility tunnels are to be provided.

9) **Accessibility for Monitoring**

Unless otherwise approved by the Director, prefabricated monitoring manholes must be installed in the inflow and outflow pipes to allow chemical monitoring of the inflow water and effluent. See Appendix 2-8 for details.

D) Design Procedures (Original DCSF Single Pool Configuration)

1) Determine Governing Site Parameters

Determine the Impervious area on the site (I_a in acres), the water quality volume to be treated (WQV in ft.^{3} = 1816 I_a), and the site parameters necessary to establish 2h, the maximum ponding depth over the filter (storm sewer invert at proposed connection point, elevation to inflow invert to BMP, etc). If a bypass weir or pipe is to be built directly into the DCSF shell, it should be designed at this point. **Worksheet E** on page 2-A4-2 is provided to perform this step.

Figure 2-20 shows the dimensional relationships required to compute the remaining steps of the design.

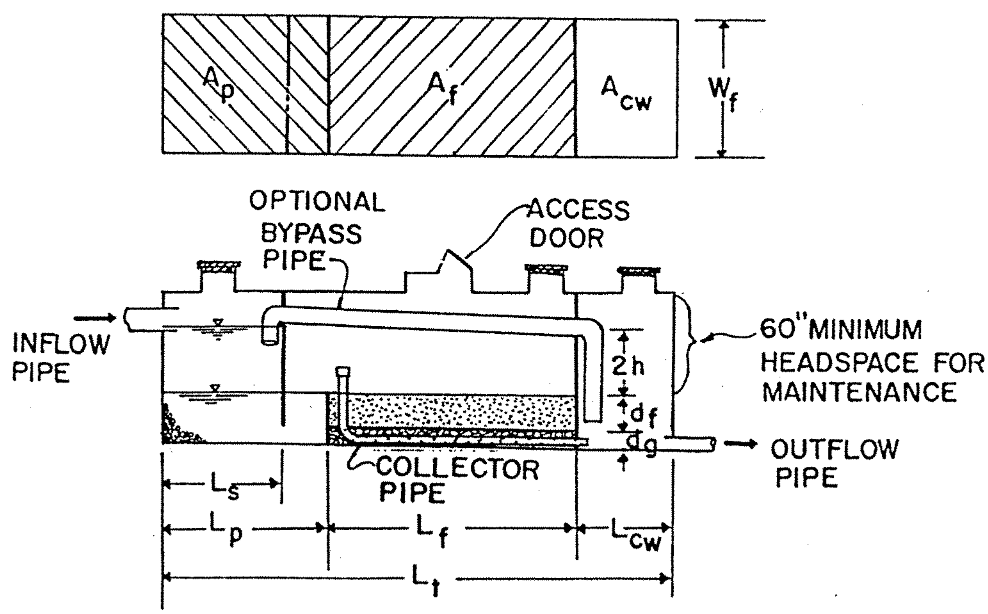


FIGURE 2-20 **DIMENSIONAL RELATIONSHIPS FOR D.C. SAND FILTER**

2) **Select Filter Depth and Determine Maximum Ponding Depth**

Considering the data from Step 1) above, select the Filter Depth ((d_f) and determine the maximum achievable ponding depth over the filter (2h).

3) **Compute the Minimum Area of the Sand Filter (A_{fm})**

D.C. utilizes a curve to determine the area of the filter **(Figure 2-21)**. For applications in Alexandria, utilize the Austin Filter Formula for partial sedimentation treatment (equation 2-13--see page 2-11):

$$A_{fm(PS)} = \frac{545 I_a d_f}{(h + d_f)} \qquad (2\text{-}13)$$

where,

A_{fm} = minimum surface area of sand bed (square feet)
I_a = impervious cover on the watershed in acres
d_f = sand bed depth (normally 1.5 to 2ft)
h = average depth of water above surface of sand media between full and empty basin conditions (ft.)

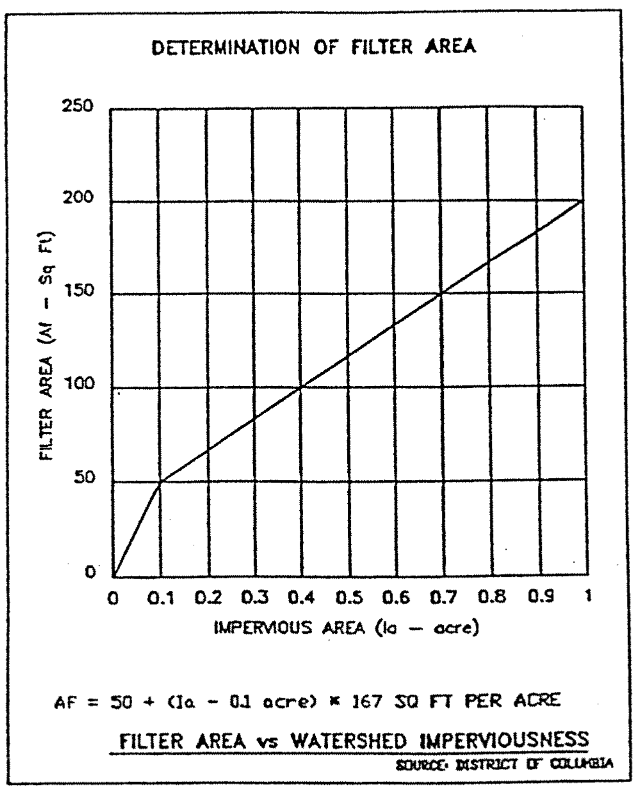


FIGURE 2-21 -- D.C. SAND FILTER CURVE AND FORMULA

4) Select Filter Width and Compute Filter Length and Adjusted Filter Area

Considering site constraints, select the Filter Width (W_f). Then compute the Filter Length (L_f) and the Adjusted Filter Area (A_f)

$$L_f = A_{fm}/W_f \qquad (2\text{-}22)$$

$$A_f = W_f \times L_f \qquad (2\text{-}23)$$

NOTE: From this point, formulae assume rectangular cross section of filter shell.

5) Compute the Storage Volume on Top of the Filter (V_{Tf})

$$V_{Tf} = A_f \times 2h \qquad (2\text{-}24)$$

6) Compute the Storage in the Filter Voids (V_v)
(Assume 40% voids in filter media)

$$V_v = 0.4 \times A_f \times (d_f + d_g) \qquad (2\text{-}25)$$

7) Compute Flow Through Filter During Filling (V_Q)
(Assume 1-hour to fill per D.C. practice)

$$V_Q = \frac{kA_f(d_f + h)}{d_f} \text{ ; use } k = 2 \text{ ft./day} = 0.0833/\text{hr.} \qquad (2\text{-}26)$$

8) Compute Net Volume to be Stored Awaiting Filtration (V_{st})

$$V_{st} = WQV - V_{Tf} - V_v - V_Q \qquad (2\text{-}27)$$

9) Compute Minimum Length of Permanent Pool (L_{pm})

$$L_{pm} = \frac{V_{st}}{(2h \times W_f)} \qquad (2\text{-}28)$$

10) Compute Minimum Length of Sediment Chamber (L_s)
(to contain 20% of WQV per Austin practice)

$$L_{sm} = \frac{0.2WQV}{(2h \times W_f)} \qquad (2\text{-}29)$$

11) Set Final Length of Permanent Pool (L_p)

If $L_{pm} \geq L_s + 2\text{ft.}$, make $L_p = L_{pm}$ (2-30)

If $L_{pm} < L_{sm} + 2\text{ft.}$, make $L_p = L_{sm} + 2\text{ft.}$ (2-31)

12) Establish Structure Dimensions and Size Clearwell (L_{cw})

It may be economical to adjust final dimensions to correspond with standard precast structures or to round off to simplify measurements during construction.

Set the length of the clearwell (L_{cw}) for adequate maintenance and/or access for monitoring flow rate and chemical composition of effluent (minimum = 3ft.)

Worksheet H1 on page 2-A3-10 is provided to assist with performing the above calculations.

E) Design Procedures (Thin Filter Configuration with Two Pools)

Figure 2-21A depicts dimensional relationships for this configuration. Design steps **1)** through **5)** are identical to those for the single pool design. Then add the following:

5A) Compute the Storage Volume on Top of the Lower Pool (V_{Tp2})

Set the length the lower pool (L_{p2}) at no less that two feet and compute the storage volume above it.

$$V_{Tp2} = L_{p2} \times W_f \times 2h \qquad (2\text{-}32)$$

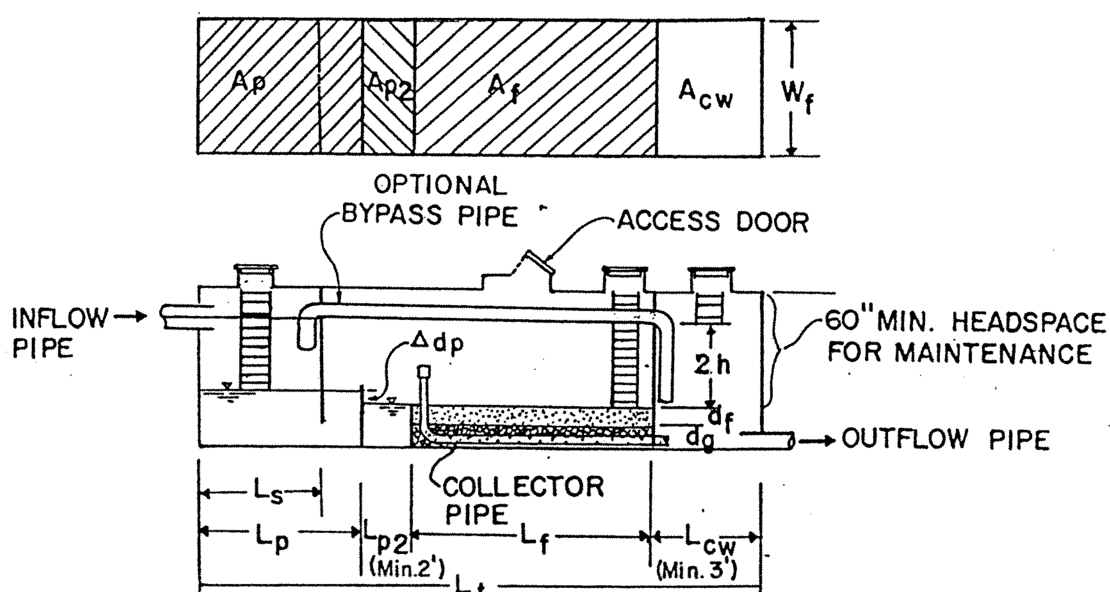


FIGURE 2-21A -- DIMENSIONAL RELATIONSHIPS FOR THIN FILTER DCSF

Design steps **6)** and **7)** are the same as for the single pool configuration. Step **8)** is modified as follows:

8) Compute Net Volume to be Stored Awaiting Filtration (V_{st})

$$V_{st} = WQV - V_{Tf} - V_v - V_Q - V_{Tp2} \qquad (2\text{-}33)$$

9) Compute Minimum Length of Permanent Pool (L_{pm})

$$L_{pm} = \frac{V_{st}}{(2h - d_p) \times W_f} \qquad (2\text{-}34)$$

10) Compute Minimum Length of Sediment Chamber (L_{sm}) (to contain 20% of WQV per Austin practice)

$$L_{sm} = \frac{0.2WQV}{(2h - d_p) \times W_f} \qquad (2\text{-}35)$$

Design steps **11)** and **12)** are the same as for the single pool configuration. **Worksheet H2** on page 2-A3-13 is provided for performing thin filter DCSF calculations.

F) Filter Specifications and Details

Figure 2-22 is a cross-section of the filter chamber.

1) Upper Aggregate Layer

The washed aggregate or gravel layer at the top of the filter shall be one inch thick and meet ASTM standard specifications (1 inch maximum diameter.)

2) Geotechnical Fabrics

The filter fabric beneath the one-inch layer of gravel on top of the filter shall be Enkadrain 9120 filter fabric or equivalent with the following specifications:.

Property	Test Method	Unit	Specification	
Material	Nonwoven geotextile fabric			
Unit Weight	ASTM D-1777	Oz/sq.yd.	4.3	(min.)
Flow Rate	Falling Head Test	gpm/sq.ft	120	(min.)
Puncture Strength	ASTM D-751(Modified)	Lb.	60	(min.)
Thickness		in.	0.8	(min.)

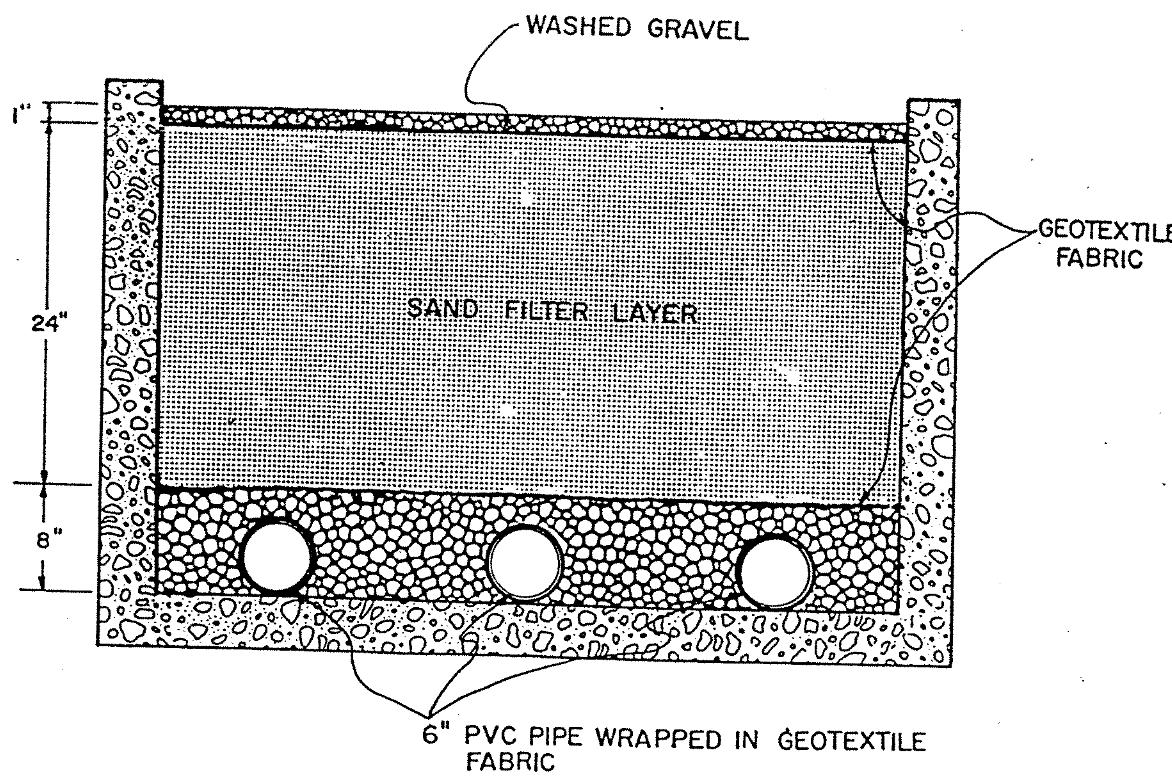


FIGURE 2-22 -- CROSS-SECTION OF DCSF FILTER

The filter cloth layer beneath the sand shall conform to the following specification (same as for Austin Sand Filter):

Property	Test Method	Unit	Specification	
Material	Nonwoven geotextile fabric			
Unit Weight		Oz/Sq.Yd.	8	(min.)
Filtration Rate		In/Sec	0.08	(min.)
Puncture Strength	ASTM D-751(Modified)	Lb.	125	(min.)
Mullen Burst Strength	ASTM D-751	Psi	400	(min.)
Tensile Strength	ASTM D-1682	Lb.	300	(min.)
Equiv. Opening Size	US Standard Sieve	No.	80	(min.)

The fabric rolls should be cut with sufficient dimensions to cover the entire wetted perimeter of the filtering area with a six-inch wall overlap.

3) Sand Filter Layer

For applications in Alexandria, use **ASTM C33 Concrete Sand**[32] or sand meeting the Grade A fine aggregate gradation standards of Section 202 of the VDOT <u>Road and Bridge Specifications</u>.[30] D.C. uses a sand filter layer 22-25 inches deep. Alexandria will allow use of 18 inches of sand with the thin filter configuration.

4) Gravel Bed Around Collector Pipes

The gravel layer surrounding the collector pipes shall be 1/2 to two (2) inch diameter gravel and provide at least two (2) inches of cover over the tops of the drainage pipes. The gravel and the sand layer above must be separated by a layer of geotextile fabric meeting the specification listed above.

5) Underdrain Piping

The underdrain piping consists of three 6-inch schedule 40 or better polyvinyl perforated pipes reinforced to withstand the weight of the overburden. Perforations should be 3/8 inch, and each row of perforations shall contain at least six (6) holes. Maximum spacing between rows of perforations shall be six (6) inches.

The minimum grade of piping shall be 1/8 inch per foot (one (1) percent slope). Access for cleaning all underdrain piping is needed. Clean-outs for each pipe shall extend at least six (6) inches above the top of the upper filter surface, e.g. the top layer of gravel.

Each pipe shall be thoroughly wrapped with 8 oz./sq.yd. geotechnical fabric meeting the above detailed specification before placement in the filter.

6) Weepholes

In addition to the underdrain pipes, weepholes should be installed between the filter chamber and the clearwell to provide relief in case of pipe clogging. The weepholes shall be three (3) inches in diameter. Minimum spacing shall be nine (9) inches center to center. The openings on the filter side of the dividing wall shall be covered to the width of the trench with 12 inch high plastic hardware cloth of 1/4 inch mesh or galvanized steel wire, minimum wire diameter 0.03-inch, number 4 mesh hardware cloth anchored firmly to the dividing wall structure and folded a minimum of 6 inches back under the bottom stone.

E) Applications in Circular Cross Sections

Single-pool underground DCSFs with a 24-inch filter may be installed in aluminum or aluminized steel corrugated metal pipe shells or fiberglass tanks. One pipe manufacturer has expressed a willingness to prefabricate a complete "drop-in" filter shell as a single unit and deliver it to a job site. **Figure 2-22A** illustrates this concept. **Worksheet H3** on page 2-A4-16 is provided for computing DCSF designs in circular cross-sections.

F) Applications in Available Structural Shells

Available structural shells with sufficient dimensions may be modified to contain sand filter systems employing D.C. concepts. **Figure 2-22B** portrays two views of an adaptation of a standard precast drop inlet to contain an inlet filter concept developed by the Alexandria Engineering staff. A built-in flow splitter is provided. The sedimentation chamber is made long and narrow, requiring a 180-degree "switch-back" in flow of the runoff, which increases energy dissipation and particle settlement. The filter illustrated fits inside a standard 8 ft by 8 ft by 20 ft precast concrete drop inlet shell and will capture and treat the WQV from 1/3 acre of new impervious cover, such as highway pavement. The filter may also be fed by a separate or integral grated inlet. **Worksheet H4** on page 2-A4-29 is provided for sizing "switch-back" sand filters.

G) Maintenance and Construction Requirements

A Maintenance Agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with the Chapter 3 of this manual must be executed by the developer/owner before the Final Site Plan for the development will be released for construction. A project-specific agreement will be forwarded by the City with the bond estimate.

Construction and Maintenance requirements for D.C. Sand Filters are delineated in detail on pages 2-A3-5 and 2-A3-6. There requirements shall be reproduced <u>verbatim</u> on the Stormwater Management Plan sheets of the Final Site Plan.

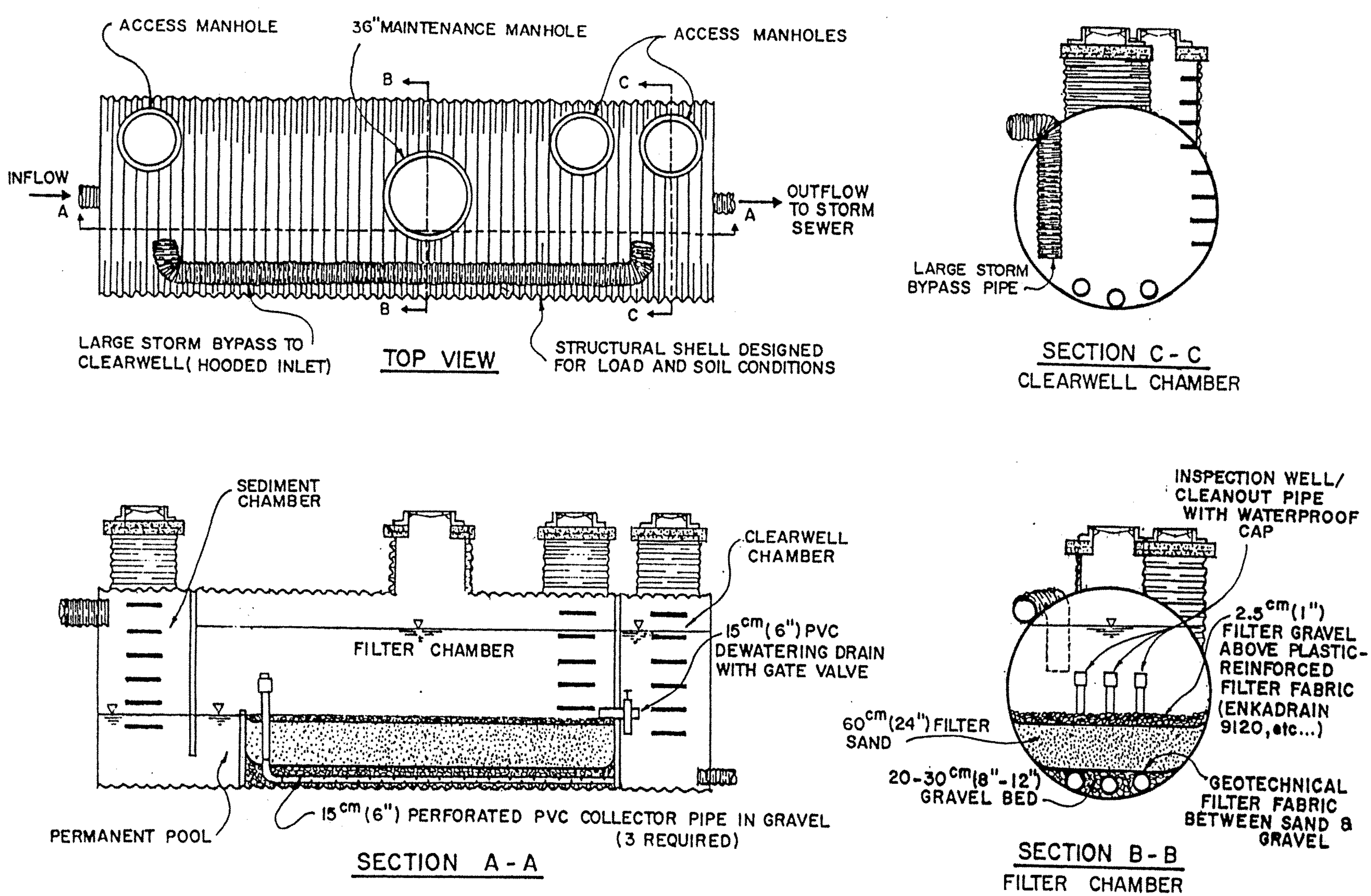
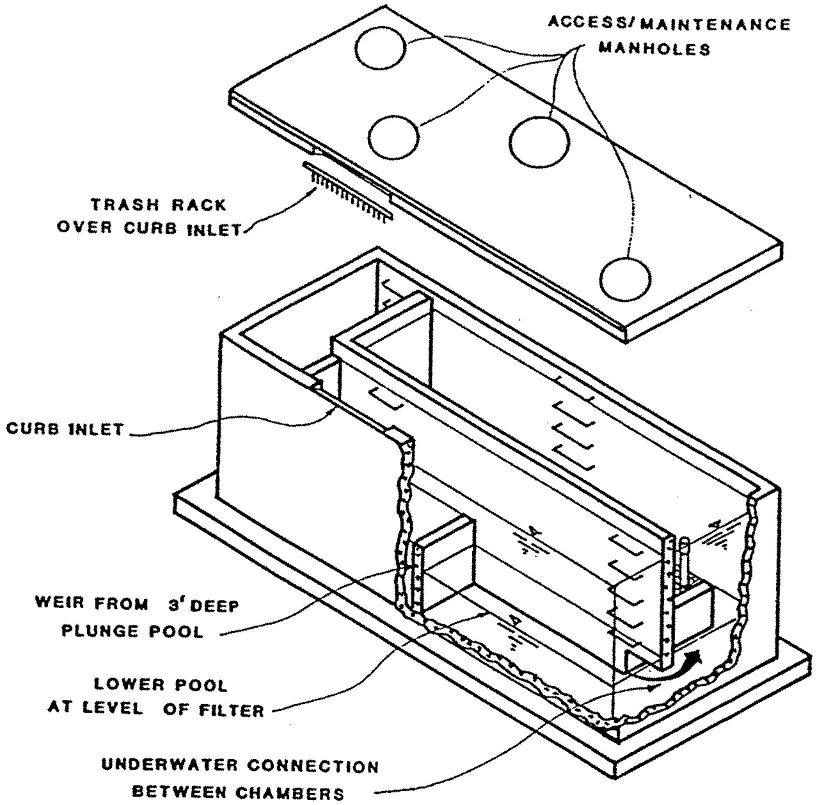
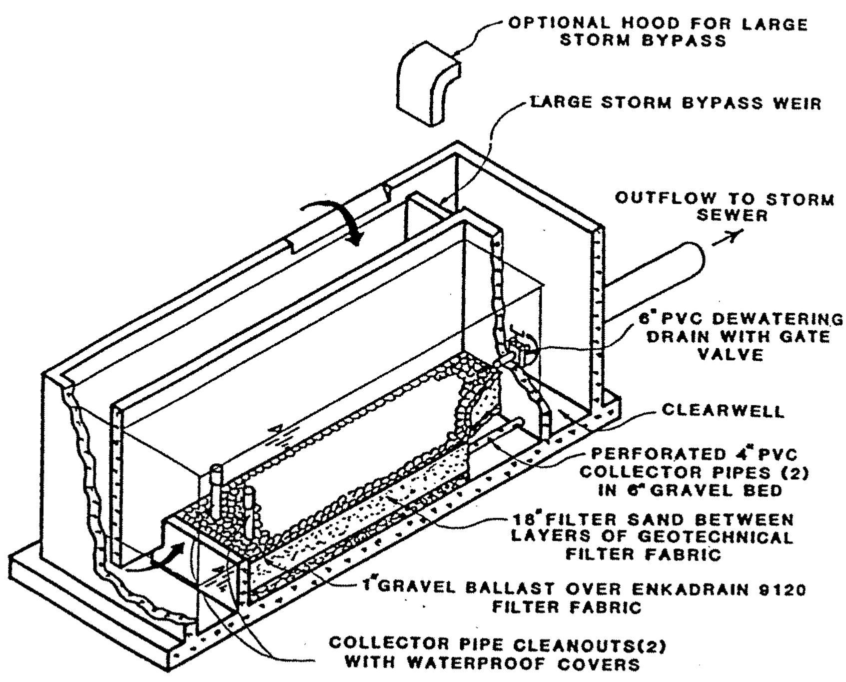


FIGURE 2-22A -- D.C. SAND FILTER IN CORRUGATED METAL SHELL



SEDIMENTATION CHAMBER CUTAWAY



FILTER CHAMBER CUTAWAY

FIGURE 2-22B --"SWITCH-BACK" SAND FILTER IN PRECAST DROP INLET SHELL

VII. DELAWARE SURFACE SAND FILTER (DSF) SYSTEMS

A) Facility Description

Mr. Earl Shaver of the Delaware Department of Natural Resources and Environmental Control has developed a surface sand filter system for use in Delaware. Most of the data in this section is extracted from information provided by Mr. Shaver,[35] and most of the End Notes are the references cited by Mr. Shaver in his paper.

As originally conceived, the Delaware Sand Filter is an **on-line** facility processing all stormwater exiting the treated site up to the point that its overflow limit is reached (Delaware provides for treating the first one inch of runoff). However, when employed in Alexandria, it will usually be provided with an integral flow-splitter to isolate and treat the Water Quality Volume.

Figure 2-23 shows a schematic drawing of the original Delaware Sand Filter. The system consists of two parallel concrete trenches connected by close-spaced wide notches in the top of the wall dividing the trenches. The trench adjacent to the site being served is the sedimentation chamber. When accepting sheet flow, it is fitted with a grated cover. Concentrated stormwater may also be conveyed to the chamber in enclosed storm drain pipes. The second chamber, which contains the sand filter, is always fitted with a solid cover.

Storm flows enter the sedimentation chamber through the grates, causing the sedimentation pool to rise and overflow into the filter chamber through the weir notches at the top of the dividing wall, assuring that the water to be treated **arrives at the filter as sheet flow.** This is essential to prevent scouring out the sand. The permanent pool in the sedimentation chamber is dead storage, which inhibits resuspension of particles that were deposited in earlier storms and prevents the heavier sediments from being washed into the filter chamber.[35] Floatable materials and hydrocarbon films, however, may reach the filter media through the surface outflow.

The second trench contains at least 18 inches of sand (Delaware specifies sand having particle size no greater that 2 millimeters). No underdrain piping is provided in Delaware practice. The water is allowed to percolate through flow nets in the sand to the lower end of the trench. The water exits the system through a grate covered with geotextile fabric to prevent sand from washing out of the filter.

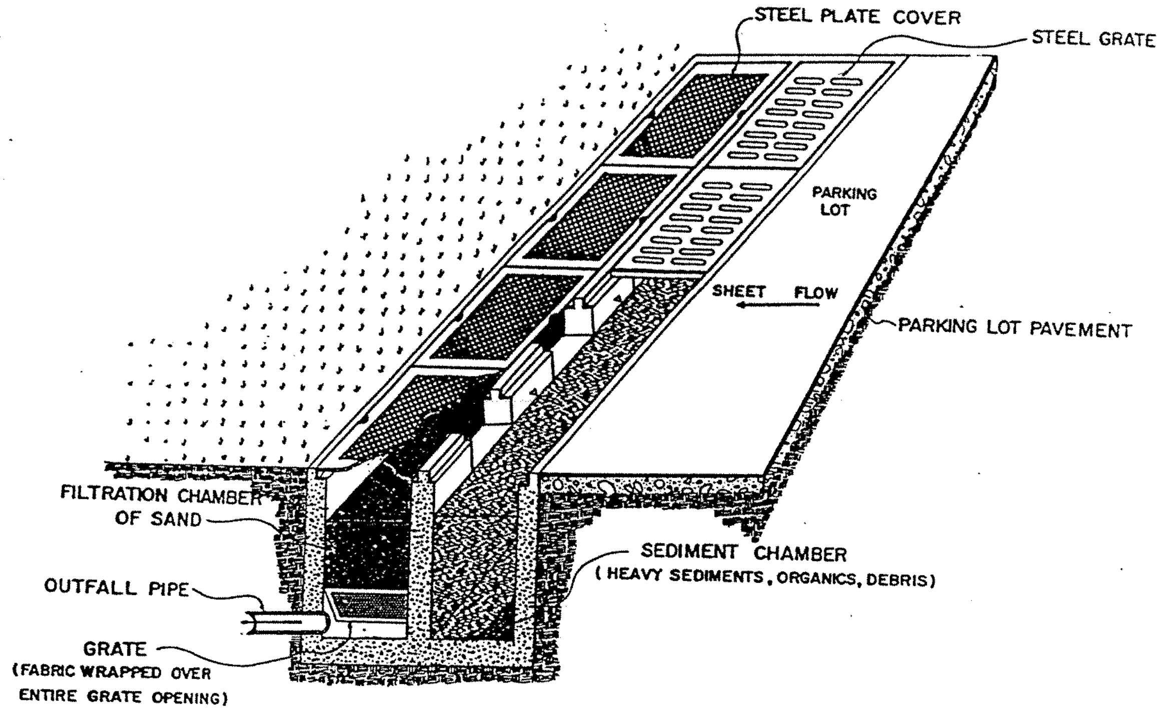


FIGURE 2-23 -- ORIGINAL DELAWARE SAND FILTER

B) Pollutant Removal Rates

Delaware does not rate these systems for nutrient removal efficiency. However, based on the results of the Austin, Texas, long term monitoring of their sand filter systems [8], Delaware has made a determination that, when treating the **first one inch of runoff**, their sand filter provides the 80 percent suspended solids removal rate required by their state environmental regulations[35].

Alexandria is monitoring a DSF to determine removal efficiencies for a number of pollutants under a grant from the Chesapeake Bay Local Assistance Department. In the interim, the City will recognize a 60 percent phosphorous removal rate for DSFs constructed according to the criteria outlined below based on the results of the Austin study.[8]

C) Design Considerations

1) Applicability

A major advantage of the Delaware Sand Filter is that it can be installed in shallow configurations, which is especially critical in the flatter regions of the City where high water tables exist. The simplicity of the system and the ready accessibility of the chambers for periodic maintenance might also prove attractive.

An obvious difference from the D.C. system is that the Delaware Sand Filter design has no provision for excluding floatable debris smaller than the grate openings and petroleum sheens from reaching the filter media. Earlier clogging of the sand filter might therefore be expected, and care would have to be exercised in disposing of clogged sand materials removed during maintenance because of their likely petroleum hydrocarbon content.

The original DSF, which was constructed in Maryland in 1986, cost approximately $10,000 and serves a one-acre watershed.[35] A large slotted curb filter in Alexandria, the prototype DSF in Northern Virginia, cost approximately $40,000 to serve watershed of 1.7 impervious acres. Two small custom-built systems which have been constructed to serve fractional cost in the $4,000-7,000 range. The City staff is working with precasters to make available standard precast units, which should considerably reduce the cost of a DSF. Designers should check with the staff early in the design process on the availability of such units.

2) Practicality

The similar sand filter system constructed in Maryland has been in service for approximately six years. It serves a parking lot that is heavily used by patrons of a courthouse.

Mr. Shaver has visited the Maryland facility on a regular basis over the six-year period. The responsible maintenance people have reported to him that there has been no instance where the sand filter has overflowed. Only recently does the system appear to be clogging to the point that the operation of the system may be impaired. Oil, grease and finer sediments have migrated into the sand to a depth of only two (2) to three (3) inches.[35]

Disposal of petroleum contaminated sand would appear to be the only potential problem with the use of this filter system. Owners of relatively lightly used parking facilities, such as church parking lots, might not have as severe a problem in this respect as might commercial establishments with high usage.

3) Modifications Resulting from Alexandria Experience

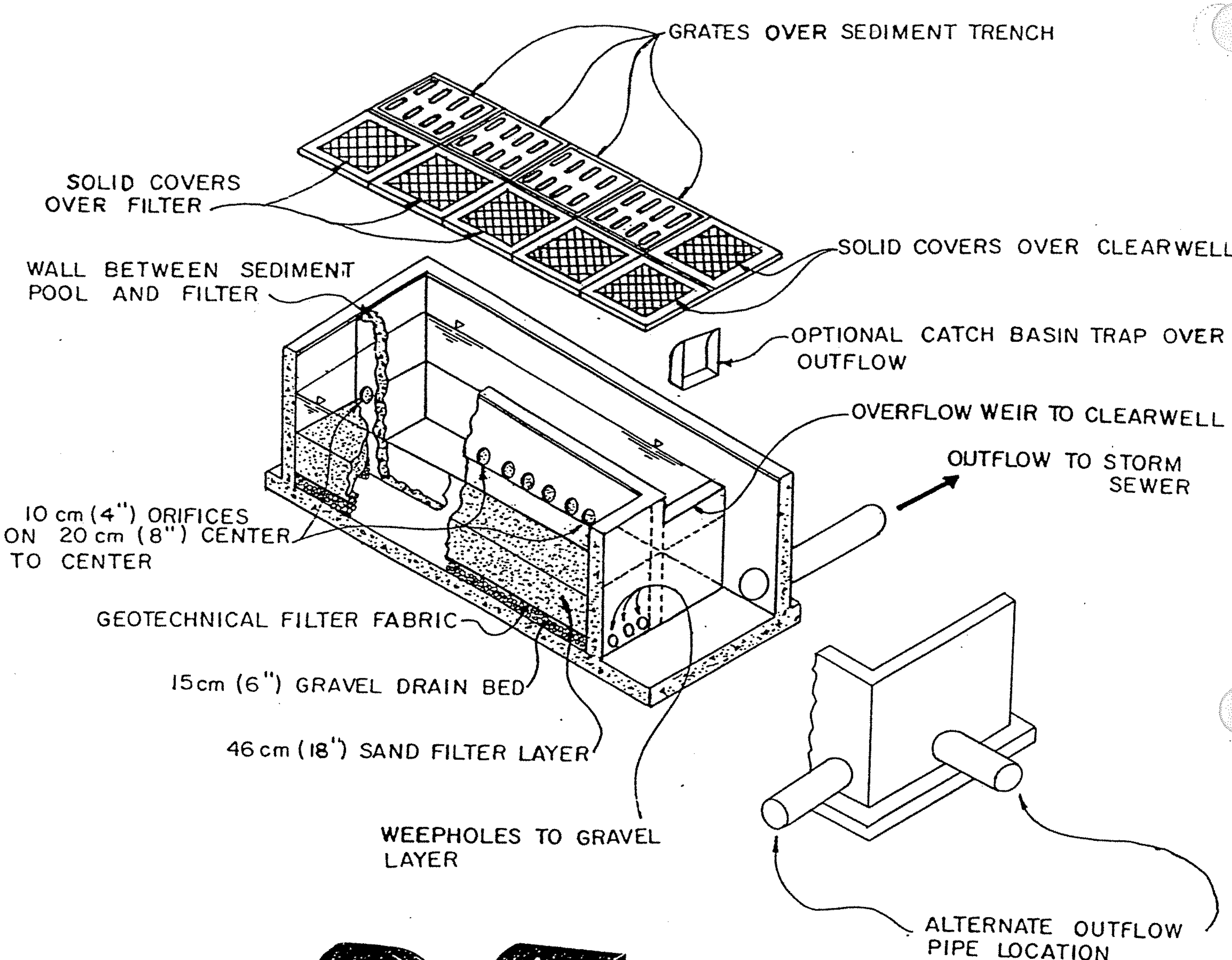
Initial experience in Alexandria with this system indicates that Delaware Sand Filters for large areas require special features not contemplated in the original design. The increased distances through which water must flow through the pores in the sand considerably reduces the outflow below the level that the normal filter formulae would indicate. Accurate flow rate computations would require the application of flow net techniques, as are utilized by Florida in some filter designs.[48]

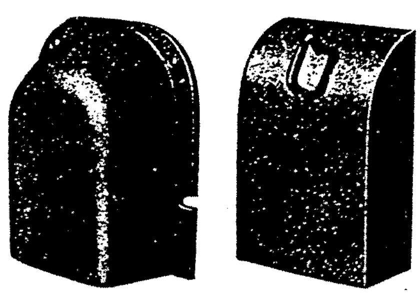
For DSFs over 20 feet in length, Alexandria will require the installation of a collector pipe or shallow rectangular drain tiles in a bed of gravel separated from the sand by a layer of geotechnical filter fabric. For DSFs with a filter length of 20 feet or less, a six-inch layer of gravel separated from the overlying sand by filter fabric will be required. In both cases, a small clearwell shall be provided at the lower end of the filter shell to collect the treated water and convey it to the storm sewer. For filters without a collector pipe or drain tile, weepholes will be utilized to allow the filtered water to flow from the voids in the gravel into the clearwell.

An overflow weir into the clearwell from the sedimentation chamber will normally be provided to convey runoff greater than the WQV directly to the storm sewer. This assures that the filter functions as an off-line BMP. The weir shall be sized to pass the 10-year storm. Where retention of hydrocarbons is a concern, the weir should be fitted with a metal hood or commercial catch basin trap. **Figure 2-24A** illustrates a small DSF with gravel underdrain and large storm bypass weir.

Experience also indicates that the grates and covers can amount to over 50 percent of the cost of this system. The City staff worked with one developer to develop a variation which introduces the water into the filter shell through a slotted curb in the side, saving almost 60 percent of the original bid costs. **Figure 2-24B** illustrates a slotted curb DSF with a collector pipe. The precast lids must be in units which can be easily removed with small lifting equipment. Curb slots should be equipped with trash grates to exclude floating debris, cans, etc.

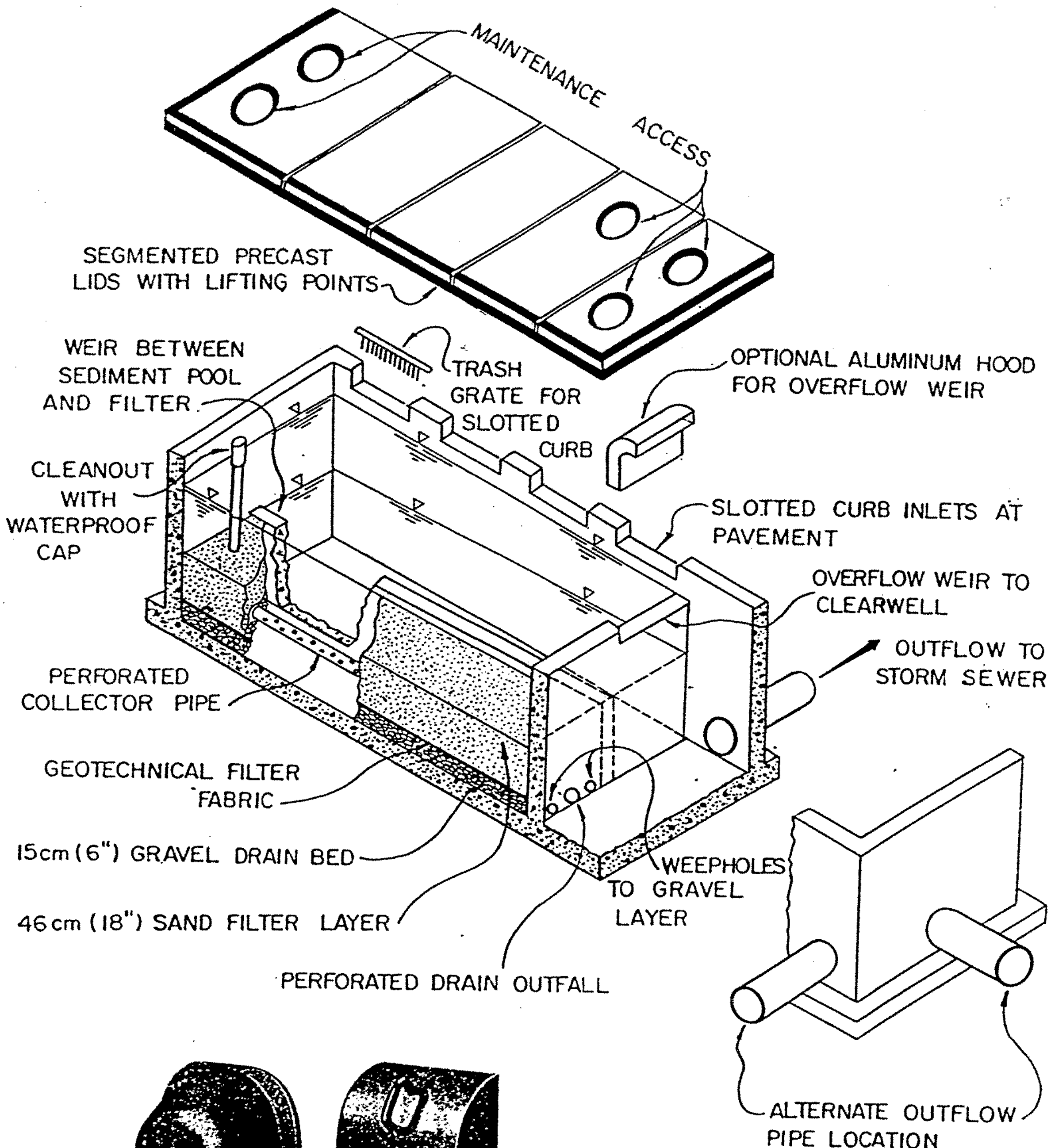
In applications where grated covers are indicated by site conditions, use of standard Virginia Department of Transportation (VDOT) grates (Grate D1-1) will usually be most cost-effective.

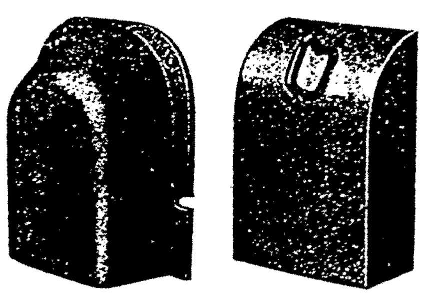




Commercial Catch Basin Traps

<u>FIGURE 2-24A</u> -- DELAWARE SAND FILTER WITH FLOW SPLITTER AND GRAVEL UNDERDRAIN





Commercial Catch Basin Traps

<u>FIGURE 2-24B</u> -- SLOTTED CURB DELAWARE SAND FILTER

4) **Structural Requirements**

The system may be placed in the street or parking area or off the pavement where lower structural loads would be involved. In Alexandria applications, the structure must be designed by a licensed professional engineer, and the design must be approved by the City.

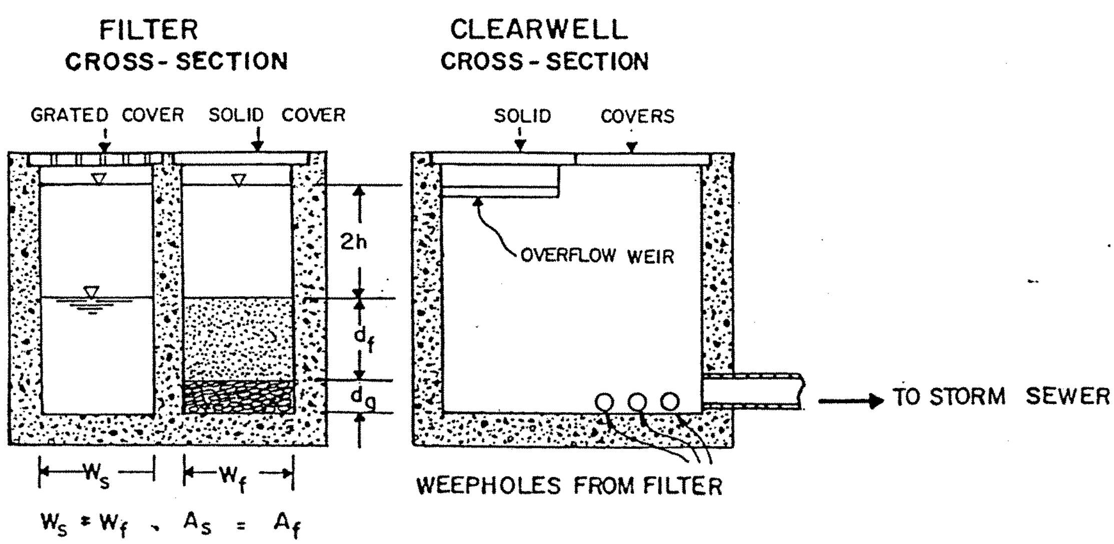
5) **Accessibility for Monitoring Equipment**

Unless waived by the Director, provision must be made for the ready installation of automatic monitoring equipment to measure both the flow rate and chemical composition of the inflow stormwater and the filtered water exiting the DSF. This will usually involve the installation of two commercially available prefabricated monitoring manholes. See Appendix 2-8 for details. A separate grate or curb inlet to capture samples of untreated runoff will also normally be required. The maintenance/monitoring agreement (see Chapter 3 of this Supplement) must provide unlimited access to the City and its contractors for the purpose of monitoring the actual pollutant removal performance of the BMP.

D) Design Procedures

Design procedures utilized by Delaware are contained on pages 2-A1-17 through 2-A1-19.

Figure 2-24C shows dimensional relationships for the Delaware Sand Filter as adopted for use in Alexandria.



<u>Figure 2-24C</u> -- Dimensional Relationships for Delaware Sand Filters

1) Calculate the Required Surface Areas of the Chambers

Considering critical site constraints (storm sewer invert at proposed connection point, minimum BMP invert to achieve drainage to connection point, site surface elevation at BMP location, required height of overflow weir to convey 10-year storm, etc.) select maximum ponding depth over filter. If an integral flow seperator is to be built into the DSF shell, size the overflow weir, orifice, or pipe using the procedures outlined on page 2-5.

Because of the shallow configuration of this BMP, resulting in low levels of hydraulic head above the filter, application of the usual partial sedimentation filter formula may not create enough storage volume to contain the WQV. With the dimensional relationships shown in **Figure 2-24** and k = 2.0 ft/day, the required DSF filter area to contain the WQV may be written as follows (derivation of formula in Technical Notes):

$$A_f = \frac{1816 I_a}{(4.1h + 0.9)} = \frac{WQV}{(4.1h + 0.9)} \qquad (2\text{-}36)$$

where:

A_f = the **area of the filter** in sq.ft.

I_a = the impervious area on the watershed in acres

h = 1/2 the maximum ponding depth over the filter (ft.)

If the maximum ponding depth above the filter (2h) is less than 2.67 feet (2'-8"), the WQV storage requirement governs and the above foumula must be used to size the filter (computation of this break-even point is contained on pages 2-a1-14 through 2-a1-16 in the technical notes at the end of this chapter). If the the maximum ponding depth above the filter (2h) is 2.67 feet or greater, use the partial sedimentation filter formula (equation 2-13--page 2-11).

$$A_f = \frac{545 I_a d_f}{(h + d_f)} \qquad (2\text{-}13)$$

where:

d_f = depth of the filter media in ft. (1.5-2.0)

Delaware (and Alexandria) make the area of the filter equal the area of the sediment chamber:[35]

$A_f = A_s$

2) **Establish Dimensions of the Facility**

Site considerations usually dictate the final dimensions of the facility. Sediment trenches and filter trenches normally be 18-30 inches wide. Use of standard VDOT D1-1 grates requires a trench width of 26". The maximum allowable trench width is 36 inches without special permission of the Director.

3) **Portland Cement Concrete**

Portland Cement concrete used for the trench structure shall conform to the A3 specification of the Virginia Department of Transportation Road and Bridge Specifications, January 1991.

4) **Sand Filter Chamber**

The top layer shall be a minimum of 18 inches of **ASTM C33 Concrete Sand**[32] **or VDTO Section 202 Grade A Fine Aggregate Sand.**[30] The top surface of the sand filter must be level, e.g. no grade is allowable. Under the sand shall be a layer of 1/2 to two (2) inch diameter gravel which provides a minimum of two inches of cover over underdrain piping or drain tiles or three (3) inches cover over weepholes. The sand and gravel must be separated by a layer of geotechnical fabric meeting the following specifications.

Property	Test Method	Unit	Specification	
Material	Nonwoven geotextile fabric			
Unit Weight		Oz/Sq.Yd.	8	(min.)
Filtration Rate		In/Sec	0.08	(min.)
Puncture Strength	ASTM D-751(Modified)	Lb.	125	(min.)
Mullen Burst Strength	ASTM D-751	Psi	400	(min.)
Tensile Strength	ASTM D-1682	Lb.	300	(min.)
Equiv. Opening Size	US Standard Sieve	No.	80	(min.)

When no underdrain piping is to be provided, the floor of the chamber must be sloped (minimum = 0.5% grade) toward the clearwell end.

5) **Geotechnical Fabric Overlayment**

In circumstances where frequent maintenance of the filter sand is to be expected, such as when treating runoff from service stations and other auto-related activities, a layer of plastic reinforced filter fabric, such as Enkadrain 9120, may be placed on top of the filter sand and secured with weights. The fabric may then be rolled up and disposed of as collection of pollutants dictates. Where a top fabric layer is used, the City will consider decreasing the sand depth requirement to 12 inches on a case-by-case basis.

6) Underdrain Piping or Drain Tiles

Where used, underdrain piping shall be four inches in diameter with 3/8 inch perforations. Piping shall be schedule 40 polyvinyl chloride or greater strength. Each row of perforations shall contain at least 4 holes and the maximum spacing between rows of perforations shall not exceed six (6) inches. The minimum grade of the piping shall be 1/8 inch per foot (1 percent slope). A vertical cleanout/inspection well extending above the surface of the sand and equipped with a waterproof cover shall be provided at the uphill end of the pipe. Drain pipes shall be completely wrapped in geotechnical filter fabric meeting the specification in (4 above before placement in the filter.

Shallow rectangular drain tiles may be fabricated from such materials as fiberglass structural channels, saving several inches of filter depth. Drain tiles shall normally be in two-foot lengths and spaced to provide gaps 1/8-inch less than the smallest gravel sizes on all four sides. Sections of tile may be cast in the dividing wall between the filter and the clearwell to provide shallow outflow oricices.

7) Weepholes

Where gravel undrains are used, the weepholes between the filter chamber and the shell shall be three (3) inches in diameter. Minimum spacing shall be eight (8) inches center to center. The openings on the filter side of the dividing wall shall be covered to the width of the trench with 12 inch high plastic hardware cloth of 1/4 inch mesh or galvanized steel wire, minimum wire diameter 0.03-inch, number 4 mesh hardware cloth anchored firmly to the dividing wall structure and folded 6 inches back under the bottom stone. Weepholes conforming to these specifications may also be provided in addition to underdrain pipes to provide a backup in case of pipe clogging.

8) Grates and Covers

When grates and cast steel covers are used, design to take the same wheel loads as the adjacent pavement. Where possible, use standard Virginia Department of Transportation grates to reduce costs (VDOT D1-1, VDOT

1-2, etc.). Grates and covers shall be supported by a gavlanized steel perimeter frame meeting the requirements for grate collars of VDOT Standard D1-1.

9) Hoods or Catch Basin Traps for Overflow Weirs

In applications where trapping of hydrocarbons and other floating pollutants is required, such as at auto-related activities, large storm overflow weirs shall be equipped with a 10-gauge aluminum hood or commercially available catch basin trap.

10) Outfall Pipe(s)

When a large storm bypass is provided, design the outfall for the 10-year storm peak flow rate. Pipe shall conform to Alexandria standards for storm sewer piping. Minimum pipe size shall usually be ten-inch pipe, but eight-inch pipe may be used with short (20 feet or under) lengths of precast filter shells.

Worksheet I1 on page 2-A4-21 is provided to assist in sizing standard DSFs.

E) DSFs With Partial External Storage for Part of WQV

When storage for part of the WQV is provided outside the filter shell, a smaller DSF structure will result. In such cases, size the filter using equation 2-13. Worksheet I2 on page 2-A4-23 is provided to assist in sizing such filters.

F) Construction and Maintenance Requirements

A maintenance agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with the Maintenance Agreement chapter of this manual must be executed by the developer before the Final Site Plan for the development will be released for construction. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected in a timely manner.

Construction and maintemance requirements for Delaware Sand Filters are delineated in detail on pages 4-A3-7 and 4-A3-8. These requirements shall be reproduced <u>verbatim</u> on the Stormwater Management Plan sheets of the Final Site Plan.

VIII. PEAT-SAND FILTRATION SYSTEMS

A) Facility Description

Owing to both its physical and chemical adsorptive/filtrative properties, peat is an excellent natural filter of many types of effluents and pollutants. Because of this, and its relative availability and low cost, peat has found several uses in both industrial and domestic wastewater treatment applications. Without question, the most widespread employment of peat as a filtering material has been for the treatment of sewage effluent. The first reported employment of peat for this purpose occurred in Finland in the 1950's; where a ditched peatland area was regularly flooded with raw sewage.

Peat-sand filters (PSF) are made-soil, filtration systems which were first developed as alternative wastewater treatment systems. Their high phosphorous (P), BOD, and pathogen removal capabilities, coupled with their simple design, low-maintenance and affordability make them an attractive alternative to many conventional treatment systems. The PSF system is a hybrid filtration system. It combines the many attributes of peat with a nutrient removing grass cover crop and a subsurface sand layer to achieve high overall pollutant removal efficiency within a single, relatively compact unit. A number of peat-sand filters are being utilized in the United States for wastewater treatment. One of the most recent of these (Summer, 1988) is the 1 mgd Mayo Peninsula PSF located in Anne Arundel County, Maryland.

Peat-sand filters are just beginning to be used for stormwater quality management applications. The first such system to be used in the Washington, D.C., area will be constructed in Montgomery County, Maryland on the Hollywood Branch watershed in the spring of 1994. The staff of the Metropolitan Washington Council of Governments (WASHCOG) has participated heavily in the development of this approach. The information presented in this section is extracted from a paper by John Galli of the WASHCOG staff (the end-notes in this section are the references cited by Galli in his paper).[36]

Several design variations exist depending upon treatment objectives and waste strength. However, most incorporate the basic design features developed by Dr. R. S. Farnham[37] which is illustrated in **Figure 2-25**.

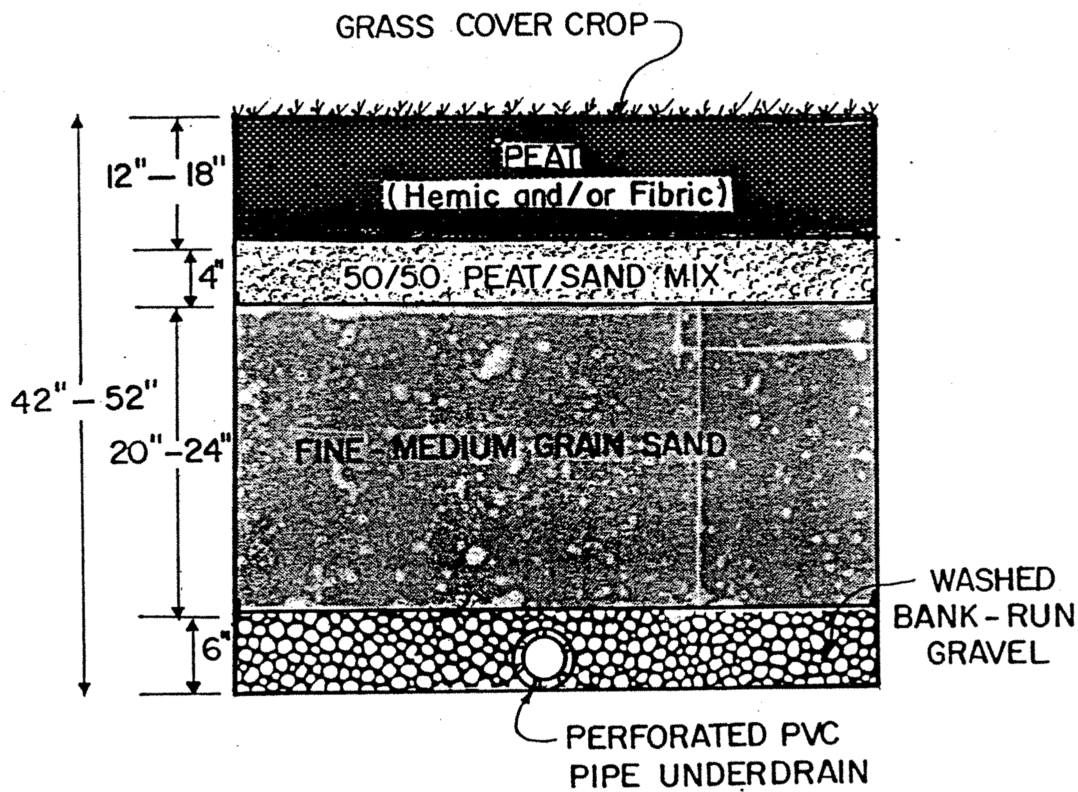


FIGURE 2 — 25 TYPICAL PEAT - SAND FILTER CROSS - SECTION

The multi-layered design of a standard PSF bed consists of five major elements: a 12 to 18 -inch thick surface layer of peat to which calcitic limestone is normally added (for greater phosphorous-removal); a 4-inch, 50-50 well-mixed layer of peat and fine-medium grain sand, a minimum 20 to 24-inch layer of clean, fine-medium grain sand; a 6 inch thick gravel underdrain (which also generally include a 4-inch diameter perforated PVC drain pipe); and a suitable, nutrient-removing grass cover crop. When used in wastewater treatment applications, the wastewater is typically applied to the surface of the peat bed via an automatic spray irrigation system.

Figure 2-26 illustrates a peat-sand filter concept developed by the staff of the Alexandria Department of Transportation and Environmental Services for use within the City. It combines features of the Austin Sand Filtration System with the peat-sand filter design proposed by Galli[36]) for use as an end-of-pipe system for a large watershed in Maryland. The Alexandria concept is intended to operate as an off-line system treating the Water Quality Volume from each storm. Any additional detention required for stormwater quantity restrictions should be provided separately downstream of the peat-sand filter system.

The sedimentation basin design shown in **Figure 2-26** is basically the same as the Austin design for full sedimentation with a sediment trap included. However, because peat-sand filter systems cannot normally operate during the more severe winter months, a gate valve is provided to shut off the flow between the sedimentation basin and the filter. Another gate valve-equipped bypass pipe is provided to pass the flow from the basin directly to the storm sewer. The invert of this pipe is placed at an elevation which will detain a permanent pool in the basin averaging at least four feet deep. In effect, this configuration converts the sedimentation basin to a small extended detention/wet pond during the winter months. Similar facilities are allowed by Seattle, Washington, in their BMP handbook[38]. As with the Austin Sand Filter, the basins may be either walled with concrete as shown or, if soil conditions permit, be soil structures.

The filtration basin is basically the Austin filtration basin design with the sand filter replaced by a Farnham peat-sand filter system.

On sites which do not provide enough vertical relief to operate the peat-sand filter by gravity flow, the system

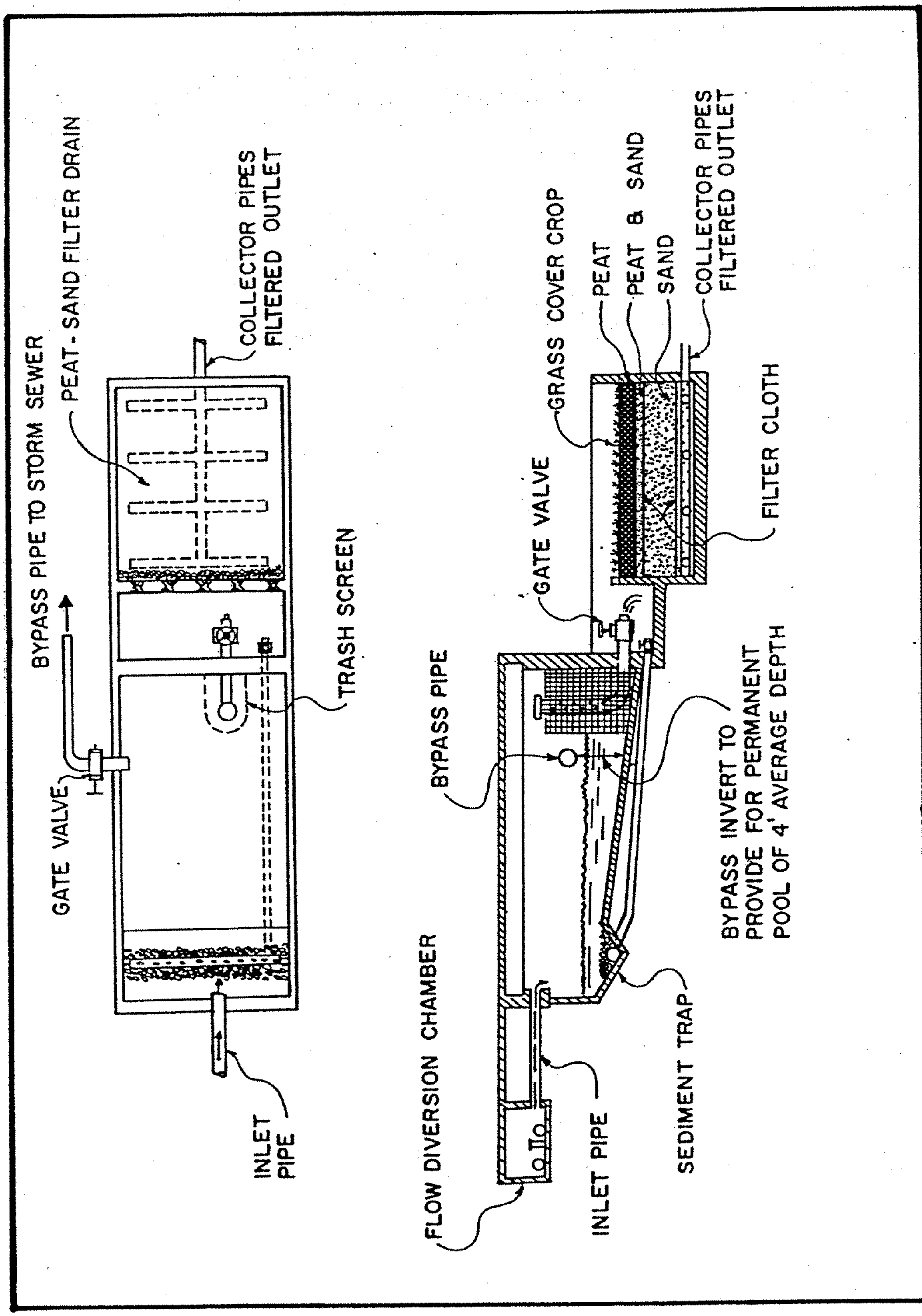


FIGURE 2-26 CONCEPTUAL PEAT-SAND FILTER SYSTEM WITH HYBRID SEDIMENTATION BASIN/WINTER PERMANENT POOL

must be augmented with a clear well and pumps to lift the stormwater from the sedimentation basin and sediment trap to the filter basin. **Figure 2-27** illustrates such a system. The permanent pool can be maintained during winter months either by providing a bypass pipe with valve directly from the sediment basin as in the gravity fed system or by adjusting the pump controls to maintain a four (4) foot deep pool in the sedimentation basin and the clear well.

B) Removal Efficiencies

Based on the known performance of various related urban BMP systems and both experimental stormwater and wastewater PSF system data, Galli estimated that the proposed PSF system should equal or exceed the pollutant removal capability of an infiltration basin system sized for 0.5 inches of storage and exfiltration per impervious acre(36). He therefore estimated that his design would have the following pollutant removal efficiency: suspended solids - 90% total phosphorus - 70% total nitrogen - 50%; BOD - 90%; trace metals - 80%; and bacteria - >90%.

However, Galli further notes that empirical stormwater total phosphorus (P) removal estimates from an EPA project (39) indicate that P-removal efficiency increases with increasing influent concentration, quoting the data shown here in **Table 2-5**.

TABLE 2-5

EMPIRICAL ESTIMATE OF TOTAL PHOSPHOROUS

REMOVAL BY A PEAT-SAND FILTER (FARNHAM AND NOONAN, 1988)(39)

Initial Phosphorous mg/L	Total Phosphorous Removal % (Range ± 95% CI)
0.1	16 (0-100)
0.2	58 (14-100)
0.3	72 (43-100)
0.4	79 (57-100)
0.5	83 (66-100)
0.6	86 (71-100)

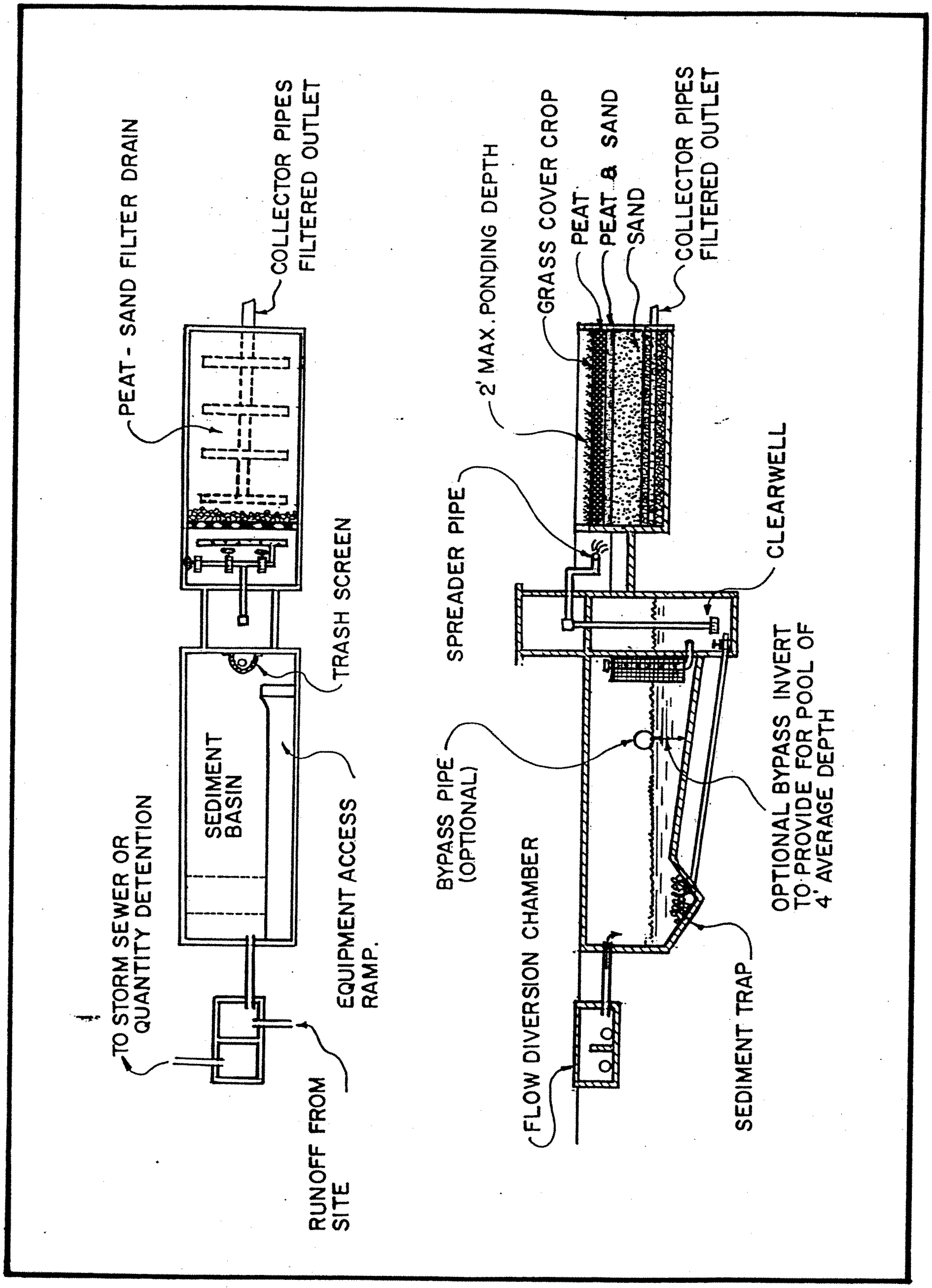


FIGURE 2-27 PEAT-SAND FILTER SYSTEM — PUMP CONFIGURATION

This data indicates that P-removal would be expected to increase with increasing watershed imperviousness/pollutant loads. In general, total P stormwater concentrations in the Washington Metropolitan Area normally range from around 0.2 mg/L to slightly over 1.0 mg/L; with higher values associated with older urban areas. [4]. It should be noted that negative nutrient removal rates may be initially experienced during PSF systems start-up. This is attributed to the possible partial washout of both P and N from the peat bed. [39,40]. The watershed for which Galli's design was intended to treat the runoff has an impervious cover of only 20 percent.

Long-term PSF systems water quality monitoring results are generally unavailable. One notable exception to this is the North Star Lake Campground PSF. This relatively small, 5,000-gal/day PSF system was constructed in 1972 to provide additional treatment of secondary sewage effluent from public campground located within Chippewa National Forest in north-central Minnesota. [41,42]. The data on this system quoted by Galli[36] indicates that over a seven-year monitoring period, the annual average phosphorous (Total P) removal efficiency of this system never fell below 98%, and the annual average nitrogen (Total N) removal efficiency remained above 50% after the first year of monitoring.

The phosphorous calculations procedures upon which the removal requirement for a developer are established assume that the stormwater to be treated has a concentration of 1.08 mg/L. **Figure 2-28** is a graph of the data from **Table 2-6** extrapolatated to 1.0 mg/l. Based on this data the Alexandria design should have a phosphorous removal efficiency approaching 90% **during the months in which the filter is in operation.** Assuming that the filter would be bypassed from mid-December to mid-March the annual phosphorous removal efficiency of the filter would approximate **67%.** Assuming a 10-15 % removal rate for the sediment basin when functioning as a small extended detention/wet pond, the total annual phosphorous removal rate of the system would be **70%.**

Contingent upon the developer agreeing to participate in a monitoring program in accordance with the monitoring protocol outlined in Chapter 3 of this manual, Alexandria provisionally recognizes a **70%** phosphorous removal efficiency for the peat sand filter design described herein.

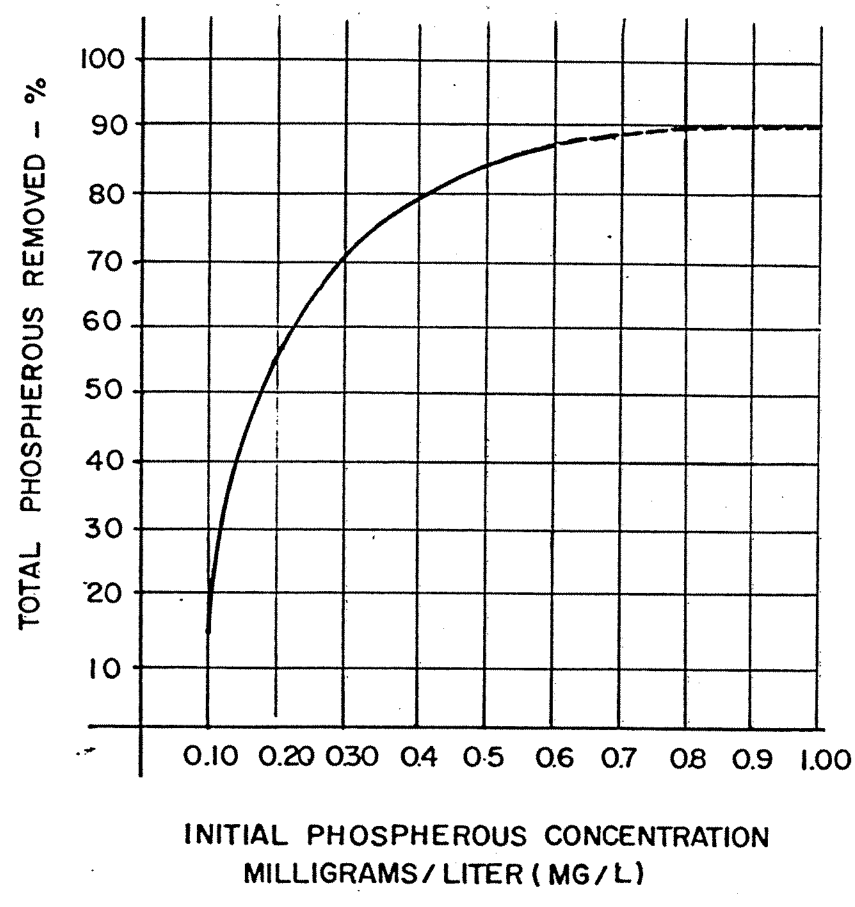


FIGURE 2 - 28—GRAPH OF EMPIRICAL ESTIMATE OF TOTAL PHOSPHOROUS REMOVAL BY A PEAT-SAND FILTER.

(DATA FROM FARNHAM AND NOONAN 1988)

C) Design Considerations

1) Applicability

A peat-sand filter system should be considered for use on developments of several acres where the pollutant removal requirement is higher than can be accommodated by other ultra-urban BMPs and insufficient space exists to install a wet pond. They should also be considered where the removal requirement is too high to be accomplished by a wet pond.

Galli[36] lists the following advantages of peat-sand filters:

- they provide a high level of water quality control;
- the filter bed can be constructed near existing ground level;
- they can be constructed in high water table areas;
- they generally not constrained by on-site soils;
- the filter bed is less prone to surface clogging due to aeration and filtering provided by grass cover crop;
- they require, vis-a-vis most traditional stormwater BMP's, less site area;
- they operates under very low head conditions;
- the general life expectancy of peat layer is <u>10-25 years;</u> and
- the projected cost is comparable with other stormwater BMP's.

2) Practicality

While peat-sand filters have been used since the 1950's to treat wastewater, they are just coming into use as stormwater quality management practices, they have a proven track record as wastewater treatment facilities.

No cost experience data exists with peat-sand filter systems approximating the design discussed here. However, except for the extra thickness of the sand layer and the addition of the peat and peat/sand layers, the other features of the design are essentially the same as the Austin Sand Filter System. The cost data from Austin contained on page 2-27 may therefore be used as a starting point.

Peat meeting the specifications for peat-sand filters is available from sources in the mid-Atlantic states. The WASHCOG staff can provide a list derived from their Water Terrace watershed experience. The cost of specification peat varies widely according to the supplier, ranging from $20 per cubic yard to $100 per cubic yard.

3) Types of Peat

Peat materials are generally differentiated on the basis of their state decomposition, acidity, absorbency, botanical origin and ash content. The USDA classifications system is normally used for specifying peat for peat-sand filters. Under thee USDA system, peats may be placed into one of three categories: fibric, hemic, or sapric.

Fibric peats include those in which the undecomposed fibrous organic materials are easily identifiable. Their bulk densities are low, often less than 0.1 g/cm 3. Because of their highly porous structure the hydraulic conductivity of slightly decomposed fibric peat can be as high as 140 cm/hr.[41]. In addition, these peats exhibit high water-holding capacities and are typically brown and/or yellow in color. The most common fibric peat is sphagnum moss, which is extremely acid.

The sapric category includes the most highly decomposed peat materials. In sapric peats, the original plant fibers have mostly disappeared. The bulk densities of the peats are relatively high, commonly 0.2 g/cm 3 or greater. Hydraulic conductivities are generally very low, with rates as slow as 0.025 cm/hr reported for highly decomposed sapric material.[41]. The water-holding capacity of sapric peat is commonly less than that of either fibric or hemic peat. Sapric peats are typically very dark gray to black in color and are quite stable in their physical properties.

Hemic peats are intermediate in their properties between those of the fibric and sapric categories. They are typically more decomposed than fibric peats but less so than sapric. In addition, they have intermediate values for both bulk density (between 0.1 and 0.2

g/cm 3) and water-holding capacity. Similarly, hydraulic conductivity and color of hemic peat are generally intermediate between those of the other two peat categories.

4) Peats for Use in Peat-Sand Filters

The proper selection of peat material (s) is essential to the successful operation and long-term functioning of the PSF system. According to Brown and Farnham [43], peat should be USDA fibric or hemic with less than 30 percent minerals by weight, and should be shredded to a uniform density before use. The **employment of sapric peat will** assuredly, as confirmed by Tomaseck et al [44], **result in system failure.**

It is most important that hemic peat not be allowed to entirely dry out, as this both destroys the bacterial microfauna and re-wetability of the material.[40] In addition, fine ground, 200-mesh `Ag-Lime' caloitic limestone is thoroughly mixed into the top 4 to 6 inches of the peat to enhance P-removal through P-sorption and precipitation through apatite formation.[43,45]

Peat will not function to remove nutrients from stormwater if anerobic conditions are allowed to develop. This is the reason that peat cannot be used in underground filter systems.

5) Accessibility for Monitoring Equipment

The isolation/diversion structure, sedimentation basin, filtered water outfflow pipe and, in pumped facilities, the clear well between the sedimentation basin and filter basin must be readily accessible for the installation of automatic monitoring equipment for measuring both flow and chemical composition of the stormwater. See Appendix 2-8 for details.

D) Design Procedures

1) Compute the Area of the Basins

The size of the sediment basin is computed exactly as that for the Austin Sand Filtration System with full sedimentation (see page 2-29).

The Austin sedimentation basin was designed to completely remove silt with a particle size of 20 microns with a specific gravity of 2.65.[1] Alternative designs for which removal of 100 percent of 30 micron silt particles with a specific gravity of 2.65 can be demonstrated will be considered.

The required area for the peat-sand filter basin was derived using both the Austin methodology employed for their sand filters and by the procedure which Galli terms "Rule 1" in his report.[36] These derivations are contained in Appendix 2-1 at the end of this chapter. Alexandria uses the Austin method, which yields:

$$A_f = 0.01I_a \tag{2-37}$$

where:

A_f is the area of the filter and I_a is the area of the impervious cover on the contributing watershed in similar units.

2) Basin Volumes

The storage capacity of the sedimentation basin shall be equal to or greater than the Water Quality Volume. A minimum freeboard of 0.5 foot of freeboard above the maximum water surface elevation shall be provided. A sediment trap shall be provided at the bottom of the basin and may be credited with up to five percent of the water quality volume.

The filtration basin should be designed for a ponding depth of two (2) feet above the filter surface. Any greater depth of ponding will risk compression of the peat layer due to the weight of the water. Provision also should be made to store up to a total of 20 percent of the Water Quality Volume to allow for backwater effects resulting from partial clogging of the filter media.

3) Sedimentation Basin Details

The basic sedimentation basin should be designed exactly as that for the Austin Sand Filtration System with full sedimentation (see pp. 2-28 through 2-38).

The PSF sedimentation chamber has the following additional requirements:

- o If the gravity flow configuration is used, a bypass connection to the storm sewer must be provided for use during the winter months shutdown of the PSF. This bypass must be equipped with a gate valve for shut-off and flow control, and the outlet invert must be placed to provide a permanent pool in the sediment chamber averaging four (4) feet deep.

- o If the pumped transfer configuration is used, providing the bypass connection is optional but highly recommended. Pump controls may be used during the winter months to maintain the required four-foot permanent pool in the sedimentation chamber and clearwell. However, installation of the bypass allows total gravity flow operation during the filter shut-down period.

4) Filter Basin Details

Figure 2-29 shows a cross section of the peat-sand filter bed.

- o **Peat Layer**

  The upper peat layer must be a minimum of 12 inches thick. In order to eliminate the possibility of saturation of the peat bed, a higher 1.0-inch/hr infiltration rate (instead of the more common 0.25-in/hr rate) is required to limit the maximum surface ponding time for the design Water Quality Volume to 24 hours. Achieving this higher hydraulic conductivity rate requires a custom-blend hemic/fibric peat mixture.

  In addition, comprehensive independent laboratory soils analyses of prospective peat material must be performed (prior to bulk purchase) to determine its cation exchange capacity; Fe, Al, $CaCo_3$, ash and nutrient content; bulk density; hydraulic conductivity; etc. It is further recommended that a soils scientist/engineer provide assistance in both the selection and certification of peat material (s) and supervision of peat bed basin construction.

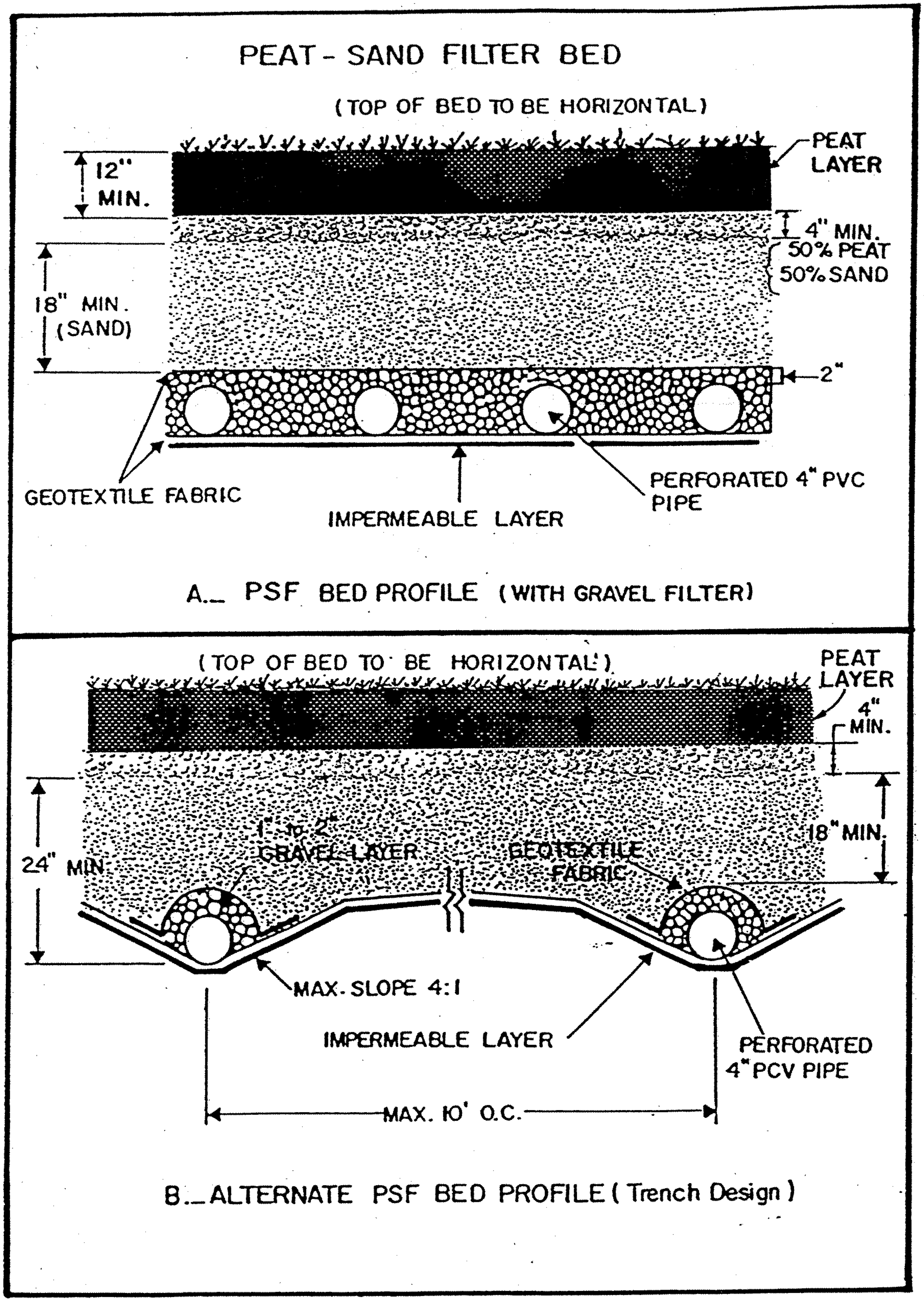


FIGURE 2-29 PEAT-SAND FILTER BED CONFIGURATIONS

Approximately 1.5 inches (or 25% by volume) of 'Ag-Lime' calcitic limestone must be mixed into the top 4 to 6 inches of peat.

During bed construction, peat must be placed in incremental two to four-inch layers, lightly compacted with a lawn roller and raked to ensure good, even contact with the next overlying layer of peat.[40,45)

- **Peat/Sand Layer**

  A minimum four-inch layer of 50% peat meeting the same specification as the upper peat layer and 50% sand meeting the same specification as the underlying sand layer must be placed immediately under the peat layer. This homogeneously mixed layer provides continuous contact between the peat and sand layers, ensuring a uniform flow of water through the bed. Proper installation of this layer is critical to the proper installation of the filter. A layer of geotechnical cloth meeting the specification described under the sand layer shall be placed between the peat/sand layer and the underlying sand.

- **Sand Layer**

  Sand is placed beneath the peat layer to serve as a vacuum pump for drawing water through the bed.The sand layer shall be a minimum of 24 inches thick. Otherwise, the details of the sand layer, the underdrain system, and the intervening geotechnical cloth layer shall be the same as for an Austin Sand Filtration System (see pp 2-39 through 2-42).

- **Filter Basin Liner**

  An impermeable geomembrane liner is required to both eliminate potential groundwater infiltration or exfiltration problems and to permit better water quality performance monitoring of the system. The geomembrane shall have a minimum thickness of 30 mils and be ultraviolet resistent.

- **Observation Wells**

  Observation wells, spaced at 50-foot intervals, must be provided for inspection and monitoring of the peat bed.

- **Grass Cover Crop**

  A cover crop of reed canary grass (Phalaris arundinacea), rough-stalked bluegrass (Poa trivialis) or an equivalent grass cover crop with characteristically high nutrient removal capability, high tolerance to regular flooding, and resistance to striped smut, brown spot and/or other fungal diseases must be plug-planted into the top surface of the peat and nurtured until it is well established. Meadow fescue and marsh foxtail are less desirable alternatives. Quackgrass is considered to be a noxious weed in several states, including Maryland, and shall not be used in PSFs in Alexandria.

  With regard to the selection of the grass cover crop, only flood and disease tolerant species should be used. For watershed areas exhibiting very high chlorides loadings, a salt-tolerant cover crop may be required.[46] Consultation with an agronomist, prior to cover crop selection, is therefore strongly recommended.

E) Construction and Maintenance Requirements

A Maintenance Agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with the Chapter 3 of this manual must be executed by the developer/owner **before** the Final Site Plan for the development will be released for construction.

Construction and maintenance requirements for Peat-Sand Filter Systems are delineated in detail on pages 2-A3-9 through 2-A3-11. These requirements shall be reproduced verbatim on the Stormwater Management Plan sheets of the Final Site Plan.

IX. D.C. MANHOLE FILTER SYSTEMS

A) Facility Description

The District of Columbia Environmental Regulation Administration staff have adapted their sand filter system to fit inside a standard precast manhole structure. Stormwater runoff is fed into the filter through a sump catchbasin to trap trash, hydrocarbons, and heavy sediment. If the manhole structure is not deep enough to store the entire WQV awaiting treatment, additional storage must be provided outside the manhole shell. **Figure 2-29A** illustrates a D.C. Manhole Filter with additional storage provided in an aluminum or aluminized steel arched corrugated metal pipe.

B) Pollutant Removal Rate

The pollutant removal efficiency of the D.C. Manhole Filter should be the same as the basic D.C. Sand Filter. fAlexandria recognizes a Total Phosphorous Removal of 40 percent for D.C. Manhole Sand Filters.

C) Design Considerations

1) Applicability

D.C. Manhole Sand Filters are especially applicable to small development sites where a BMP is required. With five feet of ponding depth over the filter, a six-foot manhole filter will serve over 5,000 square feet of impervious area, and an eight-foot manhole filter will serve over 10,000 square feet of impervious area.

2) Practicality

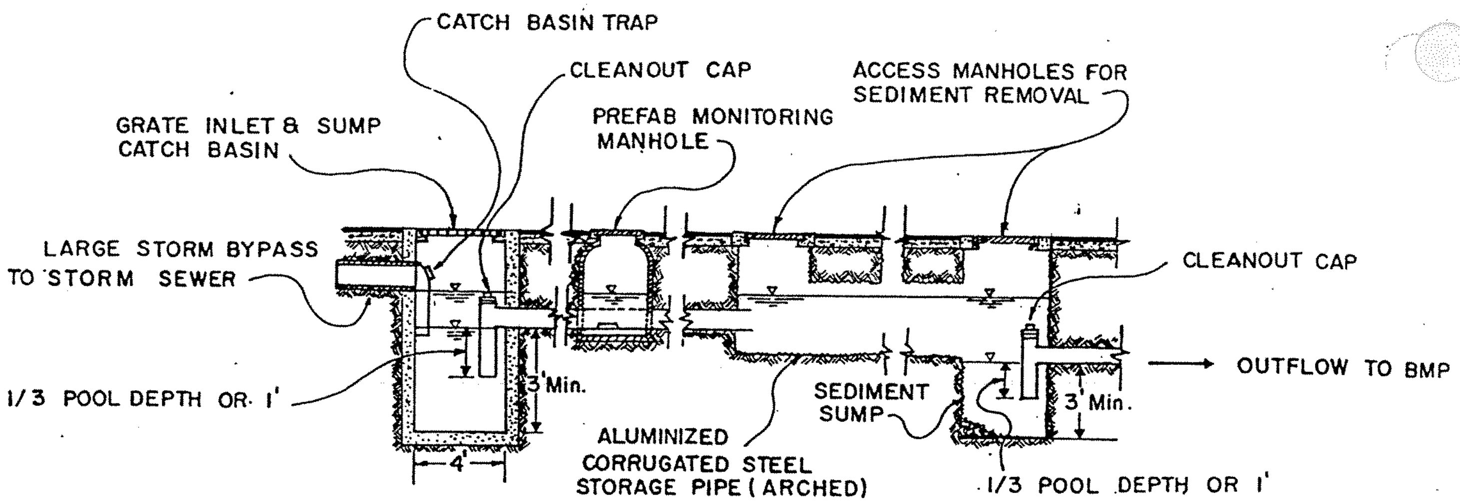
This system uses readily available precast concrete and corrugated pipe components for housing the storage and treatment components of the BMP. While no actual construction costs are as yet available, significant savings over a custom-built system should be expected.

The Design Considerations for standard D.C. Sand Filters outlined on pages 2-51 through 2-53 also apply to manhole filters.

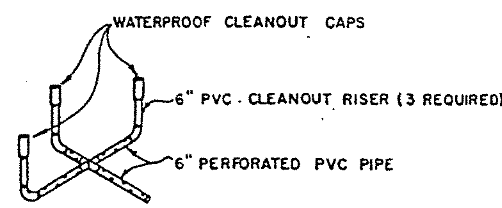
D) Design Procedures

1) Determine Governing Site Parameters

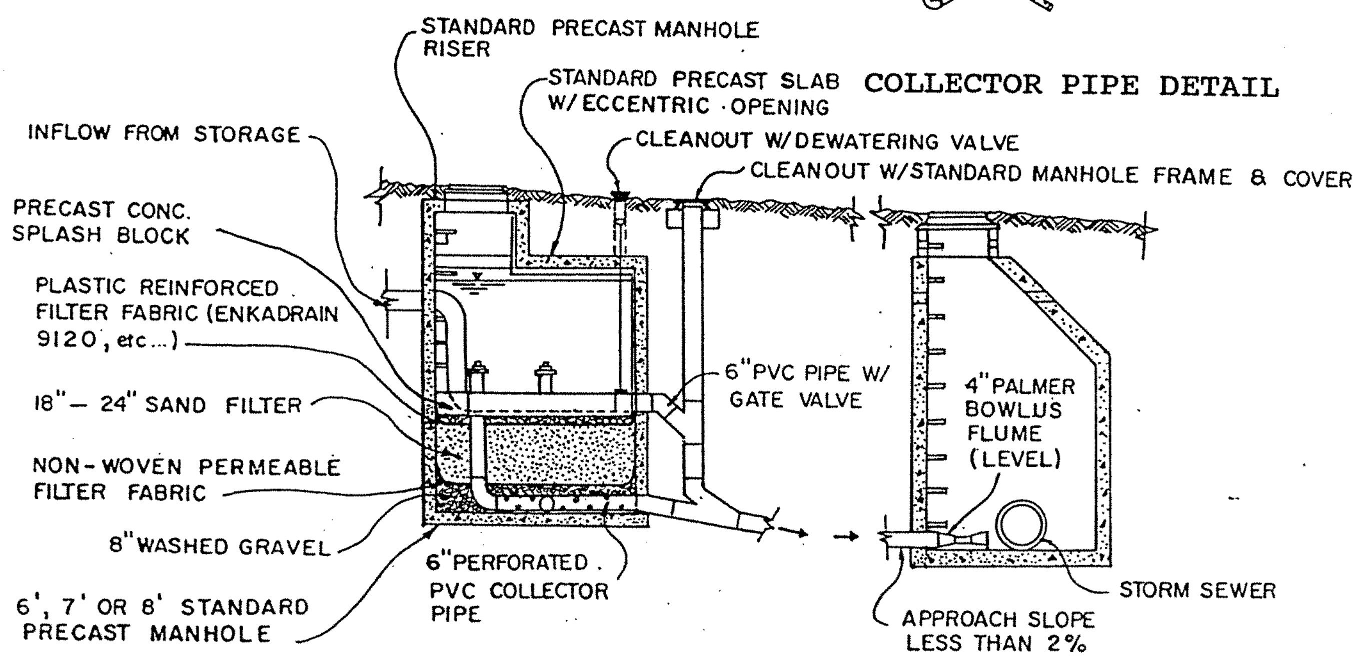
(same as for DCSF -- see page 2-53)



GRATE SUMP INLET AND STORAGE PIPE



COLLECTOR PIPE DETAIL



MANHOLE FILTER   STORM SEWER CONNECTION

FIGURE 2-29A -- D.C. MANHOLE SAND FILTER WITH PIPE STORAGE (OUTFITTED FOR MONITORING)

2) **Select Filter Depth and Determine Maximum Ponding Depth** (same as for DCSF -- see page 2-54)

3) **Compute the Minimum Area of the Sand Filter** (A_{fm}) (same as for DCSF -- see page 2-54)

4) **Select a Manhole with a Cross-Sectional Area $\geq A_{fm}$**

Precast manholes of 5, 6, 7 or 8-foot inside diameter may be used.

Design Steps 5) through 8) are identical to those of the DCSF (see page 2-55)

9) **Design Pipe Storage to Complete WQV Storage**

Worksheet J on pages 2-A3-24 and 2-A3-25 are provided to assist in the above calculations.

F) Filter Specifications and Details

Filter specifications and details are essentially the same as for the thin DCSF (see pages 2-57 through 2-59) with the addition of the following:

1) Manholes

Manholes shall normally conform to Alexandria Design and Construction Standard CSMH-2A. Precast designs with equivalent design characteristics may be proposed for approval by the Director.

G) Maintenance and Construction Requirements

A Maintenance Agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with the Chapter 3 of this manual must be executed by the developer/owner **before** the Final Site Plan for the development will be released for construction.

Construction and maintenance requirements for D.C. Manhole Sand Filter Systems are the sand as those delineated in detail on pages 2-A3-5 through 2-A3-6 for standard DCSFs. These requirements shall be reproduced <u>verbatim</u> on the Stormwater Management Plan sheets of the Final Site Plan.

X. <u>TRENCH SAND FILTER SYSTEMS</u>

A) Facility Description

The filter system concepts embodied in the Austin and District of Columbia designs may be readily adapted for small and less complex applications. This section discussed two simple trench sand filter concepts which lend themselves to use on such projects as townhouse developments or small commercial redevelopments.

1) Trench Sand Filter With Stone Reservoir

This filter system is constructed in a trench lined with impervious geomembrane sandwiched between protective layers of filter cloth. The bottom of the trench contains a simple sand filter which is connected to the storm sewer. The upper part of system is built exactly like an infiltration trench designed to treat the first one-half inch if runoff (the Water Quality Volume).

Figure 2-30 illustrates one variation this concept with optional perforated corrogated metal storage pipes. Dispersed overland sheet flow passes over a gently sloped grassed filter strip to the surface of the storage reservoir. For the grassed filter strip to perform as a primary sediment control, it must be at least 20 feet wide with a slope no greater than five (5) percent. The reservoir is further protected from sediment clogging by a layer of geotechnical filter cloth six (6) inches beneath the top surface of the aggregate. The WQV flows into the reservoir until the voids in the rock are completely full. Any overflow is directed to the storm sewer. Runoff collected in the reservoir filters down through the sand to the collector pipe, from which it is piped to the storm sewer.

2) Trench Sand Filter With Small Pond Sedimentation

Trench sand filters with a depth of only about three (3) feet are possible if sedimentation is accomplished in a small pond or pool. Such a pond for a multiple family residence project or small commercial development could be designed as a landscaping amenity with the capacity to store the WQV above a very shallow (perhaps as little as one foot deep) permanent pool. As with the Delaware Sand Filter, the permanent pool

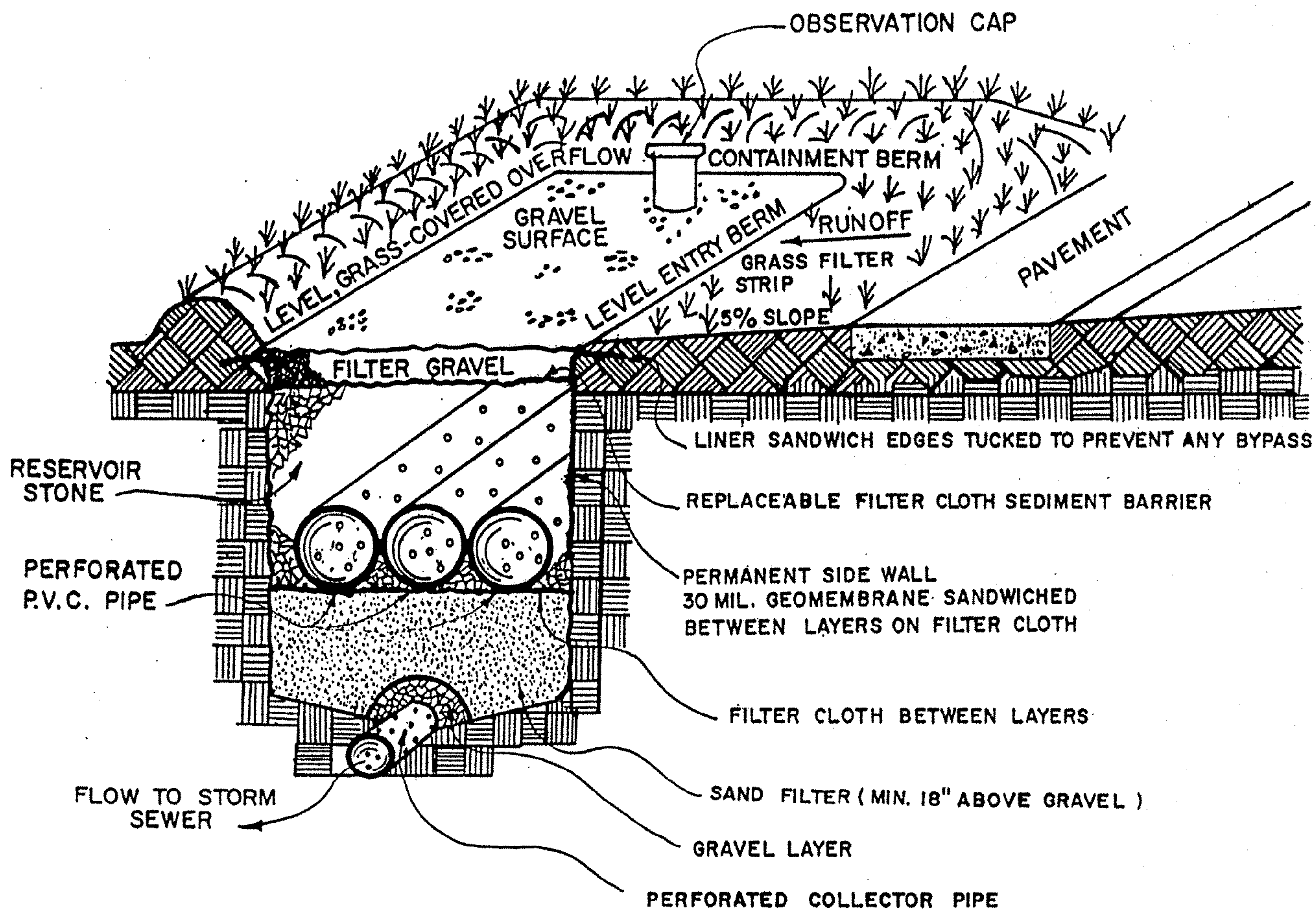


FIGURE : 2 – 30 TRENCH SAND FILTER WITH STONE RESERVOIR traps the coarser sediments and provides some small measure of nutrient removal. The filter element would essentially be a one-pipe strip of the Austin Sand Filtration Basin with a protective cover of geotechnical cloth and filter stone as with the D.C. Sand Filter.

Figure 2-31 illustrates this concept. The sediment pond is a landscape pool with an aerating fountain (a visual amenity to make it more attractive). The WQV is directed into the pond using an isolation/diversion device similar to the Austin "smart box." Any overflow goes directly to the storm sewer. The WQV is slowly released to the filter through an orifice plate or a small gate valve. The trench filter is integrated into the landscaping plan and covered with a layer of white landscaping stone to create the illusion of a small, flat rock garden. The collector pipe at the bottom of the filter is piped directly to the storm sewer.

B) POLLUTANT REMOVAL RATES

Trench sand filter systems constructed to the standards outlined below should have the same removal efficiency as an Austin Sand Filter. If the developer agrees to monitor the facility in accordance with the monitoring protocol contained in Chapter 3 of this manual, Alexandria will recognize a 40 percent removal efficiency for this BMP.

C) DESIGN CONSIDERATIONS

1) Applicability

Trench sand filters may be considered for small sites which have a pollutant removal requirement below 40 percent where the proposed development does not require a more sophisticated BMP. Small multi-family residential developments or a small commercial office plaza might be typical examples. In instances where single family homes require stormwater pollutant removal and infiltration is not practical because of soil conditions, trench sand filters may offer a practical alternative. They are not, however, recommended for treatment of stormwater runoff that is primarily from parking surfaces.

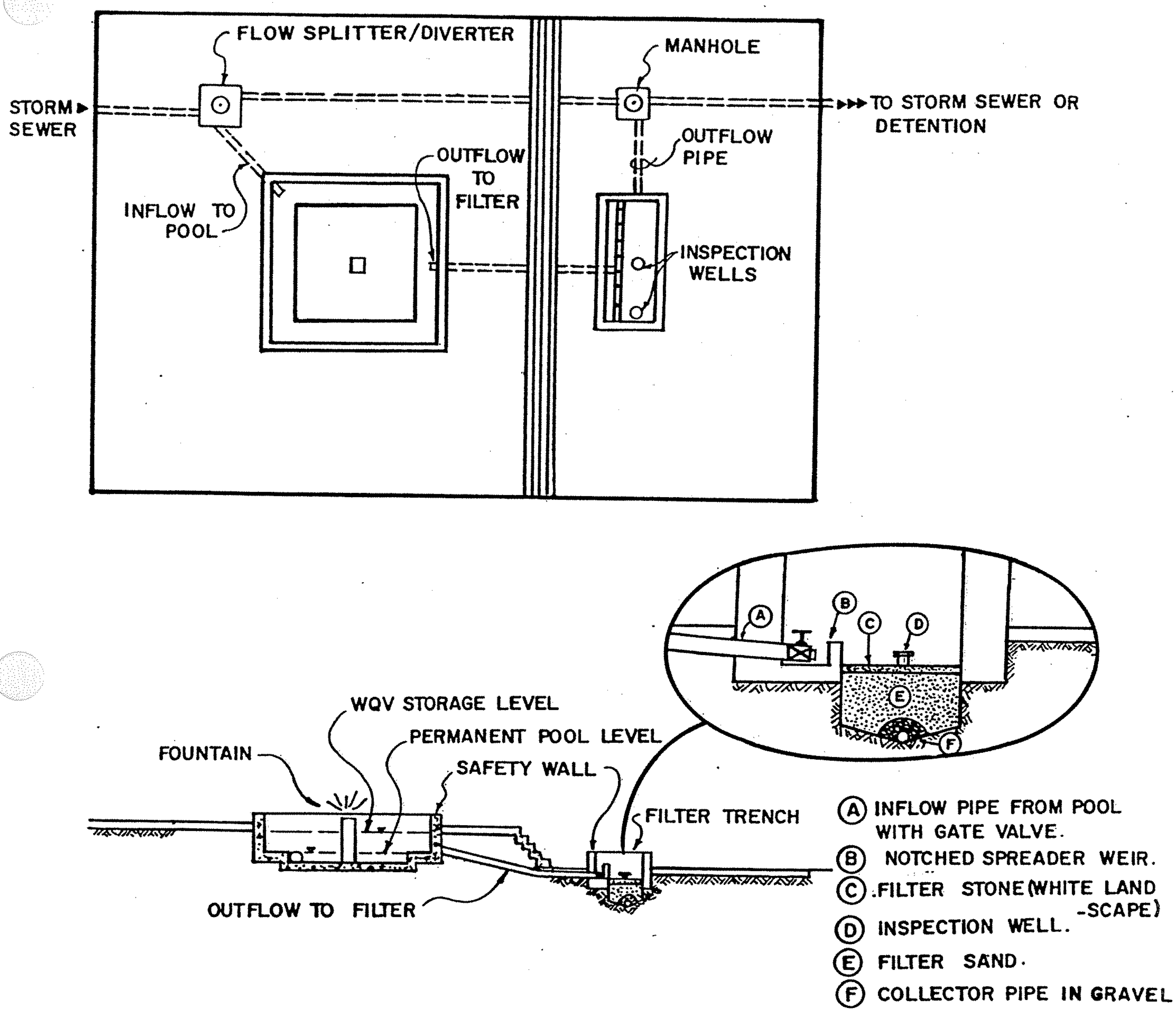


FIGURE 2-31 PLAZA TRENCH SAND FILTER WITH FOUNTAIN SEDIMENTATION POOL

2) **Practicality**

Since trench sand filters are built directly into the landscape, they should prove much less expensive that systems requiring structural concrete.

3) **Topography and Receiving Storm Sewer Elevation**

Normally, the topography should offer sufficient relief to allow a trench sand filter system to function by gravity flow. However, pumps may be employed if necessary to move the water from the sedimentation pool to the filter bed.

4) **Accessibility**

Where used, sedimentation pools will require removal of sediment accumulation on a periodic basis. Stone reservoirs will require the periodic removal and replacement of the upper filter cloth and gravel layer. Accessibility for this work must be accommodated in the original design. The flow separation/diversion chamber and the junction of the filter outflow pipe and the storm sewer must be accessible for inspection and monitoring of flow rate and the pollutant content of the stormwater.

5) **Special Considerations for Stone Reservoir Systems**

Refer to Chapter 5 of the <u>NVBMPHB</u> for detailed discussions on the design of infiltration systems. The sections concerning protection of the stone reservoir from excessive sedimentation and handling storms producing runoff greater than the design volume are especially crucial. **An observation well must be installed at the mid-point of the trench, penetrating to the bottom of the stone reservoir but not through the underlying filter cloth or into the sand bed.**

6) **Safety considerations for Small Sedimentation Ponds**

Safety barriers must be included in the design to preclude small children from wandering into the pool.

D) DESIGN PROCEDURES FOR TRENCH SAND FILTERS WITH STONE RESERVOIRS

1) Size the Stone Reservoir

Follow the procedures outlines in Chapter 5 of the NVBMPHB to size the stone reservoir for infiltration trenches. This involves computing the Water Quality Volume to be treated, selecting the stone for the reservoir, and then computing the volume of rock necessary to store the water quality volume in the voids between the stones (an alternative to an all-stone reservoir would be to place one or more perforated pipes in the stone to increase the percent of storage volume available). Dimensions of the trench portion which is to hold the rock (or rock and perforated reservoir pipes) will usually dictate the trench configuration. Single collector pipe trench filters should normally be no wider that six (6) feet. Wider trench filters should employ multiple collector pipes spaced no wider than six (6) feet center to center. The vertical dimension will also be affected by the elevation of the storm sewer into which the filter is to flow.

2) Check the Trench Bottom Dimensions against the Minimum Filter Bed Area

Use the formula for the Austin Sand Filtration System with Partial Sedimentation to compute the minimum filter bed area:

$$A_f = \frac{545 I_a d_f}{(h + d_f)} \qquad (2\text{-}13)$$

Where A_f is the area of the sand filter in ft^2, I_a is the impervious area in acres, d_f is the depth of the filter in ft, and h is the average depth of pooling in the stone reservoir (1/2 maximum depth of ponding).

Practical dimensions for the stone reservoir will usually provide a filter area several times the size of the computed minimum.

The stone reservoir may be separated from the filter either by a layer of geotechnical filter fabric or 3-4 inches of pea gravel.

3) **Complete the Design of the Filter Component**

Filter component design is identical for the various trench filter configurations and are outlined separately below.

4) **Observation Wells**

Two observation wells are required. The first shall be of the perforated pipe material as the collector pipe and be located at the mid-point of the trench; it shall extend to the bottom of the stone storage reservoir **but not pierce the geotechnical fabric between the reservoir and the filter sand** (to preclude water from bypassing the filter through the observation pipe). The second shall be a 90 degree extension of the collector pipe as used with the D.C. Sand Filter. It shall have no perforations above the geotechnical fabric which separates the stone reservoir and the filter sand. Both wells shall be equipped with vandal resistant caps.

E) DESIGN PROCEDURES FOR TRENCH SAND FILTERS WITH SMALL POND SEDIMENTATION

1) **Compute the Sedimentation Pond Surface Area**

Use the curve for the Austin Sand Filter with full sedimentation (see page 2-30). This yields a surface area of approximately 900 square feet per impervious acre if WQV ponding depth is two (2) feet above the permanent pool, 600 square feet per impervious acre for three (3) foot ponding depth, and 450 square feet per impervious acre if the additional ponding depth is four feet. Select a depth and other dimensions that are compatible with the landscaping plan and other considerations such as safety.

2) **Complete the Detailing of the Sediment Pond**

Details of sediment pond design should conform as closely as possible to the specifications for the Austin design outlined on pages 2-31 through 2-39 with modifications as necessary to fit the scale of the smaller trench system. Some specific concerns include:

- Impermeable pond liner -- at least a geomembrane liner will be required. Conform to Austin specifications.

- o Outlet Structure -- a pipe protected by a trash rack and equipped with a small gate valve for control will suffice in most cases. The gate valve can be adjusted during the first storm use to provide the required 24 hour drawdown time.

- o Sediment traps -- sediment traps are usually not necessary if a small permanent pool is to be retained for aesthetic purposes.

3) Compute the Minimum Area of the Filter Bed

Use the formula for the Austin Sand Filter with full sedimentation:

$$A_f = \frac{310 I_a d_f}{(h + d_f)} \qquad (2\text{-}12)$$

where A_f is the area of the filter bed in ft^2, d I_a is the impervious area on the watershed being served in acres, d_f is the depth of the filter, and h is the average ponding depth above the filter (1/2 the maximum ponding depth).

4) Select the Top Filter Stone

Like the D.C. Sand Filter, exposed surface trench sand filters must be protected with a layer of geotechnical cloth meeting the specifications outlined below under **Trench Liner** and a layer of filter stone at least three (3) inches thick. **Figure 2-31** shows a typical cross-section. Material specifications shall be the same as for the D.C. filter bed, except that landscaping stone may be substituted for the sake of esthetics.

F) FILTER BED DESIGN PROCEDURES COMMON TO ALL CONFIGURATIONS

1) Trench Width

Trench sand filters should normally not exceed six feet in width. Where greater widths are required for such applications as the stone reservoir configuration, multiple collector pipes spaced no more than six (6) feet center-to-center must be employed. The bottom of the trench shall be sloped toward the pipes at a maximum slope of 4 horizontal to 1 vertical.

2) Trench Lining

Trench sand filters shall be lined with ultraviolet resistant impermeable geomembrane with a minimum thickness of 30 mils. Geotechnical fabric meeting the following specification shall be placed below and on top of the geomembrane for puncture protection:

Property	Test Method	Unit	Specification	
Material	Nonwoven geotextile fabric			
Unit Weight		Oz/Sq.Yd.	8	(min.)
Filtration Rate		In/Sec	0.08	(min.)
Puncture Strength	ASTM D-751(Modified)	Lb.	125	(min.)
Mullen Burst Strength	ASTM D-751	Psi	400	(min.)
Tensile Strength	ASTM D-1682	Lb.	300	(min.)
Equiv. Opening Size	US Standard Sieve	No.	80	(min.)

Equivalent methods for protection of the geomembrane liner will be considered by the Department of Transportation and Environmental Services on a case by case basis. Equivalency will be judged on the basis of ability to protect the geomembrane from puncture, tearing and abrasion.

The trench lining sandwich shall extend to the surface, and the edges shall be tucked under the adjacent soil to anchor them in place.

When a stone reservoir is used above the filter, a layer of geotechnical fabric conforming to the above specification shall separate the stone from the sand filter.

3) Sand Bed

The sand bed shall be **ASTM C33 Concrete Sand**[32] or **VDOT Section 202 Grade A Fine Aggregate Sand**[30] at least 18 inched thick directly above the pipe. The collector pipe shall be covered by at least two inches of 1/2 to two (2) inch gravel and geotextile fabric conforming to the above specification. Internal diameter of the pipe must be four (4) i) inches or greater and perforations shall be 3/8 inch. All piping is to be schedule 40 polyvinyl chloride or greater strength. The minimum grade of piping shall be 1/8 inch per foot (one (1) percent slope).

Access for cleaning all underdrain piping is needed. A vertical cleanout and inspection well shall be provided at the upstream end of each collector pipe, and the outfall shall be accessible for cleaning and instrumentation for both flow rate and chemical monitoring where it empties into the storm sewer.

G) ROOF DOWNSPOUT SYSTEM

1) System Description

A roof downspout system is a trench sand filter or infiltration system intended only for treating or infiltering runoff from downspout drains. This BMP is not designed to treat or infiltrate any surface water that could transport sediment or pollutants such as from paved areas. **Figure 2-31A** depicts a typical roof downspout system.

Roof gutters must be covered with rigid mesh screens to exclude leaves and other large debris from entering the system. The downspouts are connected to the trench sand filter or infiltration system through a sump catchbasin to remove any debris which may wash through the gutter screens. After passing through a fine mesh screen, runoff enters the stone reservoir through a perforated distribution pipe. The infiltration trench or trench sand filter are sized and constructed in the usual manner prescribed earlier in this section or in the NVBMPHB.[3] Specific limitations on the placement of infiltration systems must also be observed.[3]

2) Applicability

Roof downspout systems may be considered for application to single family residences where the degree of impervious cover requires a BMP or for roof water treatment for townhouse or other small development projects such as condominiums.

3) Maintenance

Gutter screens and the sump catchbasin must be inspected semiannually and cleaned of any collected debris. Maintenance is the responsibility of the property owner.

G) CONSTRUCTION AND MAINTENANCE REQUIREMENTS

Construction and maintenance requirments for Trench Sand Filter systems are delineated in detail on pages 2-A3-12 through 2-A3-14. These requirements shall appear verbatim on the Stormwater Management Plan sheets of the Final Site Plan. The Maint./Monitoring agreement must be executed by the developer prior to the release of the Final Site Plan.

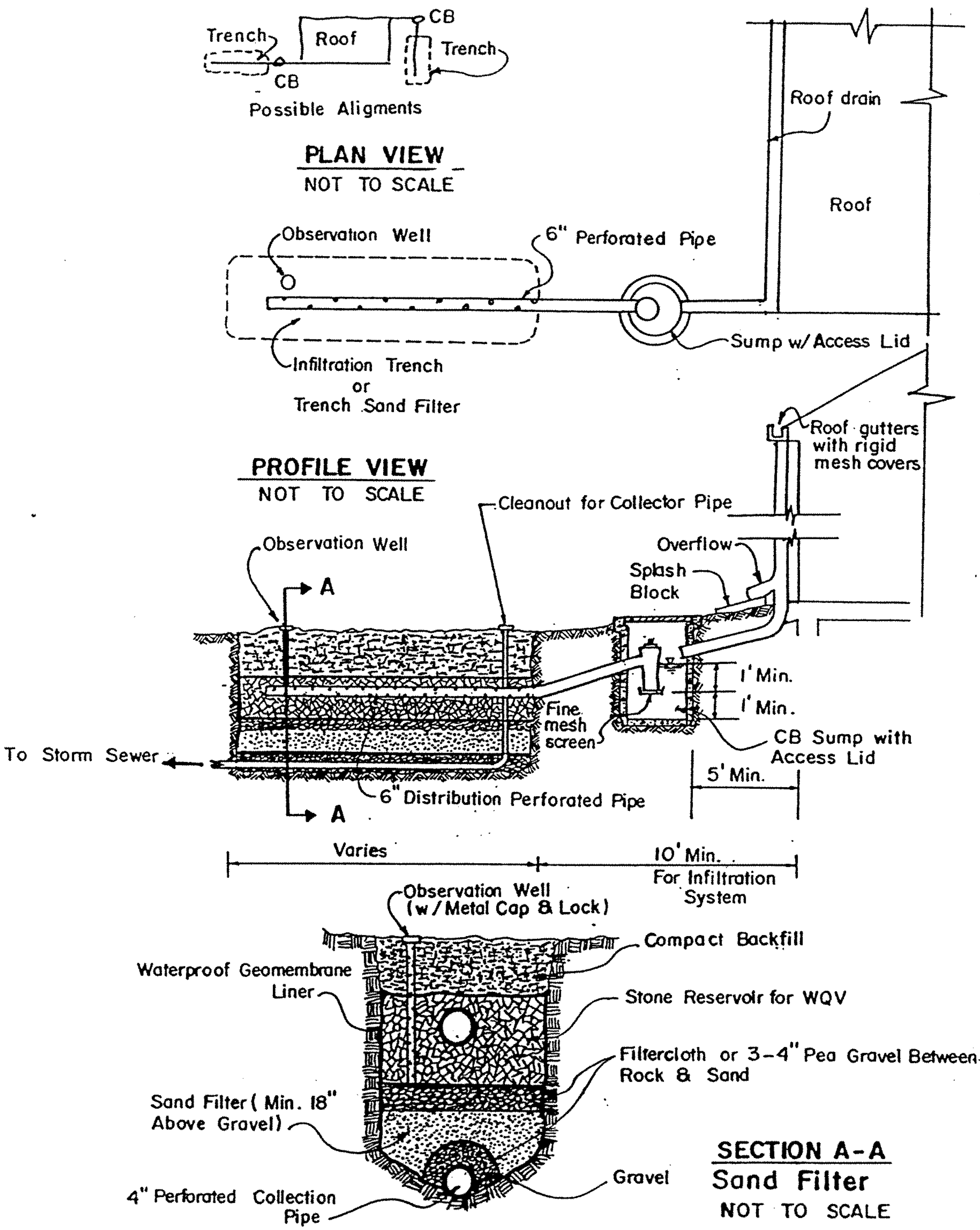


FIGURE 2-31A -- ROOF DOWNSPOUT SYSTEM

XI. <u>INFILTRATION</u> <u>WELLS</u>

A) Facility Description

Infiltration wells convey stormwater directly into deep permeable strata through well casings drilled through overlying impermeable strata. A concept used by one jurisdiction which has problems with creating successful infiltration devices at the surface because of underlying marine clay is to build a conventional infiltration trench with several sand-filled well casings spaced along the bottom. The well casings are drilled through the marine clay layer and well into a strata of sand and gravel. Commercial systems are available in which a replaceable filter of gravel, sand and activated charcoal is placed in an open casing penetrating to a permeable sand layer. Figure 2-30 illustrates such a system (note that a patent is pending on this system).

B) Removal Efficiencies

The removal efficiency of an infiltration trench with sand-filled well casings will be recognized as that of the basic infiltration trench in the <u>NVBMPHB</u>. The removal efficiency of commercial type infiltration wells will be determined for each project after review of the manufacturer's technical data, which must be submitted as part of the Stormwater Management Plan.

C) Design Considerations

1) Applicability

Infiltration wells may be considered for situations where a higher removal rate is required than can be provided by sand filtration systems, soil conditions preclude conventional infiltration devices and the site is too small to support a peat-sand filter. The geology of the site must provide a permeable strata which is capable of accepting the runoff involved.

2) Practicality

Experience with infiltration wells is fairly limited. Jurisdictions which have employed infiltration wells have successfully operated them for two (2) to three (3) years.

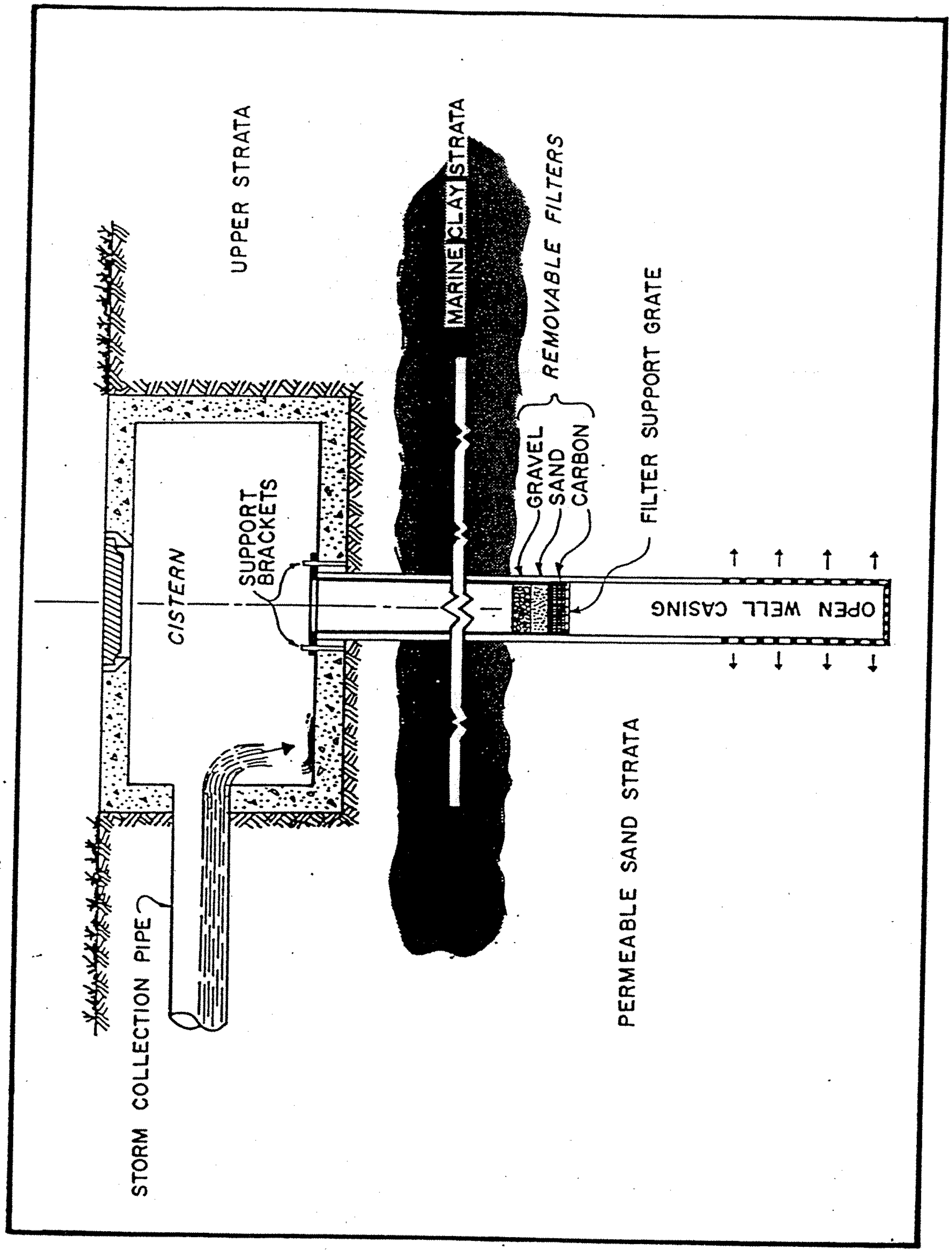


FIGURE: 2-32 COMMERCIAL INFILTRATION WELL SYSTEM
(PATENT PENDING)

One factor which must be taken into account when considering infiltration wells is that they fall under the definition of **Underground Injection Wells under 40 CFR 144.12 and are subject to regulation by EPA.** Stormwater infiltration wells appear to be Class V Underground Injection Wells, a class that does not automatically require a permit from the Underground Injection Control (UIC) administrator but are also not automatically excluded from requiring a permit. The administrator may require the submission of a permit application if:

- o the injection well is not complying with the provisions of the authorizing rules,
- o the well has ceased to be in the category of wells authorized by the rule, or
- o the protection of underground sources of drinking water requires additional regulation by permit.

Owners or operators of underground injection wells are required to report the existence of these wells to EPA, which is continually updating an inventory of these wells in each state. The UIC administrator for Virginia is Region III of EPA located in Philadelphia, Pennsylvania. The Region III staff appears open to proposals to use filtered systems such as the one illustrated in Figure 2-29 but are not encouraging concerning infiltration trenches with sand-filled well casings.

Designers/developers who are considering the use of infiltration wells should consult with the Region III staff at (215) 597-9031 or 9928.

3) Soil Suitability Investigation

Refer to Chapter 5 of the NVBMPHB for soil suitability investigation requirements for infiltration devices.

D) Design Procedures

1) Infiltration Trenches with Wells

Employ the procedures outlined in Chapter 5 of the NVBMPHB for infiltration trenches. Design the wells to hydraulically empty the stone reservoir within 48 hours.

2) **Commercial Infiltration Wells**

Obtain the manufacture's design procedures and consult with the Transportation and Environmental Services Engineering staff before finalizing the design.

E) **Maintenance Requirements**

1) **Infiltration Trenches with Sand-Filled Wells**

See Chapter 5 of the NVBMPHB.

2) **Commercial Systems**

Filters must be changed on a regular basis. Obtain the manufacture's recommendations and consult with T&ES Engineering staff to establish requirements.

3) **Maintenance Agreements**

A Maintenance Agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with the Maintenance Agreement chapter of this manual must be executed by the developer/owner before the Final Site Plan for the development will be released for construction. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services Inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected within specified time limits.

Received from Mr. Bell
8-23-95

XII. RESIDENTIAL DRY VAULT SAND FILTER

The Residential Dry Vault Sand Filter was developed by the Alexandria Transportation and Environmental Services Engineering staff to address the unique problems of vault sand filters which are owned by Homeowners Associations. This filter is not intended for use on commercial or industrial applications.

The Residential Dry Vault Sand Filter is designed to minimize maintenance costs. There are no permanent pools of water which must be periodically pumped out. No layers of geotechnical filter cloth are employed, eliminating predictable failure planes. For filter lengths of 15 feet or less, collector pipes with filter fabric wrappings are also eliminated, relying instead on the spaces between large gravel to convey the filtered water to the storm sewer.

These design concepts are based on the premise that the property owners have a vested interest in preventing introduction into the stormwater system of pollutants such as oils, antifreeze, yard wastes, and trash, which would cause premature failure of the filter. Developers are required to provide printed brochures explaining the filter and the responsibilities of the owners to each homeowner and to the association. The design also presumes that the upstream stormwater system contains positive trash-excluding features such as grate inlets or trash grates in flow-splitting devices. Such features are important since vault filters are Confined Space as defined by the U.S. Occupational Safety and Health Regulations, and operations to clean trash and sediments and make filter repairs in the vaults must comply with Confined Space requirements.

A) Facility Description

The Residential Dry Vault Sand Filter utilizes concepts from the Austin Partial Sedimentation Sand Filter, the D.C. Sand Filter, and the Delaware Sand Filter. The system is contained in a three chamber underground vault vault. **Figure 2-33** illustrates one version of the system.

The first chamber is the dry sedimentation chamber from the Austin design. Scouring and resuspention of accumulated sediments is minimized by placing a blast wall in front of the input pipe as shown or by turning the input pipe downward and providing a "T" to direct the inflow against both sidewalls. Flow of water to the filter chamber is controlled by a series of outlet ports in the intervening wall. An additional free-flow rectangular opening may be placed in the wall with a bottom elevation of at least six feet above the sediment chamber floor. A gabion wall may also be used as an alternative to the concrete dividing wall. The sediment chamber must store at least 20 percent of the Water Quality Volume.

The second chamber contains the distribution trough and the intermittent sand filter. The two-foot wide distribution trough with notched weir plate assures that flow reaching the

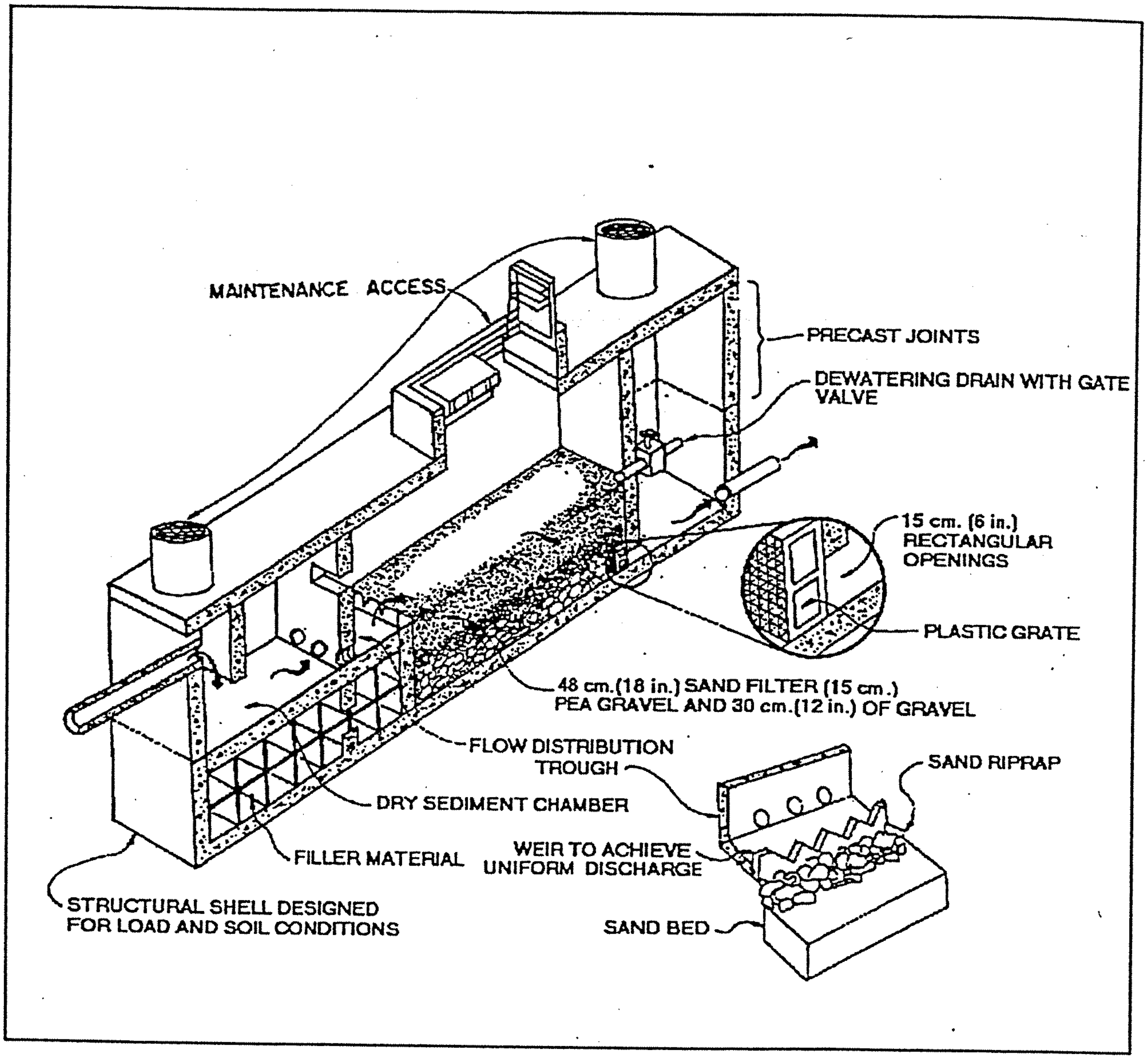


FIGURE 2-33 --RESIDENTIAL DRY VAULT SAND FILTER

filter is spread evenly and possesses low energy. The outflow side must incorporate features to prevent gouging of the filter media, such as concrete splash blocks or or rip-rap. The filter contains three layers of aggregate: 18 inches of ASTM C-33 Concrete Sand, six inches of 1/4-inch pea gravel, and 6-12 inches of 1/2-inch to 2-inch gravel. For filters longer than 15 feet, collector pipes must also be installed.

The third chamber is the clearwell, which collects the treated water from the collector pipes and/or weepholes and conveys it to the storm sewer. It also contains a dewatering valve to allow draining the filter chamber in the event of a stop-up.

The filter illustrated is a version for placement in a universal filter box system for use in either dry or wet vault applications under development by Virginia and D.C. officials. In purpose-built applications, the floor of the filter box may be stepped up at the sediment chamber, and the overall depth of the filter section may be as little as 30 inches, allowing shallower placement of the box or additional storage over the filter.

B) Pollutant Removal Rates

Alexandria recognizes 40 percent phosphorous removal efficiency for Residential Dry Vault Sand Filters if the applicant agrees to outfit the system for monitoring and grant unlimited access to the City and its contractors for monitoring purposes. 40% eff *

C) Design Considerations

1) Applicability

This system is intended only for use on small residential developments with less than two acres of impervious cover. As with the D.C. Sand Filter, the concept works best when treating one acre or less of impervious cover. Larger filters require collector pipes, an additional expense, and are also more likely to accumulate trash and sediments requiring earlier maintenance. For projects with over one acre of impervious cover, multiple filters, either in separate or the same shell, should be provided.

2) Practicability

The demonstrated success of the Austin, D.C., and Delaware Sand Filters suggest that this system should be very viable. Costs are expected to be somewhat less than current D.C. Sand Filter Costs (approximately $20,000 per impervious acre-in place and ready to use) because of the elimination of the filter cloth layers, the collector pipes, and their associated hand labor installation costs. If the filter owners practice good pollution prevention, especially prevention of hydrocarbon dumping and keeping yard debris and trash from entering the system, the maintenance costs of this design should be considerably less than those of a wet-vault filter.

3) Groundwater and Bedrock

The seasonally-high groundwater table and bedrock should be located at least two (2) to (4) feet below the footing of the filter structure.

4) Drawdown Time

As with other intermittent sand filter BMPs, drawdown time should not exceed 40 hours so that the BMP will be free to process follow-on storms.

5) Structural Requirements

The load-carrying capacity of the filter structure must be considered when it is located under parking lots, driveways, roadways, and, certain sidewalks (such as those adjacent to State highways). Traffic intensity may also be a factor. The structure must be designed by a licensed structural engineer and the plans require City approval.

6) Design Storm

The inlet design or integral large storm bypass must be adequate for isolating the WQV from the 10 year storm (7 in./hr., 10 min. TOC) and for conveying the peak flow of that storm past the filter system. Since Residential Dry Vault Sand Filters will be used only as off-line facilities in Alexandria, the interior hydraulics of the filter are not as critical as when used as an on-line facility. The system should draw down in approximately 40 hours.

7) Infrastructure Elevations

For cost considerations, it is preferable that the Residential Dry Vault Sand Filter work by gravity flow. This requires sufficient vertical clearance between the invert of the prospective inflow storm piping and the invert of the storm sewer which will receive the outflow. In cases where gravity flow is not possible, a clearwell sump and pump are required to discharge the effluent into storm sewer.

8) Accessibility and Headroom for Maintenance

All three chambers of the Residential Dry Vault Sand Filter must have personnel access manholes and built-in access ladders. The Residential Dry Vault Sand Filter must also be accessible to vacuum trucks for removing accumulated sediments and trash. Approximately every 5 years, the filter can be expected to clog to the point that removal and replacement of approximately the top 2-3 inches of sand may be required (this period may be considerably lengthened by good pollution prevention practices on the part of the owning homeowners association). **A minimum headspace of 60 inches above the filter will be required if the ceiling to the chamber is a fixed structure.** A 38-inch diameter maintenance manhole with eccentric nested covers (a 22-inch personnel access lid inside the 38-inch diameter lid) or a rectangular load bearing access door (minimum 4 ft. x 4 ft.) should be positioned directly over the center of the filter.

9) Accessibility for Monitoring

Unless otherwise approved by the Director, prefabricated monitoring manholes must be installed in the inflow and outflow pipes to allow chemical monitoring of the inflow water and effluent. See Appendix 2-8 of the *Alexandria Supplement* for details.

D) Design Procedures

1) Determine Governing Site Parameters

Determine the Impervious area on the site (I_a in acres), the water quality volume to be treated (WQV in ft.^{3} = 1816 I_a), and the site parameters necessary to establish 2h, the maximum ponding depth over the filter (storm sewer invert at proposed connection point, elevation to inflow invert to BMP, etc). If a bypass weir or pipe is to be built directly into the Residential Dry Vault Sand Filter shell, it should be designed at this point. **Worksheet E** on page 2-A4-2 *Alexandria Supplement* is provided to perform this step.

2) Design the Filter

The design logic for the Residential Dry Vault Sand Filter is essentially identical to that of the D.C. Sand Filter, Worksheet G2 on page 2-A4-33 is provided to assist in the design of dry vault filters.

E) Specifications

Specifications for the Residential Dry Vault Sand Filter are contained in Appendix 2-2 to the *Alexandria Supplement.* These specifications shall be quoted *verbatim* on the Stormwater Management Plan sheets of the Final Site Plan.

F) Construction and Maintenance Requirements

A Maintenance Agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with the Chapter 3 of this manual must be executed by the developer/owner before the Final Site Plan for the development will be released for construction. A project-specific agreement will be forwarded by the City with the bond estimate.

Construction and Maintenance requirements for Residential Dry Vault Sand Filters are delineated in detail on pages 2-A3-15 and 2-A3-16. There requirements shall be reproduced *verbatim* on the Stormwater Management Plan sheets of the Final Site Plan.

XIII. BIORETENTION AND BIORETENTION FILTERS (RAIN GARDENS)

Bioretention is an innovative BMP developed by the Prince George's County, Maryland Department of Environmental protection. The following information is drawn from their *Design Manual for Use of Bioretention in Stormwater Management* (P.G. County, 1993) unless otherwise noted. This technology is also referred to as "Rain Gardens."

As originally conceived, the bioretention (Rain Garden) concept was that of a shallow infiltration basin in which the stormwater runoff is treated by the processes of adsorption, filtration, volitization, ion exchange, microbial and decomposition prior to exfiltration into the surrounding soil mass. Microbial soil processes, evapotranspiration, and nutrient uptake in plants also come into play (Bitter and Bowers, 1995). Bioretention areas are intended to replicate the ecosystem of an upland forest floor through the use of specific shrubs, trees, ground covers, mulches and deep, rich soils (*ibid*). **Figure 2-34** illustrates the original bioretention (Rain Garden) concept.

A) Facility Description

There are six major components to the bioretention area (Rain Garden): 1) the grass buffer strip; 2) the ponding area; 3) the planting soil; 4) the sand bed; 5) the organic layer; and 6) the plant material. All are critical to the proper functioning of the BMP.

The **grass buffer strip** filters particles from the runoff and reduces its velocity. The **sand bed** further slows the velocity of the runoff, spreads the runoff over the basin, filters part of the water, provides for drainage and aeration of the planting soil and enhances exfiltration from the basin. The **ponding area** functions as storage of runoff awaiting treatment and as a presettling basin for particulates that have not been filtered out by the grass buffer. The **organic or mulch layer** acts as a filter for pollutants in the runoff, protects the soil from eroding , and provides an environment for microorganisms to degrade petroleum-based solvents and other pollutants. The **planting soil layer** nurtures the plants with stored water and nutrients. Clay particles in the soil adsorb heavy metals, nutrients, hydrocarbons, and other pollutants. The **plant species** are selected based on their documented ability to cycle and assimilate nutrients, pollutants, and metals through the interactions among plants, soil, and the organic layer (*ibid*). By replicating a forest community, monoculture susceptibilities to insect and disease infestation are avoided, and evapotranspiration is enhanced.

The minimum width for a bioretention area is 15 feet, although 25 feet are preferable. The minimum length should be 40 feet (for lengths greater than 20 feet, the length should be at least twice the width to allow dispersed sheet flow). As an infiltration BMP, the maximum ponding depth is restricted to six inches to restrict maximum ponding time to four days to preclude development of anaerobic conditions in the planting soil (which will kill the plants) and to prevent the breeding of mosquitoes and other undesirable insects in the ponded water. The planting soil should have a minimum depth of four feet in order to provide appropriate moisture capacity, create space for the root systems, and provide resistance from windthrow.

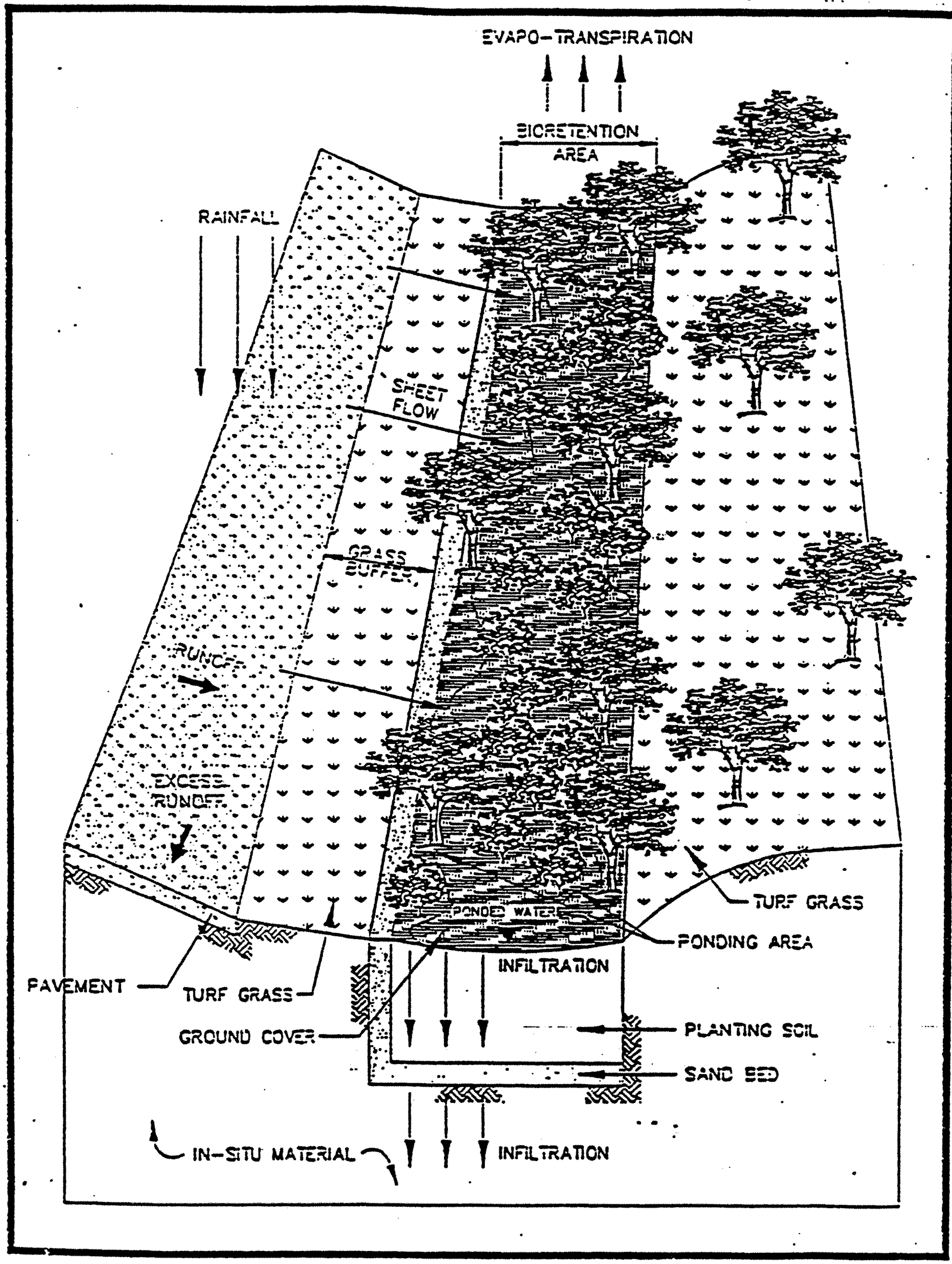


Figure 2-34 – Bioretention Area Conceptual Layout (PG Co., 1993) infiltration bioretention may not be feasible in many ultra-urban settings because of the proximity of building foundations or because soils are not conducive to exfiltration from the basin. **Bioretention Filters** were developed for use in such circumstances.

As utilized in Alexandria, Virginia, the bioretention filter is essentially the classic bioretention basin with an Austin Sand Filter collector pipe system installed beneath the basin. Because water flows through a filter much more quickly than through an infiltration facility, less surface area is required for a filter. For a dense townhouse development with 65 percent impervious cover, **less than 1.5 percent** of the land area is reqiuired for the BMPs. **Figure 2-35** illustrates a bioretention filter.

When used in areas underlain by marine clays or in proximity to building foundations, the entire basin must be provided with a dense clay or geomembrane liner. When the filter concept must be used simply because of low percolation rates of the soil, the liner may be omitted.

The gravel and collector pipe system is identical to that used on the Austin Sand Filter. Four-inch collector pipes are placed with at least two inches of cover over the pipes. Because of the high probability of eventual intrusion of roots into the collector pipes, maximum collector pipe spacing is reduced to eight feet center to center. To avoid the use of filter fabric layers to separate the sand layer from the collector pipe gravel, a six-inch layer of 1/4-inch pea gravel may be substituted for the lower six-inches of sand. The collector pipes flow into a manifold pipe, from which they are directed to the storm sewer. Cleanouts must be provided on all pipes for use should clogging occur. In order to enhance microbial removal of pollutants, outflow from the filter must be restricted by an accessible orifice or adjustable valve on the collector pipe manifold.

Since bioretention filters will drain much more quickly than infiltration basins (even when restricted by an orifice or valve), Alexandria allows a **maximum ponding depth of 12 inches**, reducing the area required to store water awaiting treatment.

B) Pollutant Removal Rates

Once a **mature** forest community is created in the bioretention areas, Prince George's County projects that the annual removal of nutrients from a typical one-acre commercial site would be approximately 2.4-4.8 pounds of nitrogen and 0.3-1.2 pounds of phosphorous. This would correspond to approximately 19.1-38.2 percent nitrogen removal and 18.4-73.5 percent removal of phosphorous.

Based on Alexandria monitoring of sand and peat-sand filter systems, the City has assigned a provisional **total phosphorous removal efficiency of 50%** to bioretention and bioretention filters. This efficiency will be revised after actual bioretention monitoring data are available.

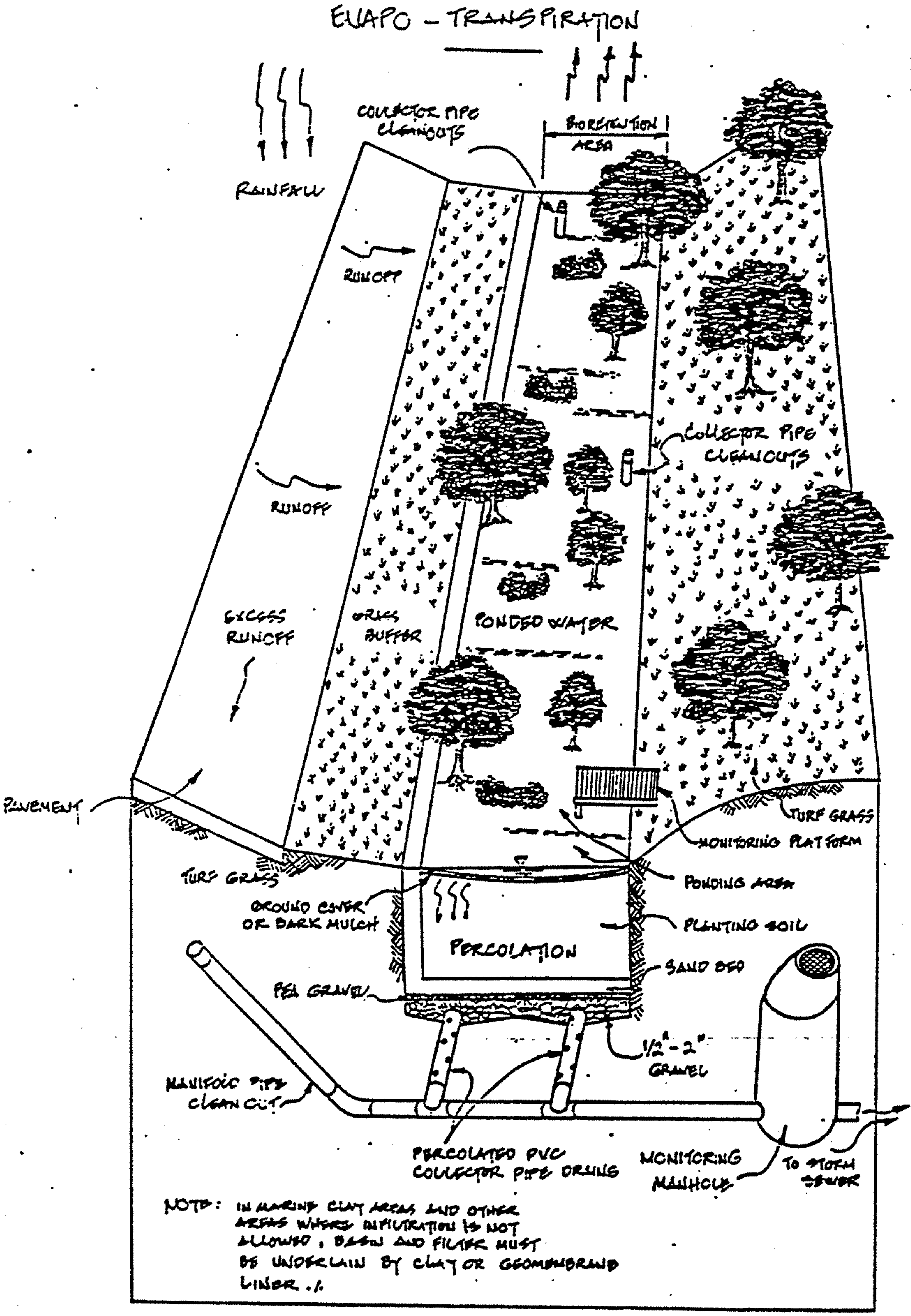


Figure 2-35 -- Bioretention Filter Conceptual Layout

C) Design Considerations

1) Applicability

Either bioretention or bioretention filters are suitable for almost any type of development. They may be installed in depressed landscaping areas in commercial parking lots, as depressed landscaping beds in the lawns of commercial or office developments, or in common use open space on residential projects. Bioretention or bioretention filters are the preferred BMP for residential development projects because of their manageable maintenance aspects (in Alexandria, surface BMPs, which also include wet and dry detention ponds, must be used on all residential developments with greater than two acres of impervious cover).

2) Practicability

Bioretention BMPs have several aspects which make them attractive for use in the ultra-urban environment. Their real estate cost may be minimized since the area may count as a portion of the vegetated open space required by zoning regulations. They are considered "Useable Open Space" as the term is used in the Alexandria Zoning Ordinance only insofar as they provide landscaping beds for part of the required landscaping plantings and tree covering. **The full complement of recreational open space must be provided in addition to open space devoted to surface BMPs.** Since a considerable volume of the annual rainfall is disposed of on-site by infiltration bioretention, infrastructure costs to collect and convey runoff may be substantially reduced.

For larger developments, the bioretention basins may be partially excavated early in the project and used as required sediment basins until the site is fully stabilized, then reexcavated and converted to bioretention facilities as the last phase of construction.

3) Inlet Configuration and Design Storm

The preferred inlet configuration for bioretention BMPs is through sheet flow. The second preferrence is to introduce runoff into the basin through openings in adjacent curbing. In some situations, however, it will be necessary to pipe the runoff into the bioretention filter. In such cases, energy dissipation must be provided to prevent erosion. Placement of bioretention BMPs in an off-line position is also preferrable. A common method is to allow runoff to overflow back into a paved gutter for conveyance to a storm inlet. **Figure 2-36** illustrates this configuration. There will be some inatances in which it is necessary to place a large storm overflow structure directly within the bioretention basin. The inlet design or integral large storm bypass must be adequate for isolating the WQV

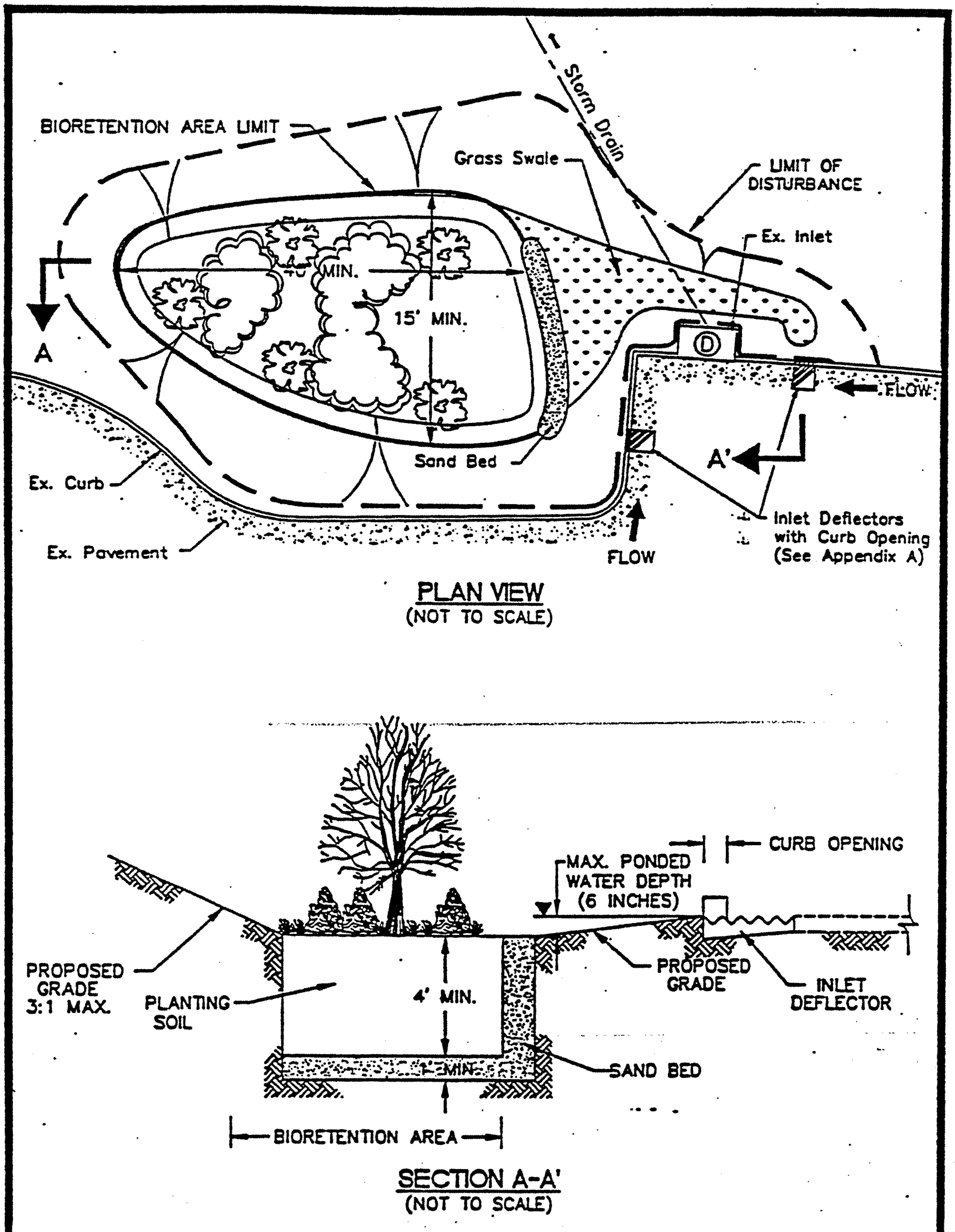


FIGURE 2-36 -- Typical Bioretention Basin Layout
Source: Prince Georges County, Md.

from the 10 year storm (7 in./hr., 10 min. TOC) and for conveying the peak flow of that storm past the bioretention filter system.

4) Minimum Dimensions and Infrastructure Elevations

Minimum width for a functional bioretention area is 15 feet. The minimum length should be forty feet (see **Figure 2-36**). For widths greater than 20 feet, the length of the bioretention area should be at least twice the width. The minimum width criteria is especially important in replicating tree and shrub distribution patterns which exist in a forest community.

For cost considerations, it is preferable that the DCSF work by gravity flow. This requires sufficient vertical clearance between the invert of the prospective inflow opening or storm piping and the invert of the storm sewer which will receive the outflow.

5) Accessibility

Bioretention filters must be accessible for periodic maintenance. Easements must be provided as necessary to assure access for the homeowners association or other owners to perform required maintenance. The fencing of any portion of a bioretention filter that serves more than one dwelling into an individual unit yard is therefore not allowed.

6) Clearwell Manhole and Orifice (Manifold Orifice Chamber)

New bioretention filters will typically empty much quicker than mature filters. To assure that the runoff being treated has sufficient contact time in the filter for the necessary biochemical reactions to occur, the outflow must be restricted while the plant community develops. A clearwell manhole must therefore be provided to receive the flow of the collector pipe manifold. The daylight end of the manifold pipe must be capped, and an orifice sized to extend filter drawdown time to a minimum of 24 hours must be drilled in the cap (for systems equipped for monitoring, a gate valve shall be provided in lieu of the cap and orifice). An alternative would be to daylight all of the collector pipes into a single chamber and provide caps with orifices (or gate valves) on each of the pipes.

7) Monitoring Manhole and Platform

Unless otherwise approved by the Director, at least one Bioretention Filter per development project must be equipped with a monitoring manhole downstream of the Clearwell (Manifold Orifice Chamber) and upstream of the large storm overflow pipe. A monitoring equipment platform above the basin pool must also be provided. See Appendix 2-8 for details.

D) Design Procedures

1) Bioretention Basin Design Features

For design features of the bioretention basin down to the lower sand layer, Alexandria utilizes the Prince George's County, Maryland ***Design Manual for Use of Bioretention in Stormwater Management*** (P.G. County, 1993)

2) Bioretention Filter Features

a) Basin Areas and Volumes – For bioretention filters in Alexandria, provide basin storage and treatment for the WQV. Water may be pooled to a maximum depth of 12 inches. An average depth of eight inches may be assumed for preliminary sizing calculations.

The mechanics of bioretention filters are similar to those of other soil media filters (sand filters, peat-sand filters, etc.). Alexandria therefore uses the Austin Filter Formula (Equation 2-11):

$$A_f = \frac{I_a H d_f}{k(d_p+d_f)t_f} \qquad \text{(Eq. 2-11)}$$

where,

A_f = surface area of the filter media (acres or sq. ft.)
I_a = Impervious drainage area contributing runoff to the basin (acres or ft^2)
H = runoff depth to be treated (ft.)
d_f = filter media depth (ft.)
k = coefficient of permeability for the filter media (ft/hr)
d_p = depth (ft.) of water ponded above surface of the filter media
t_f = time required for runoff volume to pass through filter media (hrs.)

In Alexandria, the following values shall be used when designing bioretention filter systems:

I_aH = the Water Quality Volume (WQV in $ft.^3$ = 1816 I_a). (I_a in acres)
t_f = 24 hours
k = 2.0 feet per day (0.0833 ft/hr)

When designing bioretention and bioretention filters, the slightly conservative assumption that the hydraulic gradient $[(d_p + d_f)/ d_f] = 1$ is appropriate. Inputting these factors into the filter formula and rounding off results in the following simplified formula for computing the minimum bioretention filter area (A_{fm}):

$$A_{fm} = 900I_a \qquad \text{(Eq. 2-46)}$$

Utilizing this approach, the basin is designed as follows:

1) Determine Governing Site Parameters

Determine the Impervious area on the site (I_a in acres), the water quality volume to be treated (WQV in $ft.^3 = 1816\ I_a$), and the site parameters necessary to establish d_p (2h for other stormwater filter systems), the maximum ponding depth over the filter (storm sewer invert at proposed connection point, elevation to inflow invert to BMP, etc). If a bypass or overflow pipe is to be built directly into the bioretention area, it should be designed at this point. **Worksheet E** on page 2-A4-2 is provided to perform this step.

2) Compute Minimum Area of Bioretention Filter (A_{fm}):

$$A_{fm} = 900 \times I_a$$

Note: Minimum dimensions of a bioretention filter to assure proper biochemical functioning are 15' x 40' (with irregular boundaries, approximately 450-600 ft^2). Special approval by the Director of T&ES is required for filters with smaller dimensions.

3) Considering Site Constraints, Select **Final Filter Area (A_f)**:

4) Compute Storage in Biortetention Filter Voids (V_v):
(Assume 20% voids in filter media)

$$V_v = A_f \times 0.2\ d_f \qquad \text{(Eq. 2-47)}$$

5) Compute Flow Through Filter During Filling Period (V_Q): (Assume 1-hour to fill per D.C. practice):

$$V_Q = \frac{kA_f\ (d_f + d_p)}{d_f} \qquad \text{(Eq. 2-26)}$$

Again, assuming a hydraulic gradient of unity, k = 0.0833 ft/hr, anf A_f = 900, this reduces to:

$$V_Q = 75I_a \qquad \text{(Eq. 2-48)}$$

6) Compute Net Volume to be Stored Awaiting Filtration (V_{st}):

$$V_{st} = WQV - V_v - V_Q \qquad \text{(Eq 2-49)}$$

7) Compute Minimum Area of Storage Basin (A_B):

If $V_{st} \leq A_f \times d_p$, make $A_B = A_f$

If $V_{st} > A_f \times d_p$, make $A_B = \frac{V_{st}}{d_p}$

(Alternative): Compute Excess Storage Requirement (S_E) and Store Outside Bioretention Filter Basin (i.e. underground pipe gallery, etc.):

$$S_E = V_{st} - (A_f \times d_p) \qquad \text{(Eq. 2-50)}$$

8) Compute Peak Flow Rate for Orifice (s) for 24-Hour Drawdown (formulae from ***Northern Virginia BMP Handbook***):

$$Q_p = \frac{WQV}{(0.5 \times 3600 \times 24)} = 0.000023 \times WQV \qquad \text{(Eq. 2-51)}$$

9) Compute Outflow Manifold Pipe Orifice Area to Provide 24-Hour Drawdown (formulae from ***Northern Virginia BMP Handbook***):

$$A_o = \frac{Q_p}{0.6\,(64.4 \times h_{max})^{0.5}} \qquad \text{(Eq. 2-52)}$$

10) Compute Diameter of Required Orifice (s) (D_o) (formulae from ***Northern Virginia BMP Handbook***):

$$D_o = 2.0 \times (A_o / 3.1316)^{0.5} \qquad \text{(Eq. 2-53)}$$

Worksheet L on page 2-A4-37 is provided to assist with these calculations.

b) Basin Liner – Impermeable liners are required in all cases where marine clays underlie the site. Liners may also be required in close proximity to building foundations. If an impermeable liner is required it shall meet the following specifications:

Impermeable liners may be either clay, concrete or geomembrane (such as PVC sheeting, EPDM roofing, or landfill liner such as Bentomat). If PVC geomembrane is used, suitable geotextile fabric shall be placed below and on the top of the membrane for puncture protection. If clay liners are used, the clay shall have a minimum thickness of 12 inches and meet the specifications in **Table 2-3**:

TABLE 2-3

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	%	Not less than 15
Liquid Limit of Clay	ASTM D-2216	%	Not less than 30
Clay Particles Passing	ASTM D-422	%	Not less than 30
Clay Compaction	ASTM D-2216	%	95% of Standard Proctor Density

Source: City of Austin

If a PVC geomembrane liner is used it shall be sandwiched between two layers of geotextile fabric, have a minimum thickness of 30 mils, and be ultraviolet resistant. The geotextile fabric (for protection of geomembrane) shall meet the following specifications:

Property	Test Method	Unit	Specification	
Material	Nonwoven geotextile fabric			
Unit Weight		Oz/Sq.Yd.	8	(min.)
Filtration Rate		In/Sec	0.08	(min.)
Puncture Strength	ASTM D-751(Modified)	Lb.	125	(min.)
Mullen Burst Strength	ASTM D-751	Psi	400	(min.)
Tensile Strength	ASTM D-1682	Lb.	300	(min.)
Equiv. Opening Size	US Standard Sieve	No.	80	(min.)

Equivalent methods for protection of the geomembrane liner will be considered by the Department of Transportation and Environmental Services on a case by case basis. Equivalency will be judged on the basis of ability to protect the geomembrane from puncture, tearing and abrasion.

c) Geotextile Fabrics and Drainage Matting -- Figure 2-37 shows the bioretention drainage system configuration. The underdrain system must be separated from the lower sand layer of the bioretention system by either a six-inch layer of 1/4-inch pea gravel or by a layer of geotextile fabric to prevent the filter media from infiltrating into the lateral piping. Geotextile fabric shall meet the specifications listed above under "Basin Liner".

Laterals shall be placed in trenches with a covering of at least two inches of 1/2 to two (2) inch gravel. The laterals shall be underlain by a layer of drainage matting. The drainage matting is needed to provide for adequate vertical and horizontal hydraulic conductivity to the laterals. The drainage matting shall meet the specifications contained in **Table 2-4**:

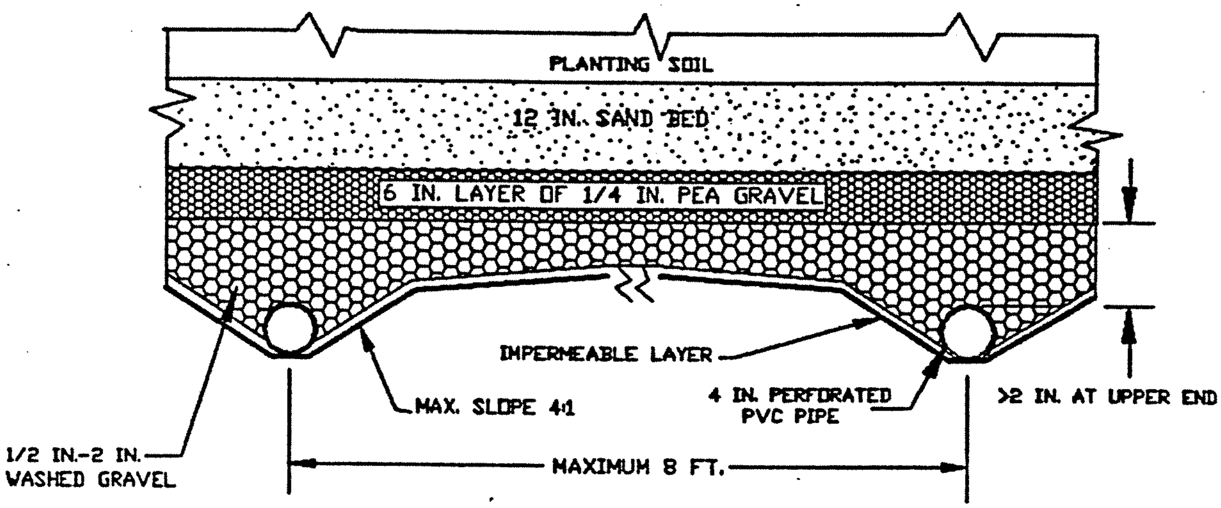


Figure 2-37 -- Bioretention Filter Drainage System Configuration

TABLE 2-4

DRAINAGE MATTING SPECIFICATIONS

Property	Test Method	Unit	Specification
Material	Nonwoven geotextile fabric		
Unit Weight		Oz/Sq.Yd.	20
Flow Rate (fabric)		GPM/Ft^2	180 (min.)
Permeability	ASTM D-2434	Cm/Sec	12.4 X 10^{-2}
Grab Strength (fabric)	ASTM D-1682	Lb.	Dry Lg.90 Dry Wd:70 Wet Lg.95 Wet Wd:70
Puncture Strength (fabric)	COE CV-02215	Lb.	42 (min.)
Mullen Burst Strength	ASTM D-1117	Psi	140 (min.)
Equiv. Opening Size	US Standard Sieve	No.	100 (70-120)
Flow Rate (drainage core)	Drexel Univ. Test Method	GPM/ft.width	14

Source: City of Austin

d) **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. Internal diameters of lateral branch pipes should be four (4) inches or greater and perforations should be 3/8 inch. Each row of perforations shall contain at least four (4) holes and the maximum spacing between rows of perforations shall be six (6) inches. All piping is to be schedule 40 polyvinyl chloride or greater strength. A maximum spacing of eight (8) feet between laterals is recommended. Lesser spacings are acceptable.

The minimum grade of piping shall be 1/8 inch per foot (one (1) percent slope). Access for cleaning all underdrain piping is needed.

e) **Clearwell Manhole or Chamber** – The main collector pipe (collector pipe manifold) must be routed through a manhole or accessible chamber where an orifice or valve is installed to restrict outflow from the filter in order to enhance microbial removal of pollutants. The usual method employed is to cap the manifold and drill the required orifice in the cap. **Figure 2-38** illustrates this configuration. On systems equipped for monitoring, a gate valve must be provided to restrict flow.

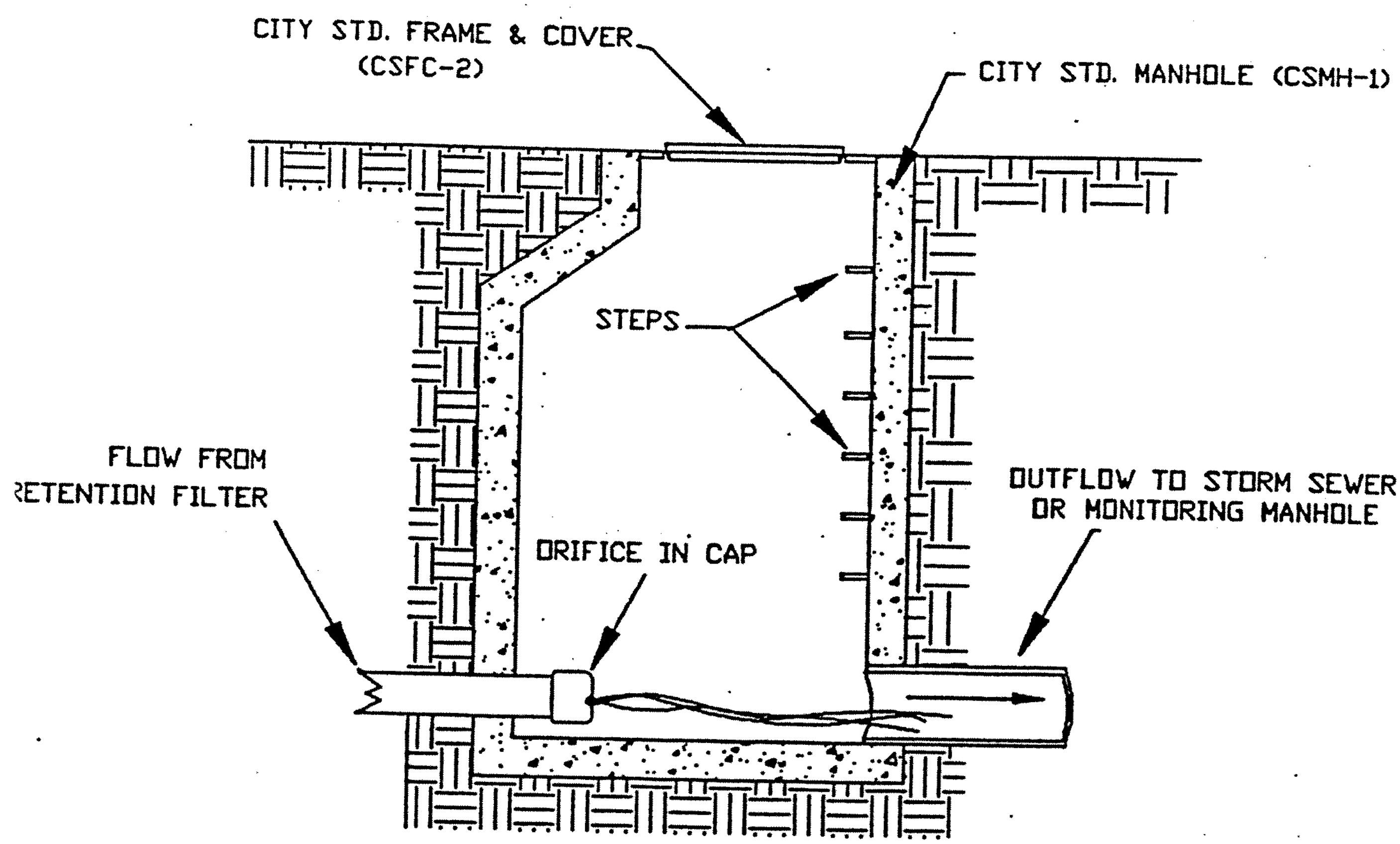


FIGURE 2-38 – Clearwell Manhole With Capped Pipe and Orifice

f) **Monitoring Manhole and Platform** – At least one Bioretention Filter per development project must be equipped with a monitoring manhole downstream of the Clearwell (Manifold Orifice Chamber) and upstream of the large storm overflow pipe. A monitoring equipment platform above the basin pool must also be provided. See Appendix 2-8 for details.

E) Specifications

Specifications for Bioretention Filters (Rain Garden Filters) are contained in Appendix 2-2. on pages 2-A2-8 through 2-A2-9. These specifications shall be quoted *verbatim* on the Stormwater Management Plan sheets of the Final Site Plan.

F) Construction and Maintenance Requirements

A Maintenance Agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with the Chapter 3 of this manual must be executed by the developer/owner before the Final Site Plan for the development will be released for construction. A project-specific agreement will be forwarded by the City with the bond estimate.

Construction and Maintenance requirements for Bioretention Basins (Rain Garden) and Bioretention Filter (Rain Garden Filter) are delineated in detail on pages 2-A3-27 through 2-A3-29. There requirements shall be reproduced *verbatim* on the Stormwater Management Plan sheets of the Final Site Plan.

ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET G2: COMPUTATIONS FOR RESIDENTIAL DRY VAULT SAND FILTER

Part 4: Considering data on Worksheet E, select maximum ponding depth over filter:

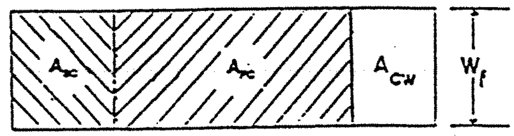
2h = ________ ft;

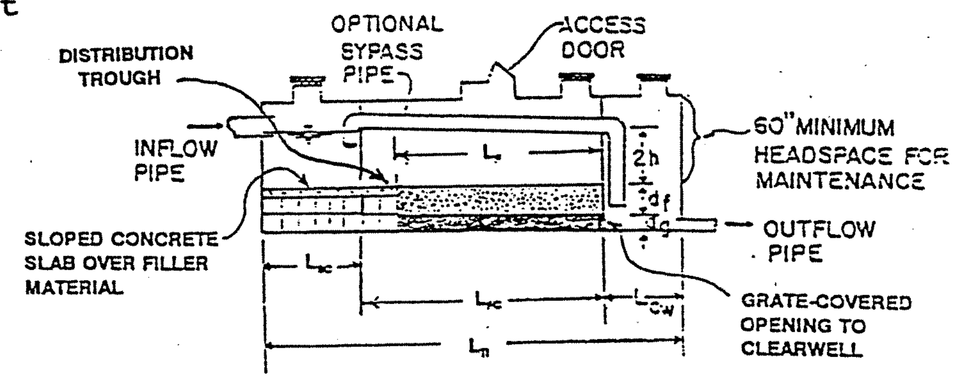
h = [ ] ft

From WORKSHEET E;

I_a = [ ] acres

WQV = [ ] ft^3





Outflow by gravity possible _____

Effluent pump required ____

Part 5: Compute Minimum Area of Filter (A_{fm}):

$$A_{fm} = \frac{545 I_a d_f}{(d_f + h)}$$

= [545 x ________ x ________] / [________ + ________]

= [ ] ft^2

Part 6: Considering Site Constraints, Select Filter Width (W_f) and Compute Filter Length (L_f) and Adjusted Filter Area (A_f):

W_f = [ ] ft;

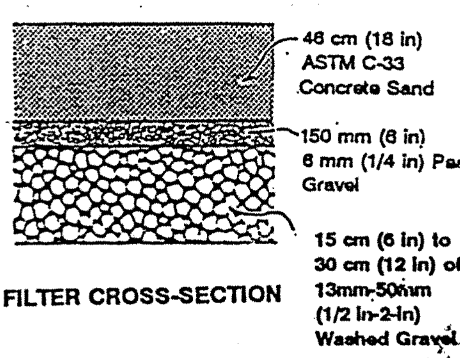
$L_f = A_{fm} / W_f$

= ________/________

= ________, say [ ] ft

$A_f = W_f \times L_f$ = ________ x ________

= [ ] ft^2



FILTER CROSS-SECTION

Part 7: Compute the Storage Volume on Top of the Filter Chamber (V_{TFC}):

Make the Distribution Trough 2.5 feet wide. Therefore, the filter chamber length (L_{FC}) = L_f + 2.5 = ______ + 2.5 = ______ ft

$A_{FC} = L_{FC} \times W_F$ = ______ x ______ = ______ ft^2

$V_{TFC} = A_{FC} \times 2h$ = ______ x ______ = $\boxed{\quad}$ ft^3

Part 8: Compute Storage in Filter Voids (V_v):
(Assume 40% voids in filter media)

$V_v = 0.4 \times A_f \times (d_f + d_g)$

= 0.4 x ______ x (______ + ______)

= $\boxed{\quad}$ ft^3

Part 9: Compute Flow Through Filter During Filling Period (V_Q):
(Assume 1-hour to fill per D.C. practice)

$V_Q = \frac{kA_f(d_f + h)}{d_f}$; use k = 2 ft/day = 0.0833 ft/hr

= [0.0833 x ______ x (______ + ______)]/______

= $\boxed{\quad}$ ft^3

Part 10: Compute Net Volume to be Stored Awaiting Filtration (V_{st}):

$V_{st} = WQV - V_{TFC} - V_v - V_Q$

= ______ - ______ - ______ - ______

= $\boxed{\quad}$ ft^3

Part 11: Compute Storage Length of Sediment Chamber (L_{SC}):

$L_{SC} = \frac{V_{st}}{(2h \times W_f)}$ = ______ / (______ x ______)

= $\boxed{\quad}$ ft

Part 12: Compute Minimum Length of Sediment Chamber (L_{sm})
(to contain at least 20% of WQV per Austin practice)

$$L_{sm} = \frac{0.2WQV}{(2h \times W_f)} = \_\_\_\_\_\_ / \_\_\_\_\_\_$$

$$= \boxed{\quad} \text{ ft}$$

Part 13: Set Final Length of Sediment Chamber (L_{SCF})

If $L_{SC} \geq L_{sm}$, Make $L_{SCF} = L_{SC} = \boxed{\quad}$ ft

If $L_{SC} < L_{sm}$, make $L_{SCF} = L_{sm} = \boxed{\quad}$ ft

Part 14: Set Length of Clearwell (L_{cw}) for Adequate Maintenance Access (Minimum = 3 ft) and Compute Final Inside Length (L_{TI}):

$L_{cw} = \boxed{\quad}$ ft ;

Sum of interior partition thicknesses (t_{pi}) = $\boxed{\quad}$ ft

$$L_{TI} = L_{FC} + L_{SCF} + L_{cw} + t_{pi}$$

$$= \_\_\_\_\_\_ + \_\_\_\_\_\_ + \_\_\_\_\_\_ + \_\_\_\_\_\_$$

$$= \boxed{\quad} \text{ ft}$$

Part 15: Design Structural Shell to Accommodate Soil and Load Conditions at Site:

It may be economical to adjust final dimensions upward to correspond with standard precast structures or to round dimensions upward to simplify layout during construction.

Part 16: Design Effluent Pump if Required:

Since pump must be capable of handling flow when filter is new, use k = 20 feet/day = 0.833 ft/hr

$$Q = \frac{kA_f(d_f + h)}{d_f}$$

$$= [0.833 \times \_\_\_\_\_\_ \times (\_\_\_\_\_\_ + \_\_\_\_\_\_)]/\_\_\_\_\_\_$$

$= \boxed{\quad}$ ft^3/hr ; /3600 = $\boxed{\quad}$ cfs;

x 448.8 = $\boxed{\quad}$ gpm

APPENDIX 2-1--TECHNICAL NOTES

I. **Technical Basis for Sand Filtration Basin Surface Area Equations** [extracted from Austin[1]]

The filtration rate through a sand filtration basin will be found using the following equation:

(1) $q_f = Q/A_f$

where

q_f = average filtration rate (e.g. gpm/ft^2)
Q = average flowrate through sand bed (e.g. gpm)
A_f = surface area of sand bed (e.g. ft^2)

The average flow rate can be determined from the following equation:

(2) $Q = V/t_f$

where

V = volume of runoff to be filtered

t_f = time required for runoff volume to pass through filter media

The volume "V" can be determined from the following equation:

(3) $V = I_aH$

where

I_a = Impervious drainage area contributing runoff to the basin

H = runoff depth

Substituting equation (3) into equation (2) gives:

(4) $Q = I_aH/t_f$

The average flow rate can also be found using Darcy's Law:

(5) $Q = kiA_f$

where

k = coefficient of permeability for filtration media
i = hydraulic gradient
= $(h+d_f)/d_f$
h = average of water above surface of sand media between full and empty basin conditions

d_f = sand bed depth

therefore,

(6) $Q = kA_f(h+d_f)/d_f$

Substituting equation (6) into (1) gives:

(7) $q_f = k(h+d_f)/d_f$

Substituting equations (4) and (7) into (1) and solving for "A_f" gives:

(8) $A_f = I_aHd_f/k(h+d_f)t_f$

Discussion

For design purposes typical values are:

d_f = 18" sand = 1.5 feet

h = 3 feet (average head of water above sand bed - i.e., 6 ft. max. ponding depth)

t_f = 40 hour filtration basin draw-down time

The coefficient of permeability "k" were primarily based on observed values for sand filtration basins in the Austin area. Actual "k" values (feet per day) have been observed to vary from approximately $0.5 < k < 2.7$ with an average value of about 1.5 feet per day. These values may appear to be conservative compared to "textbook" values but were considered realistic due to the clogging effects of accumulated sediment loads. Initial filtration runs may have higher "k" values but will typically drop to the above quoted rates after one (1) or two (2) significant storms.

For full sedimentation/filtration systems "k" is assumed to be 3.5 feet per day. This is about 30 percent higher than the upper limit of observed values but is justified because pretreatment (by full sedimentation) will reduce filter clogging and because coarse sand is specified. For partial sedimentation/filtration systems "k" is assumed to be two (2) feet per day. This is 30 percent higher than the average observed "k" of 1.5 feet per day but is justified because the pretreatment in the settling chamber will reduce some clogging. Nonetheless, clogging will be greater than for the full sedimentation system and a lower permeability will result. Therefore, it is clear that a larger surface area will be required for partial sedimentation/filtration systems in order to achieve the same draw-down period.

Surface Areas for Sand Filtration Basins

Plugging in the values from &he "Discussion" section into equation (8) for Full Sedimentation/Filtration Systems gives:

$A_f = [I_a H(1.5)/3.5(3+1.5)(40)] \text{ X } 24 \text{ hr/day}$

(9) $A_f = I_a H/18$

Doing likewise for Partial Sedimentation/Filtration Systems gives:

$A_f = [I_D H(1.5)/2(3+1.5)(40)] \text{ X } 24 \text{ hr/day}$

(10) $A_f = I_a H/10$

II. Surface Areas for Peat-Sand Filtration Systems

A) Using Austin Approach and Darcy's Law

From Galli[36]

d_f = 3.33 feet (12" peat, 4" peat/sand, 24" sand)

h = 1 foot (2 feet maximum ponding depth - greater depths risk over-compressing the peat)

k for peat = 1" per hour, or 2 feet per day

t_f = 24 hours

A "k" of 3.5 feet per day for the sand is assumed based on Austin's analysis above.

Perloff and Baron give the following formula for computing the composite "k" value for the filter.[7]

$$k_f = \frac{d_f}{\frac{d_1}{k_1} + \frac{d_2}{k_2} + \ldots \frac{d_n}{k_n}}$$

$$k_f = \frac{3.33}{\frac{1.0}{2.0} + \frac{0.33}{2.5} + \frac{2.0}{3.5}}$$

$$= \frac{3.33}{0.5 + 0.13 + 0.57} = 2.8 \text{ feet per day}$$

Again substituting in equation (8),

$A_f = [I_a\ H(3.33)/2.8(3.33 + 1)(24)] \times 24$ hrs/day

$A_f = 0.27 I_a H$

When treating the Water Quality Volume (the first 0.0417 ft. of rain), this reduces to:

$A_f = 0.011 I_a$

B) Using Mass Balance Equation for Calculating Both Areal Phosphorous Load and Hydraulic Loading Rate[44]

$A_f = [ppt \times A_{ws} \times r \times P_i]/L_p$

where:

A_f = surface area of the peat-sand filter (m^2)

ppt = annual precipitation (m/yr)

A_{ws} = total contributing drainage area (m^2)

r = runoff coefficient ("C" value from Rational Method or "R_v from Simple Method (from Worksheet A or B)

P_i = stormwater influent phosphorous concentration (mg/L or g/m^3)

L_p = specific surface phosphorous loading

$= q_s \times P_i$

where:

q_s = areal hydraulic load (maximum = 75 m/yr)

Substitution and cancellation yields:

$A_f = [ppt \times A_{ws} \times r]/q_s$

For the Washington D.C. area,

ppt = 1.02 m/yr; for maximum areal hydraulic load of 75 m/yr, the equation reduces to:

$A_f = 0.0136A_{ws} \times r$

from the Simple Method,

$r = 0.05 + .009I$

$I = [I_a/I_{ws}] \times 100$

therefore:

$r = 0.05 + (0.009)(100\ I_a/A_{ws})$

$= 0.05 = 0.9I_a/A_{ws}$

further substituting in the basic equation yields:

$A_f = 0.00068A_{ws} + 0.0136I_a$

or, for all practical purposes:

$A_f = 0.014I_a$

SAND FILTER WATER QUALITY STRUCTURE

DESIGN CRITERIA:

RAINFALL DATA: 15 year frequency, 24 hour storm. By using rational method with T_c = 5 minutes then rainfall depth, d = 0.63 inch precipitation intensity, i = 7.56 inch/hour (D.C.)

EXAMPLE:

ASSUMPPTIONS:

1. Site is a parking lot
2. Outfall to storm sewer
3. City storm sewer Invert out is 92 feet at the proposed connection point
4. Area; A = 10,000 feet^2 (0.23 acre)
5. Runoff coefficient, C = 1.0 (assuming 100% impervious surface)
6. First Flush = 0.5 inch (see sand filter water quality guide book)

STEP 1:
Determine the final surface elevation, invert in, and out of the SFWQ structure in this example:

: Invert out = 93 feet (min.)
: Invert In = 98 feet (assume 1 foot inflow pipe diameter & 2 feet minimum cover on the top of the pipe.)
: Final surface elevation from site plan at the SWQ structure location = 101 feet
: Depth of filter layer dmax = 3 feet

STEP 2:
Determine peak discharge, ie Q_{P15} from parking lot

Q_{P15} = CIA = 1 * 0.23 ac * 7.56 inch = 1.74 cfs ---(eq.1)

STEP 3:
Calculate Volume of storage needed, Vw

Vw = Q_i * A_{imp} - F * T * A_2F ----------(eq.2)
Where:

Q_i = the first flush runoff
A_{imp} = Area of impervious surface, feet^2
F = Final infiltration rate for sand = (1.18 ft/hr),
T = 1 hour filling time (ref. Maryland infiltration design practices)
A_2f = Sand filter area, square feet

Note: A_2f = 50 feet^2 to 200 feet^2 for area of 0.1 to 1 acre

Use $A_1 \le A_2/3$

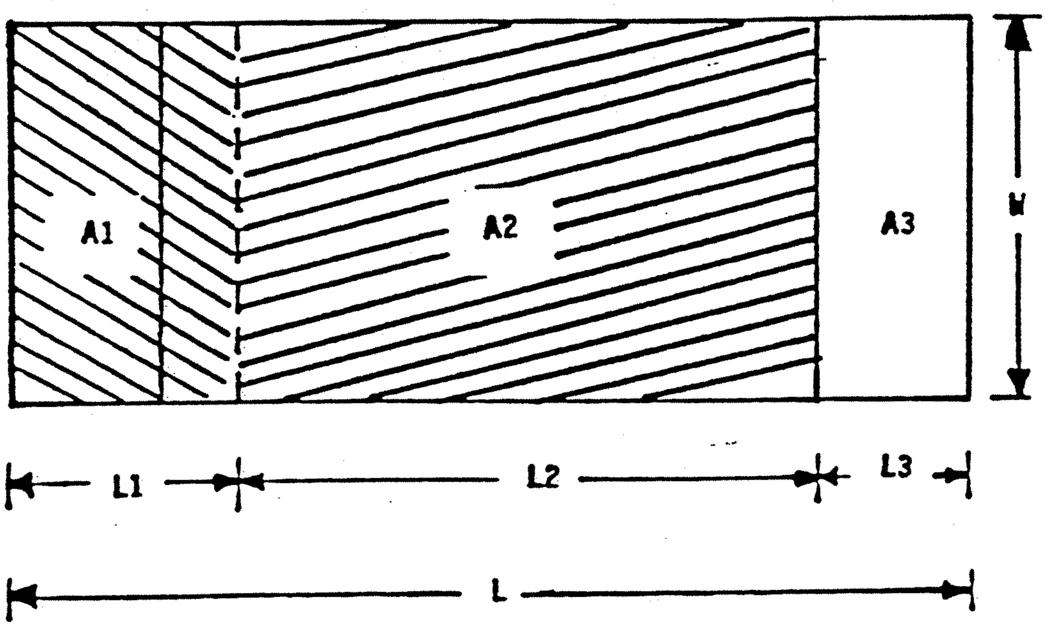
Select A_2f = 80 feet^2

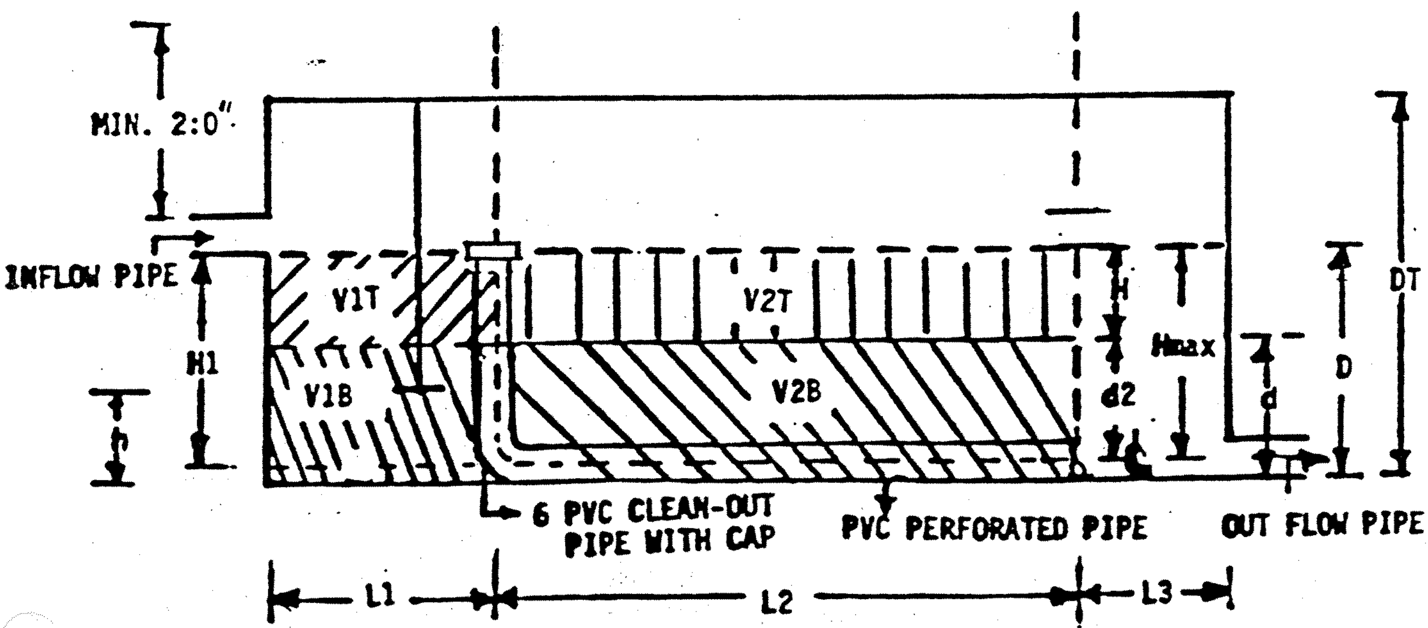
A_1 = 25 feet^2 (can also use trial and error method to select appropriate A_1 and A_2f)

$$Vw = 0.5/12 \text{ ft} * 10{,}000 \text{ Ft}^2 - 1.18 \text{ ft/hr} * 1\text{hr} * 80 \text{ ft}^2$$
$$= 345.87 \text{ ft}^3$$

STEP 4:

Design for weir height, D





MAXIMUM DEPTH AVAILABLE = 5 FT

Design for 5 ft at max. depth (allowing for 2% slope of outflow pipe)

Calculate Volume of filter layer, V_{2S}

$V_{2S} = A_{2f} * d * Vv$ -----------(eq.3)

Where:

A_{2f} = Area of filter layer
d = Depth of filter layer
V_v = Void ratio for sand
$V_{2S} = 80 \text{ feet}^2 * 3 \text{ feet} * 0.6 = 144 \text{ feet}^3$

Calculate bottom volume of first chamber, V_{1S}

$V_{1S} = A_1 * d$ --------------(eq.4)

Where:
A_1 = Area of first chamber

$V_{1S} = 25 \text{ feet}^2 * 3 \text{ feet} = 75 \text{ feet}^3$

Calculate top volume of 1st and 2nd chambers, $V_{1T} + V_{2T}$

Thus:

$V_{1T} + V_{2T} = Vw - (V_{2S} + V_{1S})$ ---------------(eq.5)

$= 345.87 \text{ feet}^3 - (144 + 75)\text{feet}^3 = 126.87 \text{ feet}^3$

$A_1H + A_2H = V_{1T} + V_{2T} = 126.87 \text{ ft}^3$

$$H = \frac{126.87 \text{ ft}^3}{A_1 + A_2} = \frac{126.87 \text{ ft}^3}{100 \text{ ft}^2} = 1.268 \text{ feet}$$ ----(eq.6)

Set Weir Height, $D = H + d = 3\text{ft} + 1.268 = 4.26 \text{ ft}$ -------(eq.7)
$D = 4.26 < 5$ ft max. depth => then OK

So, Invert out = Invert in - 4.26 ft = 98' - 4.26' = 93.74 ft

=> 93.74 > 93 then if is OK

Note: Invert of outlet pipe is at 93.74 feet elevation.

This elevation is higher than public storm sewer invert at connection point and provide more 2% slope of the outflow pipe, therefore it is OK.

STEP 5:

Calculate overflow weir opening in 2nd chamber and submerge weir in first chamber.

* overflow weir in 2nd chamber

$Q = CLH^{1.5}$ ------------ (eq.8)

where $Q = Q_{15} = 1.74$cfs (see step 2)

$C = 3.33$

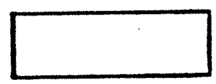
$L = 2$ft (assume)

$$H^{1.5} = \frac{Q^{15}}{CL}$$

$$H^{1.5} = \frac{1.74 \text{ cfs}}{3.33 \times 2\text{ft}} = 0.26 \text{ ft}$$

$H = 0.41$ ft use $H = 1$ ft

L = 2 ft



H = 1 ft

* submerge weir is 1st chamber

$Q = CA\sqrt{2gH}$

$Q = Q_{15} = 1.74$ cfs

$C = 0.6$ (approximately)

$A = l \times h$ (assume $h = 1$ft)

$g = 32.2\text{ft/sec}^2$

$H = [(\text{invert in} - \text{invert out}) - h/2] =$ hydraulic head above center line of the weir.

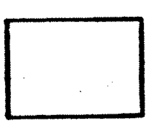
$= [(98 - 93.74) - 1/2]$ (see step 4)

$H = 3.76$ft

$$A = \frac{Q_{15}}{C\sqrt{2gH}} = \frac{1.74}{0.6\sqrt{2(32.2)(3.68)}} = \underline{0.186} \text{ sq. ft}$$

$A = l \times h = 0.186$ sq.ft

$l = 0.186 / 1 = 0.186$ ft use 1 ft. (min. requirement)



h = 1 ft

l = 1 ft

STEP 6:

FIX UP STRUCTURE DEMENSIONS

Note: L => Chamber length
W => Chamber Width. (use W = 6 ft)

$A_2 = L_2 * W = 80\ Ft^2$ ----------(eq.9)

$80\ Ft^2 = L_2 * 6\ Ft$

* $L_2 = 13.3\ Ft$ (use 13.5)

$A_1 = L_1 * W = 25\ Ft^2$ --------(eq.10)

$25\ Feet^2 = L_1 * 60\ Ft$

$L_1 = 4.16\ Ft,\ W = 6\ Ft$ 4.5

Select $L_3 = 3$ Feet
$W = 6$ Feet

∴ Total Dimension of Structure:

$L = (L_1 + L_2 + L_3)$ -----------------(eq.11)
4.5 13.5 3 = 21 Ft

W = 6 Ft

D_T = D+ inflow pipe diameter + 1 feet free board
= 4.26 + 1 + 1 = 6.26 Use 6.5 Ft

All of these dimensions are internal dimensions only

STEP 7:

DETENTION TIME AFTER STORAGE VOLUME FILLS UP

$Q = K * A_f * H\ max./d_2$ ------------(eq.12)

use: K = 0.6 ft/hr for mixed sand

$$H\ max = (d+H) - \frac{\text{pipe diameter}}{2} = 4.26 - 0.25 \text{ ---(eq.13)}$$
$$= 4.01\ ft$$

H max = 4.01 ft (for accurate hydraulic computation, assume there are 6" pvc perforated outflow pipes at the bottom).

$$d_2 = d - \frac{\text{pipe diameter}}{2} = 3 - 0.25 = 2.75ft \text{ -----(eq.14)}$$

$$Q = 0.6\ ft/hr * 80ft^2 * \frac{4.01}{2.75} = 69.9\ ft^3/hr \qquad \text{(Use eq.12)}$$

Calculate total detention time, t_s

$$t_s = V_w/Q \text{ ----------- (eq.15)}$$

$$t_s = \frac{345.87}{69.9} = 4.9 \text{ hrs or 5 hrs}$$

Since 5 < 72 hrs => OK

STEP 8:
INFLOW AND OUTFLOW HYDROGRAPHS

*Inflow hydrograph data:
Area = 10,000 ft^2 = 0.23 acre
15 Year 24 Hour Storm
Time of Concentration, T_c = 5 minutes
Runoff Coefficent, C = 1.0
Q = CIA = 1 * I * 0.23

T (MIN)	I (INCH/HOUR)	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

*TIME AT WHICH OUTFLOW HYDROGRAPH BEGINS IS GIVEN BY:

1. When T_c * Qpeak < $2V_w$

$$T = 2T_c - \sqrt{2T_c^2 - \frac{2V_w(T_c)}{Q_p}} \text{ ----------(eq.16)}$$

2. When T_c * Qpeak = 2Vw

$$T = \frac{T_c}{2} + \frac{V_w}{Q_p} \text{ --------------------(eq.17)}$$

3. When T_c * Qpeak > 2Vw

$$T = \sqrt{\frac{2V_w(T_c)}{Q_p}} \text{ ----------(eq.18)}$$

Calculate Maximum Filling Time, T

$V_W = 345.87\ ft^3$

$2V_W = 691.74\ ft^3$

$T_c * Q_p = 300\ sec * 1.74\ cfs = 522\ ft^3$

$T_c * Q_p < 2V_W$

∴ Choose equation no.16

$$T = 2T_c - \sqrt{2T_c^2 - \frac{2V_W(T_c)}{Q_p}}$$

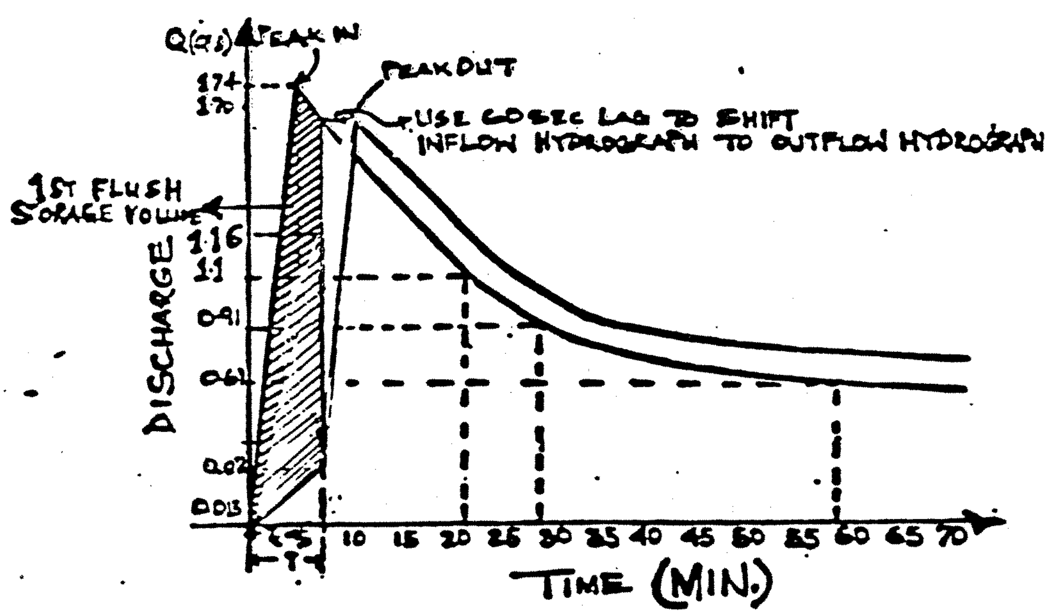
$$= 2 * 300 - \sqrt{2(300)^2 - \frac{2(345.87)\ (300)}{1.74}}$$

$= 353.56$ sec or 5.89 min

And at T => $Q = \dfrac{KH_{max}\, A_{2f}}{L}$ ---------(use eq.12)

$= 0.02\ cfs \longrightarrow 0.02\ cfs$

INFLOW - OUTFLOW HYDROGRAPH



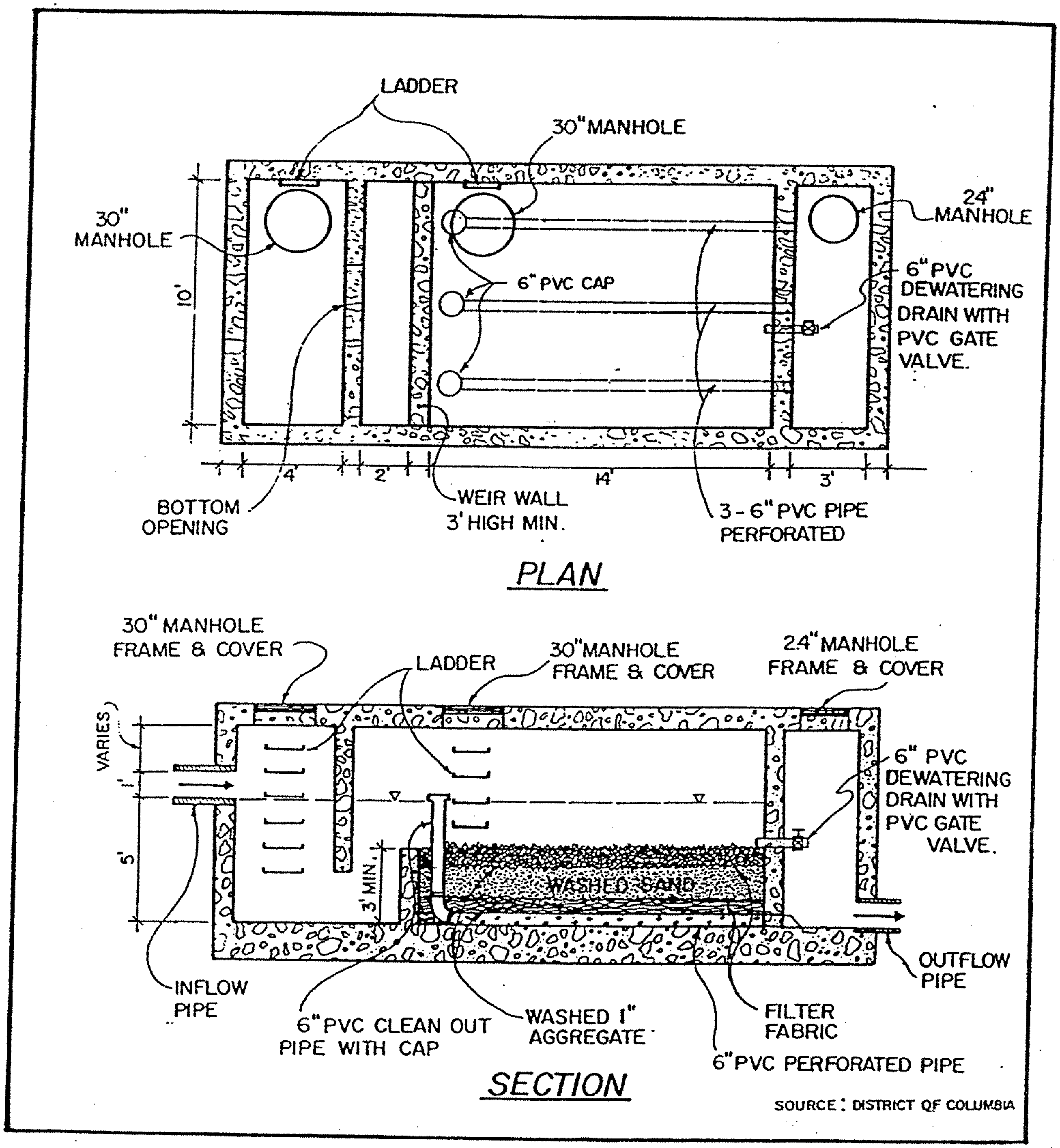
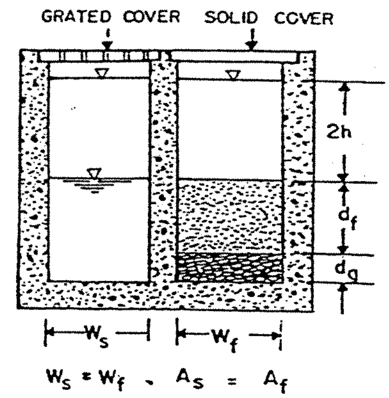


FIGURE 2-A1-1 -- DISTRICT OF COLUMBIA SAND FILTER

IV. ANALYSIS OF DIMENSIONAL RELATIONSHIPS FOR THE DELAWARE SAND FILTER:



The filter shell must have the capacity to accept and store the Water Quality Volume while it is waiting to be filtered. The dimensions must also be sized to provide a filter area which will process the WQV through the filter in the desired time frame. By setting equations for the area of the filter(A_f) expressed in terms of each requirement equal to each other, the minimum depth of ponding over the filter (2h) necessary to simultaneously meet both requirements may be determined.

GIVEN: $W_F = W_s$; therefore $A_f = A_s$

$$\text{Storage above filter and pool} = 2h(A_f + A_s) = 2h(2A_f) = 4hA_f$$

ANALYSIS A: RECOGNIZING ONLY THE VOLUME ABOVE THE FILTER AND PERMANENT POOL AS STORAGE VOLUME FOR THE WQV (NOTATION SAME FOR AUSTIN FILTER FORMULA)

Make storage above filter and permanent pool = WQV = $0.0417I_a$

$$4hA_f = 0.0417I_a$$

$$A = \frac{0.0417I_a}{4h}$$

From the Austin Filter Formula,

$$A_f = \frac{I_a H d_f}{k(h + d_f)t_f}$$

H = 1/2" = 0.0417ft of runoff
k = 2 ft./day = 0.0833 ft./hr.
t_f = 40 hrs.
d_f = 1.5 ft.

$$A_f = \frac{I_a(0.0417)(1.5)}{0.0833(h + 1.5)(40)} = \frac{1.5I_a}{(80h + 120)}$$

Therefore:

$$\frac{0.0417I_a}{4h} = \frac{1.5I_a}{(80h + 120)}$$

$$3.32h + 5 = 6h; \quad 2.68h = 5; \quad h = 1.87$$

$$2h = \overline{\underline{3.74}} = \text{Required maximum ponding depth over filter}$$

ANALYSIS B: RECOGNIZING VOIDS IN FILTER AS ADDITIONAL STORAGE

Assume 40% voids in sand filter.

therefore:

$$\text{Volume of voids} = 0.4(d_f + 0.5)A_f = 0.04(2.0A_f) = 0.8A_f$$

$$WQV = 4A_f h + 0.8A_f = A_f(4h + 0.8)$$

$$A_f = \frac{0.0417I_a}{(4h + 0.8)}$$

Using formulae from analysis I,

$$\frac{0.0417I_a}{(4h + 0.8)} = \frac{1.5I_a}{(80h + 120)}$$

$$3.32h + 5 = 6h + 1.2$$

$$2.68h = 3.8$$

$$h = 1.42 \text{ ft.}$$

$$2h = \underline{\overline{2.84 \text{ ft.}}} = \text{Required maximum pooling depth over filter and permanent pool}$$

ANALYSIS C: ALSO RECOGNIZING FLOW-THROUGH DURING FILLING PERIOD

Following D.C. practice, assume 1-hour filling period

$$WQV = A_f(4h + 0.8) + Q_{fill}$$

From Austin Filter Formula derivation above,

$$Q_f = \frac{k(A_f)(h + d_f)}{d_f} = \frac{0.0833h + 1.5)}{1.5}$$

$$= \frac{A_f(0.0833h + .125)}{1.5}$$

therefore,

$$WQV = 0.0417I_a = 4A_f h + 0.8A_f + \left[\frac{A_f(0.0833h + .125)}{1.5}\right]$$

$$= A_f[4h + 0.8 + 0.056h + 0.0833]$$

$$A_f = \frac{0.0417I_a}{[4.056h + 0.8833]}$$

therefore, using formulae from ANALYSIS II above,

$$\frac{0.0417I_a}{[4.056h + 0.8833]} = \frac{1.5I_a}{(80h + 120)}$$

$$3.32h + 5 = 6.084h + 1.32$$
$$3.68 = 2.764h$$
$$h = 1.33 \text{ ft}$$

$2h = \underline{2.66 \text{ ft.}}$ = Required maximum ponding depth above filter and permanent pool

V. Original Delaware Sand Filter Design Procedures

D) Design Procedures

Figure 2-24 provides plan and cross-sectional views of the Delaware Sand Filter System.

1) Calculate the Required Surface Areas of the Chambers
surface area of the sedimentation chamber:(35)

$A_s = I_a H \times 43{,}560/10$

where:

I_a = the impervious area on the watershed in acres

H = the design runoff depth in feet

A_s = the **area of the sediment chamber** in sq.ft.

For **one inch of runoff**, this yields:

$A_s = 360 I_a$

Delaware makes the area of the filter equal the area of the sediment chamber:(35)

$A_f = A_s = 360 I_a$

where A_f = the **area of the filter** in sq.ft.

2) Calculate the Storage Volume of the Chambers

For a storage depth of 18 inches,

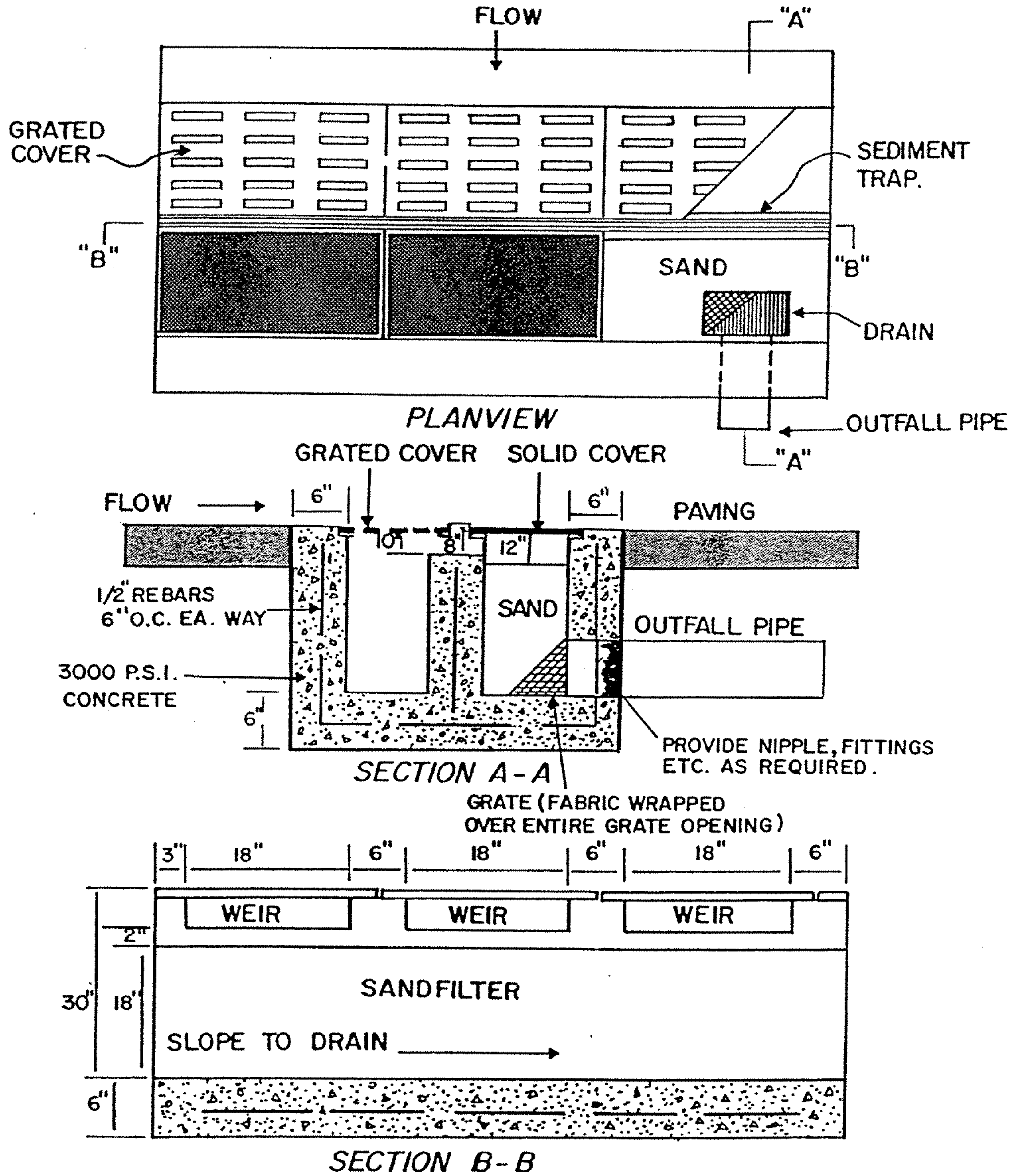
$V_s = 1.5\ A_s = 540\ I_a$

where V_s is the volume of the sediment in chamber in cu. ft.

Delaware makes the volume of the sand filter chamber equal the volume of the sedimentation chamber on the assumption that the storage volume lost to the volume of this sand is compensated for by the positive flow of water through the filter.(35)

$V_f = 540\ I_a$

where V_f = the volume of the filter chamber in cu. ft.



SOURCE: STATE OF DELAWARE (SHAVER)

FIGURE 20A1-2 -- PLAN AND CROSS-SECTIONS OF DSF

3) **Establish Dimensions of the Facility**

Site considerations will likely dictate the final dimensions of the facility. Sediment trenches and filter trenches normally be 18-24 inches wide. The maximum allowable trench width is **36** inches without special permission of the Director.

4) **Portland Cement Concrete**

Portland Cement concrete used for the trench structure shall conform to the A3 specification of the Virginia Department of Transportation Road and Bridge Specifications, January 1987.

5) **Sand Filter Chamber**

- Sand for the filter shall be **ASTM C33 Concrete Sand**[32] **or VDTO Section 202 Grade A Fine Aggregate Sand.**[30]

- Geotechnical cloth for the grate wrapping shall conform to the following:

Property	Test Method	Unit	Specification	
Material	Nonwoven geotextile fabric			
Unit Weight		Oz/Sq.Yd.	8	(min.)
Filtration Rate		In/Sec	0.08	(min.)
Puncture Strength	ASTM D-751(Modified)	Lb.	125	(min.)
Mullen Burst Strength	ASTM D-751	Psi	400	(min.)
Tensile Strength	ASTM D-1682	Lb.	300	(min.)
Equiv. Opening Size	US Standard Sieve	No.	80	(min.)

6) **Filtration Rate**

Delaware assumes a filtration rate of 0.04 gallons/minute/square foot of filter cross-sectional area on the further assumption that the average head is one (1) foot.[35]

7) **Outfall Pipe(s)**

The outfall pipe from the sand filter component shall not exceed six (6) inches in outside diameter so that there is a minimum of 12 inches of sand over the top of the pipe. If hydraulics dictate a conveyance system greater than a 6-inch pipe, several 6-inch pipes shall be used to obtain the necessary area of flow.

RESIDENTIAL DRY VAULT SAND FILTER SPECIFICATIONS

1) Structural Shell

The structural shell shall be designed to carry the load conditions anticipated at the site and take into account the specific site soil conditions. Structural shells to be placed under street, alley, parking lot, or emergency vehicle easement pavement shall be designed to resist, at a minimum, H-20 wheel loadings.

2) Inlet Energy Dissipators

A blast wall shall be provided directly in front of the inlet pipe to dissipate the energy of the inflow and minimize resuspention of collected sediments. As an alternative, the input pipe may be turned downward and a "T" placed on the end to direct the flow against both sidewalls to absorb the energy.

3) Sediment Chamber Outlet Structure

The sediment chamber and filter chamber must be separated by a concrete wall with sufficient outlet ports to discharge the flow evenly to the filtration chamber. An additional free-flow rectangular opening may be placed in the wall with a bottom elevation at least six (6) feet above the filter. No outlet port shall be placed on the central axis of the chambers in order to induce flow-spreading. As an alternative, a gabion wall containing four-inch to six-inch diameter stone may be used to separate the sediment and filter chambers (however, additional WQV storage capacity may have to be provided to compensate for the extra volume of rock). When used, the gabion shall contain stone which is four to six inches in diameter, durable, and free from seams and cracks. Weathered stone shall not be used.

4) Slope of Sediment Chamber Floor

The sediment chamber floor shall be sloped toward the filter chamber at a minimum slope of 1/2 percent to provide positive drainage. As an alternative, sloped channels in the floor may be used at each outlet port (there must be no low points where water can pool).

5) Distribution Trough Weir

A four-inch high fiberglass or aluminum weir plate shall be used to separate the distribution trough from the filter surface. Ninety-degree "V" notches shall be placed with their lowest point at the invert of the trough for complete drainage and shall be located so that no notch is directly in front of a sediment chamber outlet port. The outflow side shall incorporate features to prevent gouging of the filter media, e.g. concrete splash pad or rip-rap).

6) Geotechnical Filter Fabrics

The residential dry vault filter does not use filter fabrics to separate the aggregate layers. For wrappings for collector pipes, see the **Section (10), Underdrain Piping.**

7) Sand Filter Layer

For applications in Alexandria, use **ASTM C33 Concrete Sand.** Before construction of the filter, a laboratory analysis of the proposed sand which demonstrates that it meets this specification shall be provided to and approved by the Department of Transportation and Environmental Services for approval. The test shall include the effective size and the uniformity coefficient.

The filter sand shall be placed in six-inch lifts and lightly compacted with a lawn roller or hand tamp.

8) Intermediate Gravel Layer

The filter sand and bottom gravel layer shall be separated by a six-inch layer of washed 1/4-inch pea gravel to prevent washout of the sand. The pea gravel layer must be spread to a uniform thickness, raked to a level surface, and lightly compacted with a lawn roller or hand tamps before placement of the sand.

9) Bottom Gravel Layer

The bottom gravel layer shall be 1/2 to two (2) inch diameter gravel and provide at least two (2) inches of cover over the tops of the drainage pipes when used or above the tops of the weepholes to the clearwell when drainage pipes are not used.

10) Underdrain Piping

Underdrain piping shall be used in filters with a filter length (L_f) greater than 15 feet. The underdrain piping shall consist of three 6-inch schedule 40 or better polyvinyl perforated pipes reinforced to withstand the weight of the overburden. Perforations shall be 3/8 inch, and each row of perforations shall contain at least six (6) holes. Maximum spacing between rows of perforations shall be six (6) inches. **Only the horizontal sections of the pipes shall be perforated.**

The minimum grade of piping shall be 1/8 inch per foot (one (1) percent slope). Access for cleaning all underdrain piping is needed. Clean-outs for each pipe shall extend at least six (6) inches above the top of the upper filter surface.

Each pipe shall be thoroughly wrapped with 8 oz./sq.yd. geotechnical fabric meeting the following detailed specification before placement in the filter.

Property	Test Method	Unit	Specification	
Material	Nonwoven geotextile fabric			
Unit Weight		Oz/Sq.Yd.	8	(min.)
Filtration Rate		In/Sec	0.08	(min.)
Puncture Strength	ASTM D-751(Modified)	Lb.	125	(min.)
Mullen Burst Strength	ASTM D-751	Psi	400	(min.)
Tensile Strength	ASTM D-1682	Lb.	300	(min.)
Equiv. Opening Size	US Standard Sieve	No.	80	(min.)

11) Weepholes

In addition to the underdrain pipes, weepholes should be installed between the filter chamber and the clearwell to provide relief in case of pipe clogging. The weepholes shall be three (3) inches in diameter. Minimum spacing shall be nine (9) inches center to center. The openings on the filter side of the dividing wall shall be covered for the width of the trench with 12 inch high plastic hardware cloth of 1/4 inch mesh or galvanized steel wire, minimum wire diameter 0.03-inch, number 4 mesh hardware cloth anchored firmly to the dividing wall structure and folded back 6 inches back under the bottom stone.

12) Weepholes for Filters Without Underdrain Piping

For filters without underdrain piping (maximum L_f = 15 ft), large weepholes or rectangular slots may be used to drain the treated water from the filter chamber. Four-inch weepholes spaced 10 inches center to center or rectangular openings providing equivalent cross-sectional area shall be provided. Weephole openings on the filter side of the dividing wall shall be covered for the width of the filter chamber with 14 inch high plastic hardware cloth of 1/4 inch mesh or galvanized steel wire, minimum wire diameter 0.03-inch, number 4 mesh hardware cloth anchored firmly to the dividing wall structure and folded back 6 inches back under the bottom stone. When rectangular openings are used, a plastic or aluminum grate with openings sized to contain the bottom gravel layer shall be placed between the openings and the stone.

13) Pipe Penetration Sealer

The dewatering drain penetration in the wall between the filter chamber and the clearwell shall be sealed with a flexible strip joint sealer which swells in contact with water to form a tight pressure seal.

Water Quality Volume (WQV) Storage Tank Construction and Maintenance Requirements

A maintenance agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with Chapter 3 of the Alexandria Supplement to the Northern Virginia BMP Handbook must be executed by the developer before the Final Site Plan for the development will be released for construction. A site-specific agreement will be forwarded by the City with the bond package. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected in a timely manner.

1) Construction Requirements for WQV Storage Tanks

- The site erosion and sediment control plan must be configured to permit construction of the tank while maintaining erosion and sediment control.

- No runoff is to enter the WQV Storage Tank prior to completion of all construction and site revegetation. Construction runoff shall be treated in separate sedimentation basins and routed to by-pass the tank. Should construction runoff enter the tank prior to site revegetation, all contaminated materials must be removed and the tank thoroughly cleaned before it is placed in service.

- Access manholes to the filtration WQV Storage Tank shall conform to Alexandria standards.

- After completion of the filter shell entrances to the structure shall be plugged and the shell completely filled with water to demonstrate watertightness. Should the structure fail this test, it shall be made watertight and successfully retested prior to being placed in service.

- Electrical and telemetry lines servicing the WQV Storage Tank shall be undergrounded.

- Monitoring manholes, flumes, and other facilities shall be made complete and ready for use before release of the final Certificate of Occupancy for the project.

2) Maintenance Requirements for WQV Storage Tanks

- The BMP shall be inspected semiannually by representatives of the owner and the City to assure continued proper functioning.

- The sediment chamber must be pumped out after each joint owner-City semiannual inspection. 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 vacuum pump and disposed of in an appropriate landfill. **After each cleaning, refill the first chamber to a depth of three feet with clean water to reestablish the water seals.**

- Monitoring manholes, flumes, and other facilities shall be kept clean and ready for use.

Austin Sand Filter Construction and Maintenance Requirements

A maintenance agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with Chapter 3 of the Alexandria Supplement to the Northern Virginia BMP Handbook must be executed by the developer before the Final Site Plan for the development will be released for construction. A site-specific agreement will be forwarded by the City with the bond package. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected in a timely manner.

1) Construction Requirements for Austin 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.

- Sediment removed from the basins as a result of maintenance may be disposed of on-site if properly stabilized according to the practices outlined in the Virginia Erosion and Sedimentation Control Manual.(33) An off-site disposal site must either be an approved landfill or be issued a permit through the Department of Transportation and Environmental Services.

- Design should minimize susceptibility to vandalism by use of strong materials for exposed piping and accessories.

- Side slopes for earthen embankments structures shall 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 revegetation. Construction runoff may be routed to the sedimentation basin/chamber but outflow from this structure shall by-pass the sand filter basin.

- The top of the sand filter must be completely level. No grade is allowable.

- Monitoring manholes shall be made complete and ready for use by City forces or contractors.

2) **Major Maintenance Requirements for Sedimentation Basins**

- Removal of silt when accumulation exceeds six (6) inches in sediment basins without sediment traps. In basins with sediment traps, removal of silt shall occur when the accumulation exceeds four (4) inches in the basins, and sediment traps shall be cleaned when full.

- Removal of accumulated paper, trash and debris every six (6) months or as necessary.

- Vegetation growing within the basin is not allowed to exceed 18 inches in height at any time.

- Corrective maintenance is required any time a sedimentation basin does not drain the equivalent of the Water Quality Volume within 40 hours (i.e., no standing water is allowed).

- Corrective maintenance is required any time the sediment trap (optional) does not drain down completely within 96 hours (i.e., no standing water allowed).

3) **Major Maintenance Requirements for Filtration Components**

- Removal of silt when accumulation exceeds 1/2 inch. - Removal of accumulated paper, trash and debris every six (6) months or as necessary.

- Vegetation growing within the basin is not allowed to exceed 18 inches in height.

- Corrective maintenance is required any time draw-down does not occur within 36 hours after the sedimentation basin has emptied.

- When a dry vault filter will no longer draw down within the required 36-hour period because of clogging with silt (approximately every 3-5 years), the upper layer of gravel and geotechnical cloth must be replaced with new clean materials meeting the original specifications.

- Monitoring manholes, flumes, and other facilities shall .lm1"
be kept clean and ready for use.

Maintenance and Construction Requirements for D.C. Sand Filters

A maintenance agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with Chapter 3 of the Alexandria Supplement to the Northern Virginia BMP Handbook must be executed by the developer before the Final Site Plan for the development will be released for construction. A site-specific agreement will be forwarded by the City with the bond package. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected within specified time limits.

1) Construction Requirements for D.C. Sand Filters

- The site erosion and sediment control plan must be configured to permit construction of the filter system while maintaining erosion and sediment control.

- No runoff is to enter the sand filtration system prior to completion of all construction and site revegetation. Construction runoff shall be treated in separate sedimentation basins and routed to by-pass the filter system. Should construction runoff enter the filter system prior to site revegetation, all contaminated materials must be removed and replaced with new clean materials.

- The top of the sand filter must be completely level. No grade is allowable.

- Access manholes to the filtration system shall conform to Alexandria standards.

- After completion of the filter shell but before placement of the filter layers, entrances to the structure shall be plugged and the shell completely filled with water to demonstrate watertightness. Maximum allowable leakage is 10 percent of the filter shell volume in 24 hours. Should the structure fail this test, it shall be made watertight and successfully retested prior to placement of the filter layers.

2) Maintenance Requirements for D.C. Sand Filters

- o 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 and a log shall be maintained of the results indicating the rate of dewatering after each storm and the water depth for each observation. Once the City staff indicates that satisfactory performance of the structure has been demonstrated, the monitoring schedule can be reduced to an semiannual basis.

- o The BMP shall be inspected semiannually by representatives of the owner and the City to assure continued proper functioning.

- o The sediment chamber must be pumped out after each joint owner-City semiannual inspection. 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 vacuum pump and disposed of in an appropriate landfill. **After each cleaning, refill the first chamber to a depth of three feet with clean water to reestablish the water seals.**

- o When the filter will no longer draw down within the required 40-hour period, the top layer of filter cloth and ballast gravel must be removed and replaced with new materials conforming to the original specifications. Any discolored or sediment contaminated sand shall also be removed and replaced.

- o Monitoring manholes, flumes, and other facilities shall .lml" be kept clean and ready for use.

Construction and Maintenance Requirements for Delaware Sand Filters

A maintenance agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with Chapter 3 of the Alexandria Supplement to the Northern Virginia BMP Handbook must be executed by the developer before the Final Site Plan for the development will be released for construction. A site-specific agreement will be forwarded by the City with the bond package. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected in a timely manner.

1) Construction Requirements for Delaware Sand Filters

- Erosion and sediment control measures must be configured to prevent any inflow of stormwater into the DSF until construction on site is complete and all soil surfaces on the drainage watershed have been stabilized with vegetation.

- The DSF must not be placed in service until all soil surfaces in the drainage watershed have been stabilized with vegetated cover.

- The top of the sand filter must be completely level. No grade is allowable.

- The inverts of the notches, multiple orifices, or weirs dividing the sedimentation chamber from the filter chamber must also be completely level. Otherwise, water will not arrive at the filter as sheet flow and only the downhill end of the filter will function.

- 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 and filter chambers are completely level.

- If precast concrete lids are used, lifting rings or threaded sockets must be provided to allow easy removal with lifting equipment. Lifting equipment must be readily available to the facility operators.

- Where underdrains are used, the minimum slope of the pipe shall be 0.5%. Where only gravel filtered water conveyance is provided, the bottom of the filter chamber must be sloped towards the weepholes at a minimum slope of 0.5%.

- Monitoring manholes, flumes, and other facilities shall be made complete and ready for use before release of the final Certificate of Occupancy for the project.

2) **Maintenance Requirements for Delaware Sand Filters**

- o During the first year of operation, the cover grates or precast lids on the chambers must be removed quarterly and a joint owner-T&ES inspection made to assure that the system is functioning. Once the T&ES inspectors are satisfied that the system is functioning properly, this inspection may be made on an semiannual basis for other than auto-related activities.

- o When deposition of sediments in the filtration chamber indicate that the filter media is clogging and not performing properly, sediments must be removed (a small shovel may be all that is necessary) along with the top two to three inches of sand. The coloration of the sand will provide a good indication of what depth of removal is required. Clean sand must then be placed in the filter to restore the design depth. Where a layer of geotechnical fabric overlays the filter, the fabric shall be rolled up and removed and a similar layer of clean fabric installed. Any discolored sand shall also be removed and replaced.

- o Grass clippings from landscape areas on the drainage watershed flowing into the DSF must be bagged and removed from the site to prevent them washing into and contaminating the sediment chamber and filter.

- o Disposal of petroleum hydrocarbon contaminated sand or filter cloth may present a problem in the future. Advice on disposal may be obtained from the Environmental Quality office of the Alexandria Health Department.

- o Trash collected on the grates protecting the inlets shall be removed no less frequently than weekly to assure preserving the inflow capacity of the BMP.

- o Monitoring manholes, flumes, and other facilities shall .lm1" be kept clean and ready for use.

Construction and Maintenance Requirements for Peat Sand Filters

A maintenance agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with Chapter 3 of the Alexandria Supplement to the Northern Virginia BMP Handbook must be executed by the developer before the Final Site Plan for the development will be released for construction. A site-specific agreement will be forwarded by the City with the bond package. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected in a timely manner.

1) Construction Requirements for Peat Sand Filters

- Provisions must be made for access to the basins for maintenance purposes. A maintenance vehicle access ramp is necessary. The slope of the ramp should not exceed 4:1.

- Sediment removed from the basins as a result of maintenance may be disposed of on-site if properly stabilized according to the practices outlined in the Virginia Erosion and Sedimentation Control Handbook.[33] An off-site disposal site must either be an approved landfill or be issued a permit through the Department of Transportation and Environmental Services.

- Design should minimize susceptibility to vandalism by use of strong materials for exposed piping and accessories. A security fence such as chain-link should be placed around all surface system components. Provision should be made for locking gate valves to prevent tampering.

- Side slopes for earthen embankments structures shall not exceed 3:1 to facilitate mowing.

- The temporary erosion and sedimentation control plan must be configured to permit construction of the PSF while maintaining erosion and sedimentation control.

- No runoff is to enter the PSF system prior to completion of construction and site revegetation. Construction runoff may be routed to the sedimentation basin/chamber but outflow from this structure shall by-pass the PSF system.

2) Major Maintenance Requirements for Sedimentation Basins of Peat-Sand Filters

- o Removal of silt when accumulation exceeds six (6) inches in sediment basins without sediment traps. In basins with sediment traps, removal of silt shall occur when the accumulation exceeds four (4) inches in the basins, and the sediment traps shall be cleaned when full.

- o Removal of accumulated paper, trash and debris every six (6) months or as necessary.

- o Vegetation growing within the basin is not allowed to exceed 18 inches in height at any time.

- o Annual inspection and repair of the structure.

- o Corrective maintenance is required any time a sedimentation basin does not drain the equivalent of the Water Quality Volume within 60 hours (i.e., no standing water is allowed except during the winter months when a permanent pool is retained).

- o Corrective maintenance is required any time the sediment trap (optional) does not drain down completely within 96 hours (i.e., no standing water allowed).

3) Major Maintenance Requirements for the PSF Basin

Proper maintenance and optimal performance of the PSF system requires that the grass cover crop be periodically mowed and that cuttings/clippings be removed to prevent nutrients released by the decaying clippings from reentering the filter. Maximum nutrient removal from the system is achieved through the removal and harvesting of plant biomass. Mowing frequency and height will vary depending upon the grass species selected, climatic conditions aesthetic needs, available mowing equipment and budget constraints. An excellent summary of mowing considerations, developed by Elling [47], for five filter bed grasses (reed canary grass, quackgrass, marsh foxtial, meadow fescue and rough stalked bluegrass) is presented as follows:

Points to consider when maintaining grass at a height of <15 cm (6 in)

- o Any of the five species will produce good yields and remove large amounts of nutrients.

- o Filter bed looks neat and clean.

- A normal (large wheeled) rotary lawn mower with a 1m1" grass catcher is all that is needed for cutting the grass.

- Frequent mowing is required (about once a week during peak grass growing periods). The bed will require raking to remove cuttings following each mowing.

Points to consider when allowing grass to grow taller than 15 cm (6 in)

- Mowing is less frequent (maximum 3 to 4 times a year).

- Meadow fescue, marsh foxtail, and reed canary grass will be less likely to lodge and die than rough-stalked bluegrass or quackgrass.

- A sickle mower or some device other than a rotary lawn mower is needed to cut the grass.

- The bed will need to be raked following each mowing to remove the cut grass.

- Most people feel that tall grass is less aesthetic than the shorter, lawn-like grass.

F) Operations Requirements for Peat-Sand Filters

1) Winter Shut-Down

The PSF system is not designed to operate during winter months (i.e., mid-December to mid-March in Northern Virginia). Winter shut-down of the peat bed basin is strongly recommended so as to avoid the possible detachment of the grass cover crop via floating ice. Provision of gate valves for shut-off of flow between the basins coupled with the auxiliary gate valve-controlled bypass line to the storm sewer allow use of the sedimentation chamber as a extended detention/wet pond during the winter months. The gate valve on the bypass line should be adjusted to allow drawdown of a full WQV to the bypass invert in 48 hours.

2) Watering Grass Cover Crop During Droughts

During periods of severe droughts, the cover crop will require irrigation to keep it alive. During winter shut down months, the filter may be irrigated when temperatures are above freezing by releasing a sufficient amount of water from the permanent pool in the sediment chamber. During summer months, water from the sediment trap may be retained and used for irrigation if the drain line is equipped with a valve. If neither of these sources is available, the filter must be irrigated with fresh water.

Construction and Maintenance Requirements for Trench Sand Filters

A maintenance agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with Chapter 3 of the Alexandria Supplement to the Northern Virginia BMP Handbook must be executed by the developer before the Final Site Plan for the development will be released for construction. A site-specific agreement will be forwarded by the City with the bond package. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected in a timely manner.

1) Construction Requirements for Trench Sand Filters with Stone Reservoirs

- Erosion and sediment control measures must be configured to prevent any inflow of stormwater into the Trench Sand Filter until construction on-site is complete and all soil surfaces on the drainage watershed have been stabilized with vegetation.

- The Trench Sand Filter must not be placed in service until all soil surfaces in the drainage watershed have been stabilized with vegetated cover.

- During excavation of the trench to design dimensions, the excavated materials must be placed away from and downstream of the excavation to prevent redeposition during subsequent runoff events. Large tree roots shall be trimmed flush with the sides to protect the filter fabric and geomembrane during its installation.

- There shall be no voids between the filter fabric-geomembrane "sandwich." If boulders or similar obstacles are removed from the excavation sides, the voids shall be filled with natural soils before the "sandwich" is installed.

- The rolls of filter fabric and geomembrane for the "sandwich" and material layer separations must be cut to proper width before installation. Width shall allow for perimeter irregularities plus a minimum of six (6) inches of overlap at the top. When fabric overlap is required elsewhere, the upstream section shall overlap the downstream section by a minimum of two (2) feet to insure that the fabric conforms to the excavation during aggregate placement.

- The collector gravel, filter sand, and crushed stone aggregate shall be placed in the trench using a backhoe or front-end loader with a drop height near the bottom of the trench. Aggregates shall <u>not</u> be dumped into the trench by a truck.

- The reservoir stone shall be clean, washed crushed aggregate and shall be placed in loose lifts of about 12 inches and lightly compacted with plate compactors. Compaction assures fabric conformity to the sides and should reduce the potential for clogging and settlement problems.

- After the sand filter is placed, the filter fabric should be installed and at least a six (6) inch overlap folded up the sides. Small amounts of aggregate should be used to temporarily secure the side overlaps until the first layer of crushed aggregate is installed.

- After the aggregate is placed, the top filter fabric should be installed with at least a six (6) inch overlap folded up the sides. Small amounts of aggregate should be used to temporarily hold the side overlaps in place until the last layer (6-12 inches) of smaller sized aggregate (3/4") is placed on top. The top aggregate shall not be compacted.

- There shall be no mixing of clean aggregate with natural or fill soils. All contaminated aggregate shall be removed and replaced with clean aggregate.

- Inspection wells and monitoring manholes or access ports shall be made complete and ready for use by City forces or contractors before release of the final Certificate of Occupancy for the project.

2) Maintenance Requirements for Sedimentation Basins of Trench Sand Filters

- Removal of silt when accumulation exceeds six (6) inches.

- Removal of accumulated paper, trash and debris every six (6) months or as necessary.

  - Vegetation (if any) growing within the basin should be kept neatly trimmed and must not be allowed to exceed six (6) inches in height at any time.

- Annual inspection and repair of the basin as necessary. Corrective maintenance is required any time a sedimentation basin does not drain the equivalent of the Water Quality Volume within 60 hours.

3) Maintenance Requirements for Filtration Components

- Removal of silt when accumulation exceeds 1/2 inch. Removal of accumulated paper, trash and debris as necessary to maintain a neat appearance.
- Vegetation should not be allowed to grow within the filter bed.
- Corrective maintenance is required any time draw-down does not occur within 36 hours after the sedimentation basin has emptied. This usually involves removing the top filter gravel layer and top layer of geotechnical cloth and replacing them with new, clean materials conforming to the original specifications.
- Annual inspection and repair of the basin as required.

4) Special Maintenance Requirements for Stone Reservoir Systems

Maintenance plays a very important part in the proper functioning of any infiltration system. Without adequate sediment control, the effective life of a stone reservoir system can be less than two years. Clogging of soil surfaces and the stone aggregate within the trench by sediments reduces the capacity of the facility to provide expected benefits and requires an extensive repair. A clogged stone reservoir usually must have the stone aggregate removed and the sediment removed from the bottom of the reservoir. If the sand filter becomes clogged, the sand would also have to be removed. Clean aggregate and filter fabric must then be installed and the vegetated filter strip areas revegetated.

Explicit procedures for the routine inspections, routine maintenance, and, if necessary, eventual repairs and replacement of the facility components must be submitted as part of the Stormwater Management Plan in the Plan of Development.

The detailed requirements for the maintenance of infiltration systems contained in Chapter 6 of the <u>NVBMPHB</u> shall be applicable to stone reservoir sand filter trenches.

Construction and Maintenance Requirements for Dry Vault Sand Filters

A maintenance agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with Chapter 3 of the Alexandria Supplement to the Northern Virginia BMP Handbook must be executed by the developer before the Final Site Plan for the development will be released for construction. A site-specific agreement will be forwarded by the City with the bond package. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected in a timely manner.

1) Construction Requirements for Dry Vault Sand Filters

- The site erosion and sediment control plan must be configured to permit construction of the filter system while maintaining erosion and sediment control.

- No runoff is to enter the sand filtration system prior to completion of all construction and site revegetation. Construction runoff shall be treated in separate sedimentation basins and routed to by-pass the filter system. Should construction runoff enter the filter system prior to site revegetation, all contaminated materials must be removed and replaced with new clean materials.

- The top of the sand filter must be completely level. No grade is allowable.

- Access manholes to the filtration system shall conform to Alexandria standards.

- After completion of the filter shell but before placement of the filter layers, entrances to the structure shall be plugged and the shell completely filled with water to demonstrate watertightness. Should the structure fail this test, it shall be made watertight and successfully retested prior to placement of the filter layers.

2) Maintenance Requirements for Dry Vault Sand Filters

- 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 and a log shall be maintained of the results indicating the rate of dewatering after each storm and the water depth for each observation. Once the City staff indicates that satisfactory performance of the structure has been demonstrated, the monitoring schedule can be reduced to an semiannual basis.

- The BMP shall be inspected semiannually by representatives of the owner and the City to assure continued proper functioning.

- Removal of silt when accumulation exceeds six (6) inches in sediment basins without sediment traps. In basins with sediment traps, removal of silt shall occur when the accumulation exceeds four (4) inches in the basins, and sediment traps shall be cleaned when full.

- Removal of accumulated paper, trash and debris every six (6) months or as necessary.

- When the filter will no longer draw down within the required 40-hour period, the top layer of filter cloth and ballast gravel must be removed and replaced with new materials conforming to the original specifications. Any discolored or sediment contaminated sand shall also be removed and replaced.

- Monitoring manholes, flumes, and other facilities shall be kept clean and ready for use.

Construction and Maintenance Requirements for Extended Dry Detention Ponds (Dry Ponds)

A maintenance agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with Chapter 3 of the Alexandria Supplement to the Northern Virginia BMP Handbook must be executed by the developer before the Final Site Plan for the development will be released for construction. A site-specific agreement will be forwarded by the City with the bond package. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected in a timely manner.

1) Construction Requirements for Extended Dry Detention Ponds (Dry Ponds)

(To be issued at a later date)

2) Maintenance Requirements for Extended Dry Detention Ponds (Dry Ponds)

- Remove accumulated paper, trash, and debris every six months or as necessary.
- Remove accumulated sediment as determined by the Director of Transportation and Environmental Services or his representative (approximately every two (2) to ten (10) years), and restore the dry pond to the design condition. The owner will have sixty (60) calendar days to accomplish the sediment removal.
- Corrective maintenance is required any time the dry pond does not drain completely within the design period (i.e. No Standing Water is Allowed). Exception: in the shallow marsh area of a two stage dry detention pond a pool, is allowed in accordance with the approved plan.
- Vegetation in the dry pond shall be kept between 8" minimum and 12" maximum in height. Mowing shall occur at least twice annually.
- The grass in the emergency spillway shall not be cut to less than 8" in height.
- Eroded or denuded areas shall be backfilled with topsoil, compacted, and reseeded or sodded immediately.
- Rip rap displacement or failure shall be corrected immediately.
- All vegetative clippings and cuttings shall be removed.

- Animal burrows, nests or lodges shall be removed and the burrows shall be backfilled and compacted.

- Manholes, flumes, pipes, grates, risers, rip rap, inlet/outlet controls and other structures shall be kept clean and ready for use.

- Dam settling, woody growth, and any signs of piping, seepage, slumping or failure shall be corrected immediately.

- Any sign of a deficiency in the structural integrity of the dam shall be reported to the Alexandria City Department of Transportation and Environmental Services, Engineering Division telephone number (703)838-6470.

- Principal flow paths, emergency spillways, and all safety features of the facility shall operate in accordance with the design requirements.

- Access to the facility as required in the approved plan must be maintained.

- A joint inspection of the dry pond by a representative of the Director of Transportation and Environmental Services and a representative of the owner shall occur every 6 months.

- Whenever required maintenance is not performed within 30 days from receipt of a notice from the City (60 days for sediment removal), the City may perform the work and bill the owner as allowed by Section 11-412 of the Alexandria Zoning Code and the BMP Maintenance and Monitoring agreement. Failure of the owner or his agent to take remedial action constitutes grounds for revocation of all City approvals regarding the land involved (Section 11-412 (B)(4) Alexandria Zoning Code).

Maintenance and Construction Requirements for Alexandria Compound Stormwater Filters

A maintenance agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with Chapter 3 of the Alexandria Supplement to the Northern Virginia BMP Handbook must be executed by the developer before the Final Site Plan for the development will be released for construction. A site-specific agreement will be forwarded by the City with the bond package. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected within specified time limits.

1) Construction Requirements for Alexandria Compound Stormwater Filters

- This stormwater Best Management Practice (BMP) shall be installed and constructed under the direct supervision of the design engineer or his/her designated representative. The design engineer shall make a written certification to the City that the Compound Stormwater Filter is installed and constructed as designed and in accordance with the approved site plan.

- The site erosion and sediment control plan must be configured to permit construction of the filter system while maintaining erosion and sediment control.

- No runoff is to enter the compound filtration system prior to completion of all construction and site revegetation. Construction runoff shall be treated in separate sedimentation basins and routed to by-pass the filter system. Should construction runoff enter the filter system prior to site revegetation, all contaminated materials must be removed and replaced with new clean materials.

- The top of the gravel filter and sand filter must be completely level. No grade is allowable.

- Access manholes to the filtration system shall conform to Alexandria standards.

- After completion of the filter shell but before placement of the wet and dry filters, entrances to the structure shall be plugged and the shell completely filled with water to demonstrate watertightness. Maximum allowable leakage is 10 percent of the filter shell volume in 24 hours. Should the structure fail this test, it shall be made watertight and successfully retested prior to placement of the filter layers.

- o Fill the headbox and upflow gravel filter to a depth of 3 (three) feet to establish the water hydrocarbon seal before placing the filter system in service.

2) **Maintenance Requirements for Alexandria Compound Stormwater Filters**

- o 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 and a log shall be maintained of the results indicating the rate of dewatering after each storm and the water depth for each observation. Once the City staff indicates that satisfactory performance of the structure has been demonstrated, the monitoring schedule can be reduced to an semiannual basis.

- o The BMP shall be inspected semiannually by representatives of the owner and the City to assure continued proper functioning.

- o The sediment chamber must be pumped out after each joint owner/City semiannual inspection. If the chamber contains an oil sheen, it should be removed by a firm specializing in oil recovery and recycling. The remaining material may then be removed by vacuum pump and disposed of in an appropriate landfill. **After each cleaning, refill the first chamber and wet filter to a depth of three feet with clean water to reestablish the water seals and restore the wet filter.**

- o When determined necessary by the joint City/owner inspection the upflow gravel filter shall be backwashed with a high pressure hose while simultaneously vacuum pumping the first chamber (head box). The wet filter shall also be backwashed each time the filter cloth on top of the sand filter is changed. When the filter is cleaned to the satisfaction of the City inspector, **the head box and wet filter shall be refilled to a depth of three feet with clean water to reestablish the water seal and restore the wet filter.**

- o When the filter will no longer draw down within the required 40-hour period, the top layer of filter cloth on top of the sand filter must be removed and replaced with new materials conforming to the original specifications. Any discolored or sediment contaminated sand shall also be removed and replaced. Ballast pea gravel may be stored on top of the wet filter and reused after backwash cleaning along with the wet filter gravel.

- Monitoring manholes, flumes, and other facilities shall be kept clean and ready for use.

XIII. COMPOUND STORMWATER FILTERS

A) Background and Facility Description

While monitoring Delaware Sand Filters in 1994, the Alexandria engineering staff discovered that enhanced nitrogen removal was apparently occurring in one of the filters as a result of anaerobic activity caused by permanent flooding of the filter bottom (the outfall pipe was installed too high)(Bell, Stokes, Gavan, and Nguyen, 1995). Mr. Larry Gavan, the staff Environmental Scientist, suggested that this phenomenon might be replicated by installing a separate flooded gravel filter beneath the sand media in the vault sand filters that are becoming the BMP of choice by Alexandria developers. His research discovered several papers on similar applications in combination filters for small wastewater flow treatment..

Placement of a flooded gravel filter in a trench or box beneath an intermittent sand filter is a proven nitrogen removal technique in home wastewater systems (Piluk and Hao, 1989; Rock and Pinkham, 1986). A 30 centimeter deep anaerobic zone is sufficient for such denitrification reactors if sufficient organic carbon is present (Rock and Pinkham, 1986). Compound anaerobic gravel/aerobic sand filter systems treating home wastewater flows have demonstrated total nitrogen removal 20-60 percent higher than intermittent sand filters alone (Laak, 1986; The Cadmus Group, 1991). Some systems have approached 100 percent TN removal (Piluk and Hao, 1989; Gold, Lamb, Loomis, and McKiel, Undated). Increases in TP removal have also been observed (Laak, 1986; Piluk and Hao, 1989). However, this technology has not previously been applied to treatment of stormwater runoff.

Wet gravel filters employ the natural biochemical processes occurring in the flooded gravel to remove the nitrogen.

A number of these filters utilize an upflow configuration, e.g. the influent enters at the bottom and flows upward through the filter media. Since most of these wet filters remain in an anaerobic state, they are usually referred to as Upflow Anaerobic Filters, or UAFs. While higher nitrogen removal efficiencies occur when the gravel filter is downstream of the sand filter, UAFs utilized as pretreatment have also been shown to significantly enhance the pollutant removal efficiencies of intermittent sand filters (Marlar, 1984; Mitchell, 1985, Viraraghavan, 1986).

The Alexandria engineering staff developed a compound filter system employing such principles. The Compound Stormwater Filter is essentially a D.C. Sand Filter with a 33 centimeter (13 inch) flooded anoxic filter below a 46 centimeter (18 inch) aerobic sand filter. Ten centimeters (4 inches) of dry gravel separate the two filters to keep the sand in an aerobic state. The preferred configuration

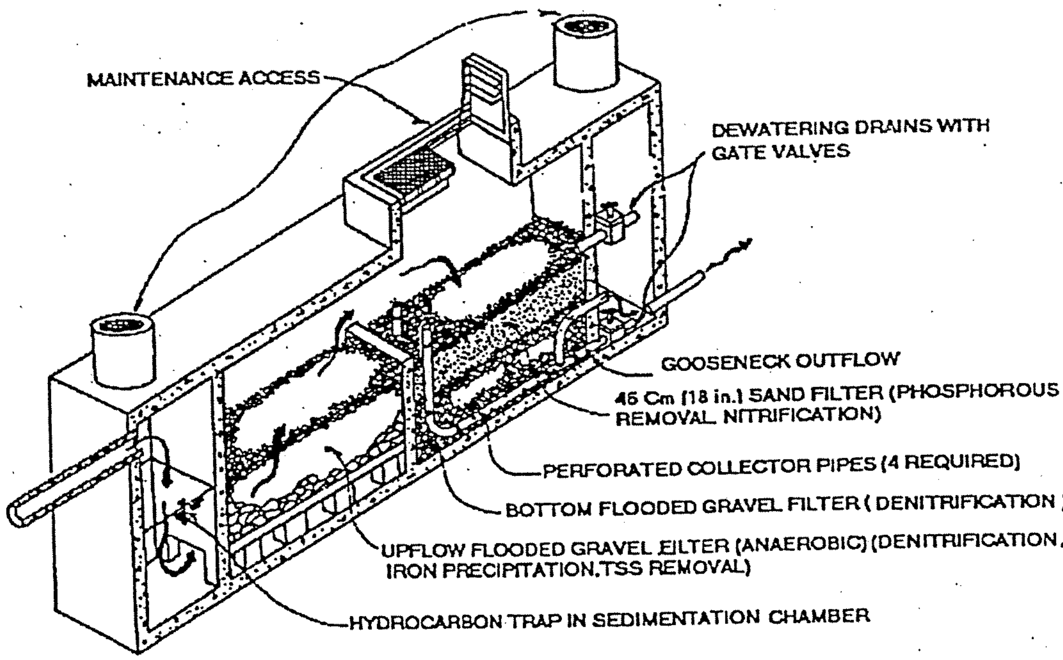


FIGURE 2-34 -- **Compound Stormwater Filter System**

also places upstream of the sand filter an upflow anaerobic filter (UAF) similar to the concept developed by Mr. R. Patel of the Prince George's County, Maryland, for pretreating runoff prior to infiltration or infiltration. The UAF is expected to further enhance phosphorous removal by precipitating more iron on the sand filter (a phenomenon observed by Mr. Patel on a sand filter system with UAF treatment) as well as remove nitrates in the raw runoff. **Figure 2-34** shows a compound filter with an upflow anaerobic filter. **Figure 2-35** is a cross-section of the UAF. **Figure 2-36** shows alternative configurations of the sand filter/bottom anoxic gravel filter. Flooding of the bottom gravel may be achieved by a system of "gooseneck" drains and dewatering valves as shown or by placing the BMP outflow pipe 13 (thirteen) inches above the filter bottom. In the latter case, a 14 (fourteen) inch high, 30 (thirty) inch wide maintenance step with a depressed channel for outflow shall be placed against the downstream end of the clearwell. The Alexandria compound filter design logic produces a filter vault of almost exactly the same size as that of a D.C. Sand Filter to treat the same impervious area. Costs of the two systems should therefore be approximately equal.

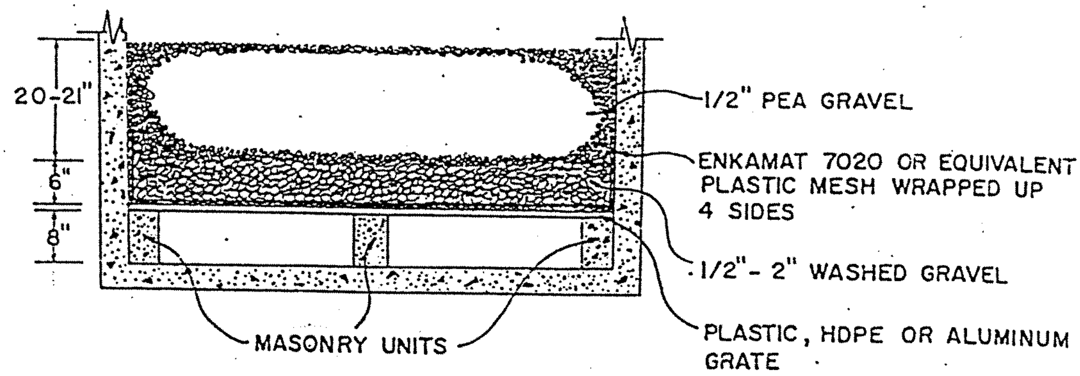


FIGURE 2-35 -- **Cross-Section of Upflow Anaerobic Filter**

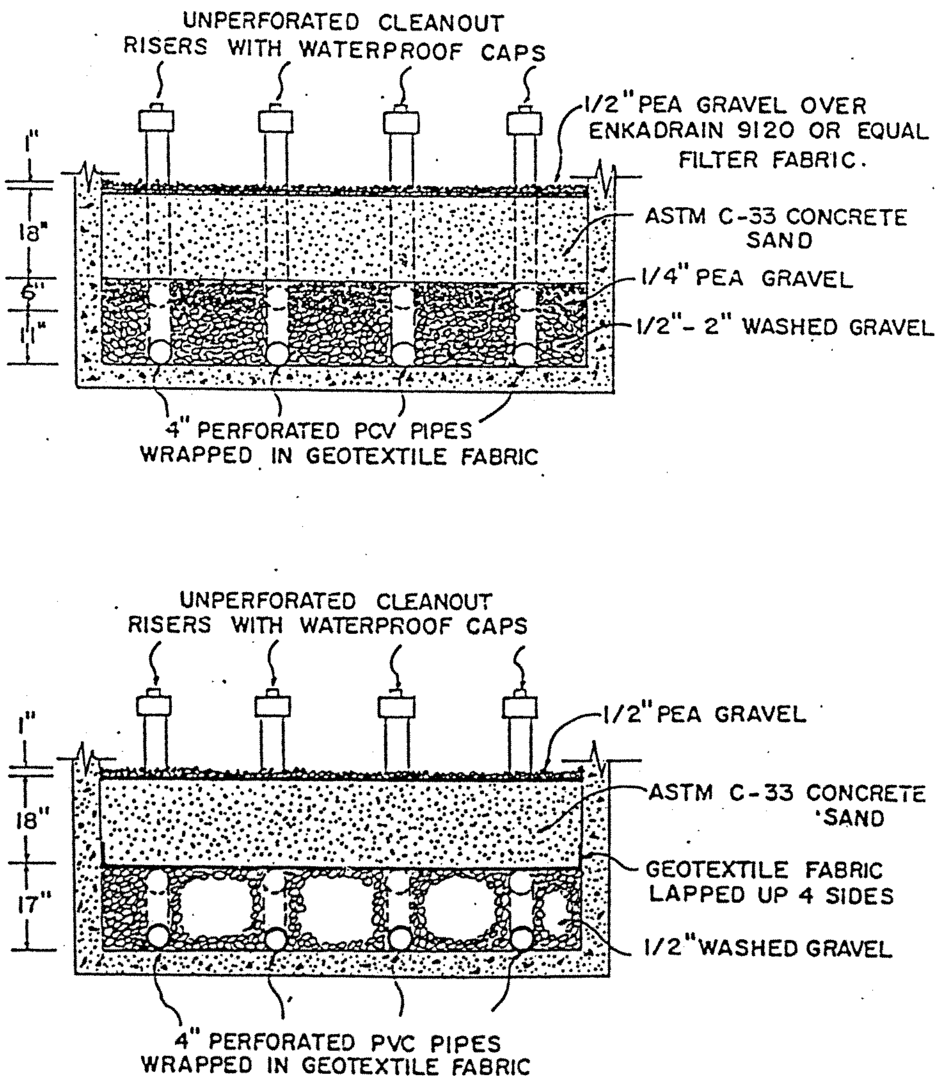


FIGURE 2-36 -- **Alternative Cross-Sections of Sand Filter/Anoxic Bottom Filter**

B) Removal Efficiencies

A Compound Filter System has been approved for installation in the Carlyle Towers condominium project in south Alexandria. The Northern Virginia Planning District Commission and Alexandria has received an EPA 319 grant to monitor this system for 18 months. In the interim, Alexandria is rating compound filters at 60 percent Total Phosphorous removal. Total nitrogen removal is expected to approach 50 percent.

C) Design Considerations

1) Applicability

Compound stormwater filters should be considered for use on projects in the ultra-urban sections of the City where a high level of impervious cover is proposed. They require no dedicated real estate since, like other vault sand filters, they may be placed beneath private streets or parking lots.

In Alexandria, these systems will be utilized only for off-line applications to treat the WQV. If a flow splitter is not installed ahead of the DCSF, an integral large storm bypass pipe(s) from the sediment chamber to the clearwell must be provided. The bypass pipe(s) must be located to one side to avoid blocking the access manholes or maintenance access doors. When required, quantity detention must be provided in a separate facility.

2) Practicability

Several years of success with sand filter systems have demonstrated their practicality for use in the Middle Atlantic States area. Costs vary with the size of the structure and the character of the site. Current custom-built vault sand filter systems in Alexandria cost approximately $20,000-25,000 per impervious acre treated. Use of precasting may reduce these costs. When compared to the average Alexandria raw real estate cost of approximately $100,000 per impervious acre treated for a wet pond or approximately $60,000 per impervious acre treated for a dry pond, vault filters, which have no real estate cost, are very competitive.

3) Groundwater and Bedrock

The seasonally-high groundwater table and bedrock should be located at least two (2) to (4) feet below the footing of the filter structure. Otherwise, additional weight must be provided to resist floatation.

4) **Drawdown Time**

As with intermittent sand filter BMPs, drawdown time should not exceed 40 hours so that the BMP will be free to process follow-on storms.

5) **Structural Requirements**

The load-carrying capacity of the filter structure must be considered when it is located under parking lots, driveways, roadways, and, certain sidewalks (such as those adjacent to State highways). Traffic intensity may also be a factor. The structure must be designed by a licensed structural engineer and the plans require City approval.

6) **Design Storm**

The inlet design or integral large storm bypass must be adequate for isolating the WQV from the 10 year storm (7 in./hr., 10 min. TOC) and for conveying the peak flow of that storm past the filter system. Since DCSFs will be used only as off-line facilities in Alexandria, the interior hydraulics of the filter are not as critical as when used as an on-line facility. The system should draw down in approximately 40 hours.

7) **Infrastructure Elevations**

For cost considerations, it is preferable that the compound stormwater filter work by gravity flow. This requires sufficient vertical clearance between the invert of the prospective inflow storm piping and the invert of the storm sewer which will receive the outflow. In cases where gravity flow is not possible, a clearwell sump and pump are required to discharge the effluent into storm sewer. If used, pumps must be configured to retain a permanently flooded zone of 33 centimeters (13 inches) in the gravel below the sand filter.

8) **Accessibility and Headroom for Maintenance**

All three compound filter chambers must have personnel access manholes and built-in access ladders .The filter must also be accessible to vacuum trucks for removing accumulated sediments and hydrocarbons at least every six months. Approximately every 3-5 years, the filter can be expected to clog to the point that replacement of the top layer of washed gravel and the top layer of filter cloth will be required. **A minimum headspace of 60 inches above the filter will be required if the ceiling to the chamber is a fixed structure.** A 38-inch diameter maintenance manhole with a nested 22-inch center lid (Neenah R-1742-D or equivalent) or a rectangular load bearing access door (minimum 4 ft. x 4 ft.) should be positioned directly over the center of the filter. When site conditions will not allow 60 inches of headspace, City staff will consider allowing reduced clearance if load bearing access doors or removable covers, such are sometimes employed over underground utility tunnels are to be provided.

9) Accessibility for Monitoring

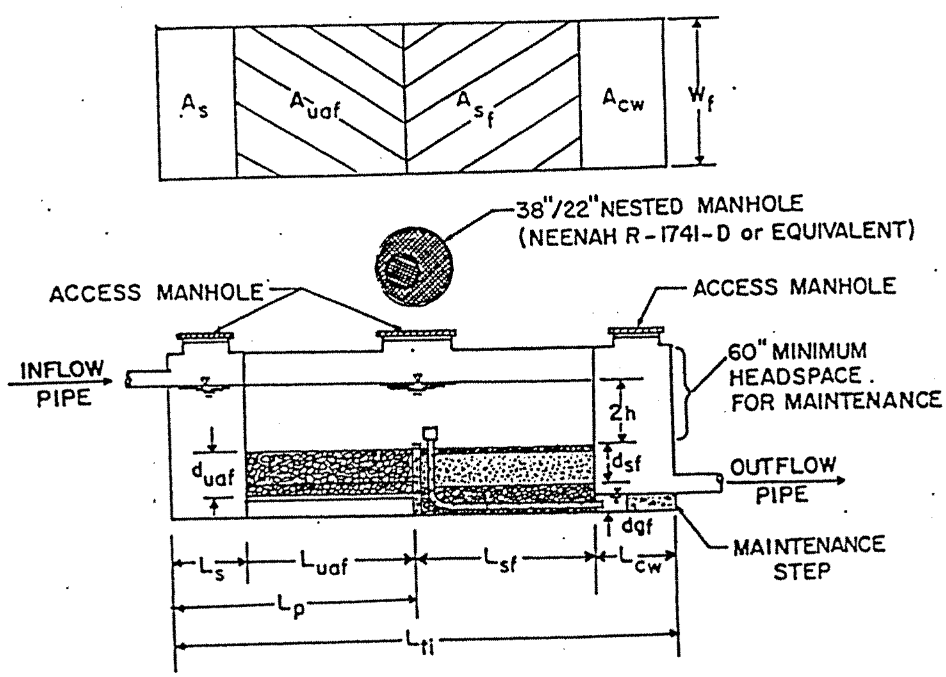
Unless otherwise approved by the Director, prefabricated monitoring manholes must be installed in the inflow and outflow pipes to allow chemical monitoring of the inflow water and effluent. See Appendix 2-8 of the *Alexandria Supplement to the Northern Virginia BMP Handbook.*

D) Design Procedures

1) Determine Governing Site Parameters

Determine the Impervious area on the site (I_a in acres), the water quality volume to be treated (WQV in ft.^{3} = 1816 I_a), and the site parameters necessary to establish 2h, the maximum ponding depth over the filter (storm sewer invert at proposed connection point, elevation to inflow invert to BMP, etc). If a bypass weir or pipe is to be built directly into the DCSF shell, it should be designed at this point. **Worksheet E** on page 2-A4-2 is provided to perform this step.

Figure 2-37 shows the dimensional relationships required to compute the remaining steps of the design.



<u>FIGURE 2-37</u> **Dimensional Relationships for Compound Filter**

2) Select Filter Depth and Determine Maximum Ponding Depth

Considering the data from Step 1) above, select the Filter Depth ((d_{sf}) and determine the maximum achievable ponding depth over the filter (2h).

3) Compute the Minimum Area of the Sand Filter (A_{sfm})

For applications in Alexandria, utilize the Austin Filter Formula with a coefficient of permeability of 3.0 ft/day (0.125 ft/hr) and a drawdown time of 40 hours. This results in:

$$A_{sfm(PS)} = \frac{363 I_a d_f}{(h + d_f)} \qquad (2\text{-}37)$$

where,

A_{sfm} = minimum surface area of sand bed (square feet)
I_a = impervious cover on the watershed in acres
d_{sf} = sand bed depth (normally 1.5 to 2ft)
h = average depth of water above surface of sand media between full and empty basin conditions (ft.)

4) Select Filter Width and Compute Filter Length and Adjusted Filter Area

Considering site constraints, select the Filter Width (W_f). Then compute the Sand Filter Length (L_{sf}) and the Adjusted Sand Filter Area (A_{sf})

$$L_{sf} = A_{sfm}/W_f \qquad (2\text{-}38)$$

$$A_{sf} = W_f \times L_{sf} \qquad (2\text{-}39)$$

NOTE: From this point, formulae assume rectangular cross section of filter shell.

5) Compute the Storage Volume on Top of the Sand Filter (V_{Tsf})

$$V_{Tsf} = A_{sf} \times 2h \qquad (2\text{-}40)$$

6) Compute the Storage in the Unflooded Filter Voids (V_v)

(Assume 40% voids in filter media)

$$V_v = 0.4 \times A_{sf} \times [d_{sf} + (d_g - 1 \text{ foot})] \qquad (2\text{-}41)$$

7) Compute Flow Through Filter During Filling (V_Q)

(Assume 1-hour to fill per D.C. practice)

$$V_Q = \frac{kA_f(d_{sf} + h)}{d_{sf}} \text{ ; use } k = 3 \text{ ft./day} = 0.125\text{ft/hr.} \qquad (2\text{-}42)$$

8) Compute Net Volume to be Stored Awaiting Filtration (V_{st})

$$V_{st} = WQV - V_{Tff} - V_v - V_Q \qquad (2\text{-}43)$$

9) **Compute Minimum Length of Permanent Pool (including upflow anaerobic filter) (L_{pm})**

$$L_{pm} = \frac{V_{st}}{(2h \times W_f)} \qquad (2\text{-}44)$$

10) **Compute the Minimum Length of the Upflow Anaerobic Filter (L_{uaf})**

Make the area of the UAF (A_{uaf}) $\geq$ the area of the sand filter (A_{sf}). Therefore, the length of the UAF (L_{uaf}) $\geq L_{sf}$.

Note: the Alexandria Engineering staff has computed that this size UAF will pass the peak flow rate of the Design Storm with a head differential of 1.5 inches between the headbox and the filter chamber.

11) **Compute Minimum Length of Sediment Chamber (Headbox) (L_s)** (Minimum length = four feet)

$$L_{sm} = L_{pm} - L_{uaf} \qquad (2\text{-}45)$$

12) **Set Length of Clearwell (L_{cw}) for Adequate Maintenance Access (Minimum Length = four feet) and Establish Structure Dimensions.**

It may be economical to adjust final dimensions to correspond with standard precast structures or to round off to simplify measurements during construction.

Worksheet K on page 2-A4-33 is provided to assist with performing the above calculations.

E) Specifications

Specifications for the Compound Stormwater Filter are contained in Appendix 2-2. to the *Alexandria Supplement* on pages 2-A2-5 through 2-A2-7. These specifications shall be quoted *verbatim* on the Stormwater Management Plan sheets of the Final Site Plan.

F) Construction and Maintenance Requirements

A Maintenance Agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with the Chapter 3 of this manual must be executed by the developer/owner before the Final Site Plan for the development will be released for construction. A project-specific agreement will be forwarded by the City with the bond estimate.

Construction and Maintenance requirements for Alexandria Compound Stormwater Filters are delineated in detail on pages 2-A3-23 through 2-A3-26. There requirements shall be reproduced ***verbatim*** on the Stormwater Management Plan sheets of the Final Site Plan.

ALEXANDRIA COMPOUND STORMWATER FILTER SPECIFICATIONS

(Revised 3/27/95)

A. Upflow Gravel Filter (if used)

1) Grate

The grate shall be fiberglass, polyvinyl chloride, or aluminum with sufficient strength to support the weight of the filter and openings close enough together to contain the large gravel. A strong plastic mesh such as Enkamat 7020 may also be used to contain the gravel above the grate.

2) Lower Gravel

The lower gravel shall be 6 (six) inches thick and be off sufficient size to be contained above the grate. If a strong plastic mesh such as Enkamat 7020 is placed above the grate, the same 1/2 inch to 2 inch diameter washed gravel as utilized in the sand filter may be used.

3) Filter Gravel

The wet gravel filter shall consist of 24-26 inches of washed 1/2-inch pea gravel.

B. Sand Filter With Anoxic Lower Zone

1) Upper Aggregate Layer

The washed aggregate or gravel layer at the top of the filter shall be the same 1/2-inch diameter washed pea gravel that was used in the Upflow Gravel Filter.

2) Geotechnical Fabrics

The filter fabric beneath the layer of gravel on top of the filter must be cut with sufficient dimensions to cover the entire wetted perimeter of the filtering area with a minimum overlap up the side and end walls to the depth of the gravel and shall be Enkadrain 9120 filter fabric or equivalent with the following specifications:.

Property	Test Method	Unit	Specification
Material	Nonwoven geotextile fabric		
Unit Weight	ASTM D-1777	Oz/sq.yd.	4.3 (min.)
Flow Rate	Falling Head Test	gpm/sq.ft	120 (min.)
Puncture Strength	ASTM D-751(Modified)	Lb.	60 (min.)
Thickness		in.	0.8 (min.)

If a filter cloth layer is used beneath the sand, the filter cloth layer beneath the sand shall be cut with sufficient dimensions to cover the entire wetted perimeter of the filtering area **with a minimum six-inch overlap up the side and end walls** and must conform to the following specification (same as for Austin Sand Filter):

Property	Test Method	Unit	Specification
Material	Nonwoven geotextile fabric		
Unit Weight		Oz/Sq.Yd.	8 (min.)
Filtration Rate		In/Sec	0.08 (min.)
Puncture Strength	ASTM D-751(Modified)	Lb.	125 (min.)
Mullen Burst Strength	ASTM D-751	Psi	400 (min.)
Tensile Strength	ASTM D-1682	Lb.	300 (min.)
Equiv. Opening Size	US Standard Sieve	No.	80 (min.)

3) Sand Filter Layer

For applications in Alexandria, use **ASTM C33 Concrete Sand.**[32] Obtain a laboratory gradation report demonstrating that the proposed sand meets this specification for approval of the Alexandria Transportation and Environmental Services Department prior to placement of the sand in the filter. Results shall include effective size and uniformity coefficient.

4) Gravel Bed Around Collector Pipes and Anoxic Filter Zone

The lower gravel layer surrounding the collector pipes shall be 1/2 to two (2) inch diameter washed gravel. If a filter fabric is used to separate the sand from the large gravel, the large gravel depth shall be 17 inches. If a filter fabric layer is not used, the large gravel shall be 11 inches thick, and a six inch thick layer of 1/4 inch pea gravel shall be placed between the sand and the larger gravel.

5) Underdrain Piping

The underdrain piping consists of four 4-inch schedule 40 or better polyvinyl perforated pipes reinforced to withstand the weight of the overburden. **Only the horizontal sections between front and rear risers (goosenecks) shall be perforated.** Perforations shall be 3/8 inch diameter, and each row of perforations shall contain at least four (4) holes. Maximum spacing between rows of perforations shall be six (6) inches.

Access for cleaning all underdrain piping is needed. Clean-outs with watertight caps for each pipe shall extend at least twelve (12) inches above the top of the upper filter surface, e.g. the top layer of gravel.

Each pipe shall be thoroughly wrapped with 8 oz./sq.yd. geotechnical fabric meeting the above detailed specification before placement in the filter.

Each pipe penetration in the wall between the filter chamber and the clearwell chamber shall be sealed with a flexible strip joint sealant which swells in contact with water to form a tight pressure seal.

When "gooseneck" drains are used to contain the permanent pool in the gravel beneath the sand filter, dewatering drains with gate valves shall be installed on each collector pipe at the level of the filter box floor.

C. Dewatering Drain for Filter Chamber

A 6 (six) inch dewatering drain between the filter chamber and the clearwell shall be provided at the elevation of the top of the sand filter. The drain shall be equipped with a gate valve in the clearwell chamber.

6) Weepholes

NO WEEPHOLES SHALL BE PROVIDED IN THE COMPOUND FILTER.

C. Access Requirements

Access to the headbox (sediment chamber) and clearwell shall be provided through 30-inch manholes. Access to the filter compartment shall be provided by a rectangular door (minimum size: 4 [four] feet by 4 [four] feet) of sufficient strength to carry prospective imposed loads or by a manhole of at least 38 inch diameter with an offset concentric 22 inch lid inside the large lid (Neenah R-1741-D or equivalent)..

Maintenance and Construction Requirements for Alexandria Compound Stormwater Filters

A maintenance/monitoring agreement with the City concerning the site stormwater quantity/quality management facilities prepared in accordance with Chapter 3 of the *Alexandria Supplement to the Northern Virginia BMP Handbook* must be executed by the developer before the Final Site Plan for the development will be released for construction. A site-specific agreement will be forwarded by the City with the bond package. Maintenance will normally be accomplished by developers/owners, with periodic inspections by Transportation and Environmental Services inspectors to assure compliance. Sanctions may be imposed if citations of improper maintenance are not corrected within specified time limits.

1) Construction Requirements for Alexandria Compound Stormwater Filters

- This stormwater Best Management Practice (BMP) shall be installed and constructed under the direct supervision of the design engineer or his/her designated representative. The design engineer shall make a written certification to the City that the Compound Stormwater Filter is installed and constructed as designed and in accordance with the approved site plan.

- The site erosion and sediment control plan must be configured to permit construction of the filter system while maintaining erosion and sediment control.

- No runoff is to enter the compound filtration system prior to completion of all construction and site revegetation. Construction runoff shall be treated in separate sedimentation basins and routed to by-pass the filter system. Should construction runoff enter the filter system prior to site revegetation, all contaminated materials must be removed and replaced with new clean materials.

- The top of the gravel filter and sand filter must be completely level. No grade is allowable.

- Access manholes to the filtration system shall conform to Alexandria standards.

- After completion of the filter shell but before placement of the wet and dry filters, entrances to the structure shall be plugged and the shell completely filled with water to demonstrate watertightness. Maximum allowable leakage is five (5) percent of the filter shell volume in 24 hours. Should the structure fail this test, it shall be made watertight and successfully retested prior to placement of the filter layers.

- After placement of the underdrain pipes and dewatering drains (including flexible strip joint sealant) but before placement of the filter aggregates, the valves shall be closed and the filter chamber flooded for 24 hours to maximum pooling depth (2h) to test for watertightness. **No leakage around the pipes or through the valves is acceptable. If leakage is observed, repairs must be made and the test rerun until no leakage occurs.**

- Fill the headbox and upflow gravel filter to a depth of 3 (three) feet to establish the water hydrocarbon seal before placing the filter system in service.

2) Maintenance Requirements for Alexandria Compound Stormwater Filters

- 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 and a log shall be maintained of the results indicating the rate of dewatering after each storm and the water depth for each observation. Once the City staff indicates that satisfactory performance of the structure has been demonstrated, the monitoring schedule can be reduced to an semiannual basis.

- The BMP shall be inspected semiannually by representatives of the owner and the City to assure continued proper functioning.

- The sediment chamber must be pumped out after each joint owner/City semiannual inspection. If the chamber contains an oil sheen, it should be removed by a firm specializing in oil recovery and recycling.. The remaining material may then be removed by vacuum pump and disposed of in an appropriate landfill. **After each cleaning, refill the first chamber and wet filter (if used) to a depth of three feet with clean water to reestablish the water seals and restore the wet filter.**

- When determined necessary by the joint City/owner inspection the upflow gravel filter (if used) shall be backwashed with a high pressure hose while simultaneously vacuum pumping the first chamber (head box). The wet filter shall also be backwashed each time the filter cloth on top of the sand filter is changed. When the filter is cleaned to the satisfaction of the City inspector, **the head box and wet filter shall be refilled to a depth of three feet with clean water to reestablish the water seal and restore the wet filter.**

- When the filter will no longer draw down within the required 40-hour period, the top layer of filter cloth on top of the sand filter must be removed and replaced with new materials conforming to the original specifications. Any discolored or sediment contaminated sand shall also be removed and replaced. Ballast pea gravel may be stored on top of the wet filter and reused after backwash cleaning along with the wet filter gravel.

- Monitoring manholes, flumes, and other facilities shall be kept clean and ready for use.

ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET K: COMPUTATIONS FOR ALEXANDRIA COMPOUND STORMWATER FILTER WITH UPFLOW GRAVEL PREFILTER

4: Considering data on Worksheet E, select maximum ponding depth over filter:

2h = ______ ft;

h = [ ] ft

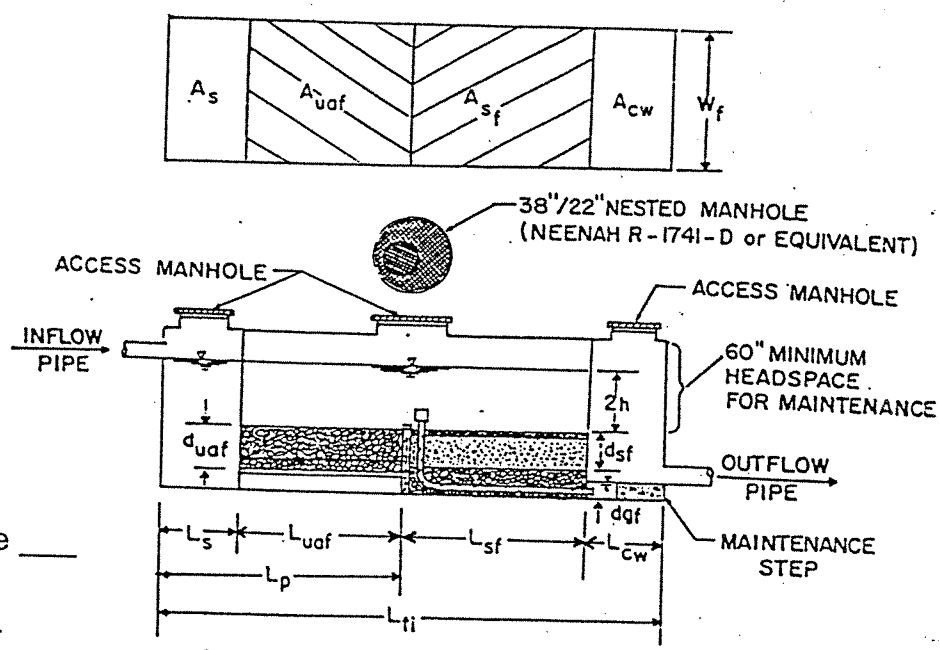
From WORKSHEET E;

I_a = [ ] acres

WQV = [ ] ft^3

Outflow by gravity possible ___

Effluent pump required ___



Part 5: Compute Minimum Area of Sand Filter (A_{sfm}):

$$A_{sfm} = \frac{363 I_a d_{sf}}{(d_{sf} + h)}$$

= [363 x ______ x ______] / [______ + ______]

= [ ] ft^2

Part 6: Considering Site Constraints, Select Filter Width (W_f) and Compute Sand Filter Length (L_{sf}) and Adjusted Filter Area (A_{sf}):

W_f = [ ] ft;

$L_{sf} = A_{sfm} / W_f$

= ______ / ______

= ______, say [ ] ft

$A_{sf} = W_f \times L_{sf}$ = ______ x ______

= [ ] ft^2

Part 7: Compute the Storage Volume on Top of the Sand Filter (V_{Tsf}):

$V_{Tsf} = A_{sf} \times 2h =$ _____ x _____

$= \boxed{}$ ft^3

Part 8: Compute Storage in Sand Filter Voids (V_v):
(Assume 40% voids in filter media)

$V_v = 0.4 \times A_{sf} \times (d_{sf} + d_g)$

$= 0.4 \times$ _____ x (_____ + _____)

$= \boxed{}$ ft^3

Part 9: Compute Flow Through Sand Filter During Filling Period (V_Q): (Assume 1-hour to fill per D.C. practice)

$V_Q = \dfrac{kA_{sf}(d_{sf} + h)}{d_{sf}}$; use k = 3 ft/day = 0.125 ft/hr

= [0.0833 x _____ x (_____ + _____)] / _____

$= \boxed{}$ ft^3

Part 10: Compute Net Volume to be Stored Awaiting Filtration (V_{st}):

$V_{st} = WQV - V_{Tsf} - V_v - V_Q$

= _____ - _____ - _____ - _____

$= \boxed{}$ ft^3

Part 11: Compute Minimum Length of Permanent Pool (Including Upflow Anaerobic Filter) (L_{uaf}):

$L_{pm} = \dfrac{V_{st}}{(2h \times W_f)} =$ _____ / (_____ x _____)

$= \boxed{}$ ft

Part 12: Compute Minimum Length of Upflow Anaerobic Filter (L_{uaf}):

Make the area of the gravel filter (A_{uaf}) $\geq$ the area of the sand filter (A_{sf}). Therefore, the length of the gravel filter (L_{uaf}) $\geq L_{sf}$)

L_{uaf} = [ ] ft

Part 13: Set Final Length of the Sedimentation Chamber (Headbox) (L_s):
(Minimum length = 4 feet)

$L_s = L_{pm} - L_{uaf}$ = ______ - ______

= ______ ft or 4 feet, whichever is larger

= [ ] ft

Part 14: Set Length of Clearwell (L_{cw}) for Adequate Maintenance Access (Minimum = 4 ft.) and Compute Final Inside Length (L_{ti})::

L_{cw} = [ ];

Sum of interior partition thicknesses (t_{pi}) = [ ] ft

$L_{ti} = L_{sf} + L_p + L_{cw} + t_{pi}$

= ______ + ______ + ______ + ______ = [ ] ft

Part 15: Design Effluent Pump if Required:

Since pump must be capable of handling flow when filter is new, use k = 12 feet/day = 0.5 ft/hr

$$Q = \frac{kA_f(d_f + h)}{d_f}$$

= [0.5 x ______ x (______ + ______)] /______

= [ ] ft^3/hr ; /3600 = [ ] cfs;

x 448 = [ ] gpm

Note: Pump must be configured to retain 33 cm (13 inches) of flooded gravel below the sand filter area.

Part 16: Design Structural Shell to Accommodate Soil and Load conditions at Site:

It may be economical to adjust final dimensions upward to correspond with standard precast structures or to round dimensions upward to simplify layout during construction.

REFERENCES

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APPENDIX 2-4--ULTRA-URBAN BMP (UUBMP) COMPUTATIONS WORKSHEETS

ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET E: COMPUTATIONS COMMON TO ALL BMPS

Part 1: Compute Post-Development Site Impervious Acreage (I_a):

structures = ________ ft^2
parking lot = ________ ft^2
roadway = ________ ft^2
sidewalk = ________ ft^2
other = ________ ft^2
= ________ ft^2
= ________ ft^2

Total = ________ $ft^2/43,560 = I_a$ = [ ] acres

(Note: This value may already be computed on Worksheet A)

Part 2: Compute Water Quality Volume to be Treated:

$WQV = 1816I_a$ = ________ x ________

= [ ] ft^3

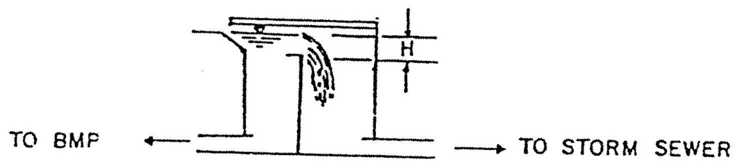
Part 3: Identify Critical Site Parameters:

Storm sewer invert at proposed connection point = ________ ft
Length of outflow line (BMP - storm sewer) = ________ ft
Minimum BMP outflow invert @ minimum 0.5% grade = ________ ft
Site Plan surface elevation at BMP location = ________ ft
Inflow invert to BMP from drainage system plan = ________ ft
Flow splitter weir or bypass pipe invert = ________ ft
(usually set at maximum BMP ponding depth)
BMP outflow possible by gravity ___; Pumped BMP Required ____

In Alexandria, overflow weirs and orifices or bypass pipes shall be designed to pass the peak flow rate of the 10-year storm (7 in./hr., 10 min. TOC) using the Rational Method (Q = CIA). The Peak Flow Rate may be selected from the following table (for a totally impervious watershed):

Area(acres)	Q(cfs)
0.25	1.75
0.33	2.31
0.50	3.50
0.75	5.25
1.00	7.00
1.25	8.75
1.50	10.50
1.75	12.25
2.00	14.00

1) When designing overflow weirs, size the weir by solving the following formula for H:(2)



$Q_{10} = 3.33LH^{1.5}$

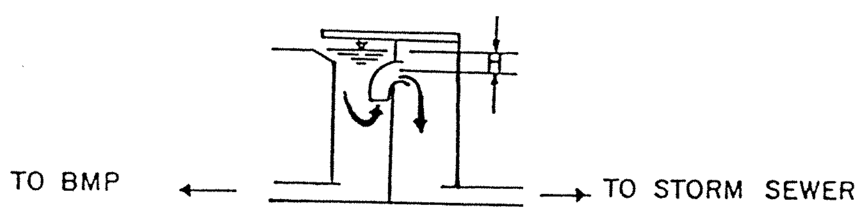
where,

Q_{10} = peak flow rate for the 10-year storm in cfs
H = the depth of ponded water above the crest of the weir in ft. (a minimum of 2" in Alexandria)
L = length of the weir in ft.

__________ = 3.33 x __________ x $H^{1.5}$

$H^{1.5}$ = __________ ; H = $\boxed{}$ ft

2) When a hooded overflow orifice is employed, use the orifice formula to size the overflow:(2)



$Q_{10} = C_dA\ (2gh_{10})^{0.5}$

where:

Q_{10} is the peak flow rate for the 10-year storm
g is the acceleration of gravity (32.2 ft/sec^2)
C_d is the coefficient of discharge (use 0.62)
A = area of the orifice in ft.^{2}
h_{10} = depth of ponded water above the flow line of the orifice

__________ = 0.62A x (64.4 x ____)$^{0.5}$

A = __________ / (0.62 x __________ = $\boxed{}$ ft^2

3) When a bypass pipe is employed, use Manning's equation to size the overflow pipe:(2)

$$V = \frac{1.49}{n} \times (R_h^{0.667}) S^{0.5}$$

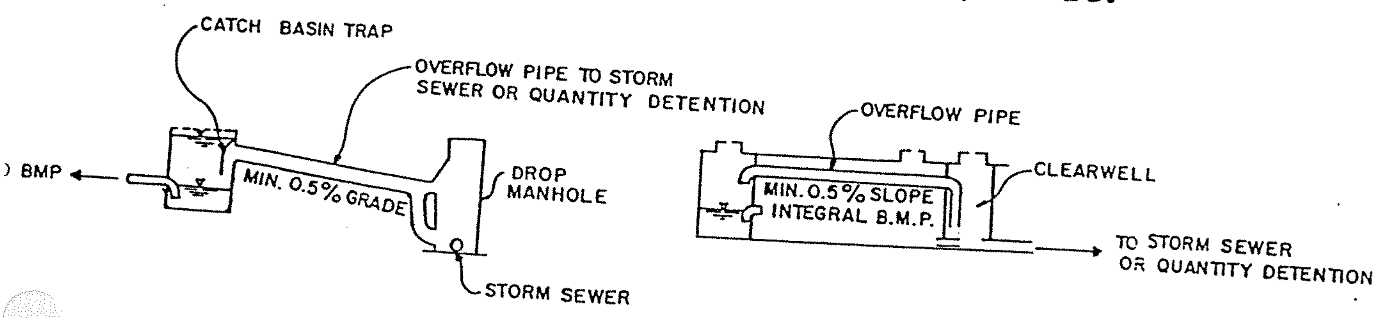
where,

V = velocity of flow in fps

n = roughness coefficient (use 0.013 for concrete, 0.015 for PVC pipe and 0.021 for corrugated metal)

S = slope of the pipe (energy gradient)(minimum 0.005)

R_h = the hydraulic radius in ft. = Area of the pipe in ft.^{2} divided by the inside circumference of the pipe (wetted perimeter) in ft.



The following table may be utilized to select bypass pipe sizes when the pipe is assumed to have a 0.5 percent grade and be flowing full:

	Q (cfs)		
Pipe Diameter	Conctrete n = 0.013	Corrugated Metal n = 0.021	PVC n =0.015
10	1.56	0.96	1.34
12	2.53	1.57	2.19
15	4.59	2.84	3.96
18	7.46	4.62	6.44
21	11.25	6.97	9.70
24	16.07	9.95	13.87

Selected bypass pipe material ______________

Selected bypass Pipe Diameter = [ ] in

WORKSHEET F: COMPUTATIONS FOR AUSTIN SAND FILTERS

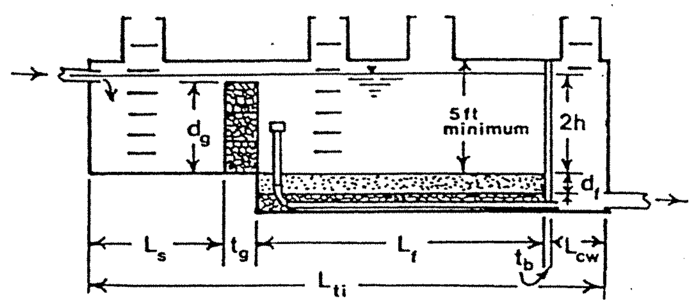
TO BE ISSUED AT A LATER DATE

TO BE ISSUED AT A LATER DATE

ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET G: COMPUTATIONS FOR ALEXANDRIA DRY VAULT SAND FILTER

Part 4: Considering data on Worksheet E, select maximum ponding depth over filter:



2h = ________ ft;

h = $\boxed{}$ ft

From WORKSHEET E;

I_a = $\boxed{}$ acres

WQV = $\boxed{}$ ft^3

Outflow by gravity possible ____

Effluent pump required ____

Part 5: Compute Minimum Area of Filter (A_{fm}):

$$A_{fm} = \frac{545 I_a d_f}{(d_f + h)}$$

= [545 x ________ x ________] / [________ + ________]

= $\boxed{}$ ft^2

Part 6: Considering Site Constraints, Select Filter Width (W_f) and Compute Filter Length (L_f) and Adjusted Filter Area (A_f):

W_f = $\boxed{}$ ft; $L_f = A_{fm} / W_f$

= ________/________

= ________, say $\boxed{}$ ft

$A_f = W_f \times L_f$ = ________ x ________

= $\boxed{}$ ft^3

Part 7: Compute the Storage Volume on Top of the Filter (V_{Tf})

$V_{Tf} = A_f \times 2h$ = ________ x ________

= $\boxed{}$ ft^3

Part 8: Compute Storage in Filter Voids (V_v):
(Assume 40% voids in filter media [sand + gravel (d_{gr})]

$V_v = A_f \times (d_f + d_{gr}) \times 0.4 =$ ________ x ________ x 0.4

$=$ [ ] ft^3

Part 9: Compute Flow Through Filter During Filling Period (V_Q):
(Assume 1-hour to fill per D.C. practice)

$V_Q = \frac{kA_f(d_f + h)}{d_f}$; use k = 2 ft/day = 0.0833 ft/hr

= [0.0833 x ________ x (________ + ________)]/________

$=$ [ ] ft^3

Part 10: Compute Storage Volume in Gabion Wall (V_g).
(Assume 40% voids in gabion stone)

$V_g = d_g \times t_g \times W_f \times 0.4 =$ ________ x ________ x ________ x 0.4

$=$ [ ] ft^3

Part 10: Compute Net Volume to be Stored Awaiting Filtration (V_{st}):

$V_{st} = WQV - V_{Tf} - V_v - V_Q - V_g$

= ________ - ________ - ________ - ________ - ________

$=$ [ ] ft^3

Part 11: Compute Minimum Length of Sediment Basin (L_s):

20% of WQV = 0.2WQV = 0.2(________) = [ ] ft^3

If $V_{st} \geq 0.2WQV$, use

$L_s = \frac{V_{st}}{(2h \times W_f)}$ = ________/(________ x ________)

$=$ [ ] ft

If $V_{st} < 0.2WQV$, use $L_s = \frac{0.2WQV}{(2h \times W_f)}$ = ______/(______ x ______)

$=$ [ ] ft

Part 12: Set Length of Clearwell (L_{cw}) for Adequate Maintenance Access (Minimum = 3 ft) and compute final inside length (L_{ti}):

$L_{cw} = \boxed{\quad}$

$L_{ti} = L_f + t_g + L_s + t_b + L_{cw}$

$= \_\_\_\_\_ + \_\_\_\_\_ + \_\_\_\_\_ + \_\_\_\_\_ + \_\_\_\_\_$

$= \boxed{\quad}$ ft

Part 13: Design Effluent Pump if Required:

Since pump must be capable of handling flow when filter is new, use k = 20 feet/day = 0.833 ft/hr

$$Q = \frac{kA_f(d_f + h)}{d_f}$$

$= [0.833 \times \_\_\_\_\_ \times (\_\_\_\_\_ + \_\_\_\_\_)]/\_\_\_\_\_$

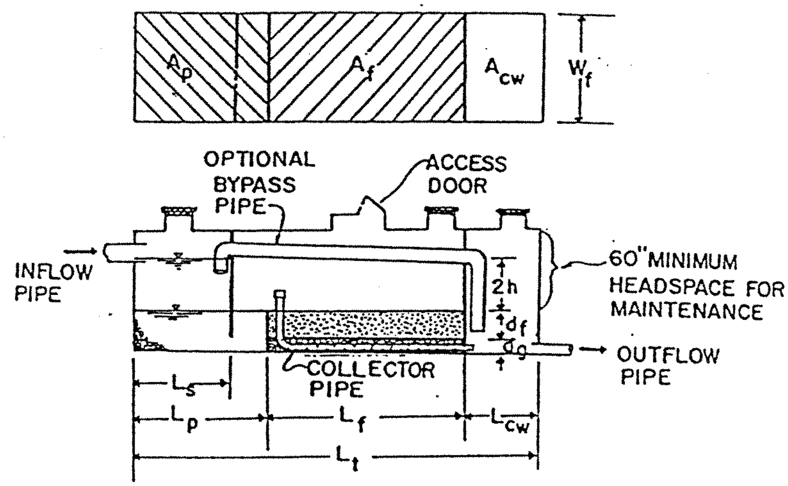
$= \boxed{\quad}$ ft^3/hr ; /3600 = $\boxed{\quad}$ cfs;

x 448.8 = $\boxed{\quad}$ gpm

Part 14: Design Structural Shell to Accommodate Soil and Load Conditions at Site:

ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET H1: COMPUTATIONS FOR D. C. SAND FILTER (ORIGINAL SINGLE POOL CONFIGURATION)



Part 4: Considering data on Worksheet E, select maximum ponding depth over filter:

2h = ________ ft;

h = [ ] ft

From WORKSHEET E;

I_a = [ ] acres

WQV = [ ] ft^3

Outflow by gravity possible _____

Effluent pump required _____

Part 5: Compute Minimum Area of Filter (A_{fm}):

$$A_{fm} = \frac{545 I_a d_f}{(d_f + h)}$$

= [545 x ________ x ________] / [________ + ________]

= [ ] ft^2

Part 6: Considering Site Constraints, Select Filter Width (W_f) and Compute Filter Length (L_f) and Adjusted Filter Area (A_f):

W_f = [ ] ft;

$L_f = A_{fm} / W_f$

= ________/________

= ________, say [ ] ft

$A_f = W_f \times L_f$ = ________ x ________

= [ ] ft^2

Part 7: Compute the Storage Volume on Top of the Filter (V_{Tf})

$$V_{Tf} = A_f \times 2h = \_\_\_\_\_\_\_\_ \times \_\_\_\_\_\_\_\_$$

$$= \boxed{}\ ft^3$$

Part 8: Compute Storage in Filter Voids (V_v):
(Assume 40% voids in filter media)

$$V_v = 0.4 \times A_f \times (d_f + d_g)$$

$$= 0.4 \times \_\_\_\_\_\_\_\_ \times (\_\_\_\_\_\_\_\_ + \_\_\_\_\_\_\_\_)$$

$$= \boxed{}\ ft^3$$

Part 9: Compute Flow Through Filter During Filling Period (V_Q):
(Assume 1-hour to fill per D.C. practice)

$$V_Q = \frac{kA_f(d_f + h)}{d_f}\ ;\ \text{use } k = 2 \text{ ft/day} = 0.0833 \text{ ft/hr}$$

$$= [0.0833 \times \_\_\_\_\_\_\_\_ \times (\_\_\_\_\_\_\_\_ + \_\_\_\_\_\_\_\_)]/\_\_\_\_\_\_\_\_$$

$$= \boxed{}\ ft^3$$

Part 10: Compute Net Volume to be Stored Awaiting Filtration (V_{st}):

$$V_{st} = WQV - V_{Tf} - V_v - V_Q$$

$$= \_\_\_\_\_\_\_\_ - \_\_\_\_\_\_\_\_ - \_\_\_\_\_\_\_\_ - \_\_\_\_\_\_\_\_$$

$$= \boxed{}\ ft^3$$

Part 11: Compute Minimum Length of Permanent Pool (L_{pm}):

$$L_{pm} = \frac{V_{st}}{(2h \times W_f)} = \_\_\_\_\_\_\_\_ / (\_\_\_\_\_\_\_\_ \times \_\_\_\_\_\_\_\_)$$

$$= \boxed{}\ ft$$

Part 12: Compute Minimum Length of Sediment Chamber (L_{sm}) (to contain at least 20% of WQV per Austin practice)

$$L_{sm} = \frac{0.2WQV}{(2h \times W_f)} = \_\_\_\_\_\_\_\_ / \_\_\_\_\_\_\_\_$$

$$= \boxed{\quad} \text{ ft}$$

Part 13: Set Final Length of Permanent Pool (L_p)

$L_{sm} + 2ft =$ ________ $+ 2 =$ $\boxed{\quad}$ ft

If $L_{pm} \geq L_{sm} + 2ft$, Make $L_p = L_{pm} =$ $\boxed{\quad}$ ft

If $L_{pm} < L_{sm} + 2ft$, make $L_p = L_{sm} + 2ft =$ $\boxed{\quad}$ ft

Part 14: Set Length of Clearwell (L_{cw}) for Adequate Maintenance Access (Minimum = 3 ft) and Compute Final Inside Length (L_{ti}):

$L_{cw} =$ $\boxed{\quad}$ ft ;

Sum of interior partition thicknesses (t_{pi}) = $\boxed{\quad}$ ft

$$L_{ti} = L_f + L_p + L_{cw} + t_{pi}$$

$$= \_\_\_\_\_\_\_\_ + \_\_\_\_\_\_\_\_ + \_\_\_\_\_\_\_\_ + \_\_\_\_\_\_\_\_$$

$$= \boxed{\quad} \text{ ft}$$

Part 15: Design Structural Shell to Accommodate Soil and Load Conditions at Site:

It may be economical to adjust final dimensions upward to correspond with standard precast structures or to round dimensions upward to simplify layout during construction.

Part 16: Design Effluent Pump if Required:

Since pump must be capable of handling flow when filter is new, use k = 20 feet/day = 0.833 ft/hr

$$Q = \frac{kA_f(d_f + h)}{d_f}$$

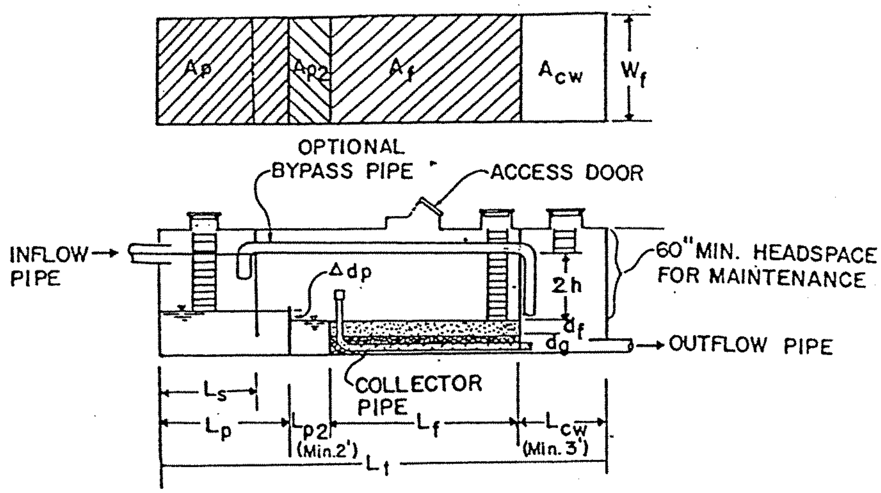
$$= [0.833 \times \_\_\_\_\_\_\_\_ \times (\_\_\_\_\_\_\_\_ + \_\_\_\_\_\_\_\_)]/\_\_\_\_\_\_\_\_$$

$= \boxed{\quad}$ ft^3/hr ; /3600 = $\boxed{\quad}$ cfs;

x 448.8 = $\boxed{\quad}$ gpm

ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET H2: COMPUTATIONS FOR D. C. SAND FILTER (THIN FILTER CONFIGURATION WITH TWO POOLS)



Part 4: Considering data on Worksheet E, select maximum ponding depth over filter:

2h = __________ ft;

h = [ ] ft

From WORKSHEET E;

I_a = [ ] acres

WQV = [ ] ft^3

Part 5: Compute Minimum Area of Filter (A_{fm}):

$$A_{fm} = \frac{545 I_a d_f}{(d_f + h)}$$

= [545 x __________ x __________] / [__________ + __________]

= [ ] ft^2

Part 6: Considering Site Constraints, Select Filter Width (W_f) and Compute Filter Length (L_f) and Adjusted Filter Area (A_f):

W_f = [ ] ft;

$L_f = A_{fm} / W_f$

= __________/__________

= __________, say [ ] ft

$A_f = W_f \times L_f$ = __________ x __________

= [ ] ft^2

Part 7: Compute the Storage Volume on Top of the Filter (V_{Tf})

$V_{Tf} = A_f \times 2h =$ ________ x ________

$=$ [ ] ft^3

Part 7A: Compute the Storage Volume on Top of the Lower Pool (V_{Tp2})

Set the lower pool length (L_{p2}) at no less than 2 feet = ________

$V_{Tp2} = L_{p2} \times W_f \times 2h =$ ________ x ________ x ________

$=$ [ ] ft^3

Part 8: Compute Storage in Filter Voids (V_v):
(Assume 40% voids in filter media)

$V_v = 0.4 \times A_f \times (d_f + d_g)$

$= 0.4 \times$ ________ x (________ + ________)

$=$ [ ] ft^3

Part 9: Compute Flow Through Filter During Filling Period (V_Q):
(Assume 1-hour to fill per D.C. practice)

$V_Q = \dfrac{kA_f(d_f + h)}{d_f}$; use $k = 2$ ft/day $= 0.0833$ ft/hr

$=$ [0.0833 x ________ x (________ + ________)]/________

$=$ [ ] ft^3

Part 10: Compute Net Volume to be Stored Awaiting Filtration (V_{st}):

$V_{st} = WQV - V_{Tf} - V_v - V_Q - V_{Tp2}$

$=$ ________ - ________ - ________ - ________ - ________

$=$ [ ] ft^3

Part 11: Compute Minimum Length of Permanent Pool (L_{pm}):

$$L_{pm} = \frac{V_{st}}{(2h - d_p) \times W_f)}$$

= ________ / (________ - ________) x ________)

= $\boxed{}$ ft

Part 12: Compute Minimum Length of Sediment Chamber (L_{sm})
(to contain at least 20% of WQV per Austin practice)

$$L_{sm} = \frac{0.2WQV}{(2h - d_p) \times W_f} = \_\_\_\_\_\_\_\_ / \_\_\_\_\_\_\_\_$$

= $\boxed{}$ ft

Part 13: Set Final Length of Permanent Pool (L_p)

L_{sm} + 2ft = ________ + 2 = ________

If $L_{pm} \geq L_{sm}$ + 2ft, Make $L_p = L_{pm}$ = $\boxed{}$ ft

If $L_{pm} < L_{sm}$ + 2ft, make $L_p = L_{sm}$ + 2ft = $\boxed{}$ ft

Part 14: Set Length of Clearwell (L_{cw}) for Adequate Maintenance Access (Minimum = 3 ft) and Compute Final Inside Length (L_{ti}):

L_{cw} = $\boxed{}$;

Sum of interior partition thicknesses (t_{pi}) = ________ ft

$L_{ti} = L_f + L_p + L_{cw} + t_{pi}$

= ________ + ________ + ________ + ________

= $\boxed{}$ ft

Part 15: Design Structural Shell to accommodate Soil and Load Conditions at Site:

It may be economical to adjust final dimensions upward to correspond with standard precast structures or to round dimensions upward to simplify layout during construction.

Part 16: Design Effluent Pump if Required:

Since pump must be capable of handling flow when filter is new, use k = 20 feet/day = 0.833 ft/hr

$$Q = \frac{kA_f(d_f + h)}{d_f}$$

$= [0.833 \times$ ________ x (________ + ________)]/__________

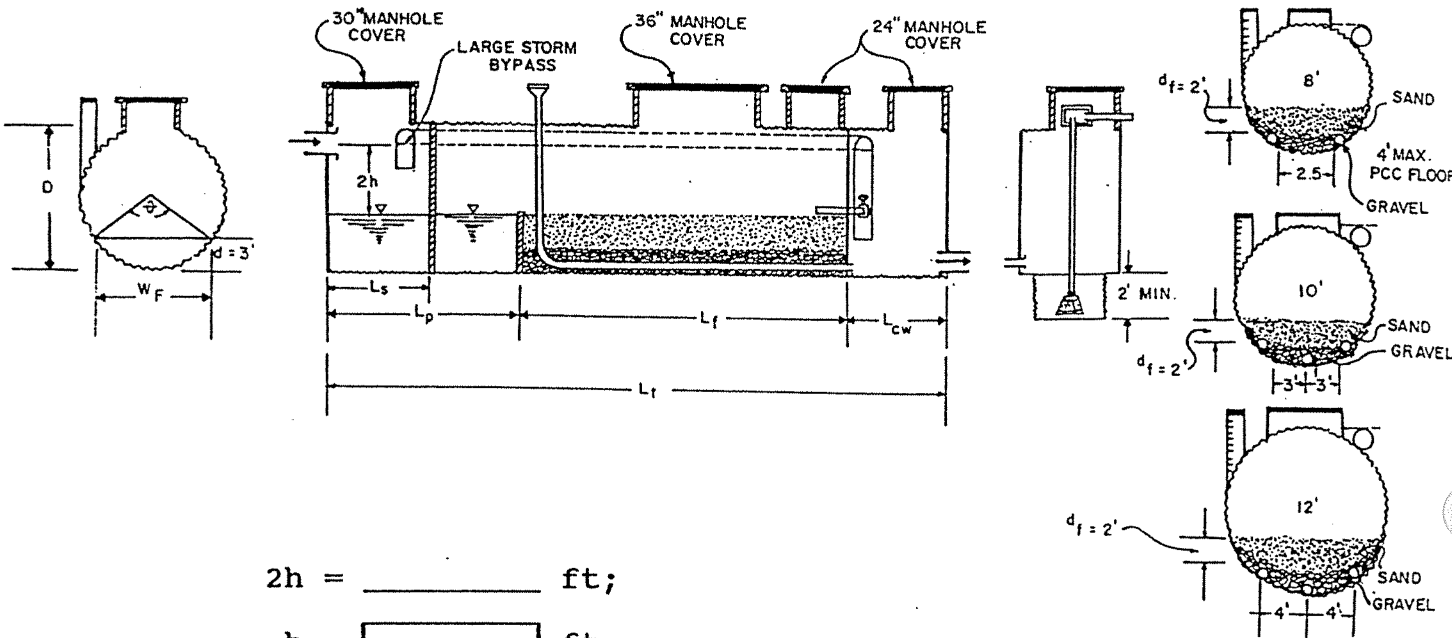
$=$ $\boxed{}$ ft^3/hr ; /3600 = $\boxed{}$ cfs;

x 448.8 = $\boxed{}$ gpm

ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET H3: COMPUTATIONS FOR D. C. SAND FILTER (CIRCULAR CROSS-SECTION SHELL CONFIGURATION)

Part 4: Considering data on Worksheet E, select maximum ponding depth over filter:



2h = ________ ft;

h = $\boxed{}$ ft

From WORKSHEET E;

I_a = $\boxed{}$ acres

WQV = $\boxed{}$ ft^3

Outflow by gravity possible _____

Effluent pump required _____

Part 5: Compute Minimum Area of Filter (A_{fm}):

$$A_{fm} = \frac{545 I_a d_f}{(d_f + h)}$$

= [545 x ________ x 2] / [________ + ________]

= $\boxed{}$ ft^2

Part 6: Considering Site Constraints, Select Pipe Diameter and Filter Width (W_f) and Compute Filter Length (L_f) and Adjusted Filter Area (A_f):

W_f = [ ] ft;

$L_f = A_{fm} / W_f$

= ________/________

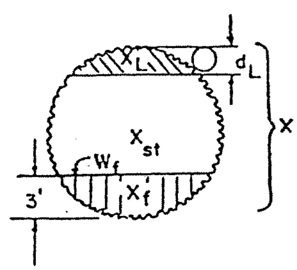
= ________, say [ ] ft

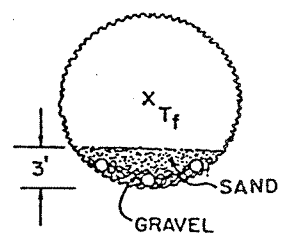
$A_f = W_f \times L_f$ = ________ x ________

= [ ] ft^2

Part 7: Compute the Storage Volume on Top of the Filter (V_{Tf})

Pipe Diam. (ft)	W_f (ft)	X (ft^2)	X_f (ft^2)	X_{Tf} (ft^2)
8	7.75	50.27	17.21	33.06
10	9.17	78.54	19.87	58.67
12	10.39	113.10	22.11	90.99





Pipe Diam. (ft)	X_L (ft^2)										
	d_L (in)										
	3	6	9	12	15	18	21	24	27	30	33
8	0.45	1.28	2.35	3.58	5.05	6.48	8.11	9.82	11.68	13.42	15.28
10	0.55	1.49	2.73	4.13	5.70	7.50	9.25	11.39	13.25	15.35	17.50
12	0.61	1.60	2.96	4.48	6.41	8.48	10.27	12.60	14.82	17.22	19.80

$X_{st} = X_{Tf} - X_L$ = ________ - ________ = [ ] ft^2

$V_{Tf} = X_{Tf} \times L_f$ = ________ x ________

= [ ] ft^3

Part 8: Compute Storage in Filter Voids (V_v):
(Assume 40% voids in filter media)

$$V_v = 0.4 \times X_f \times L_f s A_f$$

$$= 0.4 \times \_\_\_\_\_\_ \times (\_\_\_\_\_\_ + \_\_\_\_\_\_)$$

$$= \boxed{} \ ft^3$$

Part 9: Compute Flow Through Filter During Filling Period (V_Q):
(Assume 1-hour to fill per D.C. practice)

$$V_Q = \frac{kA_f(d_f + h)}{d_f} \ ; \text{ use } k = 2 \text{ ft/day} = 0.0833 \text{ ft/hr}$$

$$= [0.0833 \times \_\_\_\_\_\_ \times (2.0 + \_\_\_\_\_\_)]/\_\_\_\_\_\_$$

$$= \boxed{} \ ft^3$$

Part 10: Compute Net Volume to be Stored Awaiting Filtration (V_{st}):

$$V_{st} = WQV - V_{Tf} - V_v - V_Q$$

$$= \_\_\_\_\_\_ - \_\_\_\_\_\_ - \_\_\_\_\_\_ - \_\_\_\_\_\_$$

$$= \boxed{} \ ft^3$$

Part 11: Compute Minimum Length of Permanent Pool (L_{pm}):

$$L_{pm} = \frac{V_{st}}{X_{st}} = \_\_\_\_\_\_ / \_\_\_\_\_\_$$

$$= \boxed{} \ ft$$

Part 12: Compute Minimum Length of Sediment Chamber (L_{sm})
(to contain at least 20% of WQV per Austin practice)

$$L_{sm} = \frac{0.2WQV}{X_{st}} = 0.2 \times \_\_\_\_\_\_ / \_\_\_\_\_\_$$

$$= \boxed{} \ ft$$

Part 13: Set Final Length of Permanent Pool (L_p)

L_{sm} + 2ft = __________ + 2 = $\boxed{}$ ft

If $L_{pm} \geq L_{sm}$ + 2ft, Make $L_p = L_{pm}$ = $\boxed{}$ ft

If $L_{pm} < L_{sm}$ + 2ft, make $L_p = L_{sm}$ + 2ft = $\boxed{}$ ft

Part 14: Set Length of Clearwell (L_{cw}) for Adequate Maintenance Access (Minimum = 3 ft) and Compute Final Inside Length (L_{ti}):

L_{cw} = $\boxed{}$;

Sum of interior partition thicknesses (t_{pi}) = $\boxed{}$ ft

$L_{ti} = L_f + L_p + L_{cw} + t_{pi}$

= __________ + __________ + __________ + __________

= $\boxed{}$ ft

Part 15: Design Structural Shell to Accommodate Soil and Load Conditions at Site:

Part 16: Design Effluent Pump if Required:

Since pump must be capable of handling flow when filter is new, use k = 20 feet/day = 0.833 ft/hr

$$Q = \frac{kA_f(d_f + h)}{d_f}$$

= [0.833 x __________ x (__________ + __________)]/__________

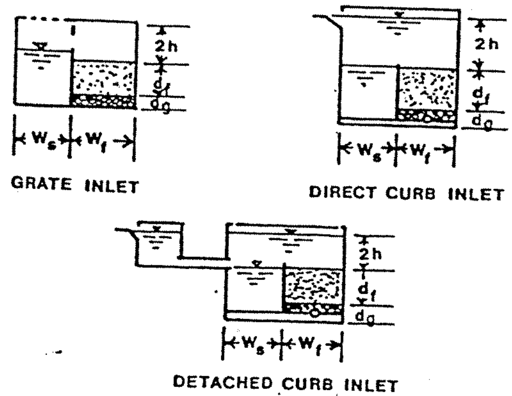
= $\boxed{}$ ft^3/hr ; /3600 = $\boxed{}$ cfs;

x 448.8 = $\boxed{}$ gpm

ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET I1: COMPUTATIONS FOR STANDARD DELAWARE SAND FILTER

Part 4: Considering data on Worksheet E, select maximum ponding depth over filter:



2h = ________ ft;

h = $\boxed{}$ ft

From WORKSHEET E;

I_a = $\boxed{}$ acres

WQV = $\boxed{}$ ft^3

Outflow by gravity possible____; Effluent pump required ____

Part 5: Compute Minimum Area of Filter (A_{fm}) and Sediment Pool (A_{sm}):

a) If 2h ≥ 2.67 feet, use the formula:

$$A_{sm} = A_{fm} = \frac{545 I_a d_f}{(d_f + h)}$$

= [545 x ________ x ________] / [________ + ________]

= $\boxed{}$ ft^2

b) If 2h < 2.67 feet, use the formula:

$$A_{sm} = A_{fm} = \frac{1816\ I_a}{(4.1h + 0.9)} = \frac{WQV}{(4.1h + 0.9)}$$

= ________ / [(4.1 x ________) + 0.9]

= $\boxed{}$ ft^2

Part 6: Considering Site Constraints, Select Filter Width (W_f) and Sediment Pool Width (W_s) and Compute Filter Length (L_f) and Adjusted Filter Area (A_f) and Sediment Chamber Area (A_s):

$W_s = W_f =$ ________ ft;

$L_s = L_f = A_{fm}/\ W_f$

$=$ ________/________

$=$ ________, say $\boxed{}$ ft

$A_s = A_f = W_f \times L_f =$ ________ x ________

$= \boxed{}\ ft^2$

Part 13: Design Structural Shell to accmmodate Soil and Load Conditions at Site:

Part 14: Design Effluent Pump if Required:

Since pump must be capable of handling flow when filter is new, use k = 20 feet/day = 0.833 ft/hr

$$Q = \frac{kA_f(d_f + h)}{d_f}$$

$= [0.833 \times$ ________ x (________ + ________)]/________

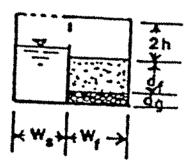
$= \boxed{}\ ft^3/hr$; /3600 = $\boxed{}$ cfs;

x 448.8 = $\boxed{}$ gpm

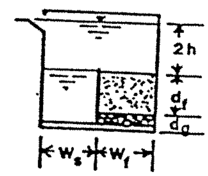
ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET I2: COMPUTATIONS FOR DELAWARE SAND FILTER WITH EXTERNAL STORAGE FOR PART OF WQV

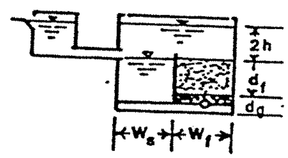
Part 4: Considering data on Worksheet E, select maximum ponding depth over filter:



GRATE INLET



DIRECT CURB INLET



DETACHED CURB INLET

2h = ____________ ft;

h = $\boxed{\qquad\qquad}$ ft

From WORKSHEET E;

I_a = $\boxed{\qquad\qquad}$ acres

WQV = $\boxed{\qquad\qquad}$ ft^3

Outflow by gravity possible _____; Effluent pump required _____

Part 5: Compute Minimum Area of Filter (A_{fm}) and Sediment Pool (A_{sm}):

$$A_{sm} = A_{fm} = \frac{545 I_a d_f}{(d_f + h)}$$

= [545 x ________ x ________] / [________ + ________]

= $\boxed{\qquad\qquad}$ ft^2

Part 6: Considering Site Constraints, Select Filter Width (W_f) and Sediment Pool Width (W_s) and Compute Filter Length (L_f) and Adjusted Filter Area (A_f) and Sediment Chamber Area (A_s):

$W_s = W_f$ = $\boxed{\qquad\qquad}$ ft;

$L_s = L_f = A_{fm} / W_f$

= ____________/____________

= ____________, say $\boxed{\qquad\qquad}$ ft

$A_s = A_f = W_f \times L_f$ = ____________ x ____________

= $\boxed{\qquad\qquad}$ ft^2

Part 7: Compute Storage in Filter Voids (V_v):
(Assume 40% voids in filter media)

$V_v = 0.4 \times A_f \times (d_f + D_G) = 0.4 \times$ ________ x ________

$= \boxed{\quad\quad} \ ft^3$

Part 8: Compute Flow Through Filter During Filling Period (V_Q):
(Assume 1-hour to fill per D.C. practice)

$V_Q = \frac{kA_f(d_f + h)}{d_f}$; use k = 2 ft/day = 0.0833 ft/hr

= [0.0833 x ________ x (________ + ________)]/________

$= \boxed{\quad\quad} \ ft^3$

Part 9: Compute Net Volume to be Stored Awaiting Filtration (V_{st}):

$V_{st} = WQV - V_v - V_Q =$ ________ - ________ - ________

$= \boxed{\quad\quad} \ ft^3$

Part 10: Compute Storage Above Filter and Sediment Pool (V_{fs}):

$V_{fs} = 2h(A_f + A_s) =$ ________(________ + ________)

$= \boxed{\quad\quad}$ ft

Part 11: Compute Storage Deficit (V_D):

$V_D = V_{st} - V_{fs} =$ ________ - ________

$= \boxed{\quad\quad} \ ft^3$

IF $V_D \leq 0$, SKIP TO PART 13. IF $V_D \geq 0$, ADJUST DESIGN TO PROVIDE ADDITIONAL STORAGE.

Part 12: Select and Compute Design Adjustment:

____ a. Increase Maximum Ponding Depth ($2h_i$):

$2h_i = V_s/A_f =$ ________/________ $= \boxed{\quad\quad}$ ft

____ b. Increase System Length (L_i)

(1) Compute storage per lineal foot above filter and sediment pool ($V_{fs/ft}$):

$V_{fs/ft} = V_{fs}/L_f =$ ________ /________

$= \boxed{}$ ft^3/ft

(2) Compute increased system length to eliminate storage deficit (L_i):

$L_i = V_{st}/V_{fs/ft} =$ ________ /________

$= \boxed{}$ ft

____c. Provide additional storage outside of filter shell (provide description and calculations):

____d. Other (provide description and calculations):

<u>Part 13: Design Structural Shell to accmmodate Soil and Load Conditions at Site</u>:

<u>Part 14: Design Effluent Pump if Required</u>:

Since pump must be capable of handling flow when filter is new, use k = 20 feet/day = 0.833 ft/hr

$$Q = \frac{kA_f(d_f + h)}{d_f}$$

= [0.833 x ________ x (________ + ________)]/________

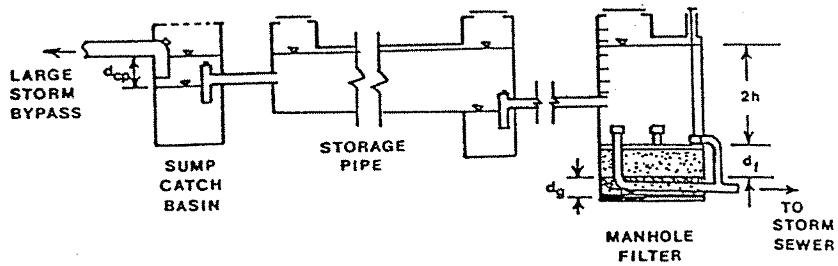
$= \boxed{}$ ft^3/hr ; /3600 = $\boxed{}$ cfs;

x 448.8 = $\boxed{}$ gpm

ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET J: COMPUTATIONS FOR D. C. MANHOLE SAND FILTER

Part 4: Considering data on Worksheet E, select maximum ponding depth over filter:



2h = ________ ft;

h = [ ] ft

From WORKSHEET E;

I_a = [ ] acres

WQV = [ ] ft^3

Outflow by gravity possible ______

Effluent pump required _____

Part 5: Compute Minimum Area of Filter (A_{fm}):
(also see table on page 3 for alternative procedure)

$$A_{fm} = \frac{545 I_a d_f}{(d_f + h)}$$

= [545 x ________ x ________] / [________ + ________]

= [ ] ft^2

Part 6: Select Manhole Size from the Following Table:

Inside Manhole Diameter (ft)	Filter Area (ft^2)
5	19.6
6	28.3
7	38.5
8	50.3

Manhole selected: [ ] ft

A_f = [ ] ft^2

Part 7: Compute the Storage Volume on Top of the Filter (V_{Tf})

$V_{Tf} = A_f \times 2h =$ ________ x ________

$= \boxed{}$ ft^3

Part 8: Compute Storage in Filter Voids (V_v):
(Assume 40% voids in filter media)

$V_v = 0.4 \times A_f \times (d_f + d_g)$

$= 0.4 \times$ ________ x (________ + ________)

$= \boxed{}$ ft^3

Part 9: Compute Flow Through Filter During Filling Period (V_Q):
(Assume 1-hour to fill per D.C. practice)

$V_Q = \dfrac{kA_f(d_f + h)}{d_f}$; use k = 2 ft/day = 0.0833 ft/hr

= [0.0833 x ________ x (________ + ________)]/________

$= \boxed{}$ ft^3

Part 10: Compute Storage in Sump Catchbasin (V_{cb}):

Cross-sectional area of catchbasin (A_{cb}) = ________

Ponding depth over catchbasin pool (d_{cp}) = ________

$V_{cb} = A_{cb} \times d_{cp} =$ ________ x ________ $= \boxed{}$ ft^3

Part 11: Compute Net Volume to be Stored Awaiting Filtration (V_{st}):

$V_{st} = WQV - V_{Tf} - V_v - V_Q - V_{cb}$

= ________ - ________ - ________ - ________ - ________

$= \boxed{}$ ft^3

Part 11: Size Outside Storage to Contain V_{st}:"

Part 12: Design Effluent Pump if Required:

Since pump must be capable of handling flow when filter is new, use k = 20 feet/day = 0.833 ft/hr

$$Q = \frac{kA_f(d_f + h)}{d_f}$$

= [0.833 x ________ x (________ + ________)]/________

= [   ] ft^3/hr ; /3600 = [   ] cfs;

x 448.8 = [   ] gpm

Maximum Ponding Depth (2h-ft)	Manhole Diameter 6 ft	7 ft	8 ft
3	0.10	0.14	0.18
3.5	0.11	0.15	0.20
4	0.12	0.16	0.22
4.5	0.13	0.18	0.23
5	0.14	0.19	0.25
5.5	0.15	0.20	0.26
6	0.16	0.21	0.28
6.5	0.16	0.22	0.29
7	0.17	0.24	0.31
7.5	0.18	0.25	0.32
8	0.19	0.26	0.34

TABLE 2-A4-1 -- MAXIMUM IMPERVIOUS AREA (I_a) TREATED BY D.C. MANHOLE FILTERS WITH VARYING DEPTHS OF PONDING ABOVE THE FILTER (WHEN d_f = 1.5 FT)

ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET H4: COMPUTATIONS FOR "SWITCH-BACK" SAND FILTER

Part 4: Considering data on Worksheet E, select maximum ponding depth over filter:

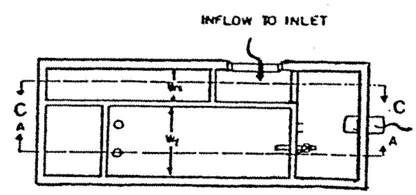
$2h$ = ____________ ft;

h = [   ] ft

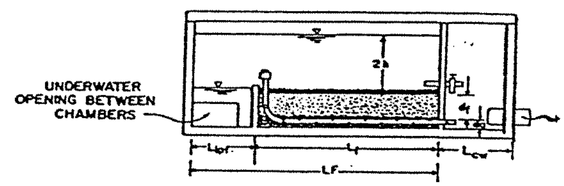
From WORKSHEET E;

I_a = [   ] acres

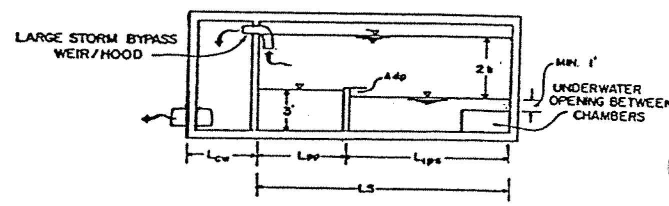
WQV = [   ] ft^3



TOP VIEW



SECTION A-A



SECTION C-C

Part 5: Compute Minimum Area of Filter (A_{fm}):

$$A_{fm} = \frac{545 I_a d_f}{(d_f + h)}$$

= [545 x ____________ x ____________] / [____________ + ____________]

= [   ] ft^2

Part 6: Considering Site Constraints, Select Filter Width (W_f) and Compute Filter Length (L_f) and Adjusted Filter Area (A_f):

W_f = [   ] ft;

$L_f = A_{fm} / W_f$

= ____________/____________

= ____________, say [   ] ft

$A_f = W_f \times L_f$ = ____________ x ____________

= [   ] ft^2

Part 7: Compute the Storage Volume on Top of the Filter (V_{Tf})

$V_{Tf} = A_f \times 2h =$ ________ x ________

$= \boxed{}$ ft^3

Part 8: Compute Storage in Filter Voids (V_v):
(Assume 40% voids in filter media)

$V_v = 0.4 \times A_f \times (d_f + d_g)$

$= 0.4 \times$ ________ x (________ + ________)

$= \boxed{}$ ft^3

Part 9: Compute Flow Through Filter During Filling Period (V_Q):
(Assume 1-hour to fill per D.C. practice)

$V_Q = \dfrac{kA_f(d_f + h)}{d_f}$; use k = 2 ft/day = 0.0833 ft/hr

= [0.0833 x ________ x (________ + ________)]/________

$= \boxed{}$ ft^3

Part 10: Select Sediment Chamber Width (W_s) and Compute the Storage Volume on Top of the 3-ft Deep Plunge Pool (V_{Tpp})

Set plunge pool length (L_{pp}) at $\geq$ 4 ft = ________ ft

$V_{Tpp} = L_{pp} \times W_s \times (2h - \Delta d_p)$

= ________ x ________ x (________ - ________)

$= \boxed{}$ ft^3

Part 11: Compute Net Volume to be Stored Awaiting Filtration (V_{st}):

$V_{st} = WQV - V_{Tf} - V_v - V_Q - V_{Tpp}$

= ________ - ________ - ________ - ________ - ________

$= \boxed{}$ ft^3

Part 12: Compute Minimum Length of Lower Pool (L_{lp}):

$$L_{lp} = \frac{V_{st}}{(2h \times W_s)}$$

$= \_\_\_\_\_\_ / (\_\_\_\_\_\_ - \_\_\_\_\_\_) \times \_\_\_\_\_\_)$

$= \boxed{}$ ft

Split Lower Pool between sedimentation and filter chambers to make chamber lengths equal (minimum L_{lpf} = 2 ft).

$L_{lp} = L_{lps} + L_{lpf}$

$L_{lps} = \boxed{}$ ft; $L_{lpf} = \boxed{}$ ft

Part 13: Check to Assure That Sediment Chamber Contains at Least 20% of WQV per Austin Practice)

$0.2WQV = 0.2 \times \_\_\_\_\_\_ = \boxed{}$ ft^3

$V_{sc} = V_{Tpp} + (2h \times L_{lps} \times W_s)$

$= \_\_\_\_\_\_ + (\_\_\_\_\_\_ \times \_\_\_\_\_\_ \times \_\_\_\_\_\_$

$= \boxed{}$ ft^3

Part 14: Set Final Length of Sediment Chamber and Filter Chamber

If $V_{sc} \geq$ WQV, $L_s = L_{lps} + L_{pp} = \_\_\_\_\_\_ + \_\_\_\_\_\_$

$= \boxed{}$ ft

If V_{sc} < WQV, increase L_{lps} until V_{sc} = WQV

New $L_{lps} = \dfrac{WQV - V_{Tpp}}{2h \times W_s}$

$= [\_\_\_\_\_\_ - \_\_\_\_\_\_]/[\_\_\_\_\_\_ \times \_\_\_\_\_\_]$

$= \boxed{}$ ft

New $L_s = \_\_\_\_\_\_ + \_\_\_\_\_\_ = \boxed{}$ ft

Make $L_F = L_s$ by increasing L_{lpf}: New $L_{lpf} = \boxed{}$ ft

Part 15: Set Length of Clearwell (L_{cw}) for Adequate Maintenance Access (Minimum = 2.5 ft) and Compute Final Inside Length (L_{ti}) and Final Inside Width (W_t):

L_{cw} = $\boxed{}$ ft;

Sum of cross full partition thicknesses (t_{pi}) = ________ ft

$L_{ti} = L_f + L_{lpf} + L_{cw} + t_{pi}$

= ________ + ________ + ________ + ________

= $\boxed{}$ ft

Sum of lengthwise full partition thicknesses (W_{pi}) = ________ ft

$W_{ti} = W_s + W_f + W_{pi}$

= ________ + ________ + ________

= $\boxed{}$ ft

Part 16: Design Structural Shell to accommodate Soil and Load Conditions at Site:

It may be economical to adjust final dimensions upward to correspond with standard precast structures or to round dimensions upward to simplify layout during construction.

Part 17: Design Effluent Pump if Required:

Since pump must be capable of handling flow when filter is new, use k = 20 feet/day = 0.833 ft/hr

$$Q = \frac{kA_f(d_f + h)}{d_f}$$

= [0.833 x ________ x (________ + ________)]/________

= $\boxed{}$ ft^3/hr ; /3600 = $\boxed{}$ cfs;

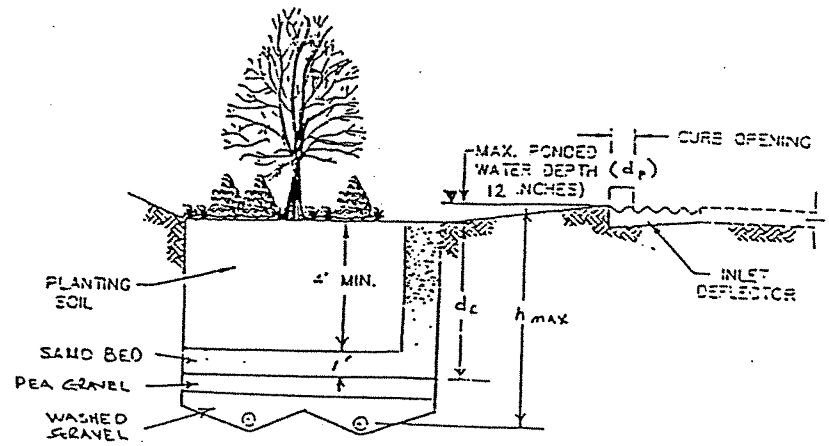
x 448.8 = $\boxed{}$ gpm

ALEXANDRIA, VIRGINIA
ULTRA-URBAN BMP COMPUTATIONS

WORKSHEET L1 : COMPUTATIONS FOR BIORETENTION FILTER

4: Considering data on Worksheet E, select maximum ponding depth over filter:

d_p = ______ ft;



From WORKSHEET E;

I_a = [ ] acres

WQV = [ ] ft^3

Outflow by gravity possible ___

Effluent pump required ___

Part 5: Compute Unit Storage Over Bioretention Filter (V_{TF}):

$V_{TF} = d_p \times 1$

= [ ] ft^3/ft^2

Part 6: Compute Unit Storage in Bioretention Filter Voids (V_{vu}):

(Assumes 20% voids in filter media)

$V_{vu} = 0.2\ d_f = 0.2 \times$ ________

= [ ] ft^3/ft^2

Part 7: Compute Unit Storage in Filter Gravel Voids (V_{gu}):

$V_{gu} = 0.4 \times d_g = 0.4 \times$ ________ = [ ] ft^3/ft^2

Part 8: Compute Unit Flow Through Filter During Filling Period (V_{QU}): (Assume 1-hour to fill per D.C. practice and unit hydraulic gradient)

$V_{QU} = k = 0.083\ ft^3/ft^2$

Part 9: Compute Total Basin Unit Storage (V_{TBU}):

$$V_{TBU} = V_{TF} + V_{vu} + V_{QU} + V_{gu}$$

$$= \_\_\_\_\_\_ + \_\_\_\_\_\_ + \_\_\_\_\_\_ + \_\_\_\_\_\_$$

$$= \boxed{} \ ft^3/ft^2$$

Part 10: Compute Minimum Area of Storage Basin (A_B):

$$A_{MB} = WQV / V_{TBU} = \_\_\_\_\_\_ / \_\_\_\_\_\_$$

$$= \boxed{} \ ft^2$$

Note: Minimum dimensions are 15' x 40' without special approval by the Director of T&ES

Part 11: Compute Peak Flow Rate for Orifice (s) for 24-Hour Drawdown:

$$O_p = \frac{WQV}{(0.5 \times 3600 \times 24)} = 0.000023 \times \_\_\_\_\_\_ = \boxed{} \ cfs$$

Part 12: Compute Outflow Manifold Pipe Orifice Area to Provide 24-Hour Drawdown:

$$A_o = \frac{Q_p}{0.6\ (64.4 \times h_{max})^{0.5}} = \_\_\_\_\_\_ / 0.6 \times (64.4 \times \_\_\_\_\_\_)^{0.5}$$

$$= \boxed{} \ ft^2$$

Part 13: Compute Diameter of Required Orifice (s) (D_o):

$$D_o = 2.0 \times (A_o / 3.1416)^{0.5} = 2.0 \times (\_\_\_\_\_\_ / 3.1416)^{0.5}$$

$$= \boxed{} \ ft$$

$$\times 12 = \boxed{} \ in$$

Note: At least one Bioretention Filter per development project must be equipped with a monitoring manhole downstream of the Clearwell (Manifold Orifice Chamber) and upstream of the large storm overflow pipe. See Appendix 2-8 of the *Alexandria Supplement to the Northern Virginis BMP Handbook.*

III-4.4.4 BMP RD.10 Presettling Basin

Purpose and Definition

A Presettling Basin provides pretreatment of runoff in order to remove suspended solids which can impact other primary treatment BMPs. A presettling basin has no "permanent pool" volume; runoff is detained so that particulates can settle out before being discharged to a another BMP. Runoff treated by a Presettling Basin *must* be further treated by a water quality infiltration or filtration BMP, a wet pond-type BMP, or a biofilter. Presettling basins may need to be located "off-line" from the primary conveyance/detention system if used to protect infiltration or filtration BMPs from siltation.

Presettling Basins are not to be used to provide streambank erosion control. If pretreatment and streambank erosion control are to be combined into one structure, see BMP RD.11, Extended Detention Dry Pond (note, however, that such a facility may have limited application in the Puget Sound Basin).

Figure III-4.12 illustrates a presettling basin.

Planning Considerations

One of the major concerns with infiltration and filtration facilities is their tendency to clog with sediment. To minimize this, all runoff entering infiltration or filtration facilities is required to be pretreated to remove the majority of particulate material. Presettling basins can be used when there is no requirement to provide streambank erosion control.

In some cases there may be greater concern than usual about sediments entering an infiltration or filtration facility (e.g. highly erodible soils). In these instances a combination of a presettling basin with a biofilter or vegetative filter strip is recommended (see BMP RB.05 and BMP RB.06).

Sediment and Debris

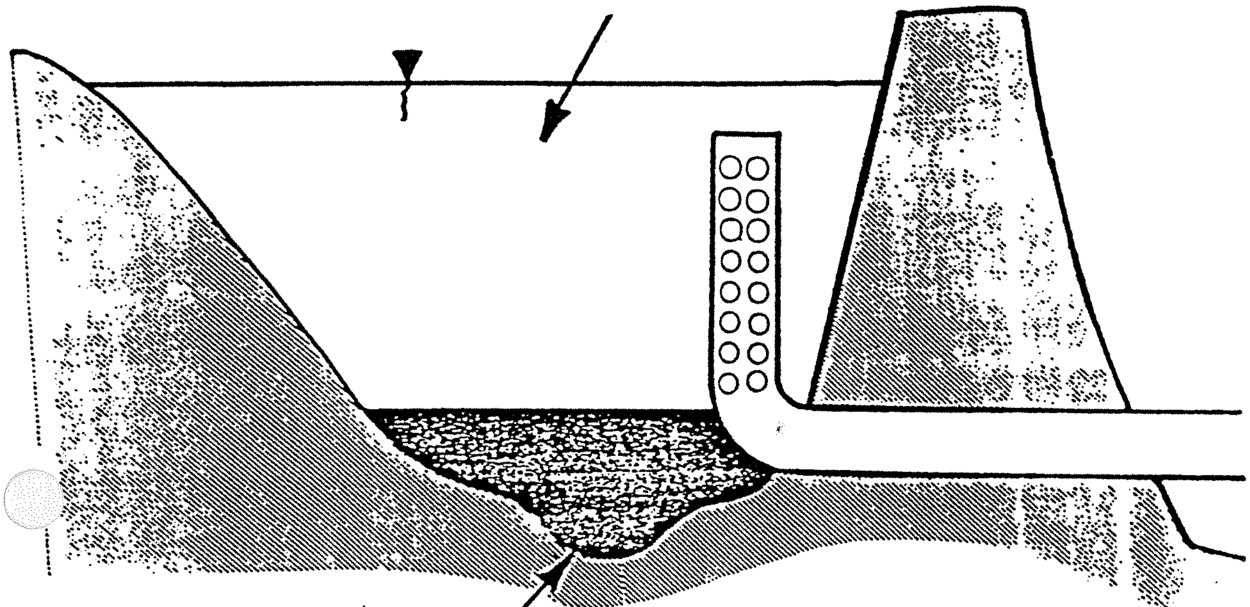
More often than not, ponds serve primarily as sedimentation basins during construction when erosion rates are particularly high. In and of itself, this situation does not present a problem. Unfortunately, these facilities are often installed without the benefit of the designer having evaluated the capacity of either the initial or the final (post-construction) design configuration to perform this type of function.

If a facility is to be used as the principal means to avoid having excessive levels of turbidity discharged from the site during construction; the engineer should evaluate the pond geometry in conjunction with the rate of outflow and grain size distribution of the soils and design the temporary sediment basin according to BMPs E3.35 or E3.40 in Volume II.

Heavy Metal Contamination

Studies have shown high accumulation rates of lead, zinc, and copper on and near heavily traveled highways and streets. Runoff from highways and streets can be expected to carry significant concentrations of these heavy metals. If a significant portion of the drainage area into a pond consists of highways, streets, or parking areas or other known sources of heavy metal contamination, there is a potential environmental health hazard. In such cases the multiple use functions of the pond should be limited and accessibility should be restricted. Additionally, liners should be provided for ponds expected to accept these types of pollutants, for certain soil types, according to Section III-4.3.2.

Figure III-4.12
BMP RD.10 Presettling Basin



Note: No streambank erosion control is provided by this BMP.

This may require that sediment dredged out of the basins during maintenance cleaning be treated as a Dangerous Waste. Investigations of sediments removed from detention ponds to date have found that many pollutants are tightly bound with only a slight possibility of leaching. To be safe, sediments to be removed should be analyzed and elutriate tests performed to verify that the sediment can be safely disposed of by conventional methods (see Volume IV, Catchbasin Sediment Disposal Policy (to be written) which deals with disposal procedures).

Site Constraints and Setbacks

Site constraints are any manmade restrictions such as property lines, easements, structures, etc. that impose constraints on development. Constraints may also be imposed from natural features such as requirements of the local government's Sensitive Areas Ordinance and Rules (if adopted). These should also be reviewed for specific application to the proposed development.

All facilities shall be a minimum of 20 feet from any structure, property line, and any vegetative buffer required by the local government, and 100 feet from any septic tank/drainfield (except wet vaults shall be a minimum of 20 feet).

All facilities shall be a minimum of 50 feet from any steep (greater than 15%) slope. A geotechnical report must address the potential impact of a wet pond on a steep slope.

Dam Safety

In urban or urbanizing areas, failure of an impoundment structure can cause significant property damage and even loss of life. Such structures should be designed only by professional engineers registered in the State of Washington who are qualified and experienced in impoundment design. Wherever they exist, local safety standards for impoundment design shall be followed. Where no such criteria exist, widely recognized design criteria such as those used by the USDA Soil Conservation Service, Ecology Dam Safety Standards, or U.S. Army Corps of Engineers are recommended.

Safety, Signage and Fencing

Ponds which are readily accessible to populated areas should incorporate all possible safety precautions. Steep side slopes (steeper than 3H:1V) at the perimeter shall be avoided and dangerous outlet facilities shall be protected by enclosure. Warning signs for deep water and potential health risks shall be used wherever appropriate. Signs should be placed so that at least one is clearly visible and legible from all adjacent streets, sidewalks or paths. A notice should be posted warning residents of potential waterborne disease that may be associated with body contact recreation such as swimming in these facilities.

If the pond surface exceeds 20,000 sq. feet, include a safety bench around the basin with a width of 5 feet, and with a depth not exceeding 1 foot during non-storm periods. Emergent vegetation such as cattails should be placed on the bench to inhibit entry by unauthorized people.

A fence is required at the maximum water surface elevation, or higher, when a pond slope is a wall. Local governments and Homeowners Associations may also require appropriate fencing as an additional safety requirement in any event.

Design Criteria

The hydrologic analysis methods in Chapter III-1 shall be used for design purposes.

Sizing Presettling Basins

Presettling basins are to be designed to settle out particulate pollutants for a range of runoff events, up to the 6-month, 24-hour design storm. The smaller storms (i.e., less than the 6-month, 24-hour storm) also need to be controlled because these frequently occurring events carry the majority of the annual pollution. Schueler (13) recommends that a maximum detention time for the maximum detention volume be 40 hours. Ecology recommends that 24 hours be used due to the Pacific Northwest rainfall pattern, with the exception of the case when presettling basins are used in tandem with water quality infiltration BMPs. For that case, the *total* detention time for *both* the presettling basin and the infiltration BMP should be 24 hours in order to maintain aerobic conditions in the infiltration BMP. Schueler also recommends that smaller events (0.1-0.2 inches) be detained no less than six hours. These are general recommendations but can be deviated from if the designer uses an appropriate equation to size the presettling basin. The Camp-Hazen equation (8) is recommended as it takes into account effects of turbulent flow, which is a typical condition during runoff events:

$E = 1 - \exp(-wA_s/Q_o)$ where

E	=	trap efficiency = fraction of suspended solids to remove; set equal to 0.8 (= 80% removal efficiency);
w	=	settling velocity of target particle; silt is recommended using a settling velocity of 0.0004 ft/sec.
A_s	=	surface area of presettling basin
Q_o	=	average release rate from the presettling basin.

The choice of a minimum 80 percent removal for suspended solids as the criterion for selecting the surface area is considered reasonable and cost-effective. However, protection of beneficial uses in receiving waters will always be required. There may be instances, depending on the nature of pollutants to be controlled and the receiving waters, when a higher removal rate, and hence larger surface area, will be required by the local government and/or Ecology or other State agencies.

Rearranging the Camp-Hazen equation and solving for A_s gives:

$A_s = -(Q_o/w) * Ln(1-E)$ where Ln is the natural logarithm

The average release rate, Q_o, can be calculated by dividing the runoff treatment volume (maximum = runoff from 6-month, 24-hour storm) by the detention time, t_d:

$$Q_o = \frac{V}{t_d}$$

The detention time will vary depending on the amount of runoff but should not exceed 24 hours for the 6-month, 24-hour storm. Longer detention times are not recommended because of the frequency of rainfall in the Northwest during the winter wet season (on the average it rains every two days from October to late March). The Camp-Hazen equation can be solved to determine the ratio of the presettling basin surface area to the total drainage area. Table III-4.5 presents the results of such an analysis and can be used for planning purposes. See Table III-4.3 for typical surface area-to-drainage area ratios for this and other detention BMPs for the maximum treatment storm (i.e., 6-month, 24-hour event).

Note that while Table III-4.5 gives recommended surface area-to-drainage area ratios it will still be necessary for the designer to size the outlet(s) for the presettling basin such that the drawdown times in Table III-4.5 are achieved for the runoff volumes shown. In some cases the minimum orifice size (0.5 inch diameter) may make it impossible to achieve the drawdown times presented. In such cases, the drawdown time can be decreased, which will increase the outflow rate and the size of the orifice, along with the surface area of the basin.

Table III-4.5
Presettling Basin Design Criteria to Treat a Range of Runoff Events*

Runoff Volume (inches)	Design Detention Time t_d (hours)	Ratio of Basin Surface Area to Drainage Area (A_s/A_d)
0.20	6	0.31%
0.50	12	0.39%
1.00	18	0.52%

* If the 6-month, 24-hour design storm runoff volume is less than the values in the first column of the table, a design detention time of 24 hours should be used.

Pond Configuration and Geometry

The shape of the presettling 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 3:1 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 by-passed by the flow regime and are, therefore, ineffective in the settling process) should be minimized. Baffles may be used to mitigate short-circuiting and/or dead storage problems. Figure III-4.13 illustrates basin geometry considerations, including the use of baffles to improve basin performance.

Interior side slopes up to the maximum water surface shall be no steeper than 3H:1V. Exterior side slopes shall be no steeper than 2H:1V.

The basin bottom shall be level to facilitate sedimentation.

Basin walls may be retaining walls, provided that the design is prepared and stamped by a structural engineer registered in the State of Washington, that they are constructed of reinforced concrete per Section III-4.6.1, that a fence is provided along the top of the wall, and that at least 25 percent of the pond perimeter will be a vegetated soil slope of not greater than 3H:1V.

Permanent 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. Sediment traps may reduce maintenance requirements by reducing the frequency of sediment removal. It is recommended that the sediment trap volume be equal to ten (10) percent of the sedimentation basin volume. Water collected in the sediment trap shall be conveyed from the basin in order to prevent standing water conditions from occurring. Water collected in the sediment trap shall drain out within 60 hours. Access for cleaning the sediment trap drain system is necessary. Figure III-4.14 illustrates a permanent sediment trap.

Inlet Structure and Isolation/Diversion Structure

The inlet structure design must be adequate for isolating the water quality volume (i.e., runoff volume from the 6-month, 24-hour storm) from the larger design storms and to convey the peak flows for the larger design storms past the basin. The water quality volume should be discharged uniformly and at low velocity into the presettling basin in order to maintain near quiescent conditions which are necessary for effective treatment. It is desirable for the heavier suspended material to drop out near the front of the basin; thus a drop inlet structure is recommended in order to facilitate sediment removal and maintenance. Energy dissipation devices may be necessary in order to reduce inlet velocities which exceed three (3) feet per second.

Note: On very small lots (approximately 1 acre) this design may result in an outlet orifice smaller than the minimum allowed (one-half inch). In this case, some of the design variables in the Camp-Hazen equation can be revised in order to increase orifice size (e.g., reduce detention time, increase treatment volume, increase trap efficiency (E)).

Off-line Isolation/Diversion Structure

Presettling basins may need to be located off-line when used to protect infiltration and filtration BMPs from siltation. Off-line systems are designed to capture and treat the 6-month, 24-hour design storm; this is typically achieved by using isolation/diversion baffles and weirs. A typical approach for achieving isolation of the water quality volume is to construct an isolation/diversion weir in the stormwater channel such that the height of the weir equals the maximum height of water in the infiltration/filtration basin during the 6-month, 24-hour design storm. When additional runoff greater than the water quality storm enters the stormwater channel it will spill over the isolation/diversion weir and mixing with the already-isolated water quality volume will be minimal. Figures III-3.24 and III-3.25 in Section III-3.4 (Filtration BMPs) illustrate two types of isolation/diversion structures which have been successfully used.

Outlet Structure

The outlet structure conveys the water quality volume from the presettling basin to the primary treatment BMP (e.g., infiltration basin, sand filtration basin). The outlet structure shall be designed to provide a range of detention times for different runoff volumes, as shown in Table III-4.5 with a maximum detention time of 24 hours for the 6-month, 24-hour design storm. A perforated pipe or equivalent is the recommended outlet structure. The 24 hour drawdown time should be achieved by installing a throttle plate or other flow control device at the end of the riser pipe (the discharges through the perforations should not be used for draw-down time design purposes). The perforated riser pipe can be selected from Table III-4.6.

A trash rack shall 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. The bottom rows of perforations of the riser pipe should be protected from clogging. To prevent clogging of the bottom perforations it is recommended that geotextile fabric be wrapped over the pipe's bottom rows and that a cone of one (1) to three (3) inch diameter gravel be placed around the pipe (see Reference 75). If a geotextile fabric wrap is not used then the gravel cone must not include any gravel small enough to enter the riser pipe perforations. Figure III-4.15 illustrates these considerations.

Table III-4.6
Perforated Riser Pipe Specifications

Riser Pipe Nominal Dia. (inches)	Vertical Spacing Between Rows (center to center - 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. This information is based on commercially available pipe. Equivalent designs are acceptable.

Other Design Considerations

Liner to Prevent Infiltration

Detention BMPs should have negligible infiltration rates through the bottom of the pond. If infiltration is anticipated then a detention facility must either not be used and an infiltration BMP used instead (see Chapter III-3) or a liner installed to prevent infiltration. If a liner is used, the specifications provided in Section III-3.7 (Filtration BMPs) can be used. When using a liner the following are recommended:

- A layer of (track) compacted top soil (minimum 18" thick shall be placed over the liner prior to seeding with an appropriate seed mixture (see BMP E1.35 in Chapter II-5).

- Other liners may be used provided the design engineer can supply support documentation that the material will provide the required performance.

Berm Embankment/Slope Stabilization

Pond embankments higher than 6 feet shall require design by a geotechnical-civil engineer licensed in the State of Washington. The embankment shall have a minimum 15 foot top width where necessary for maintenance access; otherwise, top width may vary as recommended by the geotechnical-civil engineer.

The berm dividing the pond into cells shall have a 5 foot minimum top width, a top elevation set one foot lower than the design water surface, maximum 3:1 side slopes, and a quarry spall and gravel filter "window" between the cells.

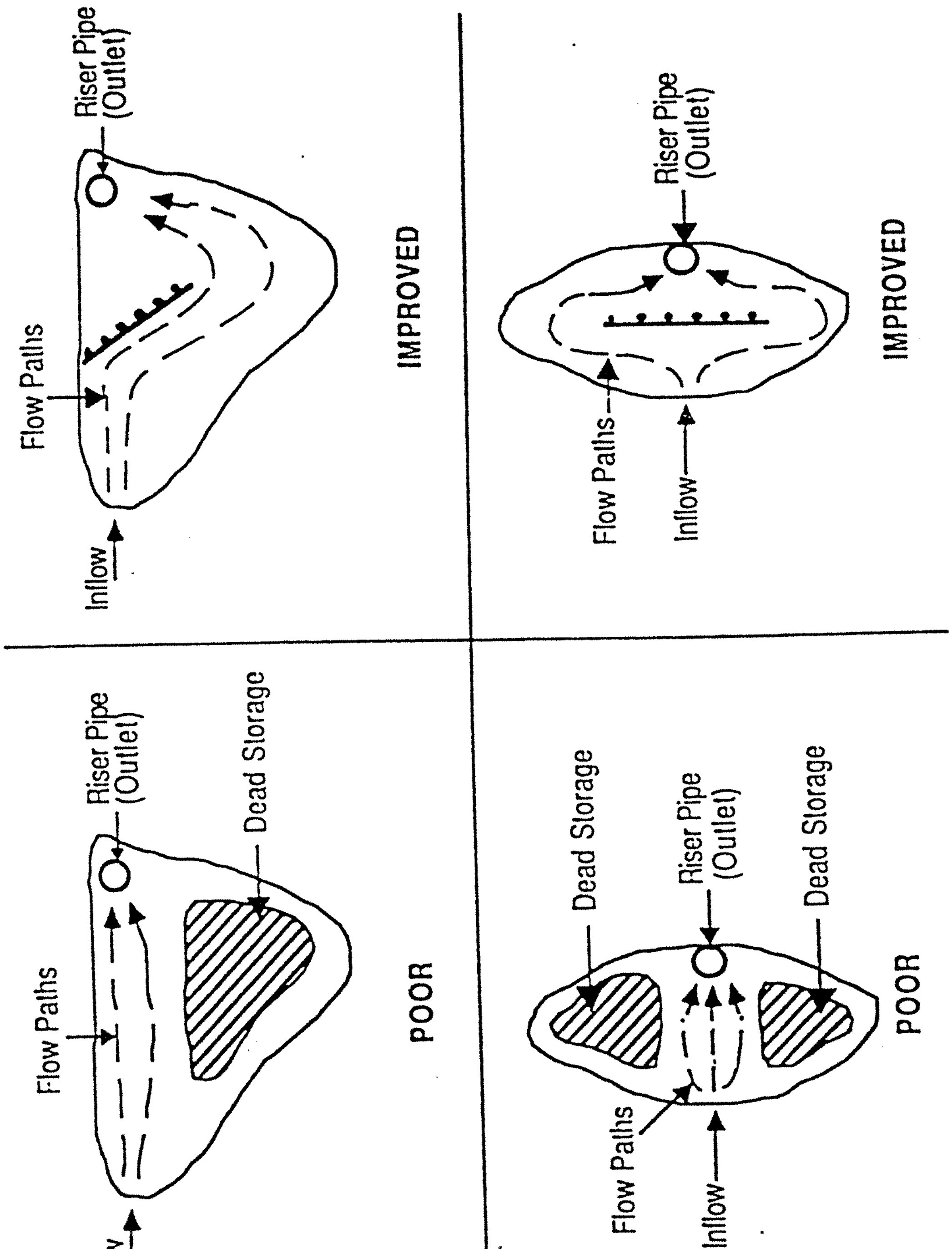
For berm embankments of 6 feet or less than (including 1-foot freeboard), the minimum top width shall be 6 feet or as recommended by the geotechnical-civil engineer.

The toe of the exterior slope of pond berm embankment must be no closer than 5 feet from the tract or easement property line.

Pond berm embankment must be constructed on native consolidated soil (or adequately compacted and stable fill soils analyzed by a geotechnical report) free of loose surface soil materials, roots and other organic debris.

Pond berm embankments must be constructed by excavating a "key" equal to 50 percent of the berm embankment cross-sectional height and width (except on highly compacted till soils where the "key" minimum depth can be reduced to 1 foot of excavation into the till).

Figure III-4.13 Use of Baffles to Improve Performance of Presettling Basins



Sedimentation Basin Baffles

Figure III-4.14 Permanent Sediment Trap for Presettling Basin

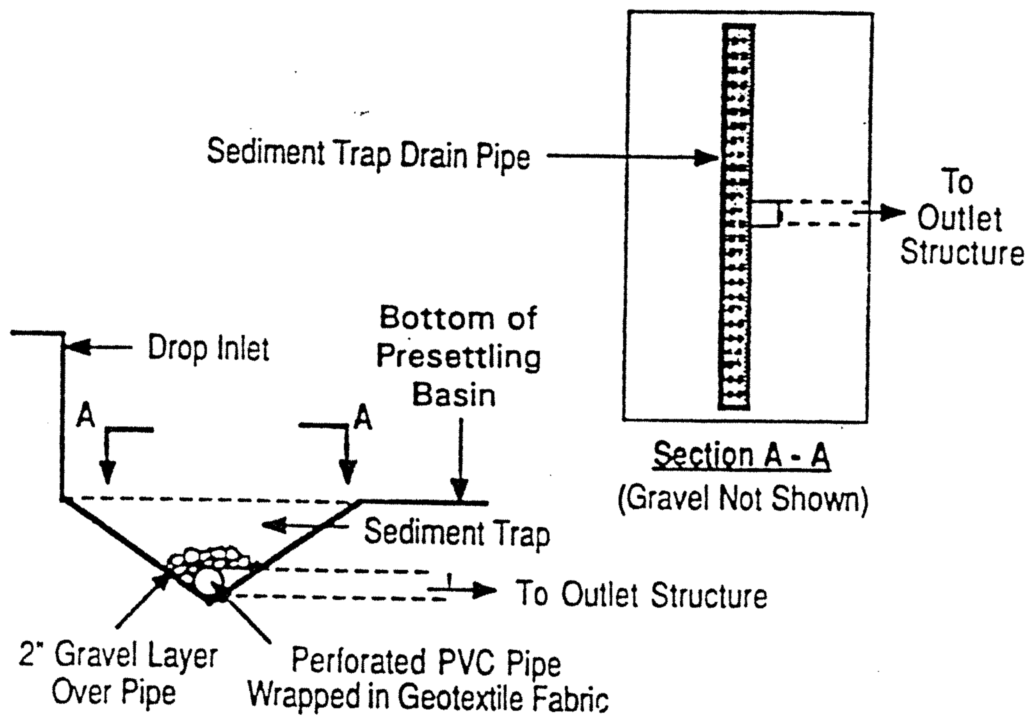
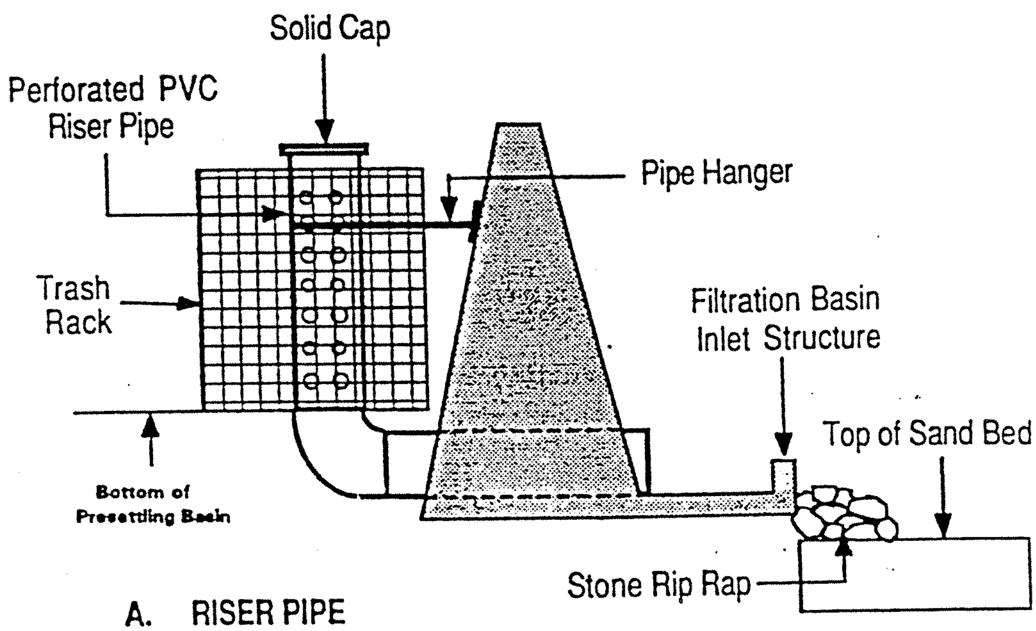


Figure III-4.15
Perforated Riser Pipe Outlet Structure with Trash Rack



The berm embankment shall be constructed on compacted soil (95 percent minimum dry density, standard proctor method per ASTM D1557), placed in 6-inch lifts, with the following soil characteristics per the United States Department of Agriculture's Textural Triangle: a minimum of 30 percent clay, a maximum of 60 percent sand, a maximum of 60 percent silt, with nominal gravel and cable content (Note, in general, excavated glacial till will be well-suited for berm embankment material).

Anti-seepage collars must be placed on outflow pipes in berm embankments impounding water greater than 8 feet in depth at the design water surface.

Exposed earth on the pond bottom and side slopes shall be sodded or seeded with the appropriate seed mixture as soon as is practicable (see Erosion and Sediment Control BMP E1.35 in Volume II). Establishment of protective vegetative cover shall be ensured with jute mesh or other protection and reseeded as necessary (see Erosion and Sediment Control BMPs E1.15 and E1.35 in Volume II).

Erosion and Sediment

Erosion and sediment control BMPs must be used to retain sediment on-site during construction (see Erosion and Sediment Control in Volume II). BMPs must be shown on the design plans and the engineer must provide instructions for proper O&M. Permanently stabilize all areas of ponds to prevent erosion and sedimentation of plantings (see Chapter II-5).

Construction and Maintenance Criteria

See BMP RD.05, Wet Pond (Conventional Pollutants).

III-4.4.5 BMP RD.11 Extended Detention Dry Pond

Purpose and Definition

An Extended Detention Dry Pond is designed to provide both pretreatment and streambank erosion control. It is similar to BMP RD.10 (Presettling Basin) except that it has an additional storage volume which provides an extended period of detention to control streambank erosion. Unlike the presettling basin, an extended detention dry pond will always be located "on-line" with the primary conveyance/detention system.

Planning Considerations

See BMP RD.10, Presettling Basin, for the following planning considerations:

- *Sediment and Debris*
- *Heavy Metal Contamination*
- *Site Constraints and Setbacks*
- *Dam Safety*
- *Safety, Signage and Fencing*

Other planning considerations are:

Multiple Uses

Multi-purpose use of the facility and aesthetic enhancement of the general area should also be major considerations. Above all, the facility should function in such a manner as to be compatible with overall stormwater systems both upstream and downstream to promote a watershed approach to providing stormwater management as well as local flood control and erosion protection.

If the facility is planned as an artificial lake to enhance property values and promote the aesthetic value of the land, pretreatment in the form of landscape retention areas or perimeter swales should be incorporated into the stormwater management facility. If possible, catchbasins should be located in grassed areas. By incorporating this "treatment train" concept into the overall collection and conveyance system, the engineer can prolong the utility of these permanently wet installations and improve their appearance. Any amount of runoff waters, regardless how small, that is filtered or percolated along its way to the final detention area can remove oil and grease, metals, and sediment. In addition, this will reduce the annual nutrient load to prevent the wet detention lake from eutrophying.

Detention system site selection should consider both the natural topography of the area and property boundaries. Aesthetic and water quality considerations may also dictate locations. For example, ponds with wetland vegetation are more aesthetically pleasing than ponds without vegetation. Ponds containing wetland vegetation also provide better conditions for pollutant capture and treatment.

A storage facility is an integral part of the environment and therefore should serve as an aesthetic improvement to the area if possible. Use of good landscaping principles is encouraged. The planting and preservation of desirable trees and other vegetation should be an integral part of the storage facility design.

Basin Planning

The design of urban detention facilities should be coordinated with a basin plan for managing stormwater runoff. In a localized situation, an individual property owner can, of course, by his or her actions alone, provide effective assistance to the next owner downstream if no other areas contribute to that owner's problems. However, uncontrolled proliferation of impoundments within a watershed can severely

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BIOFILTRATION SWALE PERFORMANCE, RECOMMENDATIONS, AND DESIGN CONSIDERATIONS

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Municipality of Metropolitan Seattle
Water Pollution Control Department
821 Second Avenue
Seattle, Washington 98104-1598

SECTION 7

RECOMMENDATIONS

In addition to the studies presented in Sections 5 and 6 of this report, members of the Biofiltration Project team and others we have consulted have acquired insights in other aspects of stormwater management associated with biofiltration swales. This information is presented here under the following categories: Institutional and planning, design and installation, operation and maintenance, and areas for further study. In addition to the discussions and recommendations given here, the Application Guide prepared in the Phase I report is updated to include these recommendations and is presented in Appendix G.

PLANNING CONSIDERATIONS

Landscaping

The relationship between biofiltration swales and landscaping, in terms of satisfying jurisdictional requirements, is purely a matter of local governmental discretion. However, there is no insurmountable constraint preventing the merging of the two uses. This said, there are several practical considerations to ensure such a melding does not compromise water quality objectives.

Swale planting. First, a word should be said about the planting of the swale itself. Biofiltration swales, otherwise known as grassy swales, typically rely on a dense planting of grass to provide the filtering mechanism responsible for water quality treatment. Most grasses tend to be very finely divided, with densely spaced blades. Pollutant removal effectiveness is related both to the density and stiffness of the blades, providing the "scrub brush" effect, and to the amount of surface per unit area provided by the individual blades. Few other herbaceous plants present the same features of finely divided and densely spaced leaf or stem surfaces.

In addition, grass has a unique ability to grow up through thin deposits of sediment or sand. Beach grasses are a good example, showing adaptation to a shifting sand environment by continuing to grow as lower stems and blades are covered with sand. The ability to grow up through a certain amount of sediment is highly desirable for water quality treatment. Besides maintaining blade density, it stabilizes the deposited sediment, preventing it from being re-suspended and washed out of the swale. Some grass species are better at dealing with deposited sediment than others, and there is, of course, a limit to how fast any grass can grow in response to burying.

Thus for typical biofiltration swales, grass is by far the most effective choice of plant material. A grass seed mix used successfully in Mountlake Terrace, Washington is given below:

- Tall fescue 67 percent
- Seaside bentgrass 16 percent
- Meadow foxtail 9 percent
- Alsike clover 6 percent
- Marshfield big trefoil 1 percent
- Inert matter 1.5 percent
- Weed seed 0.5 percent

However, swales are often positioned in shady locations, or experience self-shading due to their geometry and orientation. In these cases, as in all lawns, moss becomes a problem. Although not as finely divided as grass, moss does provide a high degree of surface area. In addition, several researchers have found that moss is a highly effective cation-exchanger, able to remove even low concentrations of metals from water (Low & Lee, 1991, Lee & Low, 1989, Ruhling & Tyler, 1970). Growing with grass, the moss tends to be supported upright by the grass stems. However, most moss species have less rigidity than grass, and when inundated, tend to lie flat rather than maintain an upright posture. Overall, moss in grassy swales is probably a benefit if grass densities are relatively high, but can be a problem if grass densities are too low, reducing the "scrub brush" effect of the vegetation.

Grass (other than reed canarygrass) will not grow under conditions of permanent inundation. For swales established on sites that intercept groundwater or with little or no slope to provide for good drainage, use of wetland vegetation is an acceptable planting alternative. However, the same considerations apply. The more finely divided the plant material in the water contact zone, the more effective its ability to provide treatment. Although cattails are inexpensive and easy to grow, they are discouraged for use in biofiltration swales for two reasons. First, in disturbed environments cattails tend to be invasive. Limiting their spread by limiting use is desirable, particularly if sensitive wetlands occur downstream from the swale location. Secondly, cattail stems tend to form tight clumps. These clumps are not finely divided like grass, but have overlapping or contiguous leaves and stems, making it difficult for water to flow through the clumps. Other smaller wetland species, such as *Juncus* (rush), *Eleocharis* (spikerush) or *Scirpus* (bulrush) may provide better filtration surface per unit area, provided the plants are healthy and produce vigorous growth.

Experience with wetland plants in a constructed wetland at Metro's South Transit Base has shown that the species listed below tend to be fairly finely divided and relatively resilient. They grew well even though subjected to fluctuating water levels and inundation stresses during typical storm events (Metro, unpublished data).

- *Juncus tenuis* (rush)
- *Juncus ensifolius* (dagger rush)
- *Scirpus microcarpus* (small-fruited bulrush)
- *Eleocharis* (spikerush)
- *Sparganium euycarpum* (burreed)

Weedy and invasive species such as cattail, purple loosestrife, reed canarygrass and giant reed (*Phragmites*) should be avoided.

Adjacent plantings. Woody or shrubby vegetation is not appropriate in the active treatment area of a biofiltration swale. However, in the area above the normal treatment area, that is, beginning with the portion of the side slopes designed as freeboard to pass larger storm flows, other kinds of landscaping material may be appropriate. The most important considerations for integrating other landscape materials without compromising the water quality objectives of the swale are shading and slope stabilization. A nonaggressive ground cover material such as *Hypericum* (St. John's wort), *Ajuga* (bugleweed) or *Vinca* (periwinkle) could be placed above the grassed treatment area of the swale without ill effect, provided the ground cover was dense enough to prevent any transport of sediment from the upper slope into the swale. Barrier shrubs, such as barberry, if continuous around the swale perimeter, could be effective in keeping out dogs.

Trees or shrubs that mature to provide a dense canopy will shade the swale. Since most grass species grow best in full sun, dense shade should be avoided. If shaded for too much of the day, grass will not grow densely enough to provide good filtration benefits. The City of Mountlake Terrace requires a 20-foot spacing of trees near swales to avoid this problem (Khan, personal communication). In addition, leaf or needle drop can contribute unwanted nutrients into the swale, can create debris jams which interfere with the evenness of water flow through the swale, clog inlet and outlet areas, and smother or even kill the grass.

Evergreen trees or shrubs, either coniferous or deciduous, planted on the east or north sides of swales could avoid most of the drawbacks mentioned above. Even so, shedding of pollen cones from coniferous trees was observed to cause minor blockage and channelization problems in the study swale in Mountlake Terrace.

Another consideration in establishing trees or shrubs in proximity to a biofiltration swale is that of stabilizing the soil in the planting beds. Often landscaped areas are mounded, with the soil surface mulched with bark. This is a particularly undesirable situation from the standpoint of good swale maintenance. Bark and mulch on mounded surfaces will inevitably be carried downstream into the swale, causing excess sedimentation, clogging of inlet or outlet areas, and unevenness in the swale bottom leading to channelized flow. The introduction of fertilizers and even pesticides and herbicides can also be problems.

If landscape beds are placed near swales, the beds themselves should be flat rather than mounded. Beds slightly lower than the ground surface are even more desirable, particularly if mulch or bark is to be applied. Bed edging can also help prevent soil from leaving the beds and being washed into the swale.

Animal manures should not be placed in the soil mix used within the swale proper. The bacteria in such a soil mix can stay alive and grow in the soil, often for long periods of time, causing water quality problems when carried downstream.

Treatment Trains

Treatment train is a term used to describe a situation when two or more treatment control devices are placed together at a site to remove pollutants from stormwater. Often a distinction is made between a treatment control device and a source control device. A treatment device, often a physical structure, is designed and constructed specifically to remove pollutants already in the water. Source control devices are prevention techniques used at the point of generation of pollutants to prevent their entry into stormwater. Source control devices could include beams and roofs erected to prevent rainwater from coming into contact with pollutants, or shut-off valves to contain spills before they leave a site.

In selecting a stormwater treatment system, several factors should be considered. The stormwater requirements of the political jurisdiction in which the business or property is located are paramount. Many jurisdictions have requirements for control of the rate of discharge (or peak runoff rate) from new or redevelopment. This control is usually accomplished by detention of the flow, discharging at a controlled release rate through an orifice (small opening). Some structures that are used for detention can also be used for stormwater treatment provided that they are properly designed for this purpose. An example is a wet detention pond which provides stormwater storage as well as water quality treatment if properly designed and maintained.

In addition, the type of land use and the potential pollutants that will be generated should be considered in determining stormwater treatment requirements. With some land uses, more than one treatment method may be needed. For example, if stormwater runoff is expected to contain high concentrations of oil, it may be necessary to use an oil/water separator to pretreat the water before it enters other treatment devices. Many jurisdictions require pretreatment in the form of solids removal or spill control by providing catch basins or gravity oil/water separators. These pretreatment measures are often used in conjunction with detention and water quality treatment devices.

In addition to grassy swales, other types of stormwater treatment methods include soil infiltration, grassed filter strips, constructed wetlands, wet detention basins, extended dry detention basins, oil/water separators (spill control catch basins, American Petroleum Institute (API) separators, and coalescing plate interceptors (CPI)).

Placement of treatment devices is also an important consideration when used in conjunction with a grassy swale. It is important to protect the vegetation in the swale against heavy loads of some pollutants like oil and grease and sediments. It may be necessary to place an oil/water separator or a sediment trap upstream from the swale. A swale may precede or follow a flow control or detention facility, but there are advantages to having the swale follow the detention pond or vault. Placing the swale downstream from a detention facility minimizes the likelihood of swale erosion. Because the detention facility will hold high flows and discharge them at a controlled rate, more of the flow from large storms events could be treated by this arrangement.

A creative placement alternative is to nest a circular grassy swale around the inner circumference of a dry detention pond. This arrangement allows for water quality treatment of low storm flows and detention of high flows within the same facility, maximizing the use of land devoted to water quality protection.

Monitoring Considerations

Often jurisdictions wish to have assurances that swales are functioning as intended and require monitoring. At other times, jurisdictions themselves may want to monitor swale performance for their own information. If monitoring may be required, jurisdictions should make these expectations clear before facilities are designed. Modifying swales for monitoring is not a casual affair. The equipment required to do flow-proportional monitoring is bulky, and flow monitoring usually requires a weir of some type (see Section 5). Manholes or catch basins may sometimes be provided at one or both ends of a swale, and may provide convenient places to sample.

Another consideration is that access to the swale should be stipulated as part of a permanent maintenance, inspection or access easement so that equipment can be serviced and samples collected. For adequate monitoring, this access is usually required well into the occupancy phase of a project, often a more difficult situation than access during construction.

DESIGN AND INSTALLATION

The effectiveness of a biofiltration swale depends largely on the accuracy of design and installation. The swale design approach in the Phase I report was first advanced by Chow (1959) for water conveyance applications and adapted or modified by others. It consists of two steps: First, designing for conveyance capacity of the swale lined with vegetative cover; and secondly, designing for channel stability to minimize erosion (Horner, 1988). This approach emphasizes the hydraulic conveyance capacity of swales rather than the water residence time. Because pollutant removal occurs as a result of sedimentation, filtration, adsorption, and other surface processes, it is logical that the emphasis on hydraulic residence time might result in optimizing swale performance.

Hydraulic residence time of a biofiltration swale depends on various components such as geometry, hydraulics, hydrology, soil type and type of vegetation. The following three design elements should be considered when constructing an efficient biofiltration system.

Hydraulics

Open channel hydraulics is one important consideration for designing a biofiltration swale. Hydraulic design of engineered swales for stormwater treatment is based on several variables, including the maximum velocity, design flow rate, depth of flow, and the channel roughness factor. The selection of proper design variables is based on several factors including hydrology, vegetation, soil type, and the goals of the treatment. The following recommendations on hydraulic design components are derived from this study.

Maximum design velocity. Geometric dimensions of most engineered biofiltration swales are designed by using Manning's Velocity Equation. A maximum permissible velocity of flow is selected to limit channel erosion and to provide a reasonable hydraulic residence time. Based on the slopes, soil types, and vegetation, a wide range of maximum allowable velocities (1.5 feet per second to 8 feet per second) have been used in earlier studies. As a part of this study, it was observed that grass (approximately 12 inches) on the bottom of the swale was flattened at velocities of about 0.9 feet per second. Flow channelization on the swale bottom accelerated with the increased degree of grass bending. Horner recommended that the design velocity be limited to a maximum of 1.5 feet per second. However, this study demonstrated that the maximum velocity should not exceed 0.9 feet per second to prevent exceeding the treatment capability of the swale.

Manning's n. A wide variety of vegetation types, heights, and densities have been used for biofiltration swales. These vegetation characteristics control the flow retardant and filtration capabilities of biofiltration swales. Manning's n is the channel roughness factor, which incorporates vegetation characteristics into the swale design. Even though many tables have been published for selecting Manning's n, the proper selection of n value is difficult as well as controversial among designers (Horner, 1988, *Guidebook Water Quality Swales*, 1990). An objective of this study was to measure Manning's n for a particular grass-lined swale. Section 6 contains the detailed description of the Manning's n determination. From results of this study, it is recommended that a Manning's n of 0.20 should be the minimum n value used for designing grassy swales intended for water quality treatment purposes. It may be that for situations in which swales are infrequently mowed, such as rural roads, a higher Manning's n value (0.235 is suggested) should be used for design. It may also be true that for denser grass, a higher Manning's n value may be appropriate for design. However, the study is not able to make recommendations for this denser grass situation.

Flow spreading and channelization. Hydraulic design of a biofiltration swale assumes that the flow will distribute evenly across the channel bottom. Flow channelization on the swale bottom reduces the effectiveness of biofilters by generating excessive velocities and scouring of the channel bottom and reducing contact with the vegetation. Swale design should incorporate a flow spreading device at the inlet. Flow spreading can be accomplished with various structures such as a shallow weir across the channel bottom, a stilling basin, perforated pipe, or other means. The flow spreader should be designed to provide a uniform flow distribution across the swale bottom. It should include a sediment clean-out area and must have low maintenance requirements.

To minimize flow channelization, it is imperative that the swale bottom is first, entirely smooth with uniform longitudinal slope, then covered with uniformly dense vegetation, and finally free from deposited sediments and debris. Swales with relatively wider bottoms are susceptible to more flow channelization. Flow channelization can be reduced by installing check dams at reasonable intervals; however, check dams make mowing more time consuming.

For best performance, check dams should be have a level upper surface rather than be made of rip-rap.

Flow bypass. Design flow rates are often established by local regulations. It was recommended in the Phase I report that the design flow rate for biofiltration swales should be estimated from the 2-year, 24-hour storm event. According to Horner (1988), this duration adequately characterizes local precipitation and represents the typical antecedent conditions prevalent in this region for most situations. Swales are designed to either convey or to bypass high flows (such as flows greater than those from the 2-year, 24-hour storm). If a swale must convey high flows, consideration should be given to the control of channel erosion and destruction of vegetation, and a stability analysis must be performed.

Flow can be bypassed by installing a pipe parallel to the swale and a flow regulating device inside the inlet structure. High flow bypasses may be of two types. During some storm events pollutants are more concentrated in the "first flush." Where space is a constraint, biofiltration swales could be designed for treating stormwater pollutants only from the initial portion of the storm. This approach would require bypassing stormwater flow around the swales during the higher portions of the flow. More typically, swale bypasses are designed to treat the design flow throughout the storm event, bypassing only the flows in excess of the design flow.

There are advantages and disadvantages to including a flow bypassing structure in the swale design. Bypassing high flows has the advantage of avoiding the carry-in of leaves, litter and heavy sediment loads dislodged by large storm events which can cause flow channelization and interfere with treatment effectiveness. High flows have also been observed to cause flattening of swale grasses for several days following a storm event, particularly if the grass is long. Flattened grass would be less effective in removing pollutants from subsequent storm events. Additionally, a bypass would allow diversion of flows in other situations, such as during swale maintenance, regrading, and vegetation establishment.

It should be noted that some of the problems experienced when high flows are transported through swales can be avoided if swales are placed after detention ponds or flow control vaults.

The disadvantage of a flow bypass is that it is more expensive. In addition to the space for the swale, additional cost for the piping and control device is necessary. However, additional space for the bypass is often not needed, as the piping can be installed within the sides of the swale, depending on the elevation required.

Hydraulic Residence Time

Based on observations made during this study, a residence time of 9 minutes which was associated with the 200-foot swale configuration was found to result in good removals of particulate pollutants, including metals, oil and grease and TPH. An attempt was made to determine an optimum residence time. However, due to an insufficient spread in the data, no reliable relationship between pollutant removal and residence time was able to be derived. There was some indication, however, that the lower residence times observed, between 4 and 5 minutes, were insufficient to assure good pollutant removal in storms with pronounced peaks, but still well below the design storm flow. Until more data are available, it is recommended that a minimum residence time of 9 minutes be used for swale design. In no case should residence time be less than 5 minutes.

Base Flow

If longitudinal slope is too flat (less than 2 percent) or if base flow is present in the swale, standing water can become a problem. With the exception of reed canarygrass, an invasive and undesirable species, grass will not grow in water-logged soils. In cases where high water tables, slight slopes or winter base flows are realities on site, use of a finely-divided wetland vegetation is recommended rather than grass. Please see the previous paragraphs on landscaping for more information.

Geometry

Natural grass-lined swales or ditches can be found in various lengths, sizes, shapes, and slopes. Most common swale cross-sections are rectangular, semi-parabolic, or trapezoidal. Engineered swales are often designed with trapezoidal cross-sections. Trapezoidal cross-sections are preferred because of relatively wider vegetative areas and ease of maintenance. They also avoid the sharp corners present in V-shaped and rectangular swales, and offer better stability than the vertical walls of rectangular swales.

Geometric dimensions of a swale determine its hydraulic residence time and flow characteristics. Because the water residence time and flow velocity are the most critical factors for pollutant removal, the geometric design should accommodate sufficient hydraulic residence time and allow for maximum design water velocities less than 0.9 feet per second.

The Phase I Biofiltration design guidelines are based on several assumptions:

- Minimum length of 200 feet

- Side slopes of 3 (horizontal) : 1 (vertical)
- Longitudinal slopes between 2 percent and 4 percent
- Design flow depths at least 2 inches below grass height

Design of the study swale used for this investigation was based on the above criteria, except a maximum water depth of 0.25 ft was specified rather than a design grass height. Performance of the 100-foot swale configuration in removing pollutants is discussed in Section 5. In general, the shorter length, which also had a reduced detention time, was more susceptible to the negative effects of siltation and short circuiting than the 200-foot configuration. The shorter configuration performed more poorly on average than the longer configuration, although this effect was not statistically discernible for pollutants other than iron and zinc.

Swale length requirements in some cases are a matter of local regulation. A scientific methodology for calculating effective swale length may be derived by analyzing optimum hydraulic residence time, width, slope, flow rate, and vegetation. As a part of this study, an attempt was made to derive the methodology for estimating swale dimensions by calculating the optimum hydraulic residence time. Due to insufficient data, it was not possible to derive the optimum residence time, although it was observed that a detention time of 9 minutes provided good pollutant removals (greater than 80 percent TSS). For better pollutant removals, a longer detention time is recommended.

Length. Previous studies have recommended that biofiltration swales be 200-feet-long for pollution control purposes. Results of a University of Washington study of grass-lined ditches along Interstate 5, and other wastewater treatment investigations reported an exponential pattern of metals removal (Horner, 1988). This study showed that, with the exception of iron and zinc, the performance of two swales of differing lengths, one being 200 feet, the other 100 feet, was not statistically different. However, in addition to length, the hydraulic residence time also varied. Because of the effect of reduced hydraulic residence time, seasonal differences and potential differences in the loading within the catchment area between the two swale configurations tested, a definite conclusion with regard to swale length cannot not be made.

However, there is also evidence to conclude that swale treatment area or residence times should be considered in swale design rather than relying on length alone. Using the Mountlake Terrace swale as an example, and specifying a 9-minute hydraulic residence time and an 8-foot maximum width, a minimum swale length would be about 125 feet, assuming all other geometric and vegetation conditions remained constant.

Width. The bottom width of parabolic swales varies with the amount of flow through the swale, but is fixed for trapezoidal swales, provided flow is adequately spread when introduced. A maximum bottom width selection should be based on the design flow depth to accommodate uniform sheet flow with average depth between 1 and 3 inches for maximum effectiveness. In theory, for a given hydraulic residence time, swale widths can be increased to compensate for reduced length. However, from the experience of members of the Project team, it has often been observed that relatively wide swales (those wider than 7 to 8 feet, a typical back hoe loader width (Irig, personal communication) are more susceptible to flow channelization and are less likely to have sheet flow across the swale bottom for the entire swale length. This occurs for several reasons:

- Inadequate flow spreading at the head of the swale
- The tendency for water to rechannelize if the swale bottom of trapezoidal swales is not perfectly level in cross section, as can occur when more than one blade pass is necessary to grade the bed
- The effect of obstructions such as leaves and branches in encouraging channelization
- The tendency for a "low flow" channel to develop in the swale bottom, which is then further intensified and channelized during higher flows

Although these problems can afflict swales of any width, they are a greater concern as channel widths increase and dominate more of the treatment area surface. Because of the tendency toward channelization, and given the reality of imperfect field installation, there is a practical upper limit for swale width. The maximum width allowed should consider the effectiveness of the flow spreading design used, the likelihood that swale bottom evenness can be assured and the frequency of mowing, which reduces the build-up of trash, leaves and other debris that tend to encourage channelization. For the swale studied, bottom evenness was about as good as possible with an ordinary construction crew, and care was taken to spread the flow over the entire 5-foot bottom width after it left the H-flume. Mowing was infrequent, however. Channelization of flow, particularly to the outer portions of the swale, was apparent during the velocity measurements for the Manning's n investigation, but not to the extent that it interfered with pollutant removal. It is recommended that unless the factors listed above can be dealt with adequately, swales wider than about 7 to 8 feet should be discouraged until more information about performance can be gathered. No recommendation is made at this time for situations in which ideal conditions of swale bottom levelness, flow spreading, and frequent mowing are assured.

A practical minimum swale width for trapezoidal swales should also be established for ease of maintenance. A minimum 2-foot bottom width is recommended to facilitate swale mowing with standard lawn mowers. However, narrower widths are possible if space is very constrained.

Longitudinal slopes. Earlier swale design standards indicate a wide range of longitudinal slopes, ranging between 0.05 percent and 8 percent. Increasing the longitudinal slope of a swale has the effect of increasing velocity. High velocity reduces the hydraulic residence time and increases erosion potential. On the other hand, stagnant water causes unhealthy grass and reduces the aesthetic values of grass-lined swales. The recommendation in the preliminary design criteria (Horner, 1988) that the longitudinal slope of swales should be between 2 percent and 4 percent, with 6 percent as an upper maximum slope, is reaffirmed based on experience gained in the field. An underdrain with perforated pipe should be installed when the slope of the swale is between 1 percent and 2 percent (Figure 7-1). The underdrain should be designed to drain standing water from the swale bottom.

On steep sites, swales can traverse grades to reduce their slope. If the site topography requires that the swale be steeper than 6 percent, then vertical drops (6 inches to 12 inches) of the swale bottom at a reasonable intervals (between 50 feet and 100 feet) should be added to minimize steepness of the slope. At the toe of a vertical drop, an energy dissipating and flow spreading structure should be installed. The performance of a swale is greatly influenced by its slope, so the grading must be accurate to ensure uniform longitudinal slope by eliminating humps and low spots.

Side slopes. Relatively flat-sided biofiltration swales are easier to mow than steep-sided swales. Selective landscape planting may also be incorporated on wide and flat side slopes to enhance the aesthetic value of parking lots or other areas. Furthermore, relatively flat side slopes reduce erosion potential and provide additional stormwater detention by increasing the conveyance flow area. Ideally, swale side slopes should be no steeper than 3 horizontal to 1 vertical. Sites with limited area to provide this slope may require slopes steeper than three to one, but maintenance and slope stability are concerns when the side slopes are steeper than 2 horizontal to 1 vertical. Rock walls are sometimes constructed above flat side slopes to accommodate space constraints. However, relatively tall rock walls may impose safety hazards, complicate maintenance, and present an awkward appearance.

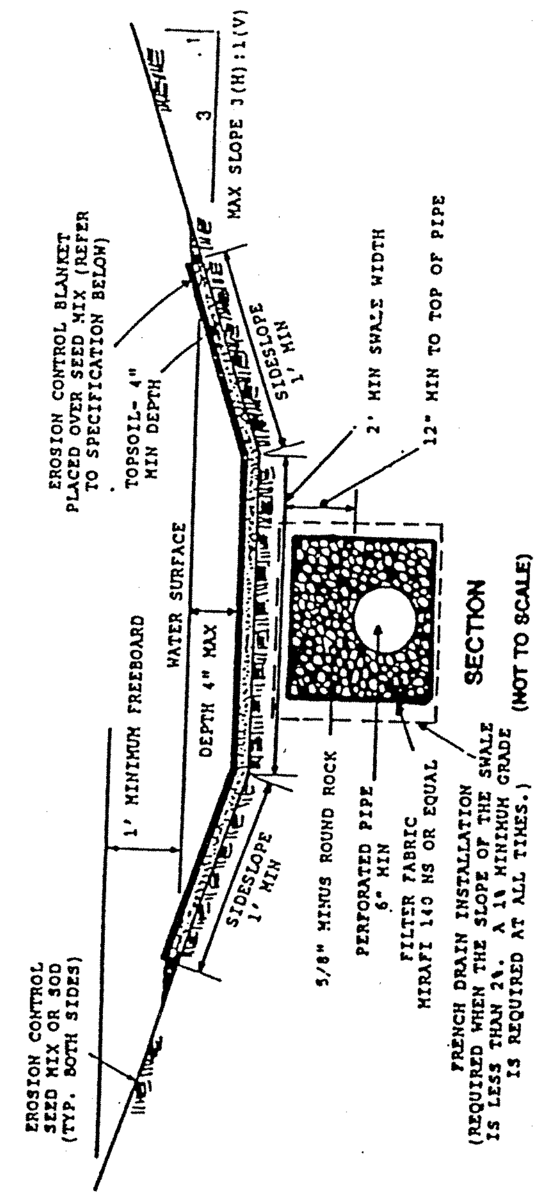

Figure 7-1. Underdrain Detail for Biofiltration Swales

Energy dissipation. Scouring of the swale near the inflow point can be a problem in grassy swales. Energy dissipation of inflow water can be achieved by installing rip-rap pads, stilling basins or other mechanisms. Rip-rap pads should be designed with swale geometry and energy generated by peak flows in mind. Often, 6- to 9-inch rock works well. However, it is important to fit the rock tightly together to avoid creation of small water pools and erosion around the rock. In general, the width of the rip-rap pad should be equal to the width of the swale bottom, with lengths between 5 and 10 feet. The flow spreader bars should be installed at the downstream end of the energy dissipater. To discourage vandalism, the rocks could be keyed into a concrete pad. Some managers recommend that rock be installed with the top flush with the bottom of the swale. However, others suggest that this can cause problems with water pooling between gaps in the rock, or that the rock can be buried due to sediment deposition.

Water depth. During the Manning's n investigations, it was observed that grass does not remain standing when water depths approached one third the height of the 12-inch grass. Therefore, the original Phase I recommendation that water depth should be at least 2 inches below the design grass height was not adequate to provide the expected biofiltration benefit. It is recommended that the design water depth be no greater than one third the height of the grass for tall grass (9 to 12 inches). It is further recommended that for mowed swales, the design water depth be no greater than one half the grass height up to a maximum water depth of 3 inches. This latter recommendation is not based on results from this research project and requires further investigation to confirm.

Soil Type Considerations

Selection of a soil type for a biofiltration swale should be based on the types of vegetation, slope of the swale, purpose of the swale, and the existing soil characteristics. Soil characteristics of a swale bottom should be conducive to grass growth. Where the longitudinal slope is less than 2 percent and the bottom is underlain with a French drain, the subgrade should be constructed with topsoil materials containing a high percentage of sand. Soils that contain large amounts of clay cause relatively low permeability and result in standing water for extended periods of time. Saturated soil causes grass to die and results in an unaesthetic appearance. Where the infiltrate from a biofiltration swale has the potential to contaminate groundwater, it is recommended that the swale bottom be sealed with clay material to protect groundwater resources. In general, for the swales located on residential and commercial development sites, use of 6 inches of the following topsoil mix is recommended:

- 50 percent to 80 percent sandy loam

- 10 percent to 20 percent clay
- 10 percent to 20 percent composted organic matter (excluding animal waste)

Topsoil must be free from materials toxic to grass growth, as well as stones and other debris and must provide adequate nutrient levels to ensure good grass growth. Onsite materials, where suitable, should be used for constructing the subgrade of biofiltration swales. Where possible, avoid using steer manures, since these are often leached into the receiving water.

OPERATION AND MAINTENANCE

During the course of monitoring the 48th Avenue W biofiltration swale, the importance of maintenance became apparent on several occasions. Areas needing particular attention include the following:

- Keeping the flow spreader even and free of leaves, rocks, and other debris
- Removing sediment from the upper portion of the swale deposited during high intensity storm events and mending channelized areas of the swale
- Removing litter, branches, rocks, and other debris which accumulate in swales
- Reseeding areas of poor grass growth
- Regular mowing

Issues related to these concerns are discussed below.

Access

In order to provide maintenance or to inspect to ensure maintenance functions are provided by the owner, access to the site is needed. Provision should be made through easements negotiated during the permitting process to require maintenance and allow access for inspection of swales. If future studies are envisioned, access for monitoring and for making modifications to the swale should also be negotiated.

Mowing

Regular mowing is important for several reasons and can affect the water quality performance of a biofiltration swale. First, biofiltration swales are often designed for a particular grass height (or water depth). If grass is mowed shorter than the design grass height, water depth may be too great relative to grass height to provide adequate biofiltration. If grass is left to grow too long, it may become lanky and grow less densely. Regular mowing encourages denser grass growth, provides for removal of vegetative debris, such as leaves and branches from swales, and avoids the tendency for formation of channels in the swale floor. If nutrient control is a treatment objective, mowing is also essential at the end of the growing season before the grass goes dormant to avoid remobilization of nutrients taken up and held by the vegetation.

In cases where nutrient removal is an objective, grass clippings should be removed from the swale and disposed of in such a way so that reentry into the receiving water is avoided. Even if nutrient control is not a primary concern, grass clippings should be removed from the swale to prevent clogging of outflow structures and to ensure flow through the swale remains even and unchannelized.

Sediment Buildup and Erosion

If swales are effective, some amount of sediment will be captured in the swale. If the rate of sediment deposition is too rapid, it can cover the grass, causing it to die, and exasperate channelization of the swale bottom. Prompt reseeding of damaged areas can prevent deterioration in effectiveness of the swale.

Before reseeding is done, the excess sediment should be removed by hand (flat-bottomed shovels work well) and grass cut short so that the bottom surface can be made as level as possible. Ideally, the same seed mix recommended for establishment of swales should be used. If possible, flow should be diverted from the swale until the grass is firmly established. Otherwise, cover the seeded areas with a high quality erosion control fabric to provide protection. It is also effective to introduce grass plugs from an area on the upper slope of the swale to further anchor the disturbed area. In general, sodding to patch damaged swale areas is not effective because of the difficulty in ensuring the sod is level with the swale bottom and the tendency for it to dry out if not watered frequently. If sod is used, it should be overseeded with a seed mix known to grow well in swale applications.

If areas are eroded, they should be filled and compacted so that the final grade is level with the bottom of the swale. Digging grass plugs from the upper slopes of the swale is preferable to filling and seeding, since the root systems already developed in the grass will do a superior job of resisting further channelization and erosion.

Byproducts and Disposal

Grass clippings. Generally, grass clippings should be removed from biofiltration swales. Upon decomposition they can contribute both nitrogen and phosphorus loads as they come in contact with stormwater. Grass can clog swale outlets, and can also collect in bunches along the swale bottom, encouraging channelization and erosion of the bottom. Unless the grass is visibly altered (for example, coated with oil or diseased) it should be disposed of in the same manner as yard waste. If the grass is visibly tainted or altered, or smells like petroleum products, it should be bagged and taken to a sanitary landfill (Burke, personal communication).

Sediment. There is much interest in whether the sediment which collects in a biofiltration swale could be a hazardous waste or a toxic material under the Model Toxics Control Act (MTCA). To be positive, chemical testing is required. Constituents to test for include total and Toxicity Characteristic Leaching Procedure (TCLP) metals (lead and zinc, perhaps others depending on land uses in the watershed) and total petroleum hydrocarbons (TPH). However, if the swale drains an area that is predominantly residential, the Seattle/King County Health Department does not typically require testing at present (Burke, personal communication).

It is possible that TPH and total metals concentrations in sediments collecting in swales could exceed the cleanup levels set by the MTCA, Chapter 173-340 WAC. More infrequently the TCLP metal concentration may be a problem. The criteria given in Table 7-1 are used by the Seattle/King County Health Department in determining if disposal of a substance of unknown origin should be specially controlled. If the substance exceeds the MTCA cleanup criteria, it should be treated to meet standards or be disposed of in a sanitary landfill. Sometimes disposal or use in controlled situations is allowed, such as for road subgrade, fill in an industrial area or a capped fill, provided no threat to health or the environment would result (Ecology, 1991). If the substance exceeds the criteria for solid waste, it must be manifested and disposed of as hazardous waste.

Table 7-1. Soil Disposal Criteria in Seattle and King County, Washington, for Selected Contaminants			
Solid Waste Criteria		Clean-up Levels (MTCA)	
TPH	< 3 percent	TPH, diesel	200 mg/kg
		TPH, gasoline	100 mg/kg
Benzene TCLP	< 0.5 ppm	Benzene TCLP	0.5 mg/kg
Toluene	< 1 percent	Toluene	100 mg/km
Xylenes	< 1 percent	Xylenes	20 mg/kg
TCLP Pb	< 5 ppm	Total Pb	250 mg/kg
TCLP Cr	< 5 ppm	Total Cr	2 mg/kg
TCLP Cd	< 1 ppm	Total Cd	no level set
TCLP Cu + Ni + Zn	< 5 ppm	Total Cu	no level set
		Total Zn	no level set
		Total Ni	no level set

Note: For more information, contact the Seattle/King County Health Department, Waste Screening staff, 296-4633.

It has been observed that sediment from catch basins can exceed the MTCA cleanup level of 200 mg/kg TPH. Work currently being done by Metro shows that vegetation in a constructed wetland has succeeded in dramatically reducing soil TPH concentrations accumulating in sediments washing off a transit base (Metro, unpublished data). Further study is needed to determine whether vegetation in a biofiltration swale may have a similar effect on reducing soil TPH.

Metals are also of potential concern. Little research has been done on soil metal contamination, but Wigington et al. (1986) and Wang et al. (1981) found that in roadside soil, most of the metals concentrated in the upper 5 cm. The only leachable metal found in the Wigington study was zinc, which was suggested to come from galvanized culverts.

A study of 21 wet and extended dry detention ponds in Virginia found that the available concentrations of trace metals were significantly less (1/1000) than the toxic thresholds of federally defined hazardous waste. However, for 19 of the 21 ponds, leachable fractions from the sediment exceeded water quality standards (Dewberry & Davis, 1990). The ages of the facilities were not given, but some may approach 12 years, the life of the management requirement for BMPs.

Trash. Trash tends to blow around and collect in low spots or against high grass. Unmaintained biofiltration swales can become unsightly, particularly if located in commercial areas. Landscape architects have found that the location of a swale can make a difference in the amount of maintenance provided. If located in the front of an establishment, better upkeep is typically provided than if located in back of a facility. An ancillary benefit of regular mowing is that the accumulated trash is cut up along with the grass and removed with minimum effort.

Yard waste is a special category of debris that is sometimes dumped into biofiltration swales by nearby residents. Specific educational efforts are recommended to inform citizens about biofiltration swales and their benefits and obtain their cooperation in maintaining them in good working order.

Animal waste. Monitoring data from this study showed that fecal coliform bacteria can be introduced into grassy swales in high numbers and take a very long time to be dispelled. In order to combat fecal coliform pollution from pet waste, public education and vigorous enforcement of scoop laws are needed. As a possible measure to protect swales from the impact of pets, a continuous planting of barrier shrubs, such as barberry, could be employed. (Barberry is a densely branched, low maintenance shrub with sharp thorns plentifully distributed along the branches. Landscapers sometimes employ this plant to manage foot traffic in some areas of public landscapes.)

Pet waste should be disposed of like human waste, by flushing to a sanitary sewer or septic tank system. Pet waste is not allowed in the garbage with other solid waste. Composting of pet waste is also an acceptable option in most locations.

Institutional and Enforcement Considerations

Operation and maintenance is critical to the effective performance of a biofiltration swale. It is important that jurisdictions be able to assure maintenance for swales they require to be installed. If no assurance of long term maintenance is identified, it may be advisable to choose another type of water quality treatment, such as a wet detention pond or constructed wetland, which requires less maintenance.

Possible approaches to maintenance include requiring a construction maintenance bond, maintenance bonds for a period of several years after project completion, or simply performing required maintenance with public resources and billing the property owner for the work. This latter approach is particularly applicable if erosion or flooding problems result from the neglect. The excerpt below describes the approach to maintenance used by the City of Mountlake Terrace:

> "A maintenance schedule, including mowing frequency, shall be included in the plans. All harvested (or mowed) vegetation shall be removed from the swale as part of the regular maintenance

program. Inspections will be performed by City of Mountlake Terrace staff, is maintenance is required, the owner shall be notified in writing of the required action. If such notice fails to produce action on the part of the owner, public works crews shall perform maintenance and the owner will be charged all costs."

As in most endeavors, thoughtful planning can alleviate maintenance problems later on. Provision of an easy to reach, perhaps concrete-lined area at the head of the swale to catch sediment can prevent having to reseed or repair a channelized-swale bottom. Adequate energy dissipation can also reduce problems with erosion and channelization.

Knowledge of the soils and groundwater regime on a site can give valuable clues about either soggy or arid conditions, both of which may influence the vigor and ease with which grass can be established. Provision for irrigation during the first summer season is important if seeding takes place in the spring.

Maintenance requirements should be considered before check dams are specified. What may seem like a water quality benefit may actually be a liability if it prevents regular mowing or maintenance. Rip-rap can plug with sediment, preventing flow of water and killing the grass. The fairly common practice of armoring the entire swale bottom with rip-rap needs to be challenged. If properly designed, armoring is not needed except for a very limited area at the head of the swale to provide energy dissipation. Rip-rapped swales also render mowing, and even maintenance, next to impossible.

At Metro and other agencies, it has been found useful to have grounds maintenance crews review landscaping and grounds designs before the final designs for capital projects are completed. A similar consultation among private sector parties would probably be equally useful.

AREAS FOR FURTHER STUDY

Hydraulic Residence Time

As in most studies, this investigation raised as many questions as it answered. One of the most promising areas for future research is to better establish the relationship between hydraulic residence time and treatment effectiveness so that more flexible basis for biofiltration facility design can be advanced. Such a study should be done on closely grouped sites having multiple swales designed to provide different water residence times. Grouping is important to assure that rainfall and runoff conditions are as similar as possible for all the swales, reducing confounding variables. The mechanics of performing such sampling are difficult and it is relatively expensive. However, the Unidata stage recorder used for this study performed better in the field than other flow instrumentation in common use. Reliable equipment greatly reduces the difficulty in collecting multiple samples. Another consideration in such a study is the impact on the site investigated. The provision of flumes, equipment barrels, and rain gages is obtrusive. In addition, the combination of flumes, water and sandbags (used to weight equipment tubing) is extremely enticing to youngsters who can recognize an ideal engineering opportunity when it presents itself. Such "assistance" was provided during the current study, causing some ideal storm samples to be rendered useless.

Manning's n Value for Mowed, Denser Grass

It is recommended that the research conducted in this project should be repeated for swales with relatively short, dense grass. The above investigation could possibly explore the performance and Manning's n for frequently mowed swales with higher grass densities at the same time that residence time considerations are explored. Manning's n for other vegetation types, such as wetland species, should also be investigated.

Maximum Width

The question of maximum swale width and criteria for effectiveness of swales wider than about 7 to 8 feet needs to be investigated. Design flexibility means little if proper field installation of those designs is not likely to occur. It may also be possible that use of plant material less densely spaced than grass but with more rigidity, such as red clover or native groundcovers, could overcome some of the channelization problems to which wide swales are susceptible.

Long-Term Pollutant Removal

The long-term effectiveness of biofiltration swales also needs to be rigorously investigated. There is some information from other parts of the country that over the long-term, swales do not perform as well as they might at first (Hartigan, personal communication). An ideal study would be to follow the swale investigated for this study periodically, say after 2, 5, and 10 years, to see how performance might vary as the swale ages. However, since the watershed would also be more densely developed, some means to account for this variable should be identified for this or any other time series study.

Swale Area Related to Watershed Area

Investigation of the correlation of swale treatment area with watershed area (or watershed impervious area) could result in a relatively simple to apply rule of thumb for provision of biofiltration swales for water quality protection, and should be investigated.

Nutrient Removal

Another important area for investigation is to enhance nutrient removal, particularly of phosphorus, in biofiltration swales. Among promising alternatives are the following:

- Investigate the effect of providing a section of bare clay soil as a mechanism to enhance the capture of dissolved phosphorus. The clay soil area should be near the end of the biofilter to minimize sedimentation on the clay surface, reducing its active treatment area.
- Investigate the use of alum, perhaps in solid form embedded in a rip-rap check dam, to facilitate removal of dissolved and bio-available phosphorus.

Both of these alternatives would require a control swale in addition to the test swale to best determine the effect of the treatment. Though not impossible, this need for paired swales would make finding a suitable site difficult.

It is recommended that these areas of investigation be considered a high priority for funding, particularly through the Centennial Clean Water Grant Program.

APPENDIX A

COMPARISON OF DESIGN METHODS

- Critique of Horner Swale Design Methodology by Gary Minton
- Comparison of Biofiltration Design Methods

APPENDIX A
COMPARISON OF DESIGN METHODS

CRITIQUE OF HORNER SWALE DESIGN METHODOLOGY BY GARY MINTON

Minton's Method uses the approach by Horner but with certain differences. Minton believes that the primary determinant of performance is surface area, rather than length of the biofilter, as indicated by the research of Professor Barfield at the University of Kentucky. Therefore, within the constraints of proper flow distribution at the end of the biofilter, the biofilter can be configured as desired to fit the site.

Also, since performance is a function of detention time, the area of the biofilter should be increased with increasing slope, reflecting that higher velocity and shorter detention time with increasing slope. However, Manning's Equation, does the opposite; the channel narrows with increasing slope and with a constant length of 200 feet, the biofilter surface area is decreased rather than increased as the slope is increased.

Minton's Method also is based on the belief that a Manning's n of 0.10 is too low for the conditions of interest; research with shallow sheet flow in thick grasses suggests the n value should be somewhere between 0.20 to 0.60.

Finally, because of the considerable uncertainty of the effect of filter geometry and highly variable turf grass quality on performance, and the uncertainty about what is the appropriate value for n, Minton's Method is based on a view that it is pointless to have design engineers and plan reviewers spend time on sizing calculations. Their time is better spent on those aspects of design that relate to facility integrity, flow spreading, energy dissipation at the inlet, etc.

Given these uncertainties it seems valid to define an "average filter area" that will be satisfactory. The figure of 500 ft^2/impervious acre is based on a series of calculations using Manning's Equation for different slopes and two values of n, 0.30 and 0.40. The calculations were done for a one-acre site with the peak rate of 0.20 cfs. The areas for the different situations varied from about 260 ft^2 (n=0.30; slope=5 percent) to about 1,00 ft^2 (n=0.40; slope=1 percent). A value of 500 ft^2 was selected. The area requirement can be used for both swales and filter strips.

COMPARISON OF BIOFILTRATION PROJECT DESIGN METHODS

Summary of Approach Differences

Feature	Horner	Minton	King County
Design Basis	Approximate or exact Manning Equation	Width * Length = 500 ft^2/acre	Exact Manning Equation
Swale Shape	Parabolic or trapezoidal	Not specified	Trapezoidal or rectangular
Swale Slope	2 to 4 percent (<2 or >4 with special provisions)	2 to 5 percent (<2 or >5 with special provisions)	Design assuming 2 percent
Flow Depth	Free choice	Assumes 4 inches	1 inch urban, 4 inches rural (8 inches wetland veg.)
Manning's n	Choose based on veg. and depth (usually 0.05 to 0.1)	Assumes 0.3 to 0.4	Use 0.35
Basis for Filter Strip	Treates as shallow, rectangular swale	Not covered	Rule of thumb

Design Differences

The following are design cases:

- Contributing areas—range from 1 to 50 acres, assumed to be 100 percent impervious
- Contributing areas slopes—2 and 15 percent
- Design flow rate calculation basis—King County's Modified Rational Method, with contributing areas assumed to be square with longest travel path along side for time of concentration estimation
- Flow depths—1 and 4 inches
- Comparison made on the basis of swale top width times length (T * L) required. (Note: All methods are based on L=200 feet, or proportional enlargement of T if L if less than 200 feet.)
- Calculations made for both the Horner approximate method and the exact method for space-limited conditions.

The following are design results:

Case	Contrib. Area, Slope (Acre, %)	Q (cfs)	Flow Depth (inches)	Resultant Square Footage of Swale for each Design Method: Horner Approx.	Horner Exact	Minton	King County
1a	1, 2	0.57	1	4,740	1,446	—	11,900
1b			4	464	360	500	1,500
2a	5, 2	2.8	1	23,280	7,012	—	58,200
2b			4	2,280	1,793	2,500	6,200
3a	20, 2	9.9	1	82,320	24,867	—	205,000
3b			4	8,060	6,324	10,000	21,000
4a	50, 2	18.9	1	157,160	47,349	—	392,600
4b			4	15,380	12,072	25,000	39,600
5a	1, 15	0.57	1	4,740	1,446	—	11,900
5b			4	464	360	500	1,500
6a	5, 15	2.8	1	23,280	7,012	—	58,200
6b			4	2,280	1,793	2,500	6,200
7a	20, 15	11.2	1	93,140	28,120	—	232,600
7b			4	9,120	7,153	10,000	23,600
8a	50, 15	28.4	1	236,200	71,205	—	590,000
8b			4	23,120	18,108	25,000	59,400

Conclusions

The Minton and King County methods result in larger swales than either Horner method. With contributing area less than or equal to 5 acres the Minton and approximate Horner methods differ by less than 10 percent. In this case, the difference between the Minton and exact Horner methods is about 40 percent. The differentials grow to more than 60 percent (Minton versus approximate Horner) and more than 100 percent (Minton versus exact Horner) as the size and slope of the contributing area increase).

The King County method results in swales greater than or equal to 150 percent as large as those produced by the approximate Horner method at both flow depths. Comparing the King County and exact Horner methods, swales designed by the former method are more than seven times as large at the shallower flow depth an more than twice as large at the 4-inch depth. Velocities in the King County swales are less than 0.3 feet per second, compared to 0.4 to 1.1 feet per second in swales designed by the approximate Horner method and nearly 1.5 feet per second in those designed by the exact Horner method. These large differences are because of the stipulation by King County that n=0.35 be used in design. The limit in depth to 1 inch in urban areas would also make swales much larger than they would have to be.

The 200-foot-long I-5 swale that was the site of the original biofiltration work drained an area of 1.2 acres. For that area and flow at 4 inches depth, the approximate Horner method gives T=2.8 feet, which is approximately the width of that swale. The King County method yields T=9 feet at the same depth and length. The actual swale produced consistent 80 percent removals of total suspended solids and lead and 60 percent reductions of copper and zinc. Therefore, the very large swale designed by the King County method is clearly not needed to achieve high treatment efficiencies.

CHAPTER III-6

BIOFILTRATION SWALES AND VEGETATIVE FILTER STRIPS

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CHAPTER III-6

BIOFILTRATION SWALES AND VEGETATIVE FILTER STRIPS

Editor's Note: This edition of the manual has classified biofiltration swales and vegetative filter strips as two different BMPs. Though their pollutant removal mechanisms are similar, their planning and design criteria are different enough to warrant separation. However, this edition of the manual retains the previous edition's criteria; subsequent editions of this manual will likely reflect changes in planning and design criteria.

There are still uncertainties and differences of opinion on how to best design biofiltration swales and vegetative filter strips. In addition, the effectiveness of these BMPs, especially for the treatment of nutrients, is an unresolved issue. As a result of this and other issues, Ecology plans to convene a standing advisory group that will attempt to resolve key technical issues. A review of the latest findings from current biofilter monitoring projects will be conducted and recommendations made regarding the design methodology, planning considerations, construction, and maintenance of biofilters and vegetative filter strips. Subsequent editions of this manual will incorporate such findings.

III-6.1 INTRODUCTION

III-6.1.1 Background

Biofiltration swales and vegetative filter strips are two practices which have been used in stormwater management for some years. Only fairly recently have they been studied to determine their effectiveness at treating pollution from stormwater runoff and to assess their abilities to reduce peak flow rates. Because these two BMPs are non-structural, they are considered desirable alternatives to ponds, tanks, and vaults. At this time these two practices are assumed to provide runoff treatment but not streambank erosion control (the latter is an issue that needs further investigation, especially for less intensely developed sites).

III-6.1.2 Purpose and Scope

The purpose of this chapter is to present general and specific criteria for the evaluation, design, construction, and maintenance of biofiltration swales and vegetative filter strips. In particular, this chapter provides guidance on how BMPs can be designed to accomplish one of the two primary stormwater management objectives, *runoff treatment* and *streambank erosion control* (recall that *source control* is another objective which is required in all cases). While streambank erosion control is not generally provided by these BMPs, biofiltration swales can be designed to convey higher flows to BMPs used for streambank erosion control and thus may be incorporated into the primary conveyance/detention system.

Section III-6.2. should be read first as it gives a description of the pollutant removal mechanisms utilized by biofilters and vegetative filter strips to meet Ecology's runoff treatment standard. Sections III-6.3 and III-6.4 provide detailed planning, design, construction, and maintenance criteria for each BMP. A design procedure is described in Appendix AIII-6.1 for both BMPs with an example problem provided in Appendix AIII-6.2.

Figure III-6.1 Biofiltration Swale

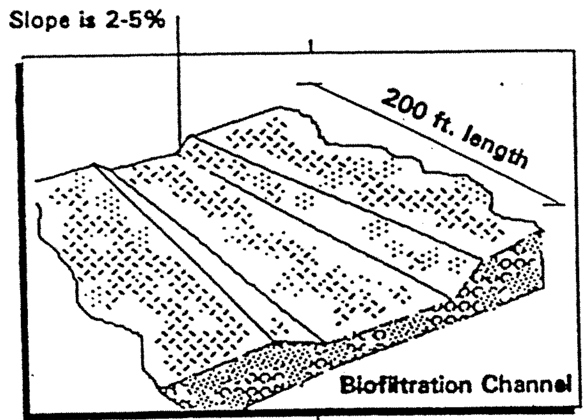
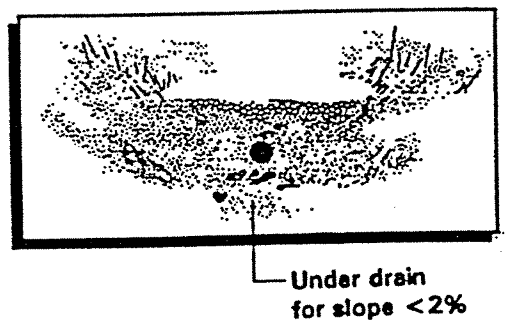


Figure III-6.2 Biofiltration Swale with Underdrain System



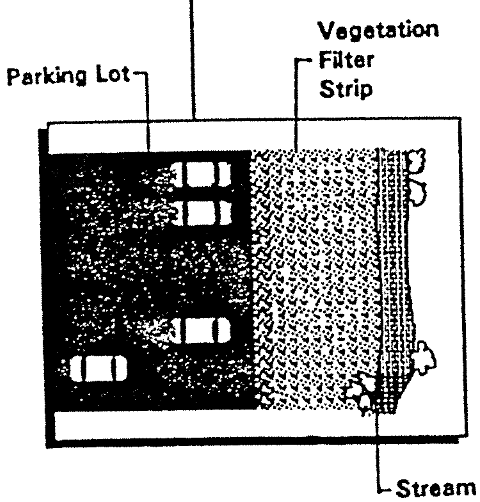


Figure III-6.3 Vegetated Filter Strip

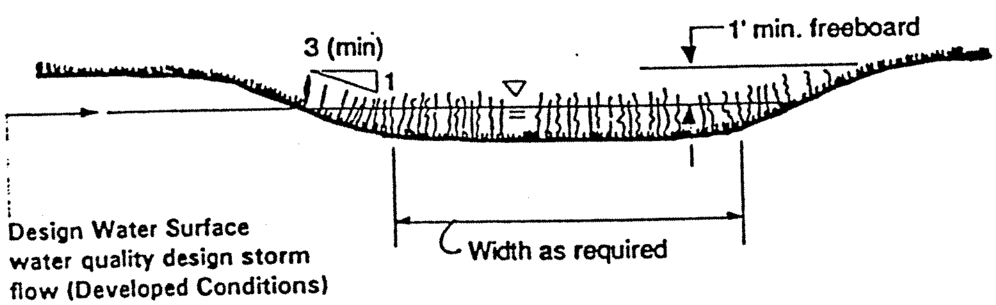


Figure III-6.4 Swale Design Showing Freeboard

III-6.2 RUNOFF TREATMENT AND CONVEYANCE

III-6.2.1 Overview

There are two types of biofiltration-type BMPs: the biofiltration swale (BMP RB.05) and the vegetated filter strip (BMP RB.10). Figures III-6.1 through III-6.4 illustrate these BMPs. A biofiltration swale is a vegetated channel that is sloped like a standard storm drain channel; stormwater enters at one end and exits at the other with treatment provided as the runoff passes through the channel. With vegetated filter strips the flow is distributed broadly along the width of the vegetated area; treatment is provided as runoff travels as sheet flow through the vegetation.

Which method to use depends upon the drainage patterns of the site. A vegetated strip would function well where the water can be spread along the length of a parking lot. Gaps in the lot curb provide the entry points. Of course, the grade of the parking lot must be flat immediately parallel to the strip.

For runoff treatment purposes, biofiltration swales and vegetative filter strips are to be designed to treat the 6-month, 24-hour design storm, as required by Minimum Requirement #4 (see Chapter I-2). *Note: This is a change from the previous edition of this manual. Formerly the design storm for biofilters was the 2-year, 24-hour event. The change has been made so that all runoff treatment BMPs will be designed in a consistent manner.*

III-6.2.2 Mechanisms of Pollutant Removal

Biofiltration swales and vegetative filter strips use similar pollutant removal mechanism, i.e., "biofiltration." The term "biofiltration" has been coined to describe the more-or-less simultaneous processes of filtration, infiltration, adsorption and biological uptake of pollutants in stormwater that take place when runoff flows over and through vegetated treatment facilities. Vegetation growing in these facilities acts as both a physical filter which causes gravity settling of particulates by regulating velocity of flow, and also as a biological sink when direct uptake of dissolved pollutants occurs. The former mechanism is probably the most important in western Washington where the period of major runoff coincides with the period of lowest biological activity.

Another means of removing pollutants occurs as the stormwater contacts the soil surface and infiltrates into the underlying soil. Dissolved pollutants are adsorbed onto soil particles. This is a potentially important removal mechanism for both dissolved heavy metals and phosphorus by undergoing ion exchange with elements in the soil. In addition, biological activity in the soil can metabolize organic contaminants. However, in highly porous soils stormwater can be a threat to shallow ground water since these soils have little treatment capacity. In such instances, biofilter BMPs must meet the General Limitations for infiltration BMPs (see Chapter III-3) or it may be necessary to install a liner to prevent infiltration.

The degree to which the above mechanisms operate will vary considerably depending upon many factors such as the depth and condition of the vegetation, the velocity of the water, the slope of the ground, and the texture of the underlying soil. However, the most important criterion that can be developed from these variables is the residence time of the stormwater in the biofilter, provided there is an adequate stand of vegetation and the underlying soil is of moderate texture. Therefore, to be effective, the biofilter must be designed such that the residence time is sufficient to permit most if not all of the particulates and at least some of the dissolved pollutants to be removed from the stormwater.

Design criteria that will maximize the effectiveness of biofiltration swales and strips are still in the developmental stage because their use for treating stormwater locally has only been applied and investigated for a relatively short time. They have been largely based on work done in the early 1980s by researchers at the University of Washington for the Washington State Department of Transportation and have relied heavily on the finding that total suspended solids and lead were reduced by at least 80 percent in 200 feet of grass swale (1).

The most recent comprehensive publication dealing with biofiltration systems locally was prepared in 1988 by Horner (2) and the reader is referred to this document for further details including a review of the literature and a survey of operating biofilters.

III-6.3 BMP RB.05 BIOFILTRATION SWALE

Purpose and Definition

A biofiltration swale is designed to provide runoff treatment of conventional pollutants but not nutrients. It does not provide streambank erosion control but can be designed to convey runoff to BMPs designed for that purpose. Biofiltration swales, when used as a primary treatment BMP, should be located "off-line" from the primary conveyance/detention system in order to enhance effectiveness (they can also be made smaller when located "off-line"). If a biofiltration swale is used to protect a water quality infiltration BMP or a sand filtration BMP (see Chapter III-3), then it will be necessary to locate it "off-line."

In cases where a biofiltration swale is located "on-line" it must be sized as both a treatment facility and as a conveyance system to pass the peak hydraulic flows of the 10 and 100-year design storm. To be effective, **the depth of the stormwater during treatment must not exceed the height of the grass.**

Planning Considerations

1. *Local governments should maintain the necessary flexibility in ordinances and regulations to permit site-by-site assessment of biofiltration alternatives, and to allow for discretionary design, installation, operating, and maintenance requirements, as long as they do not conflict with the general intent of design and maintenance requirements stated below.*

2. *Biofiltration should be regarded as one possible element of an integrated stormwater management plan for any given site or class of sites. Selection and implementation of alternatives should be based on stated water quality objectives (see Chapter I-4).*

3. *With diverse opportunities existing to apply the variety of biofilter configurations, a creative approach is recommended to obtain the best match of system and conditions.*

4. *Since biofiltration is an on-site rather than a regional technique, localized commitments must be made to maximize its application and effectiveness.*

5. *Since flexibility exists in many design features, biofiltration success depends more on proper construction and maintenance than any other factors; effective inspection and enforcement programs should be emphasized to ensure that approved plans are implemented.*

General Technical Recommendations

1. Natural drainage courses should be regarded as significant local resources that are generally to be kept in use for stormwater management, including biofiltration.

2. Roadside ditches should be regarded as significant potential biofiltration sites; road design standards and ditch maintenance programs should be developed to maximize their usefulness in biofiltration.

3. Local governments should resist proposals to enclose open channels in pipes. In addition to offering the opportunity for biofiltration, open channels generally have more capacity than pipes and are easier to inspect and maintain.

4. Retention/detention pond design requirements should recognize and assess the alternative of installing low-flow biofiltration swales within ponds where sufficient land does not exist for both.

5. Opportunities to fit biofiltration retroactively to areas already developed should be exploited whenever possible.

6. Biofilters should generally not receive construction-stage runoff; if they do, presettling of sediments should be provided (see BMPs E3.35 and E3.40 in Chapter II-5). Such biofilters should be evaluated for the need to remove sediments and restore vegetation following construction.

7. Biofilters should be protected from siltation by a permanent presettling basin when the erosion potential is high (see BMP RD.10 in Chapter III-4); otherwise, presettling is not generally needed for normal operation. However, a series arrangement of a retention/detention pond and biofilter has the ability to offer extra protection to a sensitive receiving water, due to the complementary pollutant removal mechanisms that can operate in the two devices.

8. Biofilters must be vegetated in order to provide adequate treatment of runoff. By definition, <u>bio</u>filters require vegetation, and rock-lined or vegetated channels are not biofilters.

Design Criteria

Overview

The design, planning, and operation and maintenance details that follow have been adapted directly from Horner's "general recommendations" with minor modifications, and while this is judged to be the best available information, it must be considered as interim and subject to modification. Alternative criteria is being investigated which may be reflected in future editions of this manual.

Questions remain about the nutrient-removing abilities of biofilters in the Pacific Northwest and further work needs to be done to resolve optimal geometry and slopes of swales (2). As this and other information becomes available, especially monitoring data and consequent new ideas on design, they will be incorporated into later editions of this manual.

In summary, the interim criteria have been selected to ensure that the velocity of water does not exceed 1.5 feet per second along a swale of 200 feet in length during the water quality design storm (the 6-month, 24-hour storm). Although the 1990 and 1991 versions of this manual used the 2-year, 24-hour storm, we have chosen to change it to the 6-month, 24-hour storm to make all BMP designs consistent. We do not feel that the decrease in cross-sectional area and residence time are such that the larger size storm design is necessary. An additional requirement for swales designed to convey larger storms (up to the 100-year, 24-hour event) is that the peak velocity for the maximum design storm is kept below erosive levels. Complete details of the criteria are given below, and the appendices give step-by-step procedures for designing strips and swales including an example calculation.

General Criteria

1. For biofiltration, it is important to maximize water contact with vegetation and the soil surface. Gravelly and coarse sandy soils cannot be used for biofiltration unless the bottom of the swale is lined to prevent infiltration. (Note: Sites that have relatively coarse soils may be more appropriate for stormwater infiltration for streambank erosion control purposes <u>after</u> runoff treatment has been accomplished. In any case the General Limitations in Chapter III-3 will dictate the use of coarse soils for stormwater management purposes). Also, avoid very heavy clay soils that will not support good vegetative growth.

2. Select vegetation on the basis of pollution control objectives and according to what will best establish and survive in the site conditions. Also, consider whether wildlife habitat development can occur in concert with pollution control. If so, consider the needs of such development in vegetation selection. For general purposes, select fine, close-growing, water-resistant grasses. Alternatively, where some period of soil saturation is expected, where particular pollutant uptake characteristics are desired, or both, select emergent wetland plant species. Protect these plants from predation during establishment by netting. See Appendix III-6.1 for specific vegetation selection recommendations.

3. Establish grasses as follows (all weights are per 1,000 square feet):

If hydro-seeding -	5 lb. seed mix 7 lb. 10-20-20 (N-P-K) fertilizer* 50 lb. wood cellulose fiber mulch
If broadcast seeding -	5 lb. seed mix 7 lb. 10-20-20 (N-P-K) fertilizer* 70 lb. wood cellulose fiber mulch

*Note: this is just an estimate of the amount of fertilizer necessary. Make certain that the proper amount of fertilizer for the soil type is used.

4. Based on observations in this area, select a grass height of 6 inches or less and a flow depth of less than 5 inches. Grasses over that height tend to flatten down when water is flowing over them, which prevents sedimentation. To attain this height requires regular maintenance.

5. Where grasses are to be cultivated, if possible, select an area where moisture is sufficient to provide water requirements during the dry season, but where the water table is not so high as to cause long periods of soil saturation. Irrigate if moisture is inadequate during summer drought. If saturation will be extended and/or the slope is minimal but grasses are still desired, consider subdrains. Alternatively, consider designing a constructed wetland or wet pond that has a substantially longer water residence time than a swale or filter strip (see Chapter III-4). Also see BMPs E1.35 and E1.40 in Chapter II-5 for more information on seeding and sodding.

6. The channel slope should normally be between 2 and 4 percent. A slope of less than 2 percent can be used if underdrains are placed beneath the channel to prevent ponding (Figure III-6.3). A slope of greater than 4 percent can be used if check dams (Figure III-6.4) are placed in the channel to slow the flows accordingly. (see Provisions for Swales #4, below).

7. If possible, divert runoff (other than necessary irrigation) during the period of vegetation establishment. This requirement can normally be met in the Pacific Northwest by planting during July or August. Sodding is an

alternative when rapid establishment must occur. Where runoff diversion is not possible, cover graded and seeded areas with a suitable erosion control slope covering material (see Chapter II-5).

8. Prevent bare areas in biofilters by avoiding gravel, rocks, and hardpan near the surface; fertilizing, watering, and replanting as needed; and ensuring effective drainage. Note: Fertilizer must only be used at an application rate and formula which is compatible with plant uptake, and in relation to soil type. For example, high application rates of nitrogenous fertilizer in very permeable soils can result in leaching of nitrate into ground water.

9. If flow is to be introduced via curb cuts, place pavement slightly above the biofilter elevation. Curb cuts should be at least 12 inches wide to prevent clogging.

10. Attempt to avoid compaction during construction. If compaction occurs, till before planting to restore lost soil infiltration capacity.

Specific Criteria for Biofiltration Swales

1. Design swales for hydraulic capacity and stability according to the method detailed in Appendix AIII-6.1. Base the capacity design for biofiltration on the vegetation height equal to the design flow depth and the 6-month frequency, 24-hour duration storm. Unless runoff from larger events will bypass the swale, base the capacity design for flood passage on the 100-year frequency, 24-hour duration storm, plus 1 foot freeboard (Figure III-6.5).

2. Base the design on a trapezoidal cross-section for ease of construction. A parabolic shape will evolve over time. Make side slopes no steeper than 3 horizontal:1 vertical.

3. Provide a minimum of 200 feet of swale, using a wide-radius curved path, where land is not adequate for a linear swale (avoid sharp bends to reduce erosion or provide for erosion protection). If a shorter length must be used, increase swale cross-sectional area by an amount proportional to the reduction in length below 200 feet, in order to obtain the same water residence time.

4. Install log or rock check dams approximately every 50 feet, if longitudinal slope exceeds 4 percent. Adjust check dam spacing in order not to exceed 4 percent slope within each channel segment between dams.

5. Below the design water depth, install an erosion control blanket, at least four inches of topsoil, and the selected biofiltration seed mix. Above the design water line, use an erosion control seed mix with straw mulch or sod (see BMP E1.15 in Chapter II-5).

Construction and Maintenance Criteria

Construction

See Appendix AIII-6.1.

Maintenance

- Groomed biofilters planted in grasses must be mowed regularly during the summer to promote growth and pollutant uptake. Be sure not to cut below the design flow (**maintenance personnel must be made aware of this requirement**). Remove cuttings promptly, and dispose in a way so that no pollutants can enter receiving waters.

- If the objective is prevention of nutrient transport, mow grasses or cut emergent wetland-type plants to a low height at the end of the growing season. For other pollution control objectives, let the plants stand at a height exceeding the design water depth by at least two inches at the end of the growing season.

- Remove sediments during summer months when they build up to 6 inches at any spot, cover biofilter vegetation, or otherwise interfere with biofilter operation. Use of equipment like a Ditch Master is strongly recommended over a backhoe or dragline. If the equipment leaves bare spots, re-seed them immediately.

- Inspect biofilters periodically, especially after periods of heavy runoff. Remove sediments, fertilize, and reseed as necessary. Be careful to avoid introducing fertilizer to receiving waters or ground water.

- Clean curb cuts when soil and vegetation buildup interferes with flow introduction.

- Perform special public education for residents near biofilters concerning their purpose and the importance of keeping them free of lawn debris.

- See that litter is removed in order to keep biofilters attractive in appearance.

- Base roadside ditch cleaning on an analysis of hydraulic necessity. Use a technique such as the Ditch Master to remove only the amount of sediment necessary to restore needed hydraulic capacity, leaving vegetative plant parts in place to the maximum extent possible.

III-6.4 BMP RB.10 VEGETATIVE FILTER STRIP

Purpose and Definition

A vegetative filter strip is designed to provide runoff treatment of conventional pollutants but not nutrients. This BMP is not designed to provide streambank erosion control. Also, unlike a biofiltration swale, a vegetative filter strip should not be used for conveyance of larger storms because of the need to maintain sheet flow conditions, plus the filter strip would likely be prohibitively large for this application.

Planning Considerations

See BMP RB.05, Biofiltration Swale. Additional planning considerations are provided below.

Application

Vegetative filter strips can be effective at pretreating runoff to protect infiltration and filtration BMPs from siltation. It may also be a viable treatment BMP for small, less intensely developed sites. The maximum recommended drainage area for a vegetative filter strip is 5 acres. Vegetative filter strips must not receive concentrated flow discharges as their effectiveness will be destroyed plus the potential for erosion could cause filter strips to become sources of pollution.

Slope

Vegetative filter strips should not be used on slopes greater than about 10 percent because of the difficulty in maintaining the necessary sheet flow conditions. Note: This does not mean that vegetated buffers are not suitable for slopes greater than

10 percent; it simply means that effective treatment of runoff is unlikely for slopes greater than 10 percent. Do not confuse a "buffer zone," which is used to protect streams and other environmental resources, with a "vegetative filter strip," which is a runoff treatment BMP.

Design Criteria

The design, planning, and operation and maintenance details that follow have been adapted directly from Horner's "general recommendations" with minor modifications, and while this is judged to be the best available information, it must be considered as interim and subject to modification. Alternative criteria is being investigated which may be reflected in future editions of this manual. Questions remain about the nutrient-removing abilities of biofiltration BMPs in the Pacific Northwest and further work needs to be done. As information becomes available, especially monitoring data and consequent new ideas on design, they will be incorporated into later editions of this manual.

In summary, an interim criteria have been selected to ensure that a residence time of 20 minutes for the water as it flows across (perpendicular to) the strip. Complete details of the criteria are given below, and the appendices give step-by-step procedures for designing strips and swales including an example calculation.

General Criteria

See BMP RB.05, Biofiltration Swale.

Specific Criteria for Vegetative Filter Strips

1. Design vegetative filter strips according to the same method detailed in Appendix AIII-6.1 for biofiltration swales. Calculate the necessary filter strip width (perpendicular to flow) on the basis of the 6-month frequency, 24-hour duration storm and a hydraulic radius (R) approximately equal to the design flow depth (y). *Note: The design flow depth (y) will normally be no more than 0.5" (0.04 ft) because of the need to maintain sheet flow over the strip)*

2. Calculate the necessary length (parallel to flow) to produce a water residence time of at least 20 minutes (the length should normally be in the range of 100-200 feet).

3. Install a shallow stone trench across the top of the strip to serve as a level spreader or make use of curb cuts in a parking lot. Make provisions to avoid flow bypassing the filter strip.

4. Vegetative filter strips should not be used for slopes in excess of 10 percent, and preferably less, because of the difficulty in maintaining the necessary sheet flow conditions.

Construction and Maintenance

See BMP RB.05, Biofiltration Swale.

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APPENDIX AIII-6.1
DESIGN PROCEDURE FOR BIOFILTRATION SWALE AND VEGETATIVE FILTER STRIP DESIGN

Introduction

This section has been adapted with minor modifications from Appendix D - Application Guide of "Biofiltration Systems for Storm Runoff Water Quality Control" by Dr. Richard R. Horner (2).

This guide provides biofilter design procedures in full detail, along with examples. It can be removed from the manual for convenient use alone, if desired. Refer to Sections III-6.3 and III-6.4 for design criteria and operation and maintenance details.

Procedure

Note: The procedures for swale and filter strip design are basically the same. The steps are given in full for swales, and notes are included to allow the procedure to be applied to filter strips as well. Unless specifically indicated, steps apply to both filter strips and biofilters.

Preliminary Steps (P)

Step #

P-1. Estimate runoff flow rate (Q) for the 6-month frequency, 24-hour duration storm, according to methods outlined in Chapter III-1.

P-2. Biofilters should normally be placed on slopes of 2 to 4 percent. If it can be demonstrated that adequate drainage to avoid persistent pooling will occur (using underdrains, if necessary), a slope less than 2 percent can be used. If the site slope exceeds 4 percent, the local government should make a determination of the site's suitability for a biofilter, and, if suitable, what special design features should be included. If the slope exceeds 6 percent, it is recommended that the biofilter traverse the slope or that the site topography be modified to produce a slope under 6 percent. If stepped, each section should slope at less than 6 percent. In any swale application with slope greater than 4 percent, check dams should be placed approximately every 50 feet.

P-3. Select a vegetation cover suitable for the site.

Refer to Table III-6.1 to select grasses. If the site will be persistently wet, consider wetland genera such as Typha (cattails), Scirpus (bulrushes), and Lemna (duckweed), which have relatively high rates of pollutant uptake. Other wetland plants that have been observed to serve well in biofilters are Carex (sedges), and water cresses (A. Levesque, King County, personal communication). If development of wildlife habitat is an objective, consider habitat needs in selecting vegetation.

Table III-6.1.
Characteristics of Grasses Suitable for Lining Puget Sound Region Biofilters. (a)

Common Name	Persistence/ Growth Form	Description	Rating (b)
Annual ryegrass or Italian ryegrass	Annual/bunchgrass	Common erosion control grass; establishes rapidly on bare soils but does not reseed well.	3
Kentucky bluegrass	Perennial/sod-forming	Common turf grass; may require irrigation in dry season. May need regular reseeding.	3
Tall fescue	Perennial/ bunchgrass	Common turf grass; can be used alone; may require irrigation in dry season.	4
Western wheatgrass	Perennial/ sod-forming	Tolerates drought	3

a. Adapted from Goldman et al. (3). Other recommended grasses and legumes:

Meadow foxtail	Creeping red fescue	Annual ryegrasses
Tall fescue	Timothy	White clover
Redtop	Seaside colonial bentgrass	

Other water-resistant grasses that grow well in regional conditions are Poa trivialis (roughstalk bluegrass) and Lolium perenne (perennial ryegrass) (West. D., Seattle City Light, personal communication).

The seeding mix specified for the parking lot swales at the West Willows Technical Center in Redmond was as follows:

52% perennial rye
35% winter rye
13% clover

Shapiro and Associates recommends the following seeding mix for this application (Gorski A., Shapiro and Associates, personal communication):

40% redtop bentgrass
30% red fescue
20% tall fescue
5% perennial rye
5% Russian wild rye

b. Ratings are for erosion protection: 1 - fair; 2 - good; 3 - excellent; 4 - superior.

Design for Biofiltration Capacity

Note: There are a number of ways of applying the design procedure introduced by Chow (4). These variations depend on the order in which steps are performed, what variables are established at the beginning of the process and which ones are calculated, and what values are assigned to the variables selected initially. The procedure recommended here is an adaptation appropriate for biofiltration applications of the type being installed in the Puget Sound region. This procedure reverses Chow's order, designing first for capacity and then for stability. The capacity analysis emphasizes the promotion of biofiltration, rather than transporting flow with the greatest possible hydraulic efficiency. Therefore, it is based on criteria that promote sedimentation, filtration, and other pollutant removal mechanisms. Since these criteria include a lower maximum velocity than permitted for stability, the biofilter dimensions usually do not have to be modified after a stability check.

Design Steps (D)

Step #

D-1. Establish the height of vegetation during the winter and the design depth or flow. Maximizing height advances biofiltration and allows greater flow depth, which reduces the width necessary to obtain adequate capacity. However, if nutrient capture is the principal objective, vegetation should be mowed at the end of the growing season to minimize nutrient release. The design depth of flow should be at least two inches less than the winter vegetation height. **Note: Sheet flow (<1 inch deep) generally exists in vegetative filter strips (use 0.5 inch).**

D-2. Select a value of Manning's n. Use one of the following values for an initial analysis (after U.S. Department of Commerce, (5)), or refer to Table III-2.8 in Chapter III-2.

Dense grass up to 6 inches tall - 0.07
Vegetation with coarser stems (e.g., wetland plants, woody plants) - 0.07

D-3. Select the swale shape. (Skip this step in filter strip design.) Use a trapezoidal shape for biofilter swales, as is feasible.

Rectangular and V-shapes are the least desirable from the stability standpoint. If one of these shapes is required by the site configuration, specify reinforcement for the side walls in conformance with the standards of the local government.

D-4. Use Manning's equation and first approximations relating hydraulic radius and dimensions for the selected shape to obtain a working value of a biofilter width dimension:

$$Q = \frac{1.486}{n} A R^{0.667} s^{0.5} \qquad (6\text{-}1)$$

Where:
- Q = design runoff flow rate (ft³/s, cfs)
- n = Manning's n (dimensionless)
- A = Cross-sectional area (ft²)
- R = Hydraulic radius = A/wetted perimeter (ft)
- s = longitudinal slope as a ratio of vertical rise/horizontal run (dimensionless)

Refer to Figure III-6.5 to obtain equations for A and R for the selected shape. In addition to these equations, for a rectangular shape:

$$A = Ty \qquad (6\text{-}2)$$

$$R = \frac{Ty}{T+2y} \qquad (6\text{-}3)$$

where: T = width
y = depth of flow in feet, expressed as a decimal

If these expressions are substituted in Equation 6-1 and solved for T (for previously selected y), the results are complex equations that are difficult to solve manually. However, approximate solutions can be found by recognizing that T>>y and z^2>>1, and that certain terms are nearly negligible. The approximations for the various shapes are:

Parabolic: $R \approx 0.67\ y$ (6-4)

Trapezoidal: $R \approx y$ (6-5)

V: $R \approx 0.5\ y$ (6-6)

Rectangular: $R \approx y$ (6-7)
(Also use for vegetative filter strips)

Making these substitutions and those for A from Figure III-6.5, and then solving for T gives:

Parabolic: $T \approx \dfrac{Qn}{0.76\ y^{1.667}\ s^{0.5}}$ (6-8)

Trapezoidal: $b \approx \dfrac{Qn}{1.486\ y^{1.667}\ s^{0.5}} - Zy$ (6-9)

V: $T \approx \dfrac{Qn}{0.47\ y^{0.667}\ s^{0.5}}$ (6-10)

Rectangular: $T \approx \dfrac{Qn}{1.486\ y^{1.667}\ s^{0.5}}$ (6-11)

(Also use for vegetative filter strips.)

For trapezoidal and V-shapes, select a side slope Z of at least 3.

Solve the appropriate equation for T or b. For a V-shape, check if Z = T/2y is at least 3. For a trapezoid, compute b (Step D-4a) and then top width T, where T = b + 2yZ (Step D-4b).

D-5. Compute A using the appropriate equation from Figure III-6.5 or Equation 6-2.

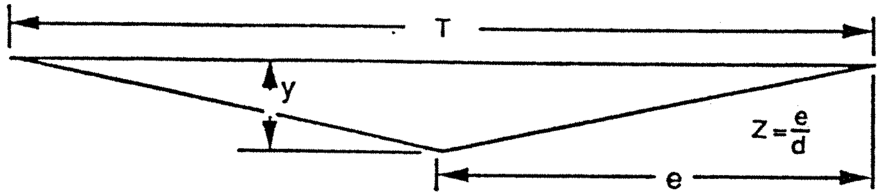
D-6. Compute the flow velocity at design flow rate:

$$V = \frac{Q}{A} \qquad (6\text{-}12)$$

This velocity should be less than 1.5 ft/s, a velocity that was found to permit the sedimentation of most particles in typical urban runoff (see (2)). However, the smallest particles (clay and much of the silt fraction) may not be removed. Also, it is not known what velocity will cause grasses to be knocked from a vertical position, thus reducing filtration. Therefore, the velocity should be as low as space allows.

CHANNEL GEOMETRY

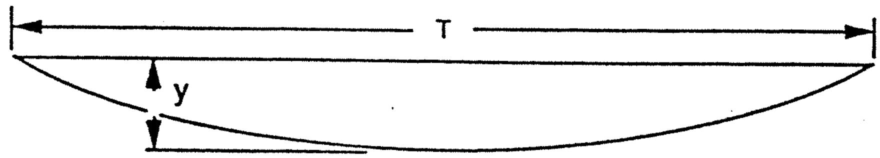
V - Shape



Cross-Sectional Area (A) = Zy^2
Top Width (T) = $2yZ$

Hydraulic Radius (R) = $\dfrac{Zy}{2\sqrt{Z^2+1}}$

Parabolic Shape.

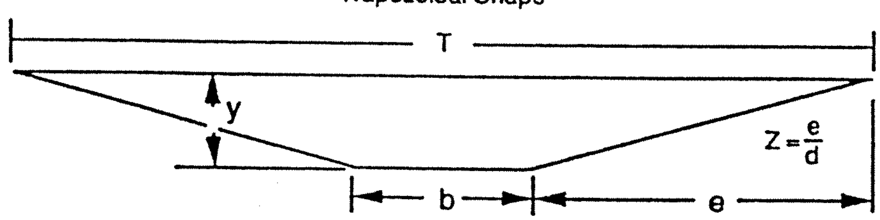


Cross-Sectional Area (A) = $\frac{2}{3}Ty$

Top Width (T) = $\dfrac{1.5A}{y}$

Hydraulic Radius (R) = $\dfrac{T^2y}{1.5T^2 + 4y^2}$

Trapezoidal Shape



Cross-Sectional Area (A) = $by + Zy^2$
Top Width (T) = $b + 2yz$

Hydraulic Radius (R) = $\dfrac{by + Zv^2}{b + 2y\sqrt{Z^2+1}}$

Figure III-6.5 Geometric Formula for Common Swale Shapes (from Livingston et al., 1984).

If V > 1.5, repeat steps D-1 to D-6 until the condition is met.

D-7. This approximate analysis tends to produce a design that results in V < 1.5, often by a substantial margin. This situation is preferred if sufficient space is available. If that is the case, proceed to the stability check. IF NOT, perform a more exact analysis according to steps D-8 to D-15, otherwise go to Step D-16.

D-8. Estimate the degree of retardance to flow created by the selected vegetation from Table III-6.2. When uncertain, be conservative by selecting a relatively high degree.

Table III-6-2. Guide for Selecting Degree of Retardance (a).

Coverage	Average Grass Height (inches)	Degree of Retardance	
Good	2-6	D.	Low
	<2	E.	Very low
Fair	2-6	D.	Low
	<2	E.	Very low

a. After Chow (4). In addition, Chow recommended selection of retardance D for a grass-legume mixture 4-5 inches high. No retardance recommendations have appeared for emergent wetland species. Therefore, judgment must be used. Since these species generally grow less densely than grasses, using a "fair" coverage would be a reasonable approach.

D-9. Refer to Figure III-6.6 and use the selected degree of retardance and Manning's n from step D-2 to obtain a first approximation of VR, the product of velocity and hydraulic radius.

D-10. Compute hydraulic radius, using V_{max} = 1.5 ft/s:

$$R = \frac{VR}{V_{max}} \qquad (6\text{-}13)$$

D-11. Use Manning's equation to solve for the actual VR associated with this R and n:

$$VR = \frac{1.486}{n} R^{1.667} s^{0.5} \qquad (6\text{-}14)$$

where VR is in units of ft^2/sec

D-12. Compare the actual VR from step D-11 and the first approximation of VR from step D-9. If they do not agree within 5 percent, select a new n and repeat steps D-9 to D-12 until acceptable agreement is reached.

D-13. Compute the actual V for the final design conditions (using the actual VR calculated in Step D-11):

$$V = \frac{VR}{R} \qquad (6\text{-}15)$$

Check to be sure V <1.5 ft/s.

D-14. Use the continuity equation to calculate the flow cross-sectional area (A):

$$A = \frac{Q}{V} \qquad (6\text{-}16)$$

D-15. Use the appropriate equation in Figure III-6.5 or Equation 6-2 to compute T or b. For trapezoidal and V-shapes, use a Z of at least 3, and for trapezoids use T = b+2yZ.

D-16. If there is still not sufficient space for the biofilter, the local government and the project proponent should consider the following solutions (listed in order of preference):

a. Divide the site drainage to flow to multiple biofilters.

b. Use infiltration to provide lower discharge rates to the biofilter (<u>only</u> if the criteria and General Limitations in Chapter III-3 are met).

c. Increase vegetation height and design depth of flow (note: the design must ensure that vegetation remains standing during design flow).

d. Reduce the developed surface area to gain space for biofiltration.

e. Increase the longitudinal slope.

f. Increase the side slopes.

Proceed to the stability check.

Check for Stability (Minimizing Erosion)

Notes:

(1) The stability check must be performed for the combination of highest expected flow and least vegetation coverage and height.

(2) Maintain the same units as in the biofiltration capacity analysis.

Stability Check Steps (SC)

(Note: Not required for biofiltration BMPs which are located "off-line" from the primary conveyance/detention system, i.e., when flows in excess of the peak flow for the 6-month, 24-hour design storm bypass the biofilter. This is the desired configuration.)

Step #

SC-1. Unless runoff from events larger than the 6-month, 24-hour storm will bypass the biofilter, perform the stability check for the 100-year, 24-hour storm. Estimate Q for that event as recommended in Preliminary step P-1.

SC-2. Estimate the vegetation coverage ("good" or "fair") and height on the first occasion that the biofilter will receive flow, or whenever the coverage and height will be least. Attempt to avoid flow introduction during the vegetation establishment period by timing of planting or bypassing.

SC-3. Estimate the degree of retardance from Table III-6.2. When uncertain, be conservative by selecting a relatively low degree.

SC-4. Establish the maximum permissible velocity for erosion prevention (V_{max}) from Table III-6.3.

Table III-6.3
Guide for Selecting Maximum Permissible
Swale Velocities for Stability Check (a)

Cover	Slope (%)	Maximum Velocity (ft/sec)
Kentucky Bluegrass Tall Fescue	0 - 5	5
Kentucky Bluegrass Tall Fescue Western Wheatgrass	5 - 10	4
Grass-legume Mixture	0 - 5 5 - 10	4 3
Red Fescue Redtop	0 - 5 5 - 10	2.5 Not Recommended

(a) Adapted from references 3, 4, and 6.

SC-5. Select a trial Manning's n. The minimum value for poor vegetation cover and low height (possibly, knocked from the vertical by high flow) is 0.033. A good initial choice under these conditions is 0.04.

SC-6. Refer to Figure III-6.6 to obtain a first approximation for VR.

SC-7. Compute hydraulic radius, using the Vmax from step SC-4:

$$R = \frac{VR}{V_{max}} \qquad (6\text{-}13)$$

SC-8. Use Manning's equation to solve for the actual VR:

$$VR = \frac{1.486}{n} R^{1.667} s^{0.5} \qquad (6\text{-}14)$$

SC-9. Compare the actual VR from step SC-8 and first approximation from step SC-6. If they do not agree within 5 percent, repeat steps SC-5 to SC-9 until acceptable agreement is reached.

SC-10. Compute the actual V for the final design conditions:

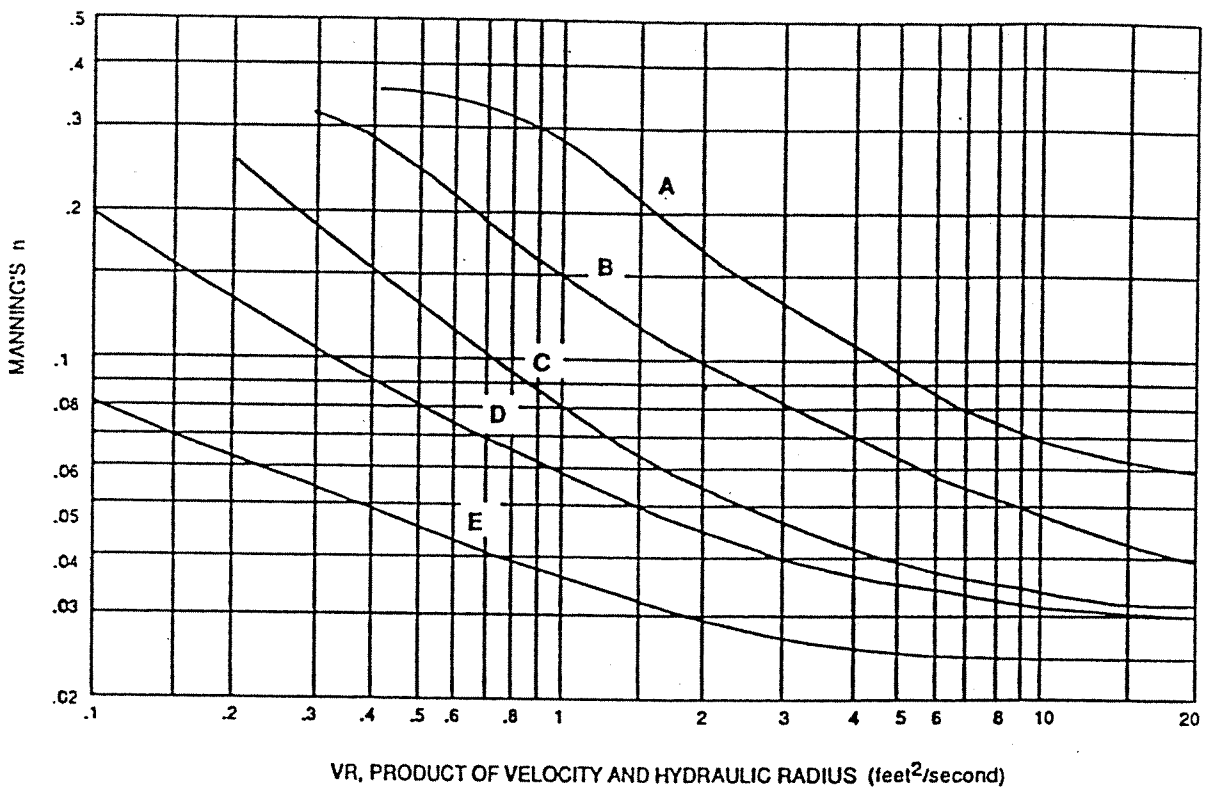
$$V = \frac{VR}{R} \qquad (6\text{-}15)$$

Check to be sure $V < V_{max}$ from step SC-4.

SC-11. Compute the required A for stability:

$$A = \frac{Q}{V} \qquad (6\text{-}16)$$

Figure III-6.6
The Relationship of Manning's n with VR for Various Degrees of Flow Retardance (from Livingston et al., 1984, after U.S. Soil Conservation Service, 1954)



SC-12. Compare the A computed in step SC-11 of the stability analysis with the A from the biofiltration capacity analysis (step D-5 or D-14).

If less area is required for stability than is provided for capacity, the capacity design is acceptable. If not, use A from step SC-11 of the stability analysis and recalculate channel dimensions (refer to Figure III-6.5 or Equation 6-2). Use y from Step D-1.

SC-13. Calculate the depth of flow at the stability check design flow rate condition for the final dimensions (refer to Figure III-6.5 or Equation 6-2). (For trapezoids use $y = (T-b)/2Z$)

SC-14. Compare the depth from step SC-13 to the depth used in the biofiltration capacity design (Step D-1). Use the larger of the two and add 1 foot freeboard to obtain the total depth (y_t) of the swale. Skip this step in filter strip design. (Editor's Note: If space is limited, calculate the depth needed for the 100-year, 24-hour storm then add this depth again for freeboard, up to a maximum freeboard of 1 foot.)

SC-15. Recalculate the hydraulic radius (trapezoidal channel - see Figure III-6.5):

$$R = \frac{by_t + zy_t^2}{b + 2y_t(Z^2 + 1)^{0.5}}$$

(use b from Step D-4 or D-15 calculated previously for biofiltration capacity, or Step SC-12, as appropriate, and y_t = total depth from Step SC-14)

SC-16. Make a final check for capacity based on the stability check design storm and maximum vegetation height and cover (this check will ensure that capacity is adequate if the largest expected event coincides with the greatest retardance). Use Equation 6-1, a Manning's n of 0.1, and the calculated channel dimensions, including freeboard, to compute the flow capacity of the channel under these conditions. Use R from step SC-15, above, and $A = by_t + Zy_t^2$ using b from Step D-4a, or D-15 or SC-12, as appropriate.

If the flow capacity is less than the stability check design storm flow rate, increase the channel cross-sectional area as needed for this conveyance. Specify the new channel dimensions.

<u>Completion Steps (CO)</u>

Step #

CO-1. If the biofilter is a swale, lay out the swale to obtain the maximum possible length. This length should be at least 200 feet. In limited spaces, attempt to attain that length by using a curved path. Use the widest radius bends possible to reduce the potential for erosion of the outside of curved sections. If a length shorter than 200 ft. must be used, increase A by an amount proportional to the reduction in length below 200 ft., in order to obtain the same water residence time. Recalculate channel dimensions from Figure III-6.5 or Equation 6-2.

If the swale is a vegetative filter strip, select a length for the calculated width that produces at least 20 minutes water residence time (normally 100-200 feet).

CO-2. If the swale longitudinal slope is greater than 4 percent, design log or rock check dams approximately every 50 feet.

APPENDIX AIII-6.2
EXAMPLE PROBLEM SHOWING APPLICATION OF DESIGN PROCEDURE FOR BIOFILTRATION SWALES AND VEGETATIVE FILTER STRIPS

Preliminary Steps

P-1. Assume that Q for the 6-month, 24-hour storm was established by one of the recommended procedures to be 3 cfs.

P-2. Assume the slope (s) is 2 percent.

P-3. Assume the vegetation will be a grass-legume mixture, with the dominant grass being red fescue.

Design for Swale Biofiltration Capacity

D-1. Set the winter grass height at 6 inches and design flow depth (y) at 4 inches (i.e. 0.33 feet) (Eq. 6-9). Recall that the design flow must be at least two inches less than the winter grass height.

D-2. Use n = 0.07

D-3. Base the design on a trapezoidal shape, with side slope (Z) equal to 3.

D-4a. Calculate the bottom width (b)

Where: n = 0.07

Q = 3 cfs
y = 0.33'
s = 0.02
Z = 3

$$b = Qn/(1.486y^{1.667}s^{0.5}) - Zy \qquad (6\text{-}9)$$

or

$$b = 5.24 \text{ feet}$$

D-4b. Calculate the top width (T)

$$T = b + 2yZ = 5.24 + [2(0.33)(3)] = 7.24 \text{ feet}$$

D-5. Calculate the cross-sectional area (A)

$$A = by + Zy^2 = (5.24)(0.33) + (3)(0.33^2) = 2.06 \text{ ft}^2$$

(from Fig. III-6.5)

D-6. Calculate the flow velocity (V)

$$V = Q/A = \frac{3}{2.06} = 1.46 \text{ ft/s} < 1.5, \text{ so OK} \qquad (6\text{-}12)$$

Proceed directly to stability check.

A top width of 6 to 10 feet is typical of many swales surveyed in the area, and should fit within most sites. For the example, assume that it does so. The calculation procedure of steps SC-8 through 15 will be demonstrated in the stability check.

Check for Channel Stability

SC-1. Base the check on passing the 100-year, 24-hour storm runoff flow through the swale. Assume that Q for that storm was established by one of the recommended procedures to be 16 cfs.

SC-2. Base the check on a grass height of 3 inches with "fair" coverage (lowest mowed height and least cover, assuming flow bypasses or does not occur during grass establishment).

SC-3. Table III-6.2: Degree of retardance = D (low)

SC-4. From Table III-6.3, set V_{max} = 3 ft/sec since the vegetation is a combination of red fescue (V_{max} = 2.5 ft/sec) and legumes (V_{max} = 4 ft/sec).

SC-5. Select trial Manning's n = 0.04

SC-6. Figure III-6.6 VR = 3 ft^2/s

SC-7. Eq. 6-13 $R = \frac{VR}{V_{max}}$

$R = 1.0$ ft

SC-8. Eq. 6-14 $VR = \frac{1.486}{n} R^{1.667} s^{0.5}$

$VR = 5.25$ ft^2/sec

SC-9. VR from step SC-8 <VR from step SC-6 by > 5%.

Select new trial n = 0.047

from Figure III-6.6 VR = 1.7 ft^2/s

Eq. 6-13 R = 0.57 ft.

Eq. 6-14 VR = 1.75 ft^2/s (within 5% of VR = 1.7)

SC-10. Eq. 6-15 $V = VR/R = 1.75/0.57$

$V = 3.07$ ft/s <5 ft/s (OK)

SC-11. Eq. 6-16 $A = Q/V = 16/3.07 = 5.21$ ft^2

SC-12. For stability check, A = 5.21 ft^2 from Step SC-11, which is greater than the capacity from Step D-5 (2.06 ft^2). Therefore, recalculate channel dimensions using A from Step SC-11 and referring to Figure III-6.5.

$$A = by + Zy^2$$

where: A = 5.21 ft^2
Z = 3
y = ?
b = ?

(Note: both depth and width dimensions can be varied to obtain needed value of A, which is 5.21 ft^2 in this example.)

For this example, choose y = 0.67 ft. (note that y was originally set at 0.33 ft. in Step D-1) then calculate value for b.

For y = 0.67 ft., b = 5.81 ft.
T = b + 2yZ = 9.81 ft.

SC-13. Calculate depth of flow at the stability design flow rate condition.

For trapezoids use y = (T-b)/2Z from Figure III-6.5, and b = 5.81 ft and T = 9.8 ft from Step SC-12.

$$y = (9.81 - 5.81)/6 = 0.67 \text{ ft.}$$

SC-14. The value for y calculated in SC-13 (0.67 ft.) is greater than that used in Step D-1. Use the greater value, and add 1 foot freeboard to give a total depth (y_t) of 1.67 feet.

SC-15. Recalculate hydraulic radius (R) where

b = 5.81 ft (from Step SC-12)
y_t = 1.67 ft (from Step SC-14)
Z = 3 (from Step D-3)

$$R = \frac{by_t + Zy_t^2}{b + 2y_t\,(Z^2 + 1)^{0.5}} = 1.1 \text{ feet}$$

SC-16. Recalculate Q where:

$$Q = \frac{1.486}{n}\,AR^{0.667}\,s^{0.5} \qquad \text{(Eq. 6-1)}$$

where: n = 0.07
A = $by_t + Zy_t^2$, using b from Step SC-12
R = 1.1 feet (from Step SC-15)
s = 0.02 (from Step P-2)

$$A = (5.81)(1.67) + (3)(1.67^2) = 18.1 \text{ ft}^2$$

$$Q = \frac{1.486}{0.07}\,(18.1)\,(1.1)^{0.667}\,(0.02)^{0.5} = 57.9 \text{ cfs}$$

This is > 16 cfs for 100-year, 24-hour storm if it coincides with maximum flow retardance. Therefore, channel dimensions are okay.

Completion Steps

CO-1 Assume 200 feet of swale length is available. The final channel dimensions are:

Bottom width = 5.81 feet
Depth = 1.67 feet
Top width = b + 2yZ = 15.8 feet

CO-2 No check dams are needed for a 2% slope.

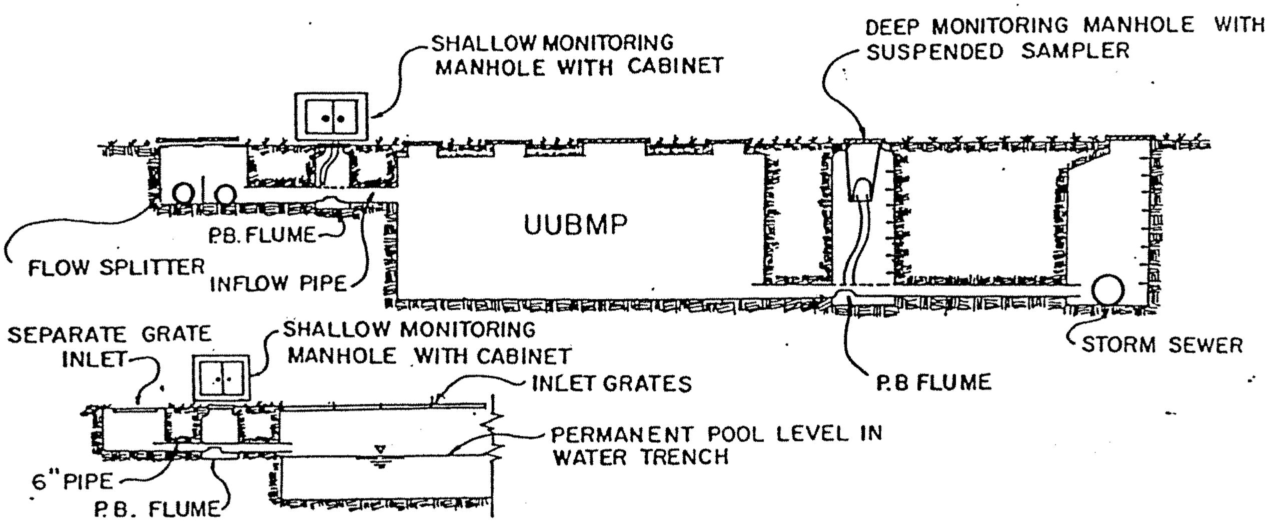
APPENDIX 2-8--STANDARD OUTFITTING OF BMPS FOR MONITORING

I. BACKGROUND

Alexandria allows the use of the innovative Ultra-Urban BMPs discussed in Chapter 2 of this manual and recognizes the phosphorous removal efficiencies discussed in Chapter 1 on the condition that the developer outfit the BMP for monitoring actual pollutant removal performance. Unlimited access by the City and its contractors for the purpose of monitoring is also required. When use of BMPs which the Director determines to be experimental are proposed, the applicant must agree to monitor and demonstrate the actual pollutant removal performance at the developer's expense.

II. REQUIREMENTS

Ultra-urban and experimental BMPs shall be outfitted with accessible points for installation of automatic monitoring equipment for measuring the flow rate and chemical composition of both the inflow water and treated effluent. These points shall be isolated from influence by large-storm bypass mechanisms. Unless otherwise approved by the Director, these accessible points shall be separate manholes equipped with Palmer-Bolus Flumes and installed in the inflow and treated effluent pipes of the BMP. **Figure 2-A8-1** illustrates the required configuration.



Inflow Configuration for
Delaware Sand Filter

FIGURE 2-A8-1--CONFIGURATION OF MONITORING OUTFITTING

A) Manholes

Manholes may be either purpose-built or precast concrete or fiberglass prefabricated monitoring units. If purpose built, they shall conform to Alexandria Construction Standard CSMH-1. If precast concrete, they shall conform to Alexandria Construction Standard CSMA-2 or CSMA-2A. The Director will entertain proposals to utilize commercial prefabricated monitoring manholes where not precluded by load or other engineering considerations. Each monitoring manhole shall have built-in ladder access and have a 4-inch Palmer-Bolus Flume installed in the flow line. **Figure 2-A8-2** illustrates the required configuration.

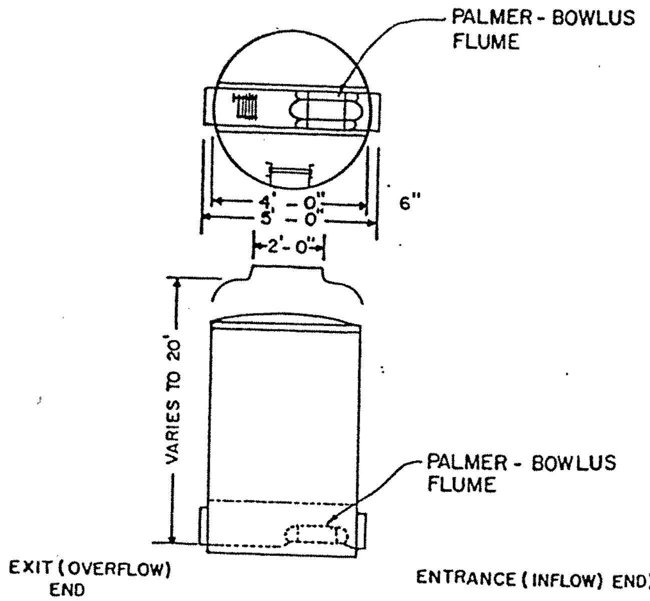


FIGURE 2-A8-2--MANHOLE WITH PALMER-BOLUS FLUME

B) Palmer-Bolus Flumes

The 4-inch Palmer-Bolus Flume shall be molded of fiberglass and reinforced polyester resin with a built-in cavity and support bracket for an American Sigma electronic sensor. The inside surface shall be smooth and free of irregularities. The entrance and exit ends shall be U-shaped, and the flume shall have an inside radius the same as the inside radius of the pipeline in which it is installed or (for oversized pipelines) the entrance and exit ends shall be supplied with end bulkheads to fit into a circular channel having a radius the same as the pipeline in which it is installed. The approach channel slope <u>must</u> be less that two (2) percent. The flume itself <u>must</u> be installed exactly level. It is highly recommended that the manufacturer be consulted before choosing the exact flume due to variations in flow rate.

C) Monitoring Equipment Cabinets

Automatic sampling and flow monitoring equipment will be suspended by a sling inside deep manholes. For monitoring manholes less than six (6) feet deep, the Director may require the provision of a matching prefabricated monitoring equipment cabinet. **Figure 2-A8-3** illustrates such a cabinet.

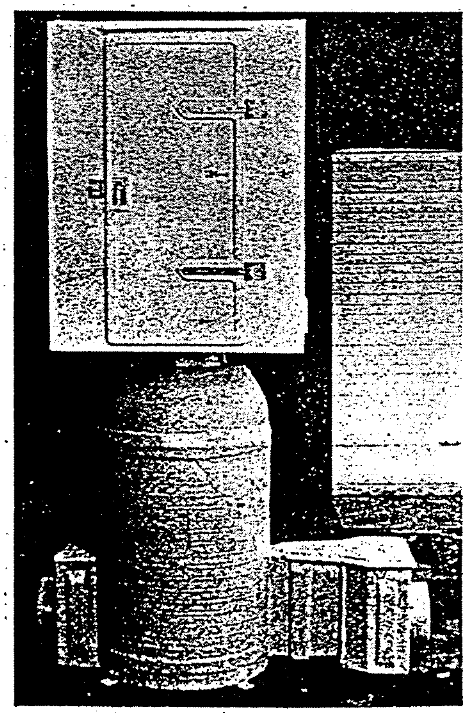


FIGURE 2-A8-3--PHOTOGRAPH OF PREFABRICATED MONITORING MANHOLE WITH MATCHING EQUIPMENT CABINET

Upon request, the Transportation and Environmental Services staff will furnish information on known manufacturers of prefabricated monitoring facilities and equipment.

RECEIVED 8/28/95

City of Alexandria, Virginia BIORETENTION FACILITIES

MEMORANDUM

ATE:	JANUARY 11, 1995
O:	DEVELOPERS, ENGINEERS, ARCHITECTS
ROM:	WARREN BELL, CITY ENGINEER, T&ES Warren Bell
UBJECT:	STORMWATER BEST MANAGEMENT PRACTICES POLICY FOR RESIDENTIAL DEVELOPMENTS

ACKGROUND

Chapter 2 of the Alexandria Supplement to the Northern Virginia MP Handbook requires that conventional BMPs (extended dry detention, et ponds, or infiltration facilities) be utilized on development rojects in Alexandria except where the Director of Transportation and nvironmental Services (T&ES) determines that their use is not easible. Since adoption of the City's Chesapeake Bay Preservation rdinance in 1992, the Director has allowed the use of underground ault sand filter systems on several fairly small (1-2 acre) townhouse edevelopment projects in the heavily built-up ("ultra-urban") areas f the City. New information on the technical nature and costs of nderground sand filter maintenance and the impact of the very stringent safety restrictions in the 1993 OSHA Confined Space Entry Regulations have raised serious questions on the suitability of these devices on residential projects for which homeowners associations must assume responsibility for future operations and maintenance. Accordingly, the following policy guidance is issued for use in planning and designing residential development projects in Alexandria.

POLICY

The general policy for residential development in Alexandria is that either conventional BMPs (wet ponds or extended detention dry ponds) built in accordance with the Northern Virginia BMP Handbook (NVBMPHB) (NVPDC AND ESI) or bioretention facilities built in accordance with the Prince George's County Design Manual for Use of Bioretention in Stormwater Management will be used. * In areas where the soil tests required by the NVBMPHB indicate that infiltration facilities are not feasible (approximately 80 percent of Alexandria), bioretention filters as discussed below may be used in lieu of bioretention. * This policy requires that approximately 3,000+ square feet per planned impervious acre on the project be set aside in locations that are hydrologically rational for use by BMPs. BMPS must be located on common open space, NOT on individual homeowner lots.

stormwater management aspects of the project must therefore be ddressed in the initial planning and layout of the project to assure hat sufficient accessible BMP space is provided to meet these equirements. Preliminary plans that do not conform to this policy ill be considered incomplete and will not be accepted for review by ransportation and Environmental Services.

The Director will continue to consider proposals for use of the nconventional BMPs discussed in Chapter 2 of the Alexandria upplement on a case by case basis for small residential edevelopment projects (two or less acres of planned impervious cover) n the heavily built up areas of the City. However, bioretention ilters will remain the preferred BMP technology for small edevelopment projects. Unconventional BMPs will also be considered or large condominium projects for which a dedicated in-house aintenance force will be employed upon occupancy. The total mpervious area to be treated by an individual underground vault ilter shall be limited to 1.5 acres. Vault sand filters will not be llowed to serve filter watersheds with less than 70 percent mpervious cover. Developers/engineers should discuss any such proposals with the T&ES Engineering staff prior to commitment of any substantial effort to the stormwater design.

This policy has been coordinated with and concurred in by the rginia Chesapeake Bay Local Assistance Department.

ORETENTION AND BIORETENTION FILTERS

Bioretention is a new concept developed by the Prince George's County environmental staff. Required vegetated open space and landscaping on development projects are concentrated into shallow (six-inches deep) depressed planting areas into which rainfall runoff to be treated for water quality is routed. Once the basins fill, remaining runoff is routed either to quantity detention facilities or directly to the storm system. Attachment 1 is an illustration of the system from the Prince George's County design manual. Bioretention areas qualify as vegetated open space under the zoning requirements, and trees and other plantings contained therein count against mandatory landscaping. The volume of water captured and treated in bioretention facilities also counts against stormwater quantity detention requirements. Developers utilizing bioretention in Maryland also report decreased costs in stormwater collection and piping facilities.

The captured runoff percolates into a thick layer of "planting soil" in which a specified number and type of trees and shrubs are planted. Pollutant removal takes place by evapotranspiration, nutrient uptake in the trees and shrubs, biochemical action facilities by bacteria in the root systems, and filtration in the soil media. A layer of sand is provided beneath the "planting soil" to facilitate infiltration of the remaining runoff into the underlying soil strata. At sites where infiltration is not feasible, a bioretention filter may be created by placing a system of perforated collector pipes in gravel draining to the storm sewer system beneath the sand layer and a geomembrane or clay liner beneath the entire bioretention basin. Runoff may be pooled 12 inches deep in a bioretention filter. Attachment 2 illustrates this concept.

Pending the issuance of a revision to the Alexandria Supplement, Alexandria will utilize the Prince George's County design manual for bioretention. For bioretention filters, basin liners and underdrain piping and gravel layers shall be as specified for Austin Sand Filtration Systems on pages 2-32 through 2-39 of the Alexandria Supplement. For projects with more than 0.5 acres of land disturbance, the minimum Water Quality Volume to be captured and treated is the first 0.5 inches of runoff from post-development impervious surfaces. Bioretention facilities must be outfitted for monitoring as outlined in Appendix 2-8 of the Alexandria Supplement. Until the actual removal efficiencies for bioretention facilities have been established by monitoring, Alexandria will provisionally recognize a rate of 60 percent Total Phosphorous removal. * 60% Removal

ATTACHMENTS

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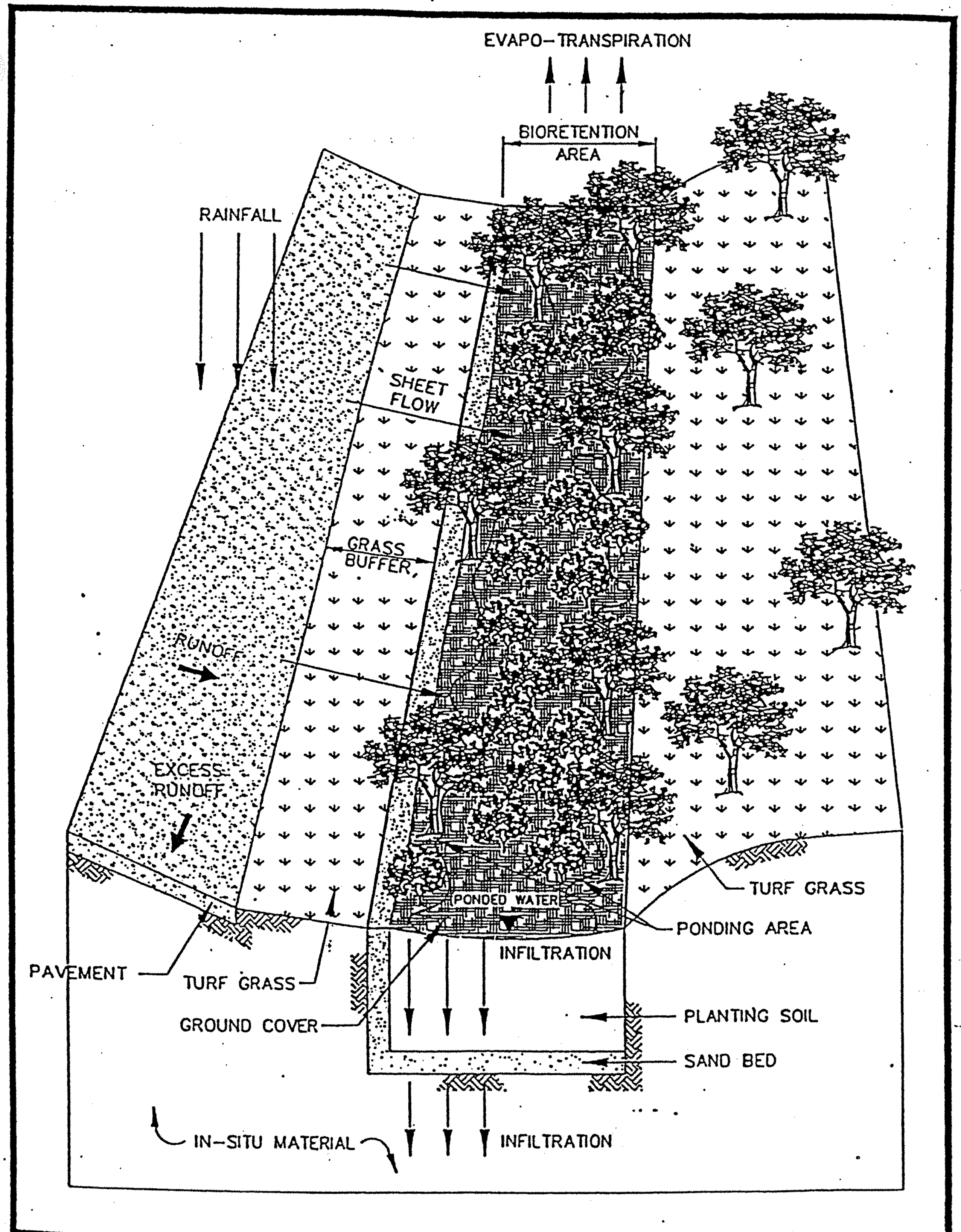
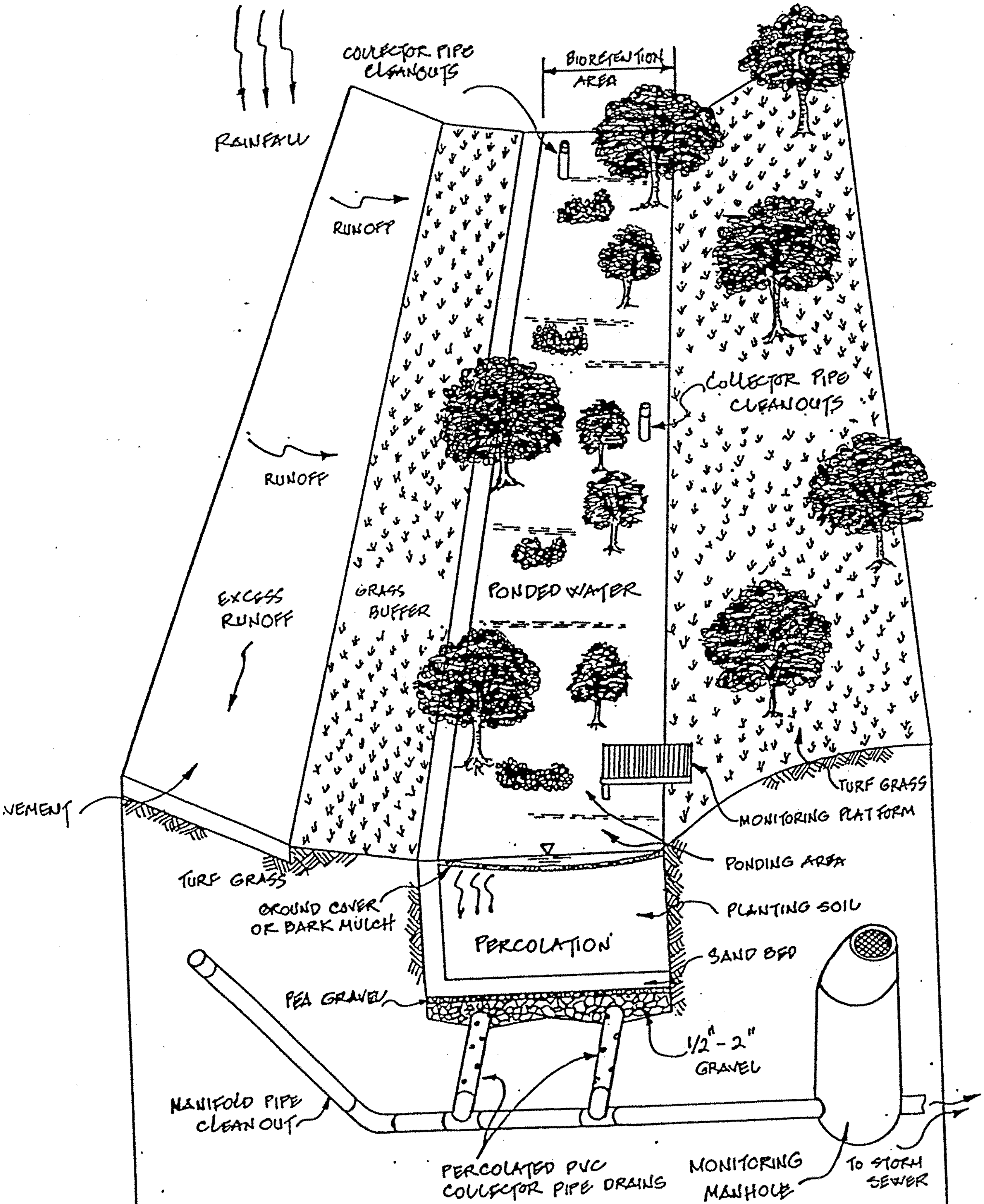


Figure II.1 Bioretention Area Conceptual Layout

EVAPO - TRANSPIRATION



NOTE: IN MARINE CLAY AREAS AND OTHER AREAS WHERE INFILTRATION IS NOT ALLOWED, BASIN AND FILTER MUST BE UNDERLAIN BY CLAY OR GEOMEMBRANE LINER.

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