3 Structural BMP Design Practices

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3.1 INTRODUCTION

The selection and successful design of structural BMPs for stormwater quality enhancement is the cornerstone of stormwater management in newly developing and redeveloping urban areas. The cost effectiveness of each control has to be considered and measured against the actual environmental benefits realized. Design objectives for BMPs can be stated in terms of technology (e.g., by specifying a particular control device) or in terms of quantitative effect (e.g., by specifying a required degree of control or a maximum allowable effect). Quantitative objectives can be defined for both hydrological parameters and constituent removal performance parameters. Some examples of objectives based on hydrological parameters include peak flow rate and retention of a defined water volume for a specific period of time, while objectives based on chemical parameters include percent removal of specific chemical constituents and effluent concentration or mass discharge targets (WEF and ASCE, 1998).

In 1990, the U.S. EPA promulgated phase I of the NPDES (National Pollutant Discharge Elimination System) program regulations for stormwater discharges from municipal separate storm sewer systems (MS4s) (U.S. EPA, 1996), which required municipalities to reduce pollutants in urban runoff to the maximum extent practicable (MEP). The definition of MEP for the control of stormwater pollutant discharges has focused primarily on the application of economically achievable management practices. Stormwater runoff rates and volumes vary highly between storms; hence, the statistical probabilities of runoff events and their management have to be considered in developing practices to meet the MEP goal. It is therefore imperative to examine the hydrology of urban runoff and the type and size of storm runoff events to result in a robust design that has a high probability of meeting these regulations (WEF and ASCE, 1998).

Stormwater management programs were developed by various states in the 1980s when the prevailing outlook was that the quality of receiving streams could be sustained by controlling the flooding caused by increases in runoff volume from new development. The objective of these stormwater management facilities therefore was to control peak flows. Efforts on stormwater management during this time addressed BMP design for flood control based on hydrological procedures with little or no satisfactory guidelines or criteria set forth for water quality management. Most of these designs were flow-based, with an emphasis on the reduction of post-construction flows of the 2- and 10-yr storm events to pre-development levels in new development; this peak management approach addressed only stream channel erosion concerns besides providing adequate flood control in receiving waters and not water quality. However, changes in land use patterns due to increasing urbanization has resulted in a large increase in impervious cover and runoff, strongly highlighting the need to control both the quality and quantity of runoff in order to prevent stream channel erosion. In response to the provisions of the Clean Water Act, (originally enacted in 1977 with subsequent revisions), a number of activities such as the Nationwide Urban Runoff Program (NURP) (U.S. EPA, 1983) were initiated to characterize and quantify the water quality impacts of WWF, and municipalities began adopting BMPs for pollutant removal.

A growing national awareness of the wide range of environmental impacts of runoff and urbanization has resulted in BMPs being designed to control larger storms as well as smaller, but more frequent storms to achieve additional ecological benefits that include stream channel protection and restoration, groundwater infiltration, protection of riparian habitat and biota, and minimized thermal impacts. Collected runoff has also been used for irrigation, toilet flushing, and other non-potable purposes, including ponds and wetlands that also enhance urban aesthetics. This redirected approach in considering a watershed in its entirety and using BMPs for water quality improvement has led to several procedures being established to achieve removal of pollutants from storm runoff. A few approaches include the mandatory requirement to remove 80% of total suspended solids (TSS) from new development in coastal zone states (Coastal Zone Management Act (CZMA), 1972), and controlling the first flush of pollutants associated with a storm, mandating the capture of the first 0.5 to 1 in. of runoff (typically generated in the first hours of the 1-yr storm).

3.2 BMP Design Criteria

3.2.1 Unified Stormwater Sizing Criteria

The objective of any stormwater design criteria is to protect receiving waters from adverse impacts associated with urban runoff. This goal can be successfully accomplished by adopting a unified approach to sizing stormwater BMPs, which is influenced by several factors. A few examples include local hydrological conditions, rainfall-runoff pattern, the type of BMP to be installed, the volume of stormwater that would be treated, degree of imperviousness, prevailing stormwater regulations to be adhered to etc. As an example, the guidelines proposed by the Maryland Department of Environment, which consists of five main quality characteristics of stormwater, is presented in Table 3-1 (MDE, 2000) and briefly explained below.

Table 3-1. Summary Example of Unified Stormwater Sizing Criteria

Sizing Criteria	Description
Water Quality Volume (WQ_v) (acre-ft)	$WQ_v = [(P)(R_v)(A)]/12$ P = 1.0 in. in Eastern Zone and 0.9 in. in Western Zone R_v = volumetric runoff coefficient A = Area in acres
Recharge Volume (Re_v) (acre-ft)	$Re_v = [(S)(R_v)(A)]/12$ S = Soil Specific Recharge Factor Re_v is a sub-volume of WQ_v
Channel Protection Storage Volume (Cp_v)	Cp_v = 24 h (12 h in USE III and IV watersheds) extended-detention of the post-developed 1-yr 24 h storm event. Not required for direct discharges to tidal waters and the Eastern Shore of Maryland.
Overbank Flood Protection Volume (Q_{px})	Local review authorities may require that the peak discharge from the 10-yr storm event be controlled to the pre-development rate (Q_{p10}). No control of the 2-yr storm event (Q_{p2}) is required.
Extreme Flood Volume (Q_f)	Consult with the appropriate local reviewing authority. Normally no control is needed if development is excluded from the 100-yr flood plain and downstream conveyance is adequate.

(MDE, 2000)

Water Quality Volume (WQ_v)

Water quality volume is the storage needed to capture and treat the runoff from 90% of average annual rainfall. Numerically this is equivalent to an inch of rainfall multiplied by the volumetric runoff coefficient (R_v) and site area. Treatment of the WQ_v shall be provided at all developments where stormwater management is required. According to (MDE, 2000), a minimum WQ_v of 0.2 in./acre shall be met at sites or drainage areas that have less than 15% impervious cover, while drainage areas having no impervious cover and no proposed disturbance during development may be excluded from the WQ_v calculations. While the WQ_v is the storage volume needed to capture and treat the runoff from 90% of the average annual rainfall, it also provides management at a critical level (one-third bankfull elevation) within stream channels

Recharge Volume (Re_v)

The criteria for maintaining recharge is based on the average annual recharge rate of the hydrologic soil group(s) present at a site as determined by the United States Department of Agriculture Natural Resources Conservation Service (USDA-NRCS) soil surveys or from detailed soil investigations. Calculation of the specific recharge factor (S) for each soil is based on the USDA-NRCS average annual recharge volume per soil type divided by the annual rainfall (42 in. in the case of Maryland) and multiplied by 90%, consistent with the WQ_v methodology.

The recharge volume is considered part of the total WQ_v that must be addressed at a site and can be achieved either by nonstructural techniques (e.g., buffers, runoff disconnection), structural practices (e.g., infiltration, bioretention), or a combination of both. Re_v and WQ_v are inclusive in that drainage areas having no impervious cover and proposed disturbance may be excluded from recharge calculations as well. The intent of the recharge requirement is to maintain existing groundwater recharge at development sites to preserve water table elevations in order to maintain the hydrology of streams and wetlands under dry-weather conditions. Re_v is dependent on slope, soil type, vegetative cover, precipitation, and evapotranspiration; sites with natural ground cover exhibit higher recharge rates when compared to impervious surfaces resulting from development.

Channel Protection Volume (Cp_v)

The channel protection storage volume(Cp_v) requirement exists to protect stream channels from excessive erosion caused by the increase in runoff from new development. The rationale for this criterion is that runoff from the 1-yr design storm will be stored and released in such a gradual manner that critical erosive velocities during bankfull and near-bankfull events will rarely be exceeded in downstream channels. The Cp_v requirement does not apply to direct discharges to tidal waters.

Of these criteria, the water quality, recharge and channel protection volumes are determined by soils, amount of imperviousness, proposed design and/or layout, and implementation of nonstructural practices. This simplifies calculations, reduces error and/or abuse, and provides direct incentives to reduce impervious areas. Another important feature of these three volumetric criteria is the relation to natural hydrologic processes. When considered together, these three criteria capture and treat the runoff from at least 95% of the average annual rainfall and mimic natural recharge and channel forming processes.

There are two primary approaches for managing stormwater runoff and addressing the unified stormwater sizing criteria requirements on a development site:

- the use of better site design practices to reduce the amount of stormwater runoff and pollutants generated and/or provide for natural treatment and control of runoff, and
- the use of structural stormwater controls to provide treatment and control of stormwater runoff (Atlanta Regional Commission, 2001).

Structural stormwater controls should be considered after all reasonable attempts have been made to minimize stormwater runoff and maximize its control and treatment through better site design methods. Once the need for structural controls has been established, all relevant stormwater sizing criteria should be applied in selecting one or more appropriate controls to meet the stormwater runoff storage and treatment requirements. Most development sites generally require a combination of structural and/or nonstructural BMPs to meet all stormwater sizing criteria (WEF and ASCE, 1998).

3.2.2 BMP Performance Objectives

A fundamental objective of stormwater management programs should be to attempt to reduce the change from the pre-development hydrology of the site. The use of structural controls for treating stormwater and to improve the quality of receiving waters has a set of objectives derived from a number of sources that include: (i) federal, state and local regulatory requirements; (ii) state or local community goals to mitigate the impacts associated with urban runoff; and, (iii) special local area needs such as trout or salmon fisheries protection, water supply and watershed protection, flood control to protect human life and property, groundwater protection, and other issues of local importance. In general, five different levels of stormwater BMP performance goals have been identified (Clar *et al.*, 2003).

- Level 1 - Flood control and peak discharge control
- Level 2 - Level 1 + 80% TSS removal
- Level 3 - Flood, peak discharge, and water quality control
- Level 4 - Unified sizing (multi-parameter) criteria
- Level 5 - Ecologically sensitive stormwater management

These goals have been discussed in detail in an earlier U.S. EPA publication on structural BMP design considerations for improving water quality(Clar *et al.*, 2003).

3.3 DESIGN OF STRUCTURAL BMPS TO IMPROVE WATER QUALITY

Stormwater BMPs can be designed for a wide range of goals and objectives, e.g., from a single parameter approach such as flood control typical in older developed watersheds, or pollutant removal typical of undeveloped watersheds , to multi-parameter ecological sustainability of receiving systems. These management goals will determine the requirement for the proper design and mix of ecological and engineering factors that must be considered. These typically include hydrology and inflow hydraulics, soil characteristics/infiltration rates, site-specific water quality concerns, location and site constraints, and the associated costs as well as the condition of the receiving waters (Clar *et al.*, 2003).

This white paper emphasizes that a clear understanding of the fundamental mechanisms at play within a BMP to reduce the effluent load is the key to properly designing BMPs. The preliminary step is the examination of the following criteria: (i) how does the BMP address the twin issues of stormwater quality and quantity control; (ii) what are the design considerations involved in achieving optimum water quality control in the BMP; (iii) how effective is the BMP in terms of meeting performance objectives; (iv) what are the cost factors associated with the design and construction of the BMP and how to be cost-effective; and more importantly, (v) how best to place it in the watershed. This white paper focuses on the design considerations of the following more commonly used structural BMPs :

3.3.1 Dry Extended-detention Ponds
3.3.2 Wet Ponds

3.3.3 Stormwater Wetland
3.3.4 Grassed Swales
3.3.5 Vegetated Filter Strips
3.3.6 Infiltration Trenches
3.3.7 Porous Pavements
3.3.8 Sand and Organic Filters

The following sections present a detailed discussion on the general design considerations and BMP design guidelines for these BMPs adopted by various state agencies for stormwater management. Several well known BMP manuals, (e.g.,Atlanta Regional Commission, 2001; Schueler, 1987; Florida DER, 1988; NVPDC and ESI, 1992; Schueler, 1992a; WEF and ASCE, 1998; VA DCR, 1999; MDE, 2000; Metro Council, 2001; NYSDEC, 2001; U. S. EPA, 2001; WA DOE, 2001; Caltrans, 2002; GeoSyntec and ASCE, 2002; CASQA, 2003) have been consulted in presenting the multitude of approaches practiced by the different agencies in designing structural BMPs for stormwater management for new and existing urban development. It must be noted that the ensuing discussion is merely a compilation of the existing design practices and should not be treated as the ultimate design guidance document. The actual design process must take into account several locally specific criteria that primarily include the long-term rainfall/hydrological and soil considerations, water quality objectives, economics, and stormwater regulatory considerations. The typical operation and maintenance requirements for these BMPs are discussed in chapter 5.

3.3.1 Dry Extended-detention (ED) Ponds

An ED pond is an impoundment that temporarily stores stormwater runoff from a water quality design storm for a specified minimum period of time (usually 24 to 48 h) and discharges it through a hydraulic outlet structure to a downstream conveyance system; it is usually dry during non-rainfall periods and does not have any permanent standing water (CASQA, 2003; VA DCR, 1999). An ED pond can be designed to provide for one or all of the following: water quality enhancement; downstream flood control; and, channel erosion control. Conventional ED ponds temporarily detain a portion of stormwater runoff for up to 24 h after a storm using a fixed orifice resulting in the settling out of urban pollutants; enhanced ED ponds are designed to prevent clogging and suspension encountered in conventional ED ponds due to frequent high inflow velocities, and thus have higher efficiencies. ED ponds provide greater flexibility in achieving target detention times. Along with a detention area, they typically include a sediment forebay near the inlet, a micropool and/or plunge pool at the outlet, and utilize an adjustable reverse-sloped pipe as the ED pond control device to prevent resuspension of particles deposited in earlier storms (WY DEQ, 1999). The detention of runoff for 24 h or more (up to 72 h) helps to remove up to 90% of particulate pollutants, while the removal of soluble forms of nitrogen and phosphorus in urban runoff can be enhanced if the normally inundated area of the pond is managed as a shallow marsh or a permanent pool (Schueler, 1987).

3.3.1.1 Stormwater Control

ED ponds can be designed for flood control by providing additional storage above the ED volume, and by reducing the peak rate of runoff from the drainage area (CASQA, 2003). These BMPs are effective in controlling post-development peak discharge rates to the desired pre-development levels for the design storm(s) specified, and the optimum level is achieved by controlling multiple design storms. The design storms are chosen based on ordinance, or specified watershed conditions (VA DCR, 1999). ED ponds are also capable of managing smaller floods that contribute to channel erosion problems and occur more frequently than the annual or 2-yr flood (Schueler, 1987).

3.3.1.2 Pollutant Removal Capability

Pollutant removal is primarily accomplished by gravitational settling that is dependent on the detention time and the fraction of the annual runoff volume that is effectively detained in the pond (Schueler, 1987) together with sorption of pollutants to particulates and the associated settling velocity distribution of these particles. Conventional ED ponds provide moderate but variable removal of particulate pollutants such as sediment, phosphorus and organic carbon, and some removal of soluble pollutants (CASQA, 2003). Increasing detention times may result in greater removal of soluble pollutants. Urban pollutants commonly of concern, including nitrate and orthophosphates, remain in solution, and may be removed by managing the lower stage of the ED pond as a shallow wetland that utilize natural biological removal processes (Schueler, 1987; VA DCR, 1999).

Positive factors influencing pollutant removal (WY DEQ, 1999) include:

- six to twelve hours of minimum detention,
- smaller treatment volumes (e.g., 0.5 watershed in.) provide the best removal rates,
- wetlands in lower stage of design prevent resuspension and augment removal of sediments,
- use of a micropool to protect the ED pond orifice, and
- soils that soak up runoff and evapotranspire the same.

Negative factors influencing pollutant removal (WY DEQ, 1999) include:

- re-suspension of previously deposited pollutants from the pilot channel of pond floor,
- large treatment volumes (acceptable ED times cannot be achieved over the broad range of expected storms), and
- difficulty in predicting ED hydraulics.

3.3.1.3 Design Considerations

General Considerations

The success of an ED pond is dependent on the designer's ability to identify any site and downstream conditions that may affect the design and function of the pond. The facility should be compatible with both upstream and downstream stormwater systems to promote a watershed approach in providing stormwater management (VA DCR, 1999). The size can be based on the volume for which BMP credit is desired and the volume being dictated by stormwater management requirements, as these facilities are usually designed for both water quality and stormwater management needs. The shape of these facilities is often dictated by site constraints and topography (WEF and ASCE, 1998).

The dimensions of the pond need to be sized appropriately in order to enhance the effectiveness of these BMPs. An effective configuration of the pond should result in a long flow path, promote the establishment of low velocities, and avoid having stagnant areas of the pond. In order to promote settling of pollutants and aesthetic appeal, the design should consider the length-to-width ratio, cross sectional areas, pond slopes and configuration, and aesthetics.

Sizing Detention Ponds

There are several ways to size an ED pond. The more common methods use either maximized volume or hydrograph routing (WEF and ASCE, 1998).

The maximized volume method is the simplest and most direct way for smaller catchments serving up to approximately 1 km^2 (0.6 mi^2). The methodology to estimate the maximized water quality capture volume is described below.

The stormwater quality capture volume may be found by using continuous hydrologic simulation and local long-term hourly (or lesser increment) precipitation records, or by obtaining a first-order estimate of the needed capture volume using simplified procedures that target the most typically occurring population of runoff events. In the U.S., this data is available for ponds that empty their entire volume in 24 and 48 h. Mean values for other emptying times can be determined by interpolating the results for the 24 and 48 h time. After an extensive analysis of the mean annual runoff-producing rainfall depths for the different meteorological regions of the U.S., simple regression equations were established to relate the mean precipitation depth to "maximized" water quality runoff capture volumes (WEF and ASCE, 1998). The analytical procedure was based on a simple transformation of each storm's volume of precipitation to a runoff volume using a coefficient of runoff. A third order regression equation, Equation 3-1 (WEF and ASCE, 1998), was derived using data from more than 60 urban watersheds (U.S. EPA, 1983). The equation has broad applicability for smaller storm events in the U.S. as it was derived from a nationwide monitoring over a 2-yr period.

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \tag{3-1}$$

where
C = runoff coefficient; and
i = watershed imperviousness ratio, namely, percent total imperviousness divided by 100.

Equation 3-2 relates mean precipitation depth to the "maximized" detention volume and is given by

$$P_0 = (a \times C) \times P_6 \qquad \textbf{(3-2)}$$

where
P_0 = maximized detention volume determined using either the event capture volume or the volume capture ratio as its basis, watershed in. (mm);
a = regression constant from least-squares analysis;
C = watershed runoff coefficient; and
P_6 = mean storm precipitation volume, watershed in. (mm).

Values of coefficient *a* have been determined based on an analysis of long-term data from seven precipitation gauging sites located in different meteorological regions of the U.S. for different drain times of 12, 24 and 48 h, respectively and shown in Table 3-2 (Guo and Urbonas, 1995). The correlation of determination coefficient, r^2, ranges from 0.80 to 0.97, implying a strong level of reliability. It is suggested that the event-capture-ratio-based coefficients in Table 3-2 be used with equation 3.2 instead of the volume capture ratio coefficients (WEF and ASCE, 1998). While the choice of the emptying or drain time rests with the designer or with the local authorities, it must be noted that suspended solids are better removed under longer emptying times. However, the disadvantage of longer drain times is that they tend to produce less attractive facilities, ones that have little or no vegetation on the bottom, or "boggy" bottoms with marshy vegetation that pose maintenance concerns.

Table 3-2. Values of Coefficient "*a*" for Finding the Maximized Detention Storage Volume*

		Drain time of capture volume		
		12 h	**24 h**	**48 h**
Event capture ratio	a r^2	1.109 0.97	1.299 0.91	1.545 0.85
Volume capture ratio	a r^2	1.312 0.80	1.582 0.93	1.963 0.85

* Approximately 85th percentile runoff event (range 82 to 88%)
(Guo and Urbonas, 1995)

The hydrograph routing method is used for detention ponds that serve areas larger than 1 km^2 by converting the maximized storm depth to a design hyetograph, to simulate a runoff hydrograph. Although the method by which this is done is dictated by the typical design storm temporal distribution in use within the region where the facility is located, it is suggested that the maximized depth be redistributed into a 2 h design storm hyetograph. The goal of reservoir routing is to balance inflow rates against outflow rates to find the needed volume, which can be accomplished with numerical methods or by using of one of the many available computer programs written for this purpose (WEF and ASCE, 1998). The needed storage volume is a time integral of the difference between inflow and outflow hydrographs from the beginning of storm runoff to the point in time where the outflow rate exceeds the inflow rate (Equation 3-3):

$$V_{max} = \int_0^1 (Q_{in} - Q_{out})dt \qquad \textbf{(3-3)}$$

where
V_{max} = storage volume;
t = time from beginning of runoff to a point of maximum storage;
$Q_{in} = Q_{out}$ on hydrograph recession limb;
Q_{in} = inflow rate; and
Q_{out} = outflow rate.

Local governments have developed a number of sizing rules for extended-detention, each specifying both a volume to be detained and a duration over which this volume is released. (CASQA, 2003) states that capture volume is determined by local requirements or sized to treat 85% of the annual runoff volume. It must be mentioned that a thorough engineering approach to sizing a pond may be the best option. This can be accomplished by using long-term dry- and wet-weather flows, pond inflow based on watershed hydrology, and particle settling velocity calculations together with necessary calculations for pond soil infiltration and evaporation. A pre-monitoring program to study the settling velocity distribution and the analysis of pollutant associations with particulates and soluble fraction is recommended.

Siting

Dry ED ponds are among the most widely applicable stormwater management practices and are especially useful in retrofit situations where their low hydraulic head requirements allow them to be sited within the constraints of the existing drainage system (CASQA, 2003). The basic guidelines for siting dry ED ponds are as follows:

- Dry ED ponds may be used for a wide range of drainage areas. However, the upper range for contributing drainage area applicable for these ponds without having to take baseflow into consideration is about 50 to 75 acres (NVPDC and ESI, 1992).

- Dry ED ponds should be used on sites with a minimum area of 5 acres. With this size catchment area, the orifice can be on the order of 0.5 in. The challenge in smaller sites is to provide channel or water quality control because the orifice diameter at the outlet needed to control relatively small storms becomes very small and is prone to clogging.

- The base of the extended-detention facility should not intersect the water table, as a permanently wet bottom may become a mosquito breeding ground (CASQA, 2003).

- Adequate access from a public or private right-of-way to the pond should be reserved. The access should be at least 10 ft wide, on a slope of 5:1 or less, and stabilized to withstand the passage of heavy equipment.

- All ED ponds should be a minimum of 20 ft from any structure or property line, and 100 ft from any septic tank/drain field. ED ponds should also be a minimum of 50 ft from any steep slope (greater than 15%). Otherwise, a geotechnical report will be required to address the potential impact of any pond that must be constructed on or near such a slope (VA DCR, 1999).

- ED ponds can be used with almost all soils and geology, with minor design adjustments for regions of rapidly percolating soils such as sand. In these areas, these BMPs may need an impermeable layer to prevent groundwater contamination. Highly permeable soils are not suitable for ED ponds, and for an enhanced ED pond, the soils must support the shallow marsh at the time of stabilization and planting.

Quantity Detained

The amount of runoff detained heavily influences the pollutant removal performance. At a minimum, ED ponds should be sized to accommodate the runoff produced by the mean storm, and preferably should be capable of storing the runoff volume of a 1.0 in. storm. Higher levels of control can be achieved when the runoff volume from the 1- or 2- yr storm is detained (Schueler, 1987). However, in many cases, the stricter storage requirements recommended above for streambank erosion control (1.0 to 1.5 in. R_v) will govern how much extra detention storage is needed.

Duration

Detention times of at least 24 to 36 h are probably necessary to achieve maximum removal of most pollutants. Although most of the settling occurs within the first 12 h in settling column experiments, it is advisable to provide further detention since several hours may be needed before ideal settling conditions develop in a pond. Slightly longer detention times of up to 40 h may be needed in larger watersheds for downstream channel erosion control. The control device must be sized so as to provide an adequate detention time for the entire spectrum of storms. The pond designer should perform several storage routing calculations e.g., TR-20 method or equivalent, to determine the approximate detention time for the smaller, more frequent runoff events. As a general rule, it is recommended that the average detention time for small runoff events (0.1 to 0.2 in.) should be no less than 6 h. As a final check, the runoff velocity of the downstream channel at the extended-detention release rate should be computed to make sure that it is not erosive (Schueler, 1987).

Pond Configuration

Minimizing the velocity of the flow through the pond greatly improves the pollutant removal efficiency of the pond, which can be effected by increasing the pond depth as well as the cross sectional area (NVPDC and ESI, 1992). The basin should gradually expand from the inlet and contract toward the outlet to reduce short circuiting and slow influent velocities by increasing the cross sectional flow area. The goal is to provide conditions where the velocity of flow through the facility for a typical storm event is less than the settling velocities of the pollutants of concern (NVPDC, 1979).

The length-to-width ratio of a pond is one design aspect that can significantly affect pollutant removal. The distance between inlet and outlet points needs to be maximized in order to promote pollutant settling. A high aspect ratio may improve the performance of detention ponds; consequently, the outlets should be placed to maximize the flowpath through the facility. While the flow path length is defined as the distance from the inlet to the outlet as measured at the surface, the average width is calculated as the surface area of the pond divided by the length (NVPDC and ESI, 1992) (Metro Council, 2001). A length-to-width ratio of two or greater, preferably up to a ratio of four is required for additional detention time for settling and biological treatment (WEF and ASCE, 1998).

Pond depths optimally range from 2 to 5 ft and may include a sediment forebay to provide the opportunity for larger particles to settle out (NVPDC and ESI, 1992).

A micropool is not recommended in the design because of vector concerns. For online facilities, the principal and emergency spillways must be sized to provide 10 ft of freeboard during the 25-yr event and to safely pass the flow from 100-yr storm (CASQA, 2003).

Pond Side Slopes

Pond side slopes need to be stable under saturated soil conditions. They also need to be sufficiently gentle to limit rill erosion, facilitate maintenance, and address the safety issue of individuals falling in when the basin is full of water. In order to promote facility effectiveness, it is highly desirable to avoid resuspension of materials collected on the pond floor; the potential for resuspension is generally minimized by reducing inflow velocities and maintaining vegetative cover. Side slopes should be no steeper than 4:1 (H:V), and no flatter than 20:1 (H:V) (Schueler, 1987; WEF and ASCE, 1998). Slopes steeper than this needs to be stabilized with an appropriate slope stabilization practice (CASQA, 2003).

Pond Lining

Ponds must be designed to prevent possible contamination of groundwater below the facility.

Pond Inlet

An ideal inflow structure should convey stormwater to the pond while preventing erosion of the pond bottom and banks, reducing resuspension of previously deposited sediment, and facilitating deposition of the heaviest sediment near the inlet. Such energy dissipation measures also reduce the tendency for short-circuiting. Inflow structures can be drop manholes, rundown chutes with an energy dissipator near the bottom, a baffle chute, a pipe with an impact basin, or one of the many other types of diffusing devices, depending on pond geometry (NVPDC and ESI, 1992; WEF and ASCE, 1998).

Outflow Structure

The outlet should be capable of slowly releasing the design capture volume over the design emptying time. ED ponds are designed to encourage sediment deposition and as stormwater has substantial quantities of settleable and floatable solids, outlets are prone to be clogged, invalidating the hydraulic function of even the best design. Each outlet therefore needs to be designed with clogging, vandalism, maintenance, aesthetics, and safety in mind (WEF and ASCE, 1998). The facility's drawdown time should be regulated by a gate valve or orifice plate. In general, the outflow structure should have a trash rack or other acceptable means of preventing clogging at the entrance to the outflow pipes. The structure should be sized in such a way to allow for complete drawdown of the WQ_v in 72 h, with no more than 50% of the water quality volume draining from the facility within the first 24 h. The outflow structure should be fitted with a valve in order to regulate the rate of discharge from the basin as well as to halt the discharge in case of an accidental spill in the watershed (CASQA, 2003). The discharge from a control orifice is given by Equation 3-4:

$$Q = CA(2gH - H_0)^{0.5} \quad \textbf{(3-4)}$$

where
Q = discharge (ft^3/s);
C = orifice coefficient;
A = area of the orifice (ft^2);
g = gravitational constant (32.2 ft/s^2);
H = water surface elevation (ft); and
H_0 = orifice elevation (ft).

Recommended values for C are 0.66 for thin materials and 0.80 when the material is thicker than the orifice diameter. This equation can be used with the pond stage/volume relationship to calculate drain time in spreadsheet form.

Storage Volume

The storage volume in the ED pond should be equal to the maximized volume discussed earlier. An additional 20% could be added to this volume to provide for sediment accumulation, and could be used to promote sedimentation of smaller particles (less than 60 µm in size), which account for approximately 80% of the suspended sediment mass found in stormwater (WEF and ASCE, 1998)

Flood Control Storage

Whenever feasible, the ED pond should be incorporated within a larger flood control facility. By doing so, both water quality and flood control functions can be combined in a single detention basin.

Two-Stage Design

This pond configuration is meant to address both water quality and quantity. A two-stage basin is recommended when extended-detention is applied to dry ponds. The top stage of the pond should have the capacity to regulate peak flow rates of large infrequent storms (10-, 25- or 100-yr), and will generally remain dry between storms. The volume in this stage is called the "flood storage volume." The second stage of the pond is designed to detain smaller storms for a sufficient period of time to remove pollutants from the runoff. The volume in this stage is called the "water quality volume." For ED ponds, WQ_v is typically the runoff from the 0.3-yr storm event, since a large fraction of the annual pollutant load is delivered by small, frequent storm events (Metro Council, 2001).

The upper stage of the pond is sized and graded (2% minimum) to remain dry except during large infrequent storms, while the bottom stage is expected to be regularly inundated. The lower portion has a micropool that fills frequently and reduces the periods of standing water and sediment deposition in the remainder of the basin. A marsh-like environment in the lower section allows for some biological uptake of soluble materials and provides quiescent conditions, which promotes sedimentation of particulates (NVPDC and ESI, 1992). However, these recommendations do not necessarily apply to large, regional extended-detention ponds (WEF and ASCE, 1998). Extra storage, over and above stormwater and extended-detention requirements, should be provided within the bottom stage, or at the inlet to account for 20 yr of sediment deposition. The main advantage of this configuration is that frequently inundated areas are localized in one section of the pond, thus allowing the upper portion of the BMP facility to be used for certain low intensity recreational uses during dry weather.

Enhancement Options - Wetland Creation

Establishing wetland vegetation in a shallow marsh component or on an aquatic bench in the lower stage of the detention basin will enhance removal of soluble nutrients, increase sediment trapping, prevent sediment resuspension, and provide wildlife and waterfowl habitat (Metro Council, 2001). The use of a shallow marsh limits the maximum range of vertical storage in the ED pond to 3 ft above the marsh's water surface elevation. However, the surface area requirements for the shallow marsh will likely force the basin's geometry to broaden at the lower stages, which will compensate for the vertical storage (VA DCR, 1999). In general, extended-detention water surface elevations greater than 3 ft, and the frequency at which those elevations can be expected, are not conducive to the growth of dense or diverse stands of emergent wetland plants. Water depths of 6 to 12 in. would be required for optimal wetland growth, and native species should be planted in the wetland (Schueler, 1987). The general guidelines in designing a shallow marsh are discussed in detail in section 3.3.3 on stormwater wetlands.

Sediment Forebay

The settling area for incoming sediments can be increased through the addition of a sediment forebay. The use of a sediment forebay, however, is only recommended for wet ponds larger than 4,000 ft^3. The forebay is an excavated settling basin or a section separated by a low weir at the head of the primary impoundment. Forebays serve to trap sediments before the sediment enters the primary pool, effectively enhancing removal rates and minimizing long-term operation and maintenance problems. Also, it is easier and more cost effective to remove sediments from the forebay periodically as compared to removal from a wet pond pool. Hard bottom forebays make sediment removal easier, and forebays should be accessible to heavy machinery, if necessary. About 10 to 25% of the surface area of the wet pond should be devoted to the forebay.

The forebay can be distinguished from the remainder of the pond by one of several means: a lateral sill with rooted wetland vegetation; two ponds in series; differential pool depth; rock-filled gabions or retaining walls; or, a horizontal rock wall filter placed laterally across the pond. Energy dissipation techniques should be used at the inlet to the sediment forebay to avoid erosion, promote settling, and minimize short-circuiting of flows. The length-to-width ratio of the forebay should be at least 2:1 to minimize short-circuiting (Metro Council, 2001).

Low Flow Channels

Low flow channels route the last remaining runoff, dry weather flow and groundwater to the permanent pool and outlet. These channels should be installed in the upper stage of the basin to ensure that the basin dries out completely. Low flow channels also serve to prevent erosion of the upper stage of the pond outside as runoff first enters the pond.

The presence of a baseflow makes the design of an extended-detention control structure difficult. An orifice designed for wet-weather baseflow, compromises the dry-weather control due to very high release rates. On the contrary, if an orifice is undersized to meet the dry-weather control, the ED pond may remain full of water during the wet-weather season and eliminate the extended-detention volume by creating an undersized permanent pool (VA DCR, 1999). When seasonal baseflow is present, an adjustable orifice should be provided in the control structure to maintain the marsh volume.

Design considerations should take into account the presence of a baseflow and the associated potential for erosion within the basin and, ideally, spread them out so they sheet flow across the bottom of the basin. A few local ordinances require the use of low-flow channels to carry baseflows. Generally, an impervious low-flow channel is not recommended in a stormwater management water quality basin, as its use is contrary to the basin's water quality function. However, an impervious ditch may be used to carry baseflow if it is designed to overflow during storm events and spread the runoff across the basin floor. The use of gabion baskets or riprap, instead of concrete, may provide the advantage of slowing the flow, encouraging spillover onto the basin floor.

Overflow

Similar to a constructed stormwater wetland, an ED overflow system should be designed to provide adequate overflow or bypass for a full range of design storms.

Liner to Prevent Infiltration

ED ponds should have negligible infiltration rates through the bottom of the pond. If infiltration is anticipated, and the area is not suspected to be underlain by karst, then an infiltration facility, rather than a detention water quality BMP should be used or a liner should be installed in the basin to prevent infiltration. The following recommendations apply when using a liner:

- A clay liner should have a minimum thickness of 12 in. and should comply with the specifications provided in Table 3-3.
- A layer of compacted soil (minimum 6 to 12 in. thick) should be placed over the liner before seeding with an appropriate seeding mixture.
- Other liner types may be used if supporting documentation is provided verifying the liner material's performance.

Table 3-3. Clay Liner Specifications

Property	**Test Method (or equal)**	**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 Particle Sieving	ASTM D-422	%	Not less than 30
Clay Compaction	ASTM D-2216	%	95% of Standard Proctor Density

(City of Austin, 1988)

Pond Buffer

A buffer strip away from the pond to the nearest lot should be reserved and landscaped using low-maintenance grasses, shrubs and trees (e.g., the minimum width should be 25 ft (Schueler, 1987). A landscaping plan for the pond and buffer should outline measures to improve the appearance for adjacent residents, meet specific design functions, and provide local wildlife habitat (Schueler, 1987).

Dam Embankment

The dam embankment should be designed not to fail during storms larger than the water quality design storm. An emergency spillway could be provided, the design of which is governed by local regulations, and the embankment should have at least one foot of freeboard above the emergency spillway. The other approach is to design the embankment to withstand overtopping commensurate with embankment size, the volume of water that can be stored in it, and the potential of downstream damages or loss of life if the embankment fails. Embankments for small onsite basins should be protected from at least the 100-yr flood, while the larger facilities should be evaluated for the probable maximum flood. Embankment slopes should be no steeper than 3:1, preferably 4:1 or flatter. They also need to be planted with turf-forming grasses. Embankment soils should be compacted to 95% of their maximum density at optimum moisture, graded to allow access for heavy equipment, and mowed twice a year to prevent woody growth. At least 10 to 15% extra fill should be allowed on the embankment to account for possible subsidence.

Vegetation

Vegetation provides erosion control and enhances sediment entrapment in a pond. The pond can be planted with native grasses or with irrigated turf, depending on the local setting, pond design, and its intended other uses such as recreation. The maintenance of a healthy grass cover on the pond bottom is difficult due to sediment deposition, along with frequent and prolonged periods of inundation. Options for an alternative bottom liner include a marshy wetland bottom, bog, layer of gravel, riparian shrub, bare soil, low-weed species, or other type that can survive the conditions existing in the bottom of the pond.

Splitter Box

Splitter structures isolate WQ_v, when ponds are designed as offline facilities. The splitter box, or other flow diverting approach, should be designed to convey the large storm event (e.g., 25- yr storm) while providing adequate (e.g., at least 10 ft) freeboard along pond side slopes.

Erosion Protection at the Outfall

For online facilities, special consideration should be given to the facility's outfall location. Flared pipe end sections that discharge at or near the stream invert are preferred. The channel below the pond outfall should be modified to conform to natural dimensions, and lined with large stone riprap placed over filter cloth. Energy dissipation may be required to reduce flow velocities from the primary spillway to non-erosive velocities. An example schematic of a dry ED pond (MDE, 2000) is shown in Figure 3-1.

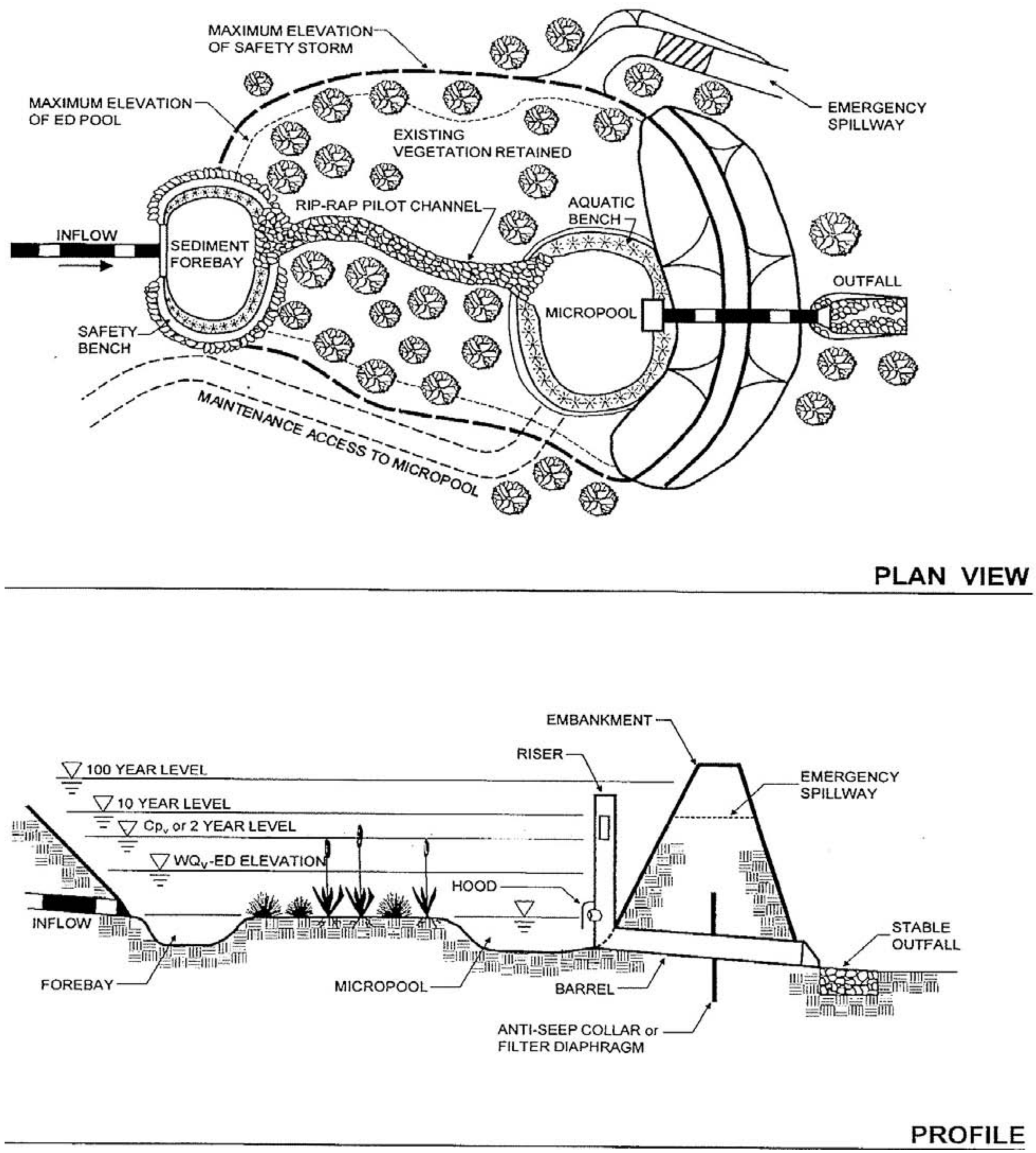

Figure 3-1. Schematic of a Dry Extended-Detention Pond (MDE, 2000)

3.3.2 Wet Ponds

A wet pond is a constructed stormwater retention basin with emergent wetland vegetation around the perimeter. It is designed to have a permanent pool of water throughout the year or at least during the wet season. Runoff from each rain event is detained and treated in the pool primarily through gravitational settling and biological uptake mechanisms until it is displaced by runoff from the next storm (Atlanta Regional Commission, 2001). The permanent pool provides a vessel for the settling of solids between storms and the removal of nutrients and dissolved pollutants. The wetland vegetation, also called the littoral zone, provides aquatic habitat, enhances pollutant removal, and reduces the formation of algal mats, and can be created by excavating an already existing natural depression or through the construction of embankments. Conventional wet ponds have a permanent water pool to treat incoming stormwater runoff. In enhanced wet pond designs, a forebay is installed to trap incoming sediments where they can be easily removed; a fringe wetland is also established around the perimeter of the pond (WY DEQ, 1999). .

3.3.2.1 Stormwater Control

Stormwater ponds are designed to control both stormwater quality and quantity and can be used to address all the unified stormwater sizing criteria for a given drainage area. Wet ponds can be effective in controlling post-development peak discharge rates to pre-development levels for desired design storms. Groundwater recharge in wet ponds is limited to the storage lost to infiltration through the pond bottom. Although the quantity of recharge is greater than that achieved in dry or extended detention ponds, it is negligible in comparison to infiltration and other volume control BMPs. The post-development increase in the total runoff volume from a site is not effectively modified by wet ponds. While some temporary control of runoff volume happens when extra dead-storage is created by evaporation or infiltration, it generally occurs during minor storms in the summer months and after prolonged droughts.

3.3.2.2 Pollutant Removal Capability

Pollutant removal in wet ponds is highly variable from storm to storm but generally high over the long-term, for well designed and maintained ponds. The degree of pollutant removal achieved by a pond is a function of the size and design of the permanent pool and the characteristics of individual urban pollutants. Suspended sediments in stormwater runoff settle out from the water column to the pond sediments. The permanent pool additionally acts as a barrier to resuspension of deposited materials and improves removal performance over that achieved by dry ponds. The greatest initial settling often occurs near the pond inlet under quiescent conditions; settling can be modeled assuming Stokes Law Type I sedimentation. However, pollutant removal rates may decline during larger storms in smaller ponds due to short-circuiting and the volume of incoming runoff being greater than the volume of the permanent pool.

A unique feature of wet ponds is the presence of aquatic plants and algae that can remove significant amounts of soluble nutrients from the water column; retention ponds can be superior to ED ponds for the control of dissolved nutrients in stormwater (Schueler, 1987). Since soluble nutrients have minimal settling velocities, biological uptake represents an important removal pathway. Retention ponds are most appropriate where nutrient loadings are of concern, especially in the following situations:

- Watersheds tributary to reservoirs and lakes - retention ponds in the watershed can help achieve eutrophication management goals in downstream reservoirs and lakes.
- Watersheds tributary to tidal embayments and estuaries - nutrient loadings into estuarine systems is a growing concern in coastal areas, including upland areas that drain into tidal waters; retention ponds can help reduce the nutrient loads.

The degree of pollutant removal is a function of pool size in relation to contributing watershed area, and is achieved by gravitational settling, algal settling, wetland plant uptake, and bacterial decomposition. Unlike ED ponds, wet ponds avoid resuspension, and nutrient cycling in these ponds is generally thought to operate much as in natural lakes; consequently, the pollutant removal capabilities can be successfully predicted by applying a controlled lake eutrophication model. The principal factors governing nutrient cycling are the loading and the decay rates for phosphorus, hydraulic residence time, and mean depth (NVPDC and ESI, 1992).

The observed pollutant removal of a wet pond is highly dependent on two factors; i.e., the volume of the permanent pool relative to the amount of runoff from the typical event in the area and the quality of the baseflow that sustains the permanent pool. If the permanent pool is much larger than the volume of runoff from an average event then the primary process is the displacement of the permanent pool by the wet-weather flow (Caltrans, 2002). The discharge quality of wet ponds during dry- and wet-weather flows is not significantly different, resulting in a relatively constant discharge quality during storms that is the same as the concentrations observed in the pond during ambient (dry weather) conditions and so are better characterized by the average effluent concentration, rather than the "percent reduction" (CASQA, 2003). The dry- and wet-weather discharge quality is thus related to the quality of the baseflow that sustains the permanent pool and the transformations of those pollutants during their residence in the basin.

Positive factors influencing pollutant removal include:

- pretreatment by sediment forebay,
- permanent pool, 0.5 to 1.0 in. per impervious acre treated,
- fringe wetlands,
- shallow wetlands and/or extended detention may improve removal efficiencies, and
- high length-to-width ratios.

Negative factors influencing pollutant removal include::

- small pool size,
- fecal contribution from large waterfowl populations,
- short-circuiting and turbulence,
- sediment phosphorus release,

- extremely deep pool depths (greater than 10 ft), and
- snowmelt conditions and/or ice.

3.3.2.3 Design Considerations

A well designed stormwater pond consists of the following:
- permanent pool of water,
- overlying zone in which runoff control volumes are stored, and
- shallow littoral zone (aquatic bench) along the edge of the permanent pool that acts as a biological filter (WEF and ASCE, 1998).

Siting

Wet ponds are a widely applicable stormwater management practice and can be used over a broad range of storm frequencies and sizes, drainage areas, and land use types. They can be constructed on- or off-line (off-line is preferred) and can be sited at feasible locations along established drainage ways with consistent baseflow. Wet basin application is appropriate in the following settings:

- where there is a need to achieve a reasonably high level of dissolved contaminant removal and/or sediment capture,
- in small to medium-sized regional tributary areas with available open space and drainage areas greater than about 10 ha (25 acre),
- where baseflow rates or other channel flow sources are relatively consistent year-round, and
- in residential settings where aesthetic and wildlife habitat benefits can be appreciated and maintenance activities are likely to be consistently undertaken (CASQA, 2003).

Soil Permeability

Highly permeable soils may not be acceptable for retention ponds because of excessive drawdown during dry periods. Where permeable soils are encountered, exfiltration rates can be minimized by scarifying and compacting a 0.3 m (12 in.) layer of the bottom soil of the pond, incorporating clay to the soil, or providing an artificial liner. Excavating the permanent pool into the groundwater table can also ensure its permanency, but seasonal fluctuations in the groundwater table need to be taken into account (Schueler, 1987).

Design Criteria

The design of permanent pools for a wet pond employs two different methods.
- The solids-settling design method relies on the solids-settling theory and assumes that all pollutant removal is because of sedimentation.
- The lake eutrophication model design method provides for a level of eutrophication by accounting for the principal nutrient removal mechanisms (WEF and ASCE, 1998).

The solids-settling method is most appropriate for situations where the control of total suspended sediments and pollutants that attach themselves to the solids is the principal objective. The method relies on rainfall and runoff statistics, pond size, and settling velocity distributions of suspended solids to calculate total suspended sediment removal. This method assumes an approximate plug flow system in the retention pond with all pollutant removal resulting from sedimentation. Testing this model using data from nine retention ponds monitored during U.S. EPA's NURP showed that it predicted removal rates reasonably well (WEF and ASCE, 1998).

The lake eutrophication model assumes that a retention pond is a small eutrophic lake that can be represented by empirical models used to evaluate lake eutrophication effects. This method is used to size a retention pond to achieve a controlled rate of eutrophication and an associated removal rate for nutrients. Retention ponds that achieve nutrient removal also removes other pollutants, and typically it is not necessary for the design process to address constituents other than nutrients.

Like most input/output lake eutrophication models, this model is an empirical approach that treats the permanent pool as a completely mixed system and assumes that it is not necessary to consider the temporal variability associated with individual storm events. While the solids-settling model accounts for the temporal variability of individual storms, the lake eutrophication model is based on annual flows and loadings.

The model is applied in two parts:

$$K = \frac{0.56 \times Q_s}{F \times (Q_s + 13.3)} \qquad \textbf{(3-5)}$$

where
K_2 = second order decay rate m^3/mg x a;
Q_s = Z/T the mean overflow rate, m/a;
Z = mean pond depth, m;
T = average hydraulic retention time, yr; and
F = inflow (ortho P/total P) ratio.

$$R = 1.0t \frac{1.0 - \sqrt{1.0 + (1.0 + 4N)}}{2N} \qquad \textbf{(3-6)}$$

where
R = total P retention coefficient, (i.e. BMP efficiency);
$N = K_2 \times P_T \times T$; and
P_T = inflow total P, µg/L.

These two equations were developed from a database for 60 U. S. Army Corps of Engineers' reservoirs and were verified for 20 other reservoirs. When this model was applied to 20 other reservoirs, 10 NURP sites and 14 other retention pond systems and small lakes, the goodness-of-fit test yielded an $R^2 = 0.8$, indicating a good job of replicating monitored total P removal (WEF and ASCE, 1998).

Sizing

State and regional stormwater management regulations and guidelines often address design criteria for the permanent pool storage volume in terms of either the average hydraulic retention time, or minimum total suspended sediment removal rate. The size of the permanent pool in relation to the contributing watershed is perhaps the single largest factor influencing pollutant removal in wet ponds. A number of wet pond sizing rules that variously specify the minimum volume of the permanent pool have been proposed to optimize pollutant removal. There is no individual rule that can be recommended as applicable in all cases, and the choice in many cases rests with local stormwater management policy makers. Sizing should take into account the objective that the pond should be sized to hold the permanent pool as well as the required water quality volume. An example (Schueler, 1987) is provided in Table 3-4. From the table, it can be seen that the choice of an appropriate pond sizing rule necessarily invites a trade-off between the degree of removal efficiency desired and the cost of achieving it.

Table 3-4. Summary of Wet Pond Sizing Rules

Sizing Rule	**Sediment removed**	**Phosphorus removed**	**Extra storage**	**Extra cost**
			(compared to 2 yr dry pond)	
RULE 1: 0.5 in. runoff per acre	60-90%	35-90%	35-200%	20-90%
RULE 2: 0.5 in. runoff per impervious acre	60%	35-40%	30%	20-25%
RULE 3: 0.1 to 0.8 in. depending on land use	55-80%	30-50%	30-70%	20-40%
RULE 4: 2.5 times the runoff of the mean storm	75%	55%	75%	40-50%

RULE 5: 4.0 times the runoff of the mean storm (app. 2 week retention)	85-90%	65%	200-250%	80-100%

(Schueler, 1987)

Pond Shape and Geometry

The wet basin should be configured as a two-stage facility with a sediment forebay and a main pool. Long, narrow and irregular shapes are also desirable for shallower ponds since they reduce surface area exposed to the wind and thereby prevent resuspension of previously deposited materials (Schueler, 1987). The basin should be wedge-shaped, narrowest at the inlet and widest at the outlet (CASQA, 2003).

The minimum length-to-width ratio for the permanent pool shape is 1.5:1, and should ideally be greater than 3:1 to avoid short-circuiting. Baffles, pond shaping or islands can be added within the permanent pool to increase the flow path (Atlanta Regional Commission, 2001).

Mean depth of the permanent pool is calculated by dividing the storage volume by the surface area. The mean depth should be shallow enough to ensure aerobic conditions and reduce the risk of thermal stratification but deep enough to ensure that algal blooms are not excessive and reduce resuspension of settled pollutants during significant storm events.

The minimum depth of the open water area should be greater than the depth of sunlight penetration to prevent emergent plant growth in this area, namely, on the order of 2 to 2.5 m (6 to 8 ft). A mean depth of approximately 1 to 3 m (3 to 10 ft) should produce a pond with sufficient surface area to promote algal photosynthesis and should maintain an acceptable environment within the permanent pool for the recommended average hydraulic retention times, although separate analyses should be performed for each locale (WEF and ASCE, 1998).
If the pond has more than 0.8 ha (2 acre) of water surface, mean depths of 2m (6.5 ft) will protect it against wind generated resuspension of sediments. A water depth of approximately 1.8 m (6 ft) over the major portion of the pond will also increase winter survival of fish (Schueler, 1987). A maximum depth of 3 to 4 m (10 to 13 ft) should reduce the risk of thermal stratification; however, in the state of Florida, pools up to 9.2 m (30 ft) deep have been successful when excavated in high groundwater areas, probably because of improved circulation at the bottom of the pond as a result of the movement of groundwater through it.

The perimeter of all permanent pool areas with depths of 4 ft or greater should be surrounded by an aquatic bench that extends inward 5 to 10 ft from the perimeter of the permanent pool and should be no more than 18 in. below normal depth. The area of the bench should not exceed about 25% of pond surface. The depth in the center of the basin should be 4 to 8 ft deep to prevent vegetation from encroaching on the pond open water surface.

Side Slopes Along Shoreline and Vegetation

Side slopes along the shoreline of the retention pond should be 4H:1V. CASQA (2003) recommends 3:1 or flatter to facilitate maintenance and reduce public risk of slipping and falling into the water. Additionally, a littoral zone should be established around the perimeter of the permanent pool to promote the growth of emergent vegetation along the shoreline and deer populations from wading. This bench for emergent wetland vegetation should be at least 3 m (10 ft) wide with a water depth of 0.15 to 0.45 m (0.5 to 1.5 ft). The total area of the aquatic bench should be 25 to 50% of the permanent pool's water surface area. The use of wetland vegetation within shallow sections of the permanent pool should adhere to guidelines issued by local agricultural agencies or commercial nurseries. Emergent plants such as bulrush, three-square and lizards tail can provide an attractive fringe habitat (Schueler, 1987), providing food and cover for wildlife and waterfowl.

Extended-detention Zone Above the Permanent Pool

Some state or local regulations require detention of a specified runoff volume as surcharge above the permanent pool, in order to reduce short-circuiting and enhance settling of total suspended sediments. Although the addition of an extended-detention zone above the permanent pool may not likely produce measurable increases in the removal of total suspended sediments, it is still recommended to have a surcharge extended detention volume, and whenever one is used or required, it is suggested in these local guidelines that the maximum event-based volume with a 12 h drain time be used..

Minimum and Maximum Tributary Catchment Areas

Stormwater ponds should have a minimum contributing drainage area of 25 acres or more for a wet pond to maintain a permanent pool. The minimum drainage area should permit sufficient baseflow to prevent excessive retention times or severe drawdown of the permanent pool during dry seasons. It is recommended that a water balance calculation be performed using local runoff, evapotranspiration, exfiltration, and baseflow data to ensure that the baseflow is adequate to keep the pond full during the dry season. The maximum tributary catchment area should be set to reduce the exposure of upstream channels to erosive stormwater flows, reduce effects on perennial streams and wetlands, and reduce public safety hazards associated with dam height (WEF and ASCE, 1998).

Construction of Retention Ponds in Wetland Areas

Although wet pond BMPs are typically designed to enhance pollutant removal by incorporating wetland areas along the perimeter, regulatory agencies may restrict their use if a significant amount of native wetlands will be submerged within the permanent pool. If field inspections indicate that a significant wetlands area will be affected at a particular site, and if the construction of a wet pond is inevitably required, the following options may be pursued during final design subject to approval: investigate moving the embankment and permanent pool upstream of the major wetland area; and if this is not feasible, a wetland mitigation plan can be developed as a part of the retention pond design.

Forebay

A sediment forebay with a hardened bottom should be constructed near the inlet to trap coarse sediment particles in order to reduce the frequency of major clean-out activities within the pool area. The forebay storage capacity should be approximately 10% of the permanent pool storage and should be at least 3 ft deep (WEF and ASCE, 1998; CASQA, 2003). Exit velocities from the forebay should not be erosive. A fixed vertical sediment depth marker should be installed in the forebay to measure sediment accumulation. Access for mechanized equipment should be provided to facilitate removal of sediment. The forebay can be separated from the remainder of the permanent pool by one of several means: a lateral sill with wetland vegetation; two ponds in series; differential pool depth; rock-filled gabions; a retaining wall; or a horizontal rock filter placed laterally across the permanent pool.

Inlet and Outlet Structures

The inlet design should dissipate flow energy and diffuse the inflow plume where it enters the forebay or permanent pool. Examples of inlet designs include drop manholes, energy dissipaters at the bottom of paved rundowns, a lateral bench with wetland vegetation, and the placement of large rock deflectors.

An outlet for a retention pond typically consists of a riser with a hood or trash rack to prevent clogging and an adequate anti-vortex device for basins serving large drainage areas. A few examples are outlet works with surcharge detention for water quality, negatively sloped pipe outlet with riser, and multiple orifice outlet. Anti-seep collars should be installed along outlet conduits passing through or under the dam embankment. If the pond is part of a larger peak-shaving detention basin, the outlet should be designed for the desired flood control performance. An emergency spillway must be provided and designed using accepted engineering practices to protect the basin's embankment. The pond embankment and spillway should be designed in accordance with federal, state, and local dam safety criteria. For on-line facilities, the principal and emergency spillways must be sized to provide 1.0 ft. of freeboard during the 25-yr event and to safely pass the 100- yr flood (CASQA, 2003). The channel that receives the discharge from the basin's outlet should be protected from erosive discharge velocities. Options include riprap lining of the channel or providing stilling basins, check dams, rock deflectors, or other devices to reduce outfall discharge velocities to nonerosive levels.

When the pond is designed as an off-line facility, a splitter structure is used to isolate the water quality volume. The splitter box, or other flow diverting approach, should be designed to convey the 25- yr event while providing at least 1.0 ft of freeboard along pond side slopes.

Each pond must have a bottom drain pipe with an adjustable valve that can completely or partially drain the pond within 24 h. However, this requirement may be waived for coastal areas, where positive drainage is difficult to achieve due to very low relief (Atlanta Regional Commission, 2001). The pond drain pipe should be sized one pipe size greater than the calculated design diameter. The drain valve is typically a handwheel-activated knife or gate valve. Valve controls shall be located inside of the riser at a point where they (i) will not normally be inundated, and (ii) can be operated in a safe manner. Figure 3-2 provides an example schematic of a wet pond (CASQA, 2003)

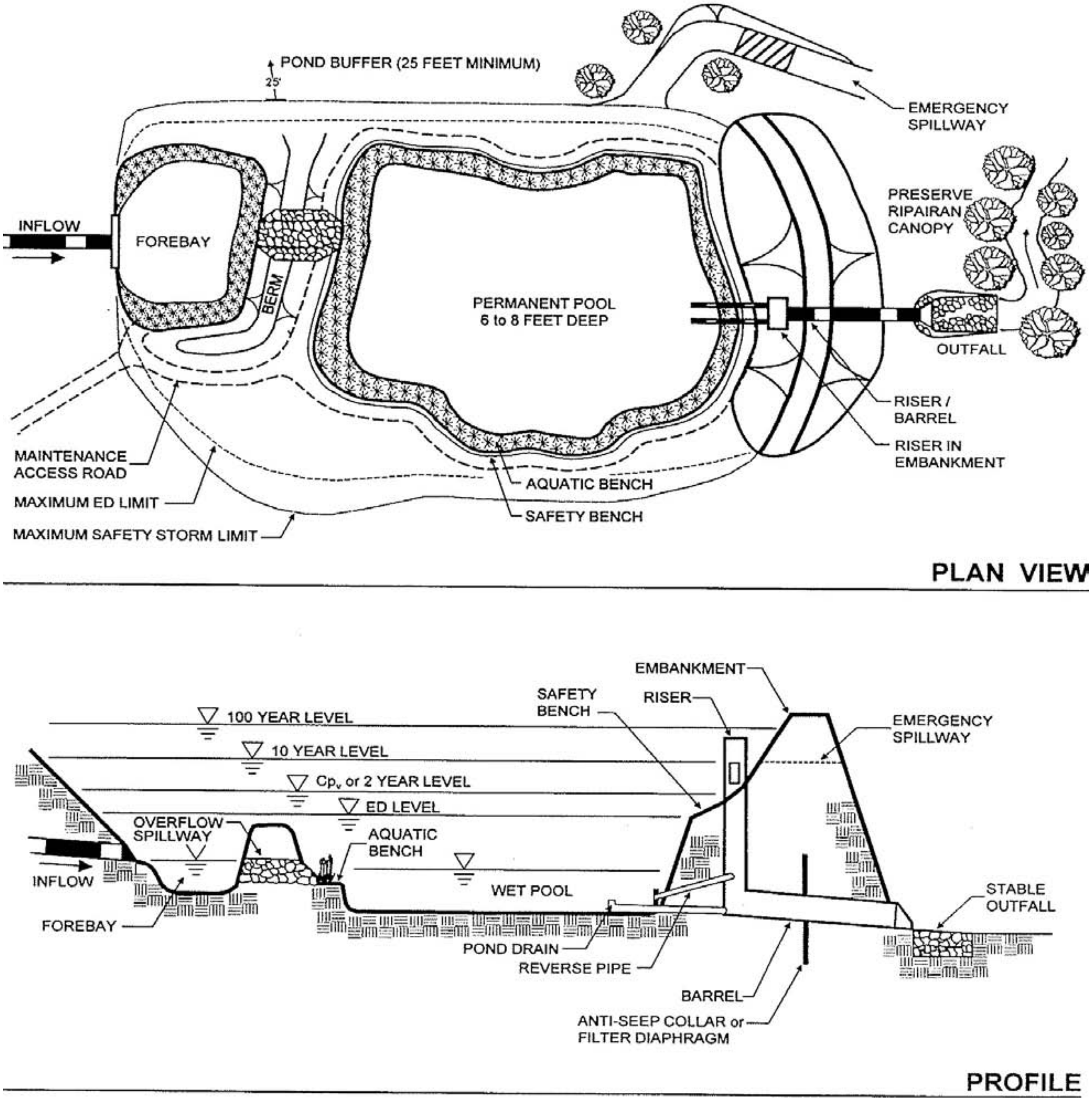

Figure 3-2. **Schematic of a Wet Pond** (CASQA, 2003)

3.3.3 Stormwater Wetlands

Stormwater wetlands can be defined as constructed wetland systems that are explicitly designed to mitigate the impacts of stormwater quality and quantity that occur during the process of urbanization. They do so by temporarily storing stormwater runoff in shallow pools that create growing conditions suitable for emergent and riparian wetland plants and routing runoff through vegetation to maximize contact. The runoff storage, complex micro topography and emergent plants in the stormwater wetland together form an ideal matrix for the removal of urban pollutants (Schueler, 1992a). Stormwater wetlands have been characterized as having one of five basic designs: (1) shallow marsh system; (2) pond/wetland system; (3) extended detention wetland; (4) pocket wetlands; and, (5) fringe wetlands.

Conventional stormwater wetlands are shallow pools that create growing conditions suitable for the growth of marsh plants. These are constructed systems and typically are not located within delineated natural wetlands. Stormwater wetlands differ from other artificial wetlands created to comply with mitigation requirements in that they may not replicate all the ecological functions of natural wetlands. However, as with natural wetlands, stormwater wetlands require a continuous baseflow or a higher table to support aquatic vegetation (Atlanta Regional Commission, 2001). Functional differences depend on the design of the wetland, interactions with groundwater and surface water, and local storm climate (WY DEQ, 1999). Enhanced stormwater wetlands designed for more effective pollutant removal and species diversity also include design elements such as a forebay, complex microtopography, and pondscaping with multiple species of wetland trees, shrubs and plants.

3.3.3.1 Stormwater Control

Constructed stormwater wetlands should generally not be used for flood control or stream channel erosion control due to the anticipated water level fluctuations associated with quantity controls (VA DCR, 1999). The clearing of vegetation and the addition of impervious surfaces may cause large and sudden surges of runoff during rain events, and may cause less than normal baseflows during dry periods. Large, sudden fluctuations in water levels can stress emergent wetland and upland edge vegetation, most of which cannot survive drought or saturation extremes, leaving wetland banks exposed to potential erosion. The large surface area requirement for constructed stormwater wetlands will help to minimize the "extreme" water level fluctuations during all but the larger storm events and the wetland design should allow for gradual increases and increases in wetland design.

3.3.3.2 Pollutant Removal Capability

Wetlands remove pollutants through gravitational settling, wetland plant uptake, adsorption, physical filtration and microbial decomposition. Primary removal of stormwater pollutants occurs during the relatively long quiescent period between storms (WY DEQ, 1999) and the degree of pollutant removal is a function of aquatic treatment volume, surface area-to-volume ratio, and the ratio of wetland surface area to watershed area. Pollutant removal is also expected to increase with longer storm water flow paths through the wetland and longer residence time within the wetland. Conventional stormwater wetlands have a high pollutant removal capability that is generally comparable to that of wet ponds. While sediment removal may be greater in well designed facilities, phosphorus removal is more variable. Some cases of negative removal for ammonia and orthophosphorous are reported; the addition of ammonia or orthophosphorous may be due, in part, to wildlife use and populations and vegetation management (WY DEQ, 1999). According to Strecker *et al.*, (1990), overall performance is greatest during the growing season and lowest during the winter months.

Positive factors influencing pollutant removal include:
- constant pool elevations,
- range of micro-topography within the watershed,
- sediment forebay,
- high surface area to volume ratio,
- constructed wetland performs better than natural wetland,
- adding greater retention volume and/or detention time to the wetland,
- effective in areas with high water table or poorly drained soils, and
- lengthy travel paths for stormwater.

Negative factors influencing pollutant removal include::
- lower removal rate during non-growing seasons,
- concentrated inflows,
- sparse wetland cover, and
- ice cover or snowmelt runoff that would require a modification in design.

3.3.3.3 Design Considerations

General Considerations

A well-designed stormwater wetland consists of:
- shallow marsh areas of varying depths with wetland vegetation,
- permanent micropool, and
- overlying zone in which runoff control volumes are stored.

In addition, all wetland designs must include a sediment forebay at the inflow to the facility to allow heavier sediments to drop out of suspension before the runoff enters the wetland marsh. Additional pond design features include an emergency spillway, maintenance access, safety bench, wetland buffer, and appropriate wetland vegetation and native landscaping (Atlanta Regional Commission, 2001).

Specific site conditions are important to the proper design of a wetland. Key site characteristics include soils, hydro period, and plant species and density. Depth to the confining layer or groundwater is important to ensure that the wetland does not dry up during extended periods of no rainfall. In addition, a constant source of surface water is recommended, taking appropriate measures to prevent the undesirable consequences of stagnant water in the wetlands. The depth and duration of maximum submergence are important because an excess of either will kill the vegetation (WEF and ASCE, 1998).

Location and Siting

A continuous baseflow or high water table is required to support wetland vegetation. A water balance must be performed to demonstrate that a stormwater wetland can withstand a 30-day drought at summer evaporation rates without completely drawing down (Atlanta Regional Commission, 2001).

Stormwater wetlands should normally have a minimum contributing drainage area of 25 acres or more and the minimum drainage area is 5 acres for a pocket wetland.

Wetland siting should also take into account the location and use of other site features such as natural depressions, buffers, and undisturbed natural areas, and should attempt to aesthetically fit the facility into the landscape. Bedrock close to the surface may prevent excavation.

Stormwater wetlands cannot be located within navigable waters of the U.S., (including wetlands), without obtaining a Section 404 permit under the Clean Water Act and any other applicable state permit. In some isolated cases, a wetlands permit may be granted to convert an existing degraded wetland through local watershed restoration efforts.

Minimum setback requirements for stormwater wetland facilities unless specified by local ordinances or criteria are as follows:

- from a property line - 10 ft,
- from a private well - 100 ft; if well is downgradient from a hotspot land use then the minimum setback is 250 ft, and
- from a septic system tank/leach field - 50 ft.

If a wetland facility is not used for overbank flood protection, it should be designed as an offline system to bypass higher flows rather than passing them through the wetland system.

Sizing

For optimal pollutant removal, a stormwater wetland must meet the following seven basic sizing criteria:

- contain a treatment volume (V_t) that is capable of capturing the runoff generated by 70 to 90% of the runoff-producing storms in the region on an annual basis,
- have a minimum surface area in relation to the contributing watershed area,
- allocate the surface area of the wetland to meet targets for certain depth zones,
- meet a minimum standard for the internal flow path through the wetland,

- demonstrate that the water supply to the wetland is greater than the expected loss rate so that water elevations can be maintained, and
- provide for extended detention for smaller storms (for ED wetlands only)(Schueler, 1992b).

Physical Specifications/Geometry

Recommended hydraulic design criteria for wetlands are as follows:

- Maintain dry-weather flow depths that vary through the wetland between 0.1 and 1.2 m (0.5 to 4 ft), depending on the types of vegetation planted, with the outlet structure designed so that the wetland can be periodically drawn down completely to dry the sediments; this provides for natural oxidation of built-up organics.

- Size the wet-weather storage volume using the methodology for ED ponds (discussed in sec 3.3.1.3) but with a maximum surcharge depth above the dry-weather flow depth of 0.6 m (2 ft) and a drawdown time of 24 h; this will reduce stress on herbaceous wetland plants. The 0.6 m depth limitation will determine the surface area required for the wetland.

- Design inlet structures to achieve sheet flow across the wetland to the maximum extent possible.

- Design the outlet structure to control the water surface and protect it from plugging by floatables common in wetlands.

- If open water is to be included in the wetland, it should be less than 50% of the total wetland area; the depth of the open water should follow the rules for the maximum permanent pool depth in retention ponds (WEF and ASCE, 1998).

In general, wetland designs are unique for each site and application. However, there are a number of geometric ratios and limiting depths for the design of a stormwater wetland that are recommended for adequate pollutant removal, ease of maintenance, and improved safety. Table 3-5 provides the recommended physical specifications and geometry for the various stormwater wetland design variants. .

Table 3-5 Recommended Design Criteria for Stormwater Wetlands

Design Criteria	**Shallow Wetland**	**ED Shallow Wetland**	**Pond/ Wetland**	**Pocket Wetland**
Length-to-width ratio (minimum)	2:1	2:1	2:1	2:1
Extended-detention (ED)	No	Yes	Optional	Optional
Allocation of WQ_v volume (pool/marsh/ED) in %	25/75/0	25/25/50	70/30/0 (includes pond volume)	25/75/0
Allocation of surface area (deep water/low marsh/high marsh/semi-wet) in %	20/35/40/5	10/35/45/10	45/25/25/5 (includes pond surface area)	10/45/40/5
Forebay	Recommended	Recommended	Recommended	Optional
Micropool	Recommended	Recommended	Recommended	Recommended
Outlet configuration	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Hooded broad-crested weir

Depth:
Deepwater: 1.5 to 6 ft below normal pool elevation
Low Marsh: 6 to 18 in. below normal pool elevation
High Marsh: 6 in. or less below normal pool elevation
Semi-wet zone: Above normal pool elevation
(Schueler, 1992a; Atlanta Regional Commission, 2001)

Depth

The stormwater wetland should be designed with the recommended proportion of the "depth zones." Each of the four wetland design variants has depth zone allocations which are given as a percentage of the stormwater wetland surface area. The four basic depth zones are given below.

Deepwater zone
The deep water zone is between 1.5 and 6 ft deep. It includes the outlet micropool and deepwater channels through the wetland facility. This zone supports little emergent wetland vegetation, but may support submerged or floating vegetation.

Low Marsh Zone
Low marsh zone is from 6 to 18 in. below the normal permanent pool or water surface elevation. This zone is suitable for the growth of several emergent wetland plant species.

High Marsh Zone
This is the zone from 6 in. below the pool to the normal pool elevation. This zone supports a greater density and diversity of wetland species than the low marsh zone and has a higher surface area to volume ratio.

Semi-wet Zone
Semi-wet zone refers to those areas above the permanent pool that are inundated during larger storm events and support a number of species that can survive flooding.

A minimum dry-weather flow path of 2:1 (length-to-width) is required from inflow to outlet across the stormwater wetland and should ideally be greater than 3:1. This path may be achieved by constructing internal dikes or berms, using marsh plantings, and by using multiple cells. Finger dikes are commonly used in surface flow systems to create serpentine configurations and prevent short-circuiting. Micro topography, or contours along the bottom of a wetland or marsh, which provides a variety of conditions for different species needs and increases the surface area to volume ratio, is encouraged to enhance wetland diversity.

A 4 to 6 ft deep micropool should be included in the design at the outlet to prevent the outlet from clogging and resuspension of sediments, and to mitigate thermal effects. In general, the maximum depth of any permanent pool areas should not exceed 6 ft.

The volume of the extended detention should not comprise more than 50% of the total WQ_v, and its maximum water surface water elevation should not extend more than 3 ft above the normal pool. Qp and/or Cpv storage can be provided above the maximum WQ_v elevation within the wetland.

The perimeter of all deep pool areas (4 ft or greater in depth) should be surrounded by safety and aquatic benches similar to those for stormwater ponds

The contours of the wetland should be irregular to provide a more natural landscaping effect.

Pretreatment/Inlets

A wetland facility should have a sediment forebay or upstream pretreatment in order to remove incoming sediment from the stormwater flow prior to dispersal into the wetland. The forebay should consist of a separate cell, formed by an acceptable barrier and should be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the wetland facility.

The forebay should be sized to contain 0.1 in. per impervious acre of contributing drainage and should be 4 to 6 ft deep. The pretreatment storage volume is part of the total WQv requirement and may be subtracted from WQv for wetland storage sizing.

A fixed vertical sediment depth marker should be installed in the forebay to measure sediment deposition over time and the bottom of the forebay may be hardened to make sediment removal easier.

Inflow channels need to be stabilized with flared riprap aprons, or the equivalent and inlet pipes to the pond can be partially submerged. Exit velocities from the forebay must be nonerosive.

Outlet Structures

Flow control from a stormwater wetland is typically accomplished with the use of a concrete or corrugated metal riser and barrel. While the riser is a vertical pipe or inlet structure that is attached to the base of the micropool with a watertight connection, the outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment. The riser is recommended to be located within the embankment for maintenance access, safety and aesthetics.

A number of outlets at varying depths in the riser provide internal flow control for routing of the water quality, channel protection, and overbank flood protection runoff volumes. The number of orifices varies and is usually a function of the pond design. Alternative hydraulic control methods to an orifice can be used and include the use of a broad-crested rectangular, V-notch, proportional weir, or an outlet pipe protected by a hood that extends at least 12 in. below the normal pool.

Higher flows pass through openings or slots protected by trash racks further up on the riser or in a separate outlet. After entering the riser, flow is conveyed through the barrel and is discharged downstream. Anti-seep collars should be installed on the outlet barrel to reduce the potential for pipe failure. Riprap, plunge pools or pads, or other energy dissipators, are to be placed at the outlet of the barrel to prevent scouring and erosion. If a wetland facility daylights to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to re-establish a forested riparian zone in the shortest possible distance.

The wetland facility should have a bottom drain pipe located in the micropool with an adjustable valve that can completely or partially de-water the wetland within 24 h. However, this requirement may be waived for coastal areas, where positive drainage is difficult to achieve due to very low relief.

The wetland drain should be sized one pipe size greater than the calculated design diameter. The drain valve is typically a handwheel activated knife or gate valve; valve controls shall be located inside of the riser at a point where they would not normally be inundated, and can be operated in a safe manner.

Emergency Spillway

An emergency spillway may be included in the stormwater wetland design to safely pass flows that exceed the design storm flows. It should be located so that downstream structures will not be affected by spillway discharges. One recommendation is to provide a minimum of 1 ft of freeboard, measured from the top of the water surface elevation for the extreme flood to the lowest point of the dam embankment, not counting the emergency spillway.

Maintenance Access

A maintenance right-of-way should be provided to the wetland facility from a public or private road. Maintenance access should be at least 12 ft wide, have a maximum slope of no more than 15%, and be appropriately stabilized to withstand maintenance equipment and vehicles. The maintenance access should extend to the forebay, safety bench, riser and outlet, and, to the extent feasible, be designed to allow vehicles to turn around. Access to the riser should be provided by lockable manhole covers and manhole steps within easy reach of valves and other controls.

Vegetation

The vegetation diversity in a constructed wetland is established by the landscape plan or volunteer vegetation. The selection of vegetation should be limited to native plant species suitable for the pool depths expected within the different depth zones (VA DCR, 1999). Care should be taken to avoid introducing exotic or invasive species. This problem can be overcome by the use of appropriate donor soil and wetland mulch. Suitable plants for created wetlands vary between different eco-regions. However, the wetland plants chosen for created wetlands should incorporate the following attributes:

- tolerance to wide ranges of water elevations, salinity, temperature, and pH,
- a mixture of perennials and annuals,
- moderate amounts of leaf production, and
- proven removal efficiencies, e.g., *Scriptus* spp. (WEF and ASCE, 1998).

Additional considerations include:

- The use of vegetation and an appropriate landscaping plan should provide elements that promote greater wildlife and waterfowl use within the wetland and buffers.

- Woody vegetation may not be planted on the embankment or allowed to grow within 15 ft of the toe of the embankment and 25 ft from the principal spillway structure.

- A wetland buffer shall extend 25 ft outward from the maximum water surface elevation, with an additional 15 ft setback to structures. The wetland buffer should be contiguous with other buffer areas that are required by existing regulations, or are part of the overall stormwater management concept plan. No structures shall be located within the buffer and an additional setback to permanent structures may be provided.

- Existing trees should be preserved in the buffer area during construction. It is desirable to locate forest conservation areas adjacent to ponds. Resident goose populations can be discouraged by planting the buffer with trees, shrubs and native ground covers.

Figure 3-3 shows a schematic of a stormwater wetland (CASQA, 2003).

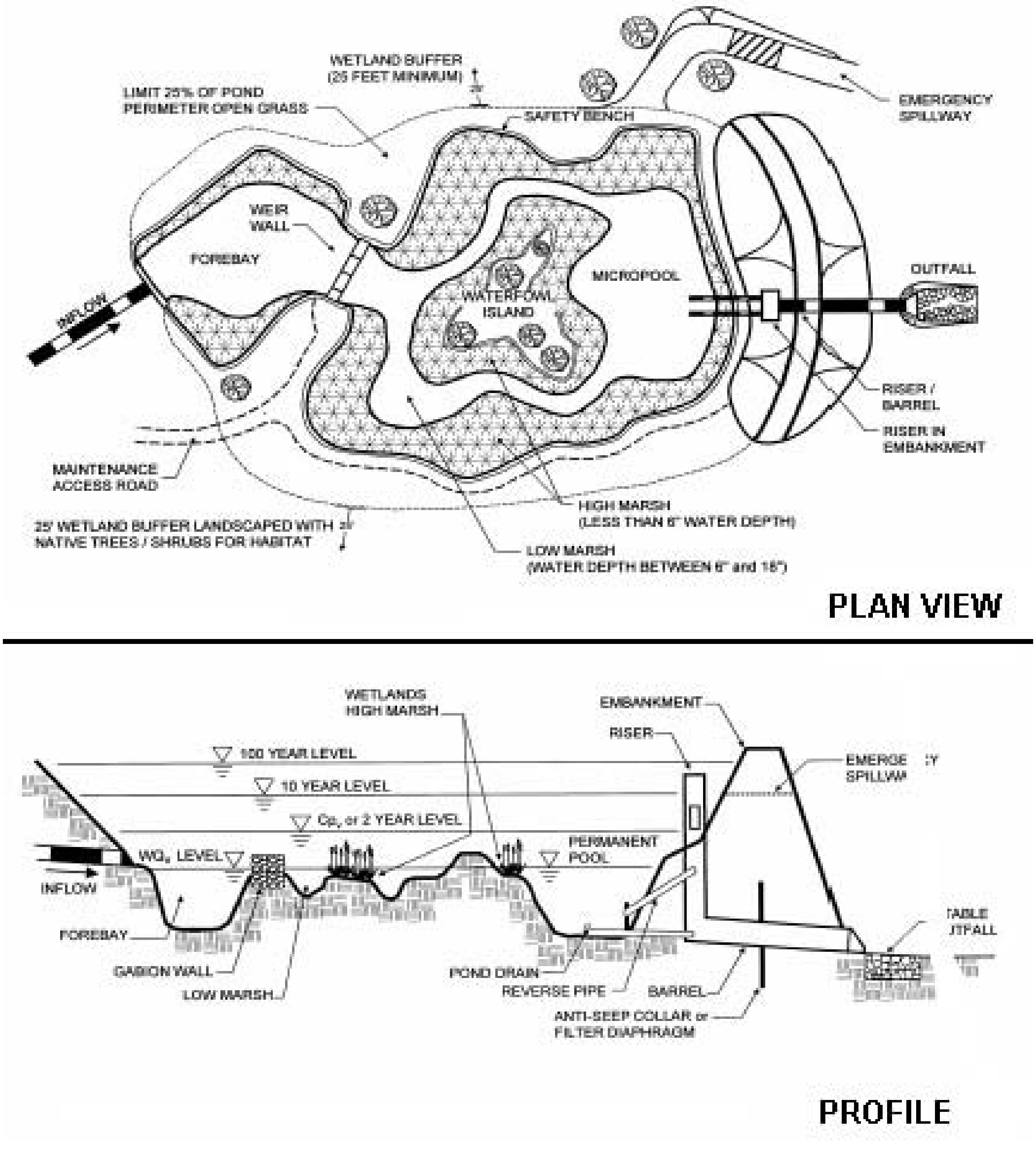

Figure 3-3. **Schematic of a Constructed Wetland** (CASQA, 2003)

3.3.4 Grassed Swales

The term "grassed swales", also known as grassed water courses or vegetated swales, refers to the use of grassed conveyances, which are essentially earthen channels vegetated with erosion resistant and flood-tolerant grasses. They are designed to infiltrate runoff from intermittent storm events or to transfer rainfall excess at a non-erosive velocity to desired locations for retention, detention, storage, or discharge. There are three variations of grassed swales: (1) traditional grass swales; (2) grass swales with a media filter; and, (3) wet swales. Although using grassed swales for the sole purpose of conveying stormwater has become a common practice in residential and institutional settings, their effective use for water quality control is a fairly recent practice, and in most cases is better accomplished in combination with other BMPs placed downstream to meet stormwater management requirements (Yousef *et al.*, 1985; MDE, 2000; CASQA, 2003). With respect to total stormwater management, the desirable attributes in vegetated grass swales include:

- slower flow velocities than pipe systems, which result in longer times of concentration and corresponding reduction of peak discharges,
- ability to disconnect directly connected impervious surfaces, such as driveways and roadways, thus reducing the computed runoff curve number (CN) and reduction of peak discharge,
- filtering of pollutants by grass media,
- infiltration of runoff into the soil profile, thus reducing peak discharges and providing additional pollutant removal, and
- uptake of pollutants by plant roots (phytoremediation) (WEF and ASCE, 1998).

A water quality swale is appropriate where greater pollutant removal efficiency is desired; the capacity to accept runoff from large design storms being limited in swales, these treatment swales must often lead into storm drain inlets to prevent large, concentrated flows from gullying/eroding the swale. Placing check dams across the flow path to temporarily pond runoff could improve the hydrologic performance of swales with regard to flow attenuation and infiltration for small design storms (Schueler, 1987).

3.3.4.1 Stormwater Control

Grassed swales and water quality swales usually provide some peak attenuation depending on the storage volume created by the check dams. However, flood control should be considered a secondary function of grassed swales since the required storage volume for flood control is usually more than the swales can provide (NVPDC and ESI, 1992). Swales act to control peak discharges in two ways:

- Reduction in runoff velocity by the grass, which depends on the length and slope of the swale, which in turn, lengthens the time needed for runoff to reach the desired control point, and can at least partially attenuate the post development peak discharge rate.
- A portion of the runoff passing through the swale infiltrates into the soil and does not appear at the downstream control point. However, this seldom exceeds a few tenths of an inch and depends on soils and slope besides the short contact time of runoff with the swale

(5-20 min.); swale soils have less infiltration capacity than undisturbed soils as they are heavily compacted to achieve the desired slope and load bearing capacity. Due to previous saturation of the swale soils by the same rain that supplies runoff to the swale, infiltration rates in a swale will almost always be near the minimum rates for the local soil type.

3.3.4.2 Pollutant Removal Capability

The primary pollutant removal mechanisms associated with grassed swales are sedimentation and infiltration into the subsoil. Adsorption and filtration mechanisms can be considered as secondary removal mechanisms (Schueler, 1987). Changes in the flow hydraulics affected by routing the flow through grassed channels increase the opportunity for infiltration of soluble pollutants, deposition of suspended solids, filtration of suspended solids by vegetation, and adsorption of soluble particles by plants. The flow rate becomes a critical design element since surface runoff must pass slowly through the filter to provide sufficient contact time for the aforementioned removal mechanisms to function effectively (NVPDC and ESI, 1992; WY DEQ, 1999). Conventional grassed swale designs have achieved mixed performance in removing particulate pollutants such as suspended solids and trace metals. They are generally unable to remove significant amounts of soluble nutrients. Design practices that increase the retention time of urban runoff will increase removal efficiencies for soluble forms of nitrogen and phosphorus (Yousef *et al.*, 1985). Biofilters that increase detention, infiltration, and wetland uptake within the swale have the potential to substantially improve swale removal rates (CASQA, 2003).

Positive factors influencing pollutant removal (WY DEQ, 1999; CASQA, 2003) include:

- check dams,
- low slopes,
- permeable subsoils and soil moisture holding capacity,
- dense grass cover or vegetation or mulches,
- long contact time,
- smaller storm events,
- coupling swales with plunge pools,
- infiltration trenches or pocket wetlands, and
- swale length greater than one hundred ft.

Negative factors influencing pollutant removal include:

- compacted subsoils,
- short runoff contact storms,
- large storm events,
- snow melt events,
- short grass heights,
- steep slope (6% or greater),
- runoff velocities greater than 1.5 fps,
- peak discharge greater than 5 cfs, and
- dry-weather flow.

3.3.4.3 Design Considerations

The basic design procedure for a swale system was developed by Chow (Chow, 1959) and has been used in a number of ways in sizing and designing grass swales of varying degrees of complexity and design robustness. A summary of the various approaches to the design procedure is presented in this white paper. Reported estimates of low pollutant removal efficiencies for grassed swales verify the need to improve standard design procedures to make them more effective for BMP purposes. Studies on swale performance are ambiguous making it hard to propose specific estimates for swale pollutant removal efficiency. The design of a grassed swale includes calculations for traditional swale parameters such as flow rate, maximum permissible velocities, etc. along with storage volume calculations for the water quality volume (VA DCR, 1999). A moderate removal of particulate pollutants can be achieved during small storms if a swale conforms to the design considerations discussed below.

General Design Recommendations

A dry swale system consists of an open conveyance channel with a filter bed of permeable soils that overlays an underdrain system. Flow passes into and is detained in the main portion of the channel where it is filtered through the soil bed. Runoff is collected and conveyed by a perforated pipe and gravel underdrain system to the outlet. A wet swale consists of an open conveyance channel that has been excavated to the water table or to poorly drained soils. Check dams are used to create multiple wetland "cells" that act as miniature shallow marshes. The dry and the wet swale are designed to treat the WQ_v through a volume-based design, and to safely pass larger storm flows. Runoff enters the channel through a pretreatment forebay or along the sides of the channel as sheet flow through the use of a pea gravel flow spreader trench along the top of the bank.

Siting

The suitability of a swale at a site will depend on land use, size of the area served, soil type, slope, imperviousness of the contributing watershed, and dimensions and slope of the swale system (Schueler *et al.*, 1992). In general, swales (dry/wet) should be sited such that the topography allows for the design of a channel with sufficiently mild slope and cross-sectional area to maintain non-erosive velocities (Atlanta Regional Commission, 2001).

Swale siting should also take into account the location and use of other site features such as buffers, undisturbed natural areas, and natural drainage courses, and should attempt to aesthetically "fit" the facility into the landscape (CASQA, 2003).

Roadside ditches should be regarded as significant potential swale/buffer strip sites and should be utilized as such whenever possible. If flow is to be introduced through curb cuts, it is recommended to place pavement slightly above the elevation of the vegetated areas. Curb cuts should be at least 12 in. wide to prevent clogging (WEF and ASCE, 1998).

Dry swales can be sited on most soils; however, native soils with low permeability need to be amended or replaced to increase infiltration. A wet swale can be used where the water table is at or near the soil surface, or where there is a sufficient water balance in poorly drained soils to support a wetland plant community (Metro Council, 2001).

The soil below the swale should not consist of too much gravel or coarse sand, as these constituents do not easily support dense vegetation. It should be undisturbed, as this area may be periodically inundated and remain wet for long periods of time. In areas with steep slopes, swales should be employed in locations where they can be parallel to the contours. Unless existing soils are highly permeable, they should be replaced with a 30 in. depth of a sand/soil mixture (approximately 50/50 mix) to ensure infiltration.

An underlying engineered soil bed and underdrain system may be utilized in areas where the soils are not permeable and the swale would remain full of water for extended periods of time (creating nuisance conditions). This soil bed should consist of a moderately permeable soil material with a high level of organic matter; e.g., 50% sand, 20% leaf mulch, 30% top soil. The soil bed should be 30 in. deep and accompanied by a perforated pipe and gravel underdrain system. In residential developments with marginal soils, it may be appropriate to provide a soil bed and underdrain system in all grassed swales to avoid possible safety and nuisance concerns.

Appropriate soil stabilization methods, such as mulch, blankets, or mats should be used before establishing vegetation. Seeding, sodding, and other items related to establishing vegetation should be in accordance with accepted erosion control and planting practices (WEF and ASCE, 1998; Metro Council, 2001).

Physical Specifications/Geometry

The detention/retention capacity of grassed swales is governed by the runoff associated with the "water quality storm." The swale length, width, depth, and slope should be designed to temporarily accommodate the WQ_v through surface ponding. The WQ_v is retained for 24 h, but ponding may continue indefinitely depending on the depth and elevation to the watertable. The WQ_v for high density residential, commercial, and industrial land uses will most likely be too high to be accommodated with most swale designs, and swales in these cases may be appropriate for pretreatment in association with other practices for these higher density land uses (Metro Council, 2001). The swale can be sized as both a treatment facility for the design storm and as a conveyance system to pass the peak hydraulic flows of the 100-yr storm if it is located online (CASQA, 2003).

Water Quality Volume

The purpose of a grassed swale used as a conveyance channel is to transport stormwater to the discharge point. However, the purpose of a water quality grassed swale is to slow the water as much as possible to encourage pollutant removal. The use of check dams will create segments of the swale that will be inundated for a period of time. The required total storage volume behind the check dams is equal to the water quality volume for the contributing drainage area to that point. However, the maximum ponding depth behind the check dams should not exceed 18 in. To insure that this practice does not create nuisance conditions, an analysis of the subsoil is recommended to verify its permeability.

Swale Geometry

A grassed swale should have a trapezoidal cross-section to spread flows across its flat bottom. Triangular or parabolic shaped sections are generally not recommended as they tend to concentrate the runoff. However, a parabolic shape could be acceptable provided the width is equal to or greater than the design bottom width for a trapezoidal cross section (Atlanta Regional Commission, 2001; Metro Council, 2001). The side slopes of the swale should be no steeper than 3H:1V to simplify maintenance and help prevent erosion (Schueler, 1987; WA DOE, 2001).

Bottom Width

The bottom width of the swale should be 2 ft minimum and 8 ft maximum in order to maintain sheet flow across the bottom and avoid concentration of low flows. The 2 ft requirement would allow for construction considerations and would ensure a minimum filtering surface for water quality treatment, while the 8 ft maximum would reduce the likelihood of flow channelization within a portion of the bottom of the swale. Widths up to 16 ft may be used if separated by a dividing berm or structure to avoid braiding (WEF and ASCE, 1998; VA DCR, 1999; Metro Council, 2001). The actual design width of the swale is determined by the maximum desirable flow depth, as discussed below.

Flow Depth

The flow depth for a water quality grassed swale should be approximately the same as the height of the grass. An average grass height for most conditions is 4 in. Therefore, the maximum flow depth for the water quality volume should be 4 in. (CWP, 1996b). According to (WEF and ASCE, 1998), the maximum depth of flow should not be greater than one-third of the gross or emergent wetland vegetation height for infrequently mowed swales or not greater than one-half of the vegetation height for regularly mowed swales, up to a maximum of approximately 75 mm (3 in.) for grass and approximately 50 mm (2 in.) below the normal height of the shortest wetland plant species in the biofilter.

Flow Velocity

The maximum velocity of the water quality volume through the grassed swale should be no greater than 1.5 fps. The maximum design velocity of the larger storms should be kept low enough so as to avoid resuspension of deposited sediments. The 2-yr storm recommended maximum design velocity is 4 fps and the 10-yr storm recommended maximum design velocity is 7 fps.

Longitudinal Slope

The slope of the grassed swale should be as flat as possible, while maintaining positive drainage and uniform flow, to permit the temporary ponding of the WQ_v within the channel without having excessively deep water at the downstream end. The minimum constructable slope is between 0.75 and 1.0% and the maximum slope depends upon what is needed to maintain the desired flow velocities as well as to provide adequate storage for the water quality volume while avoiding excessively deep water at the downstream end. Generally, a slope of between 1 and 3% is recommended. (CASQA, 2003) recommends a maximum of 2.5%, (WEF and ASCE, 1998; Metro Council, 2001) recommend 2% and (VA DCR, 1999) recommends 3%. The slope should never exceed 5%.

Swale Length

Swale length is dependent on the swale geometry and the ability to provide the required storage for the water quality volume. However, the swale should have a length that provides a minimum hydraulic residence time of at least 10 minutes (CASQA, 2003)), and regardless of the recommended detention time, the swale should be not less than 100 ft in length.

Swale Capacity

The capacity of the grassed swale is a combined function of the flow volume (the water quality volume) and the physical properties of the swale such as longitudinal slope and bottom width. The depth of flow and velocity for any given set of values can be obtained by using the Manning equation or channel flow nomographs. The Manning's 'n' value, or roughness coefficient, varies with the depth of flow and vegetative cover. An n value of 0.15 is considered appropriate for flow depths of up to 4 in.(equal to the grass height). The n value decreases to a minimum of 0.03 for grass swales at a depth of approximately 12 in. A grassed swale should have the capacity to convey the peak flows from the 10-yr design storm without exceeding the maximum permissible velocities. (It must be noted that a maximum velocity is specified for the 2-yr and 10-yr design storms to avoid resuspension of deposited sediments and other pollutants and to prevent scour of the channel bottom and side slopes). The swale should pass the 10-yr flow over the top of the check dams with 6 in. minimum of freeboard. Alternatively, a bypass structure may be engineered to divert flows from the larger storm events (runoff greater than the water quality volume) around the grassed swale. However, when the additional area and associated costs for a bypass structure and conveyance system are considered, it may be more economical to simply increase the bottom width of the grassed swale. It should then be designed to carry runoff from the 10-yr design storm at the required permissible velocity. The Manning equation can be used to adjust the longitudinal slope and bottom width to achieve the maximum allowable velocity (VA DCR, 1999). The following criteria are probably most applicable in warm and temperate non-semi-arid climates and should be met or exceeded during the biofiltration capacity design event (WEF and ASCE, 1998): "maximized" runoff hydraulic residence time of 5 min or more; maximum flow velocity less than 0.3 m/s (0.9 ft/sec); Manning's n = 0.20 for routinely mowed swales, and Manning's n = 0.24 for infrequently mowed swales.

One recommended procedure for designing grassed swales (Claytor and Schueler, 1996) outlines a series of guidelines as shown below and in Table 3-6:

- Compute the water quality treatment volume (WQ_v) for the given land surfaces as required by the local permitting agency.
- Identify the required swale bottom width, depth, length and slope necessary to store the WQ_v within a shallow ponding depth (18 in. maximum).
- Compute the WQ_v drawdown time to ensure that it is less than 24 h.
- Compute the 2-yr and 10-yr frequency storm event peak discharges.
- Check the 2-yr velocity for erosive potential (adjust swale geometry, if necessary, and reevaluate WQ_v design parameters).
- Check the 10-yr depth and velocity for capacity (adjust swale geometry, if necessary, and reevaluate WQ_v design parameters).
- Provide minimum freeboard above 10-yr stormwater surface profile (6-in. minimum recommended).

Table 3-6. Design Criteria for Dry (and Wet) Swale Systems

Parameter	**Swale Design Criteria**
Pretreatment volume	.05 in. per impervious acre, at initial flow point
Preferred shape	Trapezoidal or parabolic
Bottom width	2 ft minimum, 8 ft maximum; widths up to 16 ft are allowable if a dividing berm or structure is used
Side slopes	2:1 maximum, 3:1, or flatter preferred
Longitudinal slope	1.0% to 2.0% without check dams
Sizing criteria	Length, width, depth and slope needed to provide surface storage for WQ_v. Outlet structures, when used, should be sized to release WQ_v over 24 h
Underlying soil bed	Equal to swale width Dry Swale: Moderately permeable soils, 30 in. deep with gravel/pipe underdrain system if needed Wet Swale: Undisturbed soils, no underdrain system
Depth and capacity	Surface storage of WQ_v with a maximum depth of 18 in. for water quality treatment (12in. average depth) Safely convey 2 yr storm with non-erosive velocity ($\leq$ 4.0 ft/s) Adequate capacity for 10 yr storm with 6 in. of freeboard

(Claytor and Schueler, 1996)

Pretreatment/Inlets

Inlets to swales must be provided with erosion controls as needed (e.g., riprap, flow spreaders, energy dissipators, sediment forebays, etc.).

Pretreatment of runoff in both dry and wet swale system is typically provided by a sediment forebay located at the inlet. The pretreatment volume should be equal to 0.1 in. per impervious acre. A forebay large enough to accommodate 25% of the water quality volume is created by installing a check dam, constructed of timber or concrete, between the inlet and the main body of the swale and/or driveway crossings (Atlanta Regional Commission, 2001; Metro Council, 2001). The checkdam should overlay a stone base to prevent downstream scour. The area downstream of the checkdam should be protected from scouring with riprap or channel lining. In the undesirable event of clogging in the surface soils, a checkdam may also be installed at the downstream end of the swale, along with an optional pea gravel window to route water to the underdrain.

Enhanced swale systems that receive direct concentrated runoff may have a 6 in. drop to a pea gravel diaphragm flow spreader at the upstream end of the control. A pea gravel diaphragm and gentle side slopes should be provided along the top of the channels to provide pretreatment for lateral sheet flows.

Check dams

Check dams are utilized in swales for two reasons: to increase pollutant removal efficiency and/or to compensate for steep longitudinal slope. The dams should be installed perpendicular to the direction of flow and anchored into the slope of the channel. The side slopes of the check dams should be between 5 and 10 to 1 to facilitate mowing operations. The berm height should not exceed 0.6 m (2 ft), and water ponded behind the berm should infiltrate into the soils within 24 h (UDFCD, 2002). Check dams should be spaced so that the toe of the upstream dam is at the same elevation as the top of the downstream dam. For best performance, check dams should have a level upper surface rather than the uneven surface of a riprap check dam. Earthen check dams are also not recommended due to erosion potential and high maintenance effort.

Level Spreaders

Level spreaders are diminutive check dams used to provide a uniform flow distribution across the swale bottom. The hydraulic design of the swale assumes a uniform distribution, which is difficult to attain without the aid of level spreading devices. The device, placed at the swale inlet, may consist of a shallow weir across the channel bottom, a stilling basin, or perforated pipe. A sediment clean-up area should be provided for ease of maintenance.

Flow Bypass

Flow bypass should be considered for high flow events to avoid erosion and channelization. Flow bypass also allows diversion of flows during swale maintenance, regrading, and vegetation establishment. 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: "first-flush" treatment or design flow treatment. The "first-flush" treatment is based on the principle that storm event pollutants are more concentrated during the "first-flush." Biofiltration swales can be designed for treating stormwater only from this initial portion of the storm event, and would require bypassing stormwater flow around the swale during higher portions of 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.

Riprap

Riprap is used as an energy dissipation or erosion control device in grassy swales. Riprap pads, consisting of 152 to 228 mm (6 to 9 in.) rocks that fit tightly across the bed may be used as an energy dissipater at the swale inlet and continuing for a distance of 1.5 to 3 m (5 to 10 ft) downstream. Riprap can also be used to line the swale channel if erosion and/or channelization of the swale bottom are of concern. Riprap could also be used with check dams as described above.

Outlet Structures

Discharges from grassed swales must be conveyed at non-erosive velocities to either a stream or a stabilized channel to prevent scour at the outlet of the swale. In dry swales, the underdrain system should discharge to the storm drainage infrastructure of a stable outfall; in wet swales, outlet protection must be used at any discharge point to prevent scour and downstream erosion.

Emergency Spillway

Enhanced swales must be adequately designed to safely pass flows that exceed the design storm flows.

Landscaping and Vegetation

Landscape design should specify proper grass species and wetland plants based on specific site, soil, and hydric conditions present along the channel. A dense cover of water-tolerant, erosion-resistant grass or other vegetation must be established. Grasses used in swales should have the following characteristics:

- a deep root system to resist scouring,
- a high stem density, with well-branched top growth,
- tolerance to flooding,
- resistance to being flattened by runoff, and
- an ability to recover growth following inundation.

Recommended grasses include but are not limited to the following: Kentucky-31, tall fescue, reed canary grass, redtop, rough-stalked blue grass, switch grass, little blue stem, and big blue stem. It should be noted that these grasses can be mixed.

The selection of an appropriate vegetative lining for a grassed swale is based on several factors including climate, soils, and topography (VA DCR, 1999). Erosion control matting should be used to stabilize the soil before seed germination. This protects the swale from erosion during the germination process. In most cases, the use of sod may be required to provide immediate stabilization on the swale bottom and/or side slopes (WEF and ASCE, 1998). Figures 3-4 and 3-5 provide a representative typical section including both a cross-section and plan view of a dry and wet swale respectively (MDE, 2000).

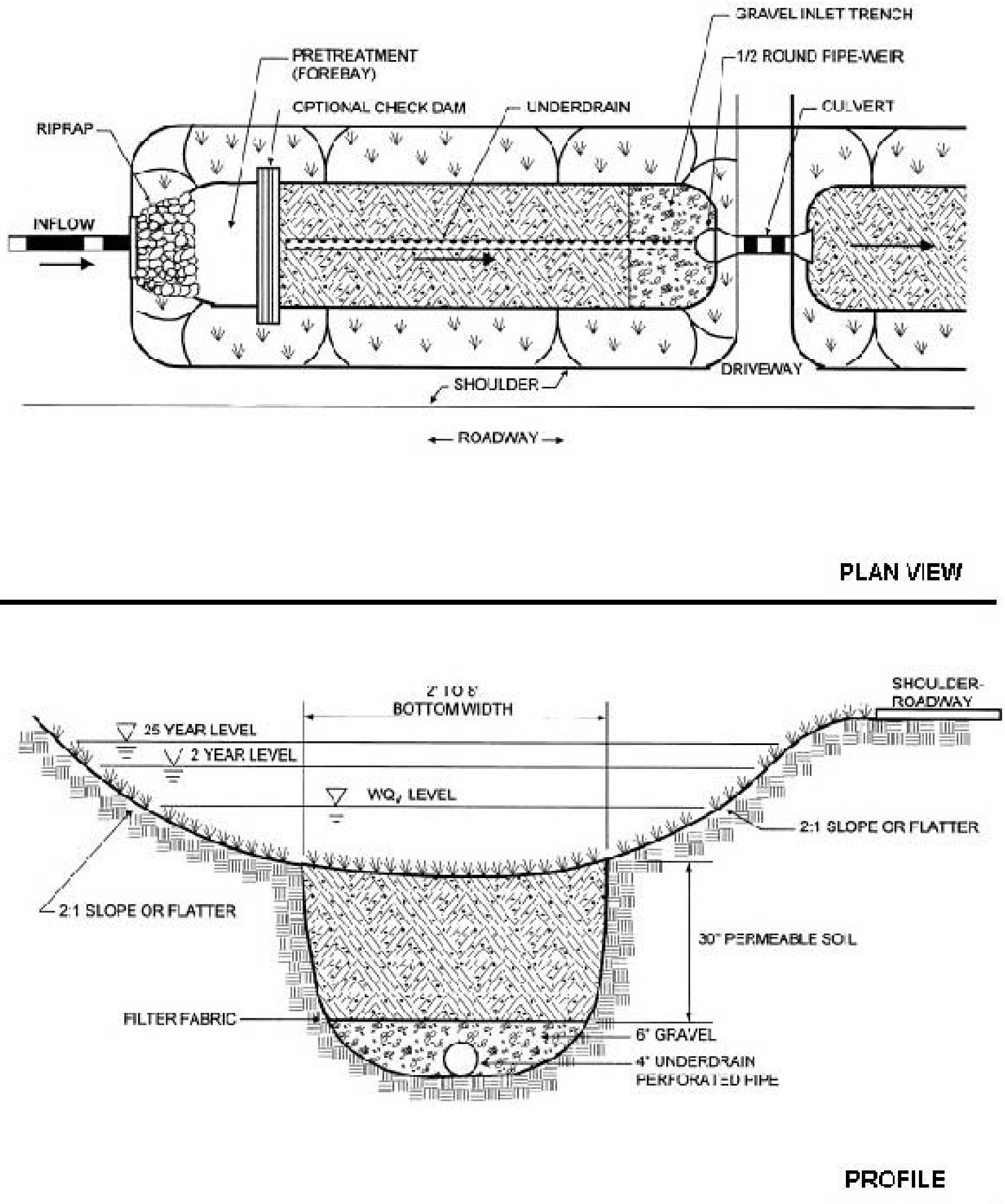

Figure 3-4 Schematic of a Dry Swale (MDE, 2000)

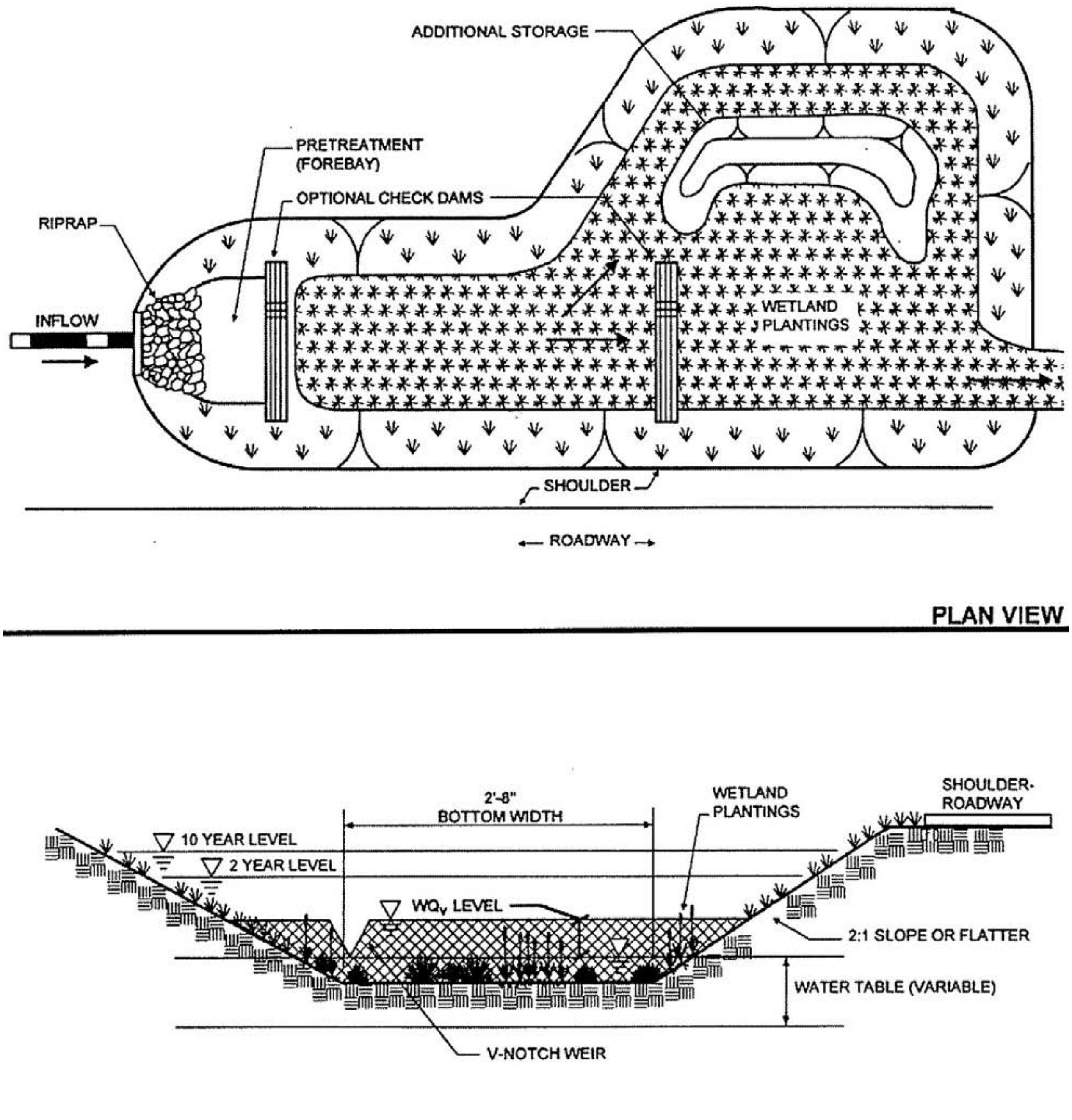

Figure 3-5 Schematic of a Wet Swale (MDE, 2000)

3.3.5 Vegetated Filter Strips (VFS)

Filter strips are uniformly graded and densely vegetated sections of land, engineered and designed to treat runoff and remove pollutants through vegetative and soil filtering, evapotranspiration, and infiltration. Filter strips are similar to grassed swales in many respects, except that they are designed to accept only overland sheet flow. The requirement that the runoff from an adjacent impervious area be evenly distributed across the filter strip is not an easy task due to the strong tendency of runoff to concentrate and form a channel, reducing the performance efficiency of the filter strip and in some cases leading to erosion of portions of the filter strip (Schueler, 1987). VFS are best suited to treat runoff from roads and highways, roof down spouts, very small parking lots, and pervious surfaces. They can function as the outer zone of a stream buffer, or as a pretreatment for other structural stormwater controls (WY DEQ, 1999; Atlanta Regional Commission, 2001). To function properly, a filter strip must be equipped with some level spreading device, densely vegetated with a mix of erosion resistant plant species that effectively bind the soil, graded to a uniform, even, and relatively low slope, and at least as long as the contributing runoff area (Knoxville, 2003). There are two filter strip designs: a simple filter strip and another design that includes a permeable berm at the bottom. The presence of the berm increases the contact time with the runoff, thus reducing the overall width of the filter strip required to treat stormwater runoff. As filter strips are typically an on-line practice, they must be designed to withstand the full range of storm events without eroding.

3.3.5.1 Stormwater Control

Filter strips do not provide enough storage or infiltration to effectively reduce peak discharges to pre-development levels for design storms (Schueler, 1987; NVPDC and ESI, 1992). The lowering of runoff velocities and runoff volume, observed sometimes in VFS, may not be typically adequate for controlling stream channel erosion or flooding (NVPDC and ESI, 1992). Little attenuation of peak runoff rates and volumes is observed for larger events, depending on soil properties, suggesting the practice of following strips with another BMP option that can reduce flooding and erosion downstream (CASQA, 2003). The increasing use of filter strips as a pretreatment BMP in integrated stormwater management systems helps lower runoff velocities and hence the watershed time of concentration, slightly reduce both runoff volumes and watershed imperviousness, and contribute to groundwater recharge.

3.3.5.2 Pollutant Removal Capability

Both swales and filter strips exhibit similar mechanisms of pollutant removal. Pollutant removal from filter strips is highly variable and depends primarily on density of vegetation and contact time for filtration and infiltration and soil moisture absorption capacity (Atlanta Regional Commission, 2001). The mechanisms include the filtering action of vegetation, deposition in low velocity areas, or by infiltration into the subsoil. The rate of removal appears to be a function of the length, slope and soil permeability of the strip, the size of the contributing runoff area, and the runoff velocity (Schueler, 1987).

Vegetated buffer strips are generally effective in reducing the volume and mass of pollutants in runoff and when designed properly, tend to provide somewhat better treatment of stormwater runoff than swales with fewer tendencies for flow concentration and the resulting erosion (CASQA, 2003). Filter strips can effectively reduce particulate pollutant levels in areas where runoff velocity is low to moderate; however, the ability to remove soluble pollutants under the same conditions is highly variable (WY DEQ, 1999). Soluble pollutants in filter strips are removed by pollutant infiltration into the soil and subsequent uptake by rooted vegetation. However, the efficiency of soluble pollutant removal may not be high since only a small portion of the incoming runoff will be infiltrated (Schueler, 1987). Filter strips are effective in removing particulate pollutants such as sediment, organic matter, and trace metals as observed from results from small test plots and several modeling studies (Schueler, 1987). They also exhibit good removal of litter and other floatables as the water depth in these systems is well below the vegetation height. Forested filter strips appear to have greater pollutant removal capability than grass filter strips because of greater uptake and long-term retention of nutrients in forest biomass.

Positive factors influencing pollutant removal include:

- minimum strip width of fifty ft,
- slope of 5% or less,
- clay soil or organic matter surface,
- contributing area of less than 5 acre,
- grass height of 6 to 12 in., and
- sheet flow.

Negative factors influencing pollutant removal include:

- runoff velocity > 2.5 fps, depending on site conditions (Horner, 1988),
- slopes greater than 15%,
- hilly terrain, and
- unmowed filter strips.

3.3.5.3 Design Considerations

General Considerations

VFS have limited feasibility as a water quality control in ultra-urban settings with a high percentage of impervious area where runoff velocities and peak discharge rates are high and flow is concentrated. Their use is therefore primarily restricted to low and medium density residential areas (16 to 21% impervious) where they can accept rooftop runoff and runoff from pervious areas such as lawns, or as a pre-treatment component for structural BMPs in higher density developments (WY DEQ, 1999). The retrofit capability is relatively simple if enough land area is available to adequately service the contributing watershed area, and soil and slope conditions are favorable.

Filter strips should be constructed outside the natural stream buffer area whenever possible to maintain a more natural buffer along the streambank (Atlanta Regional Commission, 2001).

Forests and other natural areas should not be destroyed to create a filter strip system, as such areas may already be functional or may only need to be enhanced to function properly as treatment systems. Disturbance of native vegetation in buffer areas should be avoided whenever possible (Metro Council, 2001). Other considerations include:

- Adequate pollutant removal may not be observed on slopes over 15%; filter strips require climates that can sustain vegetative cover on a year-round basis; contributing upland area must be small (1 to 5 acres) so that runoff arrives at the filter strip as overland sheet flow; use of native vegetation or vegetation appropriate for the local climate is essential to enhance plant survival (WY DEQ, 1999; CASQA, 2003).

- Filter strips should not be used on soils that cannot sustain a dense grass cover with high retardance. It is recommended to choose grasses that can withstand relatively high velocity flows at the entrances during both dry and wet periods (Atlanta Regional Commission, 2001).

Siting

The use of buffer strips is limited to gently sloping areas where the vegetative cover is robust and diffuse and where shallow flow characteristics are possible. Slopes should not exceed 15% or be less than 1% (CASQA, 2003). The vegetative surface should extend across the full width of the area being drained. The upstream boundary of the filter should be located contiguous to the developed area. The following site conditions should be considered when selecting a vegetated filter strip as a water quality BMP:

Vegetated filter strips should be used with soils having an infiltration rate of 0.52 in./h; (sandy loam, loamy sand). Soils should be capable of sustaining adequate stands of vegetation with minimal fertilization (VA DCR, 1999). The ability to remove nutrients from surface runoff improves where clay soils or organic matter are present (WY DEQ, 1999).

A shallow or seasonally high groundwater table will potentially inhibit infiltration and one recommendation from the VA DCR (VA DCR, 1999) is to have the lowest elevation in the filter strip at least 2 ft above the water table. Filter strips should be separated from the ground water by between 2 and 4 ft to prevent contamination and should not remain wet between storms (CASQA, 2003). Greater removal of soluble pollutants can be achieved where the water table is within 3 ft of the surface, i.e., within the root zone (WY DEQ, 1999). If the soil permeability and/or depth to water table are unsuitable for infiltration, the primary function of the filter strip becomes one of filtering and settling of pollutants. This requires a modified design to allow ponding of the water quality volume at the downstream end of the filter (VA DCR, 1999). Ponding area may be created by constructing a small permeable berm using a select soil mixture, with the maximum ponding depth behind the berm as 1 ft.

A natural area that is designed to serve as a vegetated filter strip should not be used for temporary sediment control.

Flows in excess of design flow must be ensured to move across or around the strip without damaging it. Higher flows can be handled by a bypass channel or overflow spillway with protected channel section (Atlanta Regional Commission, 2001).

Physical Specifications/Geometry

Slope

Filter strips appear to be a minimal design practice because they are basically no more than a grassed slope. The general requirement is that the slope should not exceed 15% and the slope should be at least 15 ft long to provide water quality treatment. Minnesota urban small sites BMP manual (Metro Council, 2001) recommends that filter strip slopes should be no less than 1 or 2% and no greater than 6%. Greater slopes will encourage concentrated flow and flatter slopes may result in ponding. Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent erosion. The top of the strip should be installed 2 to 5 in. below the adjacent pavement, so that vegetation and sediment accumulation at the edge of the strip does not prevent runoff from entering. The flat cross-slope in filter strips ensures that runoff remains as sheet flow while filtering through the vegetation (CASQA, 2003).

Length

The filter strip should stretch the entire length of the impervious surface from where stormwater originates and when adjacent to a natural water body, it should stretch the entire length of the property of shoreline (Metro Council, 2001). VFS should be long enough to provide filtration and contact time for water quality treatment. Although smaller lengths for e.g., 15 ft, 25 ft are recommended in some states, (Atlanta Regional Commission, 2001), the recommended minimum length can be 50 to 75 ft based on the assumption that runoff changes from sheet flow to shallow concentrated flow after traveling 150 ft over pervious surfaces and 75 ft over impervious surfaces (CWP, 1996b); an additional 4 ft for any one percent increase in slope is recommended. Filter strips should be used to treat small drainage areas. Flow must enter the strip as sheet flow spread out over the width (longer dimension normal to the direction of flow) of the strip, generally no deeper than 1 to 2 in. However, the length is normally dictated by design method and the minimum length of a filter strip is:

$$W_{fMIN} = Q / q \tag{3-7}$$

where
W_{fMIN} = minimum filter strip width perpendicular to flow (ft) (Atlanta Regional Commission, 2001).

The following section highlights the recommendations and minimum design guidelines for vegetated filter strips intended to enhance water quality. The responsibility rests with the designer to decide criteria applicable to each facility with any design modifications as well as to provide for the long-term functioning of the BMP.

Maximum discharge loading per ft of filter strip width (perpendicular to flow path) is found using Manning's equation:

$$q \quad \frac{0.00236}{n} Y^{5/3} S^{1/2} \qquad \textbf{(3-8)}$$

where
q = discharge per ft of width of filter strip ($ft^3/s/ft$);
Y = allowable depth of flow (in.);
S = slope of filter strip (%); and
n = Manning's "n" roughness coefficient.
0.15 for medium grass, 0.25 for dense grass, 0.35 for very dense Bermuda-type grass.
Compliance with the design parameters will result in optimal filter strip performance (NVPDC and ESI, 1992)

Width

Filter strips must be at least 15 ft wide in the direction of flow in order to be effective, however greater widths will enhance treatment. The steeper the slope, the wider the strip should be. NRCS recommends a minimum of 150 ft of filtering buffer between a land disturbance activity and a water body; depending upon soil types and slopes, it may be even greater (Metro Council, 2001).

The width of the filter strip should generally be equal to the width of the contributing drainage area and when this is not practical, a level spreader should be used to reduce the flow width to that of the filter strip. The width of the level spreader will determine the depth of flow and runoff velocity of the stormwater as it passes over the spreader lip and into the filter strip. While a wide lip will distribute the flow over a longer level section, it reduces the potential for concentrated flows across the filter (VA DCR, 1999).

A level spreader should be provided at the upper edge of a filter strip when the width of the contributing drainage area is greater than that of the filter and may extend across the width of the filter, leaving only 10 ft open on each end. Many configurations of the level spreader can be used and include a concrete sill or weir, curb stops, curb and gutter with "saw teeth" cut into it, or a level trench (12 in wide by 24 in deep), filled with pea gravel or crushed stone; the key is to have a long, continuous, and level overflow elevation to spread the concentrated flow into sheet flow upstream of the filter strip (VA DCR, 1999; Atlanta Regional Commission, 2001). An effective technique is to use a pea gravel diaphragm at the top of the slope (a small trench running along the top of the filter strip) which serves two purposes: (i) it acts as a pretreatment device settling out sediment particles before they reach the practice; and (ii) it acts as a level spreader, maintaining sheet flow as runoff flows over the filter strip.

Pervious Berm

A pervious berm may be installed to force ponding in a vegetated filter strip. A pervious berm of sand and gravel, or soils meeting USDA sandy loam or loamy sand texture, or any other moderately permeable soil could be installed at the toe of the slope to enhance the effectiveness of the filter strip. This could also include outlet pipes flowing through the berm or an overflow weir to provide an area for temporary shallow ponding and accommodate a portion or all of the water quality volume.

A pervious berm may be installed to force ponding in a vegetated filter strip. It should be constructed using a moderately permeable soil such as ASTM ML, SM or SC. Soils meeting USDA sandy loam or loamy sand texture, with a minimum of 10 to 25 % clay, may also be used. Additional loam should be used on the berm ± 25% to help support vegetation. An armored overflow should be provided to allow larger storms to pass without overtopping the berm. Maximum ponding depth behind a pervious berm is 1 ft (VA DCR, 1999; Metro Council, 2001).

Vegetation

A filter strip should be densely vegetated with a mix of erosion resistant plant species that effectively bind the soil. The selection of plants should be based on their compatibility with climate conditions, soils, and topography and their ability to tolerate stresses from pollutants, variable soil moisture conditions, and ponding fluctuations. A filter strip should have at least two of the following vegetation types:

- deep-rooted grasses, ground covers, or vines,
- deciduous and evergreen shrubs, or
- under- and over-story trees.

Native plant species should be used if possible. As newly constructed stormwater BMPs will be fully exposed for several years before the buffer vegetation becomes adequately established, plants that require full shade, or are susceptible to winter kill or prone to wind damage should be avoided and plant materials should conform to the American Standard for Nursery Stock (VA DCR, 1999). A schematic representation of a filter strip is given in Figure 3-6 (Claytor and Schueler, 1996).

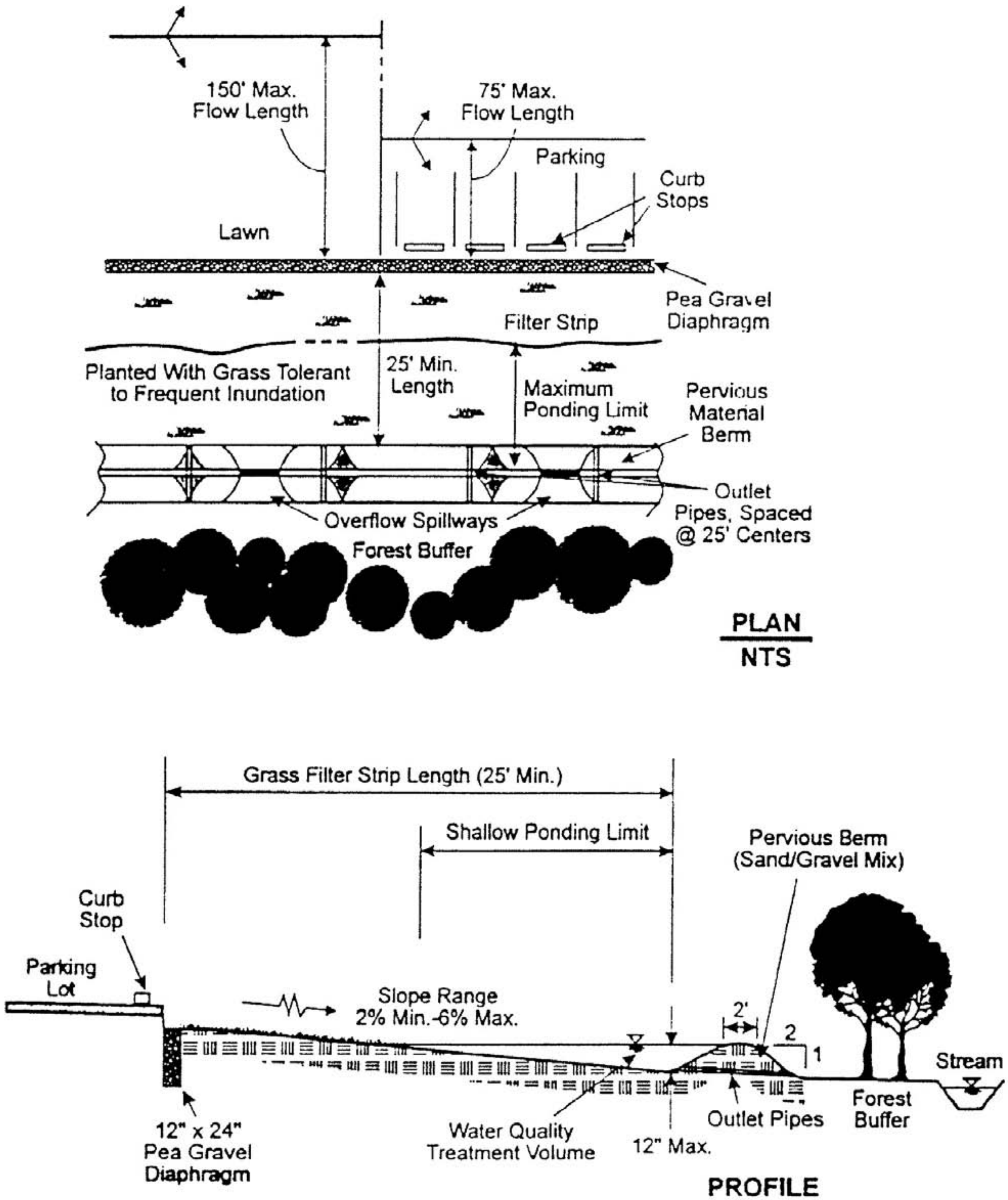

Figure 3-6. **Schematic of a Vegetated Filter Strip** (Claytor and Schueler, 1996)

3.3.6 Infiltration Trenches

A conventional infiltration trench is a shallow, excavated trench, generally 2- to 10- ft deep, that has been backfilled with a coarse stone aggregate and lined with filter fabric to create an underground reservoir for stormwater runoff from a specific design storm. Stormwater runoff diverted into the trench gradually infiltrates into the surrounding soils from the bottom and sides of the trench. This infiltration reduces the volume of runoff, removes many pollutants and provides stream baseflow and groundwater recharge. The design storm for an infiltration trench is typically a frequent, small storm such as the 1- yr event, that provides treatment for the "first flush" of stormwater runoff (0.5 in. runoff per acre of impervious surface) or even larger volumes.

The trench can be either an open surface trench or an underground facility (VA DCR, 1999) and infiltration trenches are typically implemented at the ground surface to intercept overland flows and stormwater runoff that generally enters the facility at one or more point sources. Primarily used as water quality BMPs, infiltration trench BMPs can route stormwater runoff into the aggregate filled storage chamber by two means: dispersed input or concentrated input. In dispersed input, water enters the top of the trench as overland sheet flow directed over a gently sloping grassed filter strip and flows to the surface of the storage chamber; concentrated input transports collected runoff to the storage chamber by means of gutters, curb inlets, and pipes. Some infiltration trench designs combine stormwater detention and water quality objectives by storing the entire stormwater volume with the water quality volume committed to infiltration by slowly releasing the water quality volume through an orifice set at a specified level in the storage reservoir (NVPDC and ESI, 1992).

Infiltration trenches require pretreatment of stormwater in order to remove as much of the suspended solids from the runoff as possible before it enters the trench. Also, public education with respect to street/driveway sediments may be provided in areas where an infiltration trench is proposed (Metro Council, 2001). Enhanced infiltration trenches have extensive pretreatment systems to remove sediment and oil. Conventional as well as enhanced trenches require on-site geotechnical investigations to determine appropriate design and location. Generally suited for low-to medium-density residential and commercial developments, these facilities can be incorporated in multi-use areas such as along parking lot perimeters, parking lots, residential areas, commercial areas, and open space areas. Unlike most BMPs, trenches can easily fit into the margin, perimeter, or other unused areas of developed sites, making them particularly suitable for retrofitting into existing developments or in conjunction with other BMPs (VA DCR, 1999). Infiltration may be a more promising practice in that it tends to reverse the hydrologic consequences of urban development by reducing peak discharges and increasing baseflow to local streams.

3.3.6.1 Stormwater Control

The size of the infiltration trench is determined by the volume of runoff controlled and the degree to which infiltration is used to dispose of runoff. There are three basic trench systems.

- Full exfiltration system - Runoff exits the stone reservoir by exfiltration through the underlying subsoil and the exfiltration system provides total peak discharge, volume and water quality control for all rainfall events less than or equal to the design storm (Schueler, 1987). The stone reservoir must be large enough to accommodate the entire increase in runoff volume for the design storm, less any runoff volume that is exfiltrated during a storm. Excess runoff from storms greater than the design storm should be handled by an emergency overflow channel such as a raised curb located above ground.

- Partial exfiltration system - In this design, an underground drainage system is installed that comprises regularly spaced perforated pipes located in shallow depressions to collect the runoff and direct it to a central outlet, and is generally designed to pass the 2-yr storm. Runoff from smaller storms will still be exfiltrated before it is collected, thereby providing significant water quality control. An alternative method may be to place perforated pipes on the underside near the top of the stone reservoir (NVPDC, 1987) to promote a greater degree of exfiltration, especially for design storms.

- Water quality exfiltration system - The storage volume of the stone reservoir is generally set to handle only the first flush of runoff volume during a storm, which has been variously defined as 0.5 in. of runoff per contributing impervious acre, 0.5 in. runoff per contributing total acres, and the volume of runoff produced by a 1 in. storm. Runoff volumes in excess of the first flush are not treated by the system but instead are conveyed to a stormwater management facility further downstream. While this system does not satisfy stormwater storage requirements, it may result in smaller, less costly facilities downstream (Schueler, 1987) .

3.3.6.2 Pollutant Removal Capability

The pollution removal processes that occur in infiltration systems are more complex than those occurring in wet ponds and extended detention dry ponds. Target pollutant behavior is governed by an array of factors including pH, redox potential, clay mineralogy, organic matter, microbial populations and temperature, as well as the physical characteristics of the soil environment, which change with depth and lateral distance inside the trench. While infiltration trenches are not really intended to remove a high level of coarse particulate pollutants, which need to be removed by a pre-treatment device before they enter the trench, fine particulates and soluble pollutants are effectively removed after exfiltrating through the trench and into the soil (MWCOG, 1979). Pollutant removal occurs due to sorption, precipitation, trapping, straining and bacterial degradation and transformation. It should be noted that the pollutant removal capability of water quality trenches are somewhat lower than other designs as a significant portion of the annual runoff volume will bypass a water quality trench, and is not subject to removal by exfiltration.

The pollution removal system of an infiltration system has two separate mechanisms. The sediment control system needed to maintain the function of the trench removes those pollutants associated with suspended solids such as adsorbed phosphorus, certain heavy metals and some exchangeable ions. Upon infiltration into the soil, several chemical and biological processes attenuate the levels of an array of pollutant species (NVPDC and ESI, 1992).

Infiltration trenches eliminate the discharge of the water quality volume to surface receiving waters and consequently can be considered to have 100% removal of all pollutants within this volume. Transport of some of these constituents to groundwater is likely, although the attenuation in the soil and subsurface layers will be substantial for many constituents (CASQA, 2003). The greatest sorption of nutrients and metals occurs in soils with a high content of clay and/or organic matter, with the least sorption observed in sandy soils (U.S. EPA, 1977); the same trend holds true for bacterial densities.

Positive factors influencing pollutant removal include:
- bank run or washed aggregate,
- high organic matter and loam content of subsoil,
- capture of a large fraction of annual runoff volume,
- effective pretreatment system, e.g., a sump pit; and
- pretreatment of sediments; oil; and grease.

Negative factors influencing pollutant removal include:
- sandy soils,
- trench clogging,
- high water table,
- long de-watering times,
- design considerations, and
- infiltration trench design variations.

3.3.6.3 Design Considerations

Trench designs can be distinguished as to whether they are located on the surface or below ground. Surface trenches accept diffuse runoff (sheet flow) directly from adjacent areas after it has been filtered through a grass buffer. Underground trenches accept more concentrated runoff (from pipes and storm drains), but require the installation of special inlets to prevent coarse sediment and oil/grease from clogging the stone reservoir.

Surface trenches are typically applied in residential areas, where smaller loads of sediment and oil can effectively be trapped by grass filter strips. As the surface is exposed, these trenches have a slightly higher risk of clogging than underground trenches, which could be prevented by placing a permeable filter fabric 6 to 12 in. below the surface of the trench for sediment interception. The following are a few design variations of the surface trench system.
- Median strip trench design
- Parking lot perimeter trench design
- Swale design

Underground trenches can be applied in a variety of development situations and are particularly suited to accept concentrated runoff. Pretreatment and the even distribution of concentrated runoff is an essential requirement in these systems. The top of the trench is protected by a layer of impermeable geo-textile and is covered by topsoil and planted with grass. Underground trenches may be more aesthetically pleasing, but may also be more expensive to maintain, and, more so when the trench is covered by pavement or concrete. These BMPs should only be installed when strong, enforceable maintenance agreements can be secured from the property owner. Some design variations include:

- over-sized pipe trench;
- underground trench with oil/grit inlet;
- under-the-swale design;
- dry well design; and,
- off-line trench system design.

General Considerations

Infiltration can be a very desirable method of stormwater treatment for land uses that do not heavily pollute stormwater runoff. It may be used where the subsoil is sufficiently permeable to provide a reasonable infiltration rate and where the water table is low enough to prevent pollution of groundwater. Areas containing karst topography may initially appear to have excellent infiltration, but are not recommended for planning an infiltration trench as they may cause subsurface collapse and sink-hole formation (VA DCR, 1999; Knoxville, 2003).

Paved areas subject to heavy use by motor vehicles, fueling stations, vehicle maintenance facilities, and similar areas subject to high hydrocarbon loads should be serviced by a water quality inlet as an in-line pretreatment to any infiltration structure (NVPDC and ESI, 1992).

Infiltration facilities are prone to high failure rates when designed improperly (Schueler, 1992b). This makes a strong case for designing and accepting infiltration trench systems on the basis of actual subsurface analysis and permeability tests rather than using pre-existing information on soils compiled from an array of data (VA DCR, 1999). Further, site-specific soil bores should be used to justify the use of infiltration practices. A minimum of one soil boring log is recommended for every 50 ft of trench length, with a minimum requirement of two soil boring logs for each proposed trench location (Metro Council, 2001). To identify localized soil conditions, soil boring should be done at the actual location of the proposed infiltration trench. In general, the following information should be included in a site-specific subsurface or geotechnical study.

Siting

One of the first steps in siting and designing infiltration treatment facilities is to conduct a characterization study. Geotechnical investigation data can be used for site characterization. Some of the key data and issues that need characterization include:

- surface features characterization,

- subsurface characterization,
- infiltration rate determination,
- soil testing, and
- infiltration receptor (WA DOE, 2001).

Soil Permeability

The soil types within the subsoil profile which extends a minimum of 3 ft below the bottom of the facility should be identified to verify the infiltration rate or permeability of the soil. The infiltration rate, or permeability, measured in in./h, is the rate at which water passes through the soil profile during saturated conditions, the minimum and maximum of which establish the suitability of various soil textural classes for infiltration. Each soil texture and the corresponding hydrologic properties within the soil profile are identified through analysis of a gradation test of the soil boring material. Soil textures acceptable for use with infiltration systems include those with infiltration rates between 0.52 in./h and 8.27 in./h (VA DCR, 1999), although Schueler (Schueler, 1987) recommends a minimum infiltration rate of 0.27 in./h (Table 3-7). This implies that sites with "D" soils (infiltration rates of less than 0.27 in./h), or any soil with a clay content greater than 30% (as determined from the SCS soil textural triangle) are not suitable options for infiltration trenches, nor are soils with a combined silt/clay percentage greater than 40% by weight that are susceptible to frost-heave. Silt loams and sandy clay loams ("C") soils provide marginal infiltration rates, and should only be considered for partial exfiltration systems. The stone subgrade must extend below the frost-line irrespective of the soil type, and is typically 8 to 12 in. in the Washington DC Metropolitan area. Also, trenches should not be located over fill soils that form an unstable upgrade and are prone to slope failure.

Under suitable soil conditions, soil cores or trenches to a depth of at least 5 ft below the anticipated level of the stone reservoir bottom may need to be evaluated for any impermeability in the soil strata that could impede infiltration. However, the presence of such layers does not necessarily preclude a trench, as long as the stone reservoir completely penetrates them.

Table 3-7. Soil Limitations for Infiltration Trenches

Soil Texture	**Effective Water Capacity (C_w) in. /in.**	**Minimum Infiltration Rate (*f*) (in./h)**	**SCS soil group**	**Maximum Depth of Trench (in.) 48 h**	**72 h**
*Sand	0.35	8.27	A	992	1489
**Loamy Sand	0.31	2.41	A	290	434
**Sandy Loam	0.25	1.02	B	122	183
**Loam	0.19	0.52	B	62	93

Silt Loam	0.17	0.27	C	32	49
Sandy Clay Loam	0.14	0.17	C	20	31
Clay Loam	0.14	0.09	D	11	16
Silty Clay Loam	0.11	0.06	D	7	11
Sandy Clay	0.09	0.05	D	6	9
Silty Clay	0.09	0.04	D	6	7
Clay	0.08	0.02	D	2	4

* Suitable for infiltration with typical 6' to 8' separation from seasonal high groundwater
** Suitable for infiltration with at least 3' separation from seasonal high groundwater
(Schueler, 1987; VA DCR, 1999; Knoxville, 2003)

Depth to Bedrock, Water Table, or Impermeable Layer

Typically, infiltration facilities are not recommended in areas with a high groundwater table due to the inability of the soil to adequately filter out pollutants before the stormwater enters the table. While the general requirement of various states is a distance of 2 to 4 ft, the Washington State Department of Ecology (WA DOE, 2001) recommends that the base of all infiltration trench systems shall be ≥5 ft above the seasonal high-water mark, bedrock (hardpan) or other low permeability layer. A minimum separation of 3 ft may be considered if the groundwater mounding analysis, volumetric receptor capacity, and the design of the overflow and/or bypass structures are judged to be adequate to prevent overtopping and meet the site suitability criteria (WY DEQ, 1999).

Topography

The topographic conditions of a development site represent feasibility factors that should be examined before designing an infiltration system. These factors include the slope of the land, the nature of the soil (natural/fill), and the proximity of building foundations and water supply wells. Infiltration trenches should be located in areas in which the slope does not exceed 20% (5H:1V) because steeper grade would increase the chance of water seepage from the subgrade to the lower areas of the site and reduce the volume that infiltrates. The use of infiltration trenches on fill material is not recommended due to the possibility of creating an unstable upgrade. Fill areas can be very susceptible to slope failure due to slippage along the interface of the in-situ and fill material, which could be aggravated if the fill material is allowed to become saturated by using infiltration practices (VA DCR, 1999).

Setback requirements for infiltration trenches that are required by local regulations, uniform building code requirements, or state regulations generally include the following:

- >100 ft from drinking water wells, septic tanks or drain fields, and springs used for public drinking water supplies. Infiltration trenches up gradient of drinking water supplies and within 1, 5 and 10-yr time of travel zones must comply with Health Department requirements,

- >20 ft downslope and >/100 ft upslope from building foundations,
- >20 ft from a Native Growth Protection and Easement (NGPE), and
- >50 ft from the top of slopes > 15% (WA DOE, 2001).

On-site and off-site structural stability due to extended subgrade saturation and/or head loading of the permeable layer need to be evaluated. This would include studying the potential impacts to downgradient properties, especially on hills with known side-hill seeps.

Design Criteria

Infiltration trenches are assumed to have rectangular cross-sections. Thus, the infiltration surface area (trench bottom) can be readily calculated from the trench geometry.

Sizing Procedure

The storage volume required for infiltration facilities designed for water quality enhancement is determined by the water quality volume, determined by the desired pollutant removal efficiency, and needs to be calculated using the void ratio of the backfill material that will be placed in it.

The sizing of water quality infiltration BMPs is best approached by applying Darcy's Law, which assumes that the drain time of the facility is controlled by one-dimensional flow through the bottom surface (VA DCR, 1999).

$$Q = f \times I \times SA \qquad \textbf{(3-9)}$$

where
Q = rate of exfiltration into soil ft^3/s;
f = infiltration rate of the soil in ft/h;
I = hydraulic gradient; and
SA = bottom surface area of facility in ft^2.

Infiltration Rate

Infiltration rates for treatment can be determined using either a correlation to grain size distribution from soil samples, textural analysis, or by in-situ field measurements. Short-term infiltration rates up to 2.4 in./h represent soils that typically have sufficient treatment properties, while long-term infiltration rates are used for sizing the trench based on maximum pond level and drawdown time. Long-term infiltration rates up to 2.0 in./h can also be considered for treatment if site suitability criteria are met for soil infiltration rate/drawdown time as well as soil physical and chemical suitability for treatment (WA DOE, 2001).

Historically, infiltration rates have been estimated from soil grain size distribution data using the USDA textural analysis approach. This involves conducting the grain size distribution test on soils passing the # 10 sieve (2 mm) (U. S. Standard) to determine the percentages of sand, silt and clay. The ASTM soil size distribution test procedure (ASTM D422), which considers the full range of soil particle sizes, is also being used by many laboratories to develop soil size distribution curves; however, these should not be used in conjunction with the USDA soil textural triangle (WA DOE, 2001).

The three methods for determining the long-term infiltration rate for sizing the infiltration trench are:

- USDA soil textural classification,
- ASTM gradation testing at full scale infiltration facilities, and
- in-situ infiltration measurements or pilot infiltration tests (PIT) (WEF and ASCE, 1998).

Over the life of the infiltration facility, the rate of infiltration into the soil, f, may gradually decrease due to clogging of the surface layer of the soil as a result of siltation and biomass buildup in the trench. This suggests the need for a safety or a correction factor to be built into the design of the facility to allow for future clogging, which is a factor of 2 to be applied to the infiltration rate determined from the soil analysis. The design soil infiltration rate, f_d, therefore, is equal to one-half the actual rate:

$$f_d = 0.5 f \qquad \textbf{(3-10)}$$

It must be mentioned that a value of 2 for correction factor is based on the assumption that homogeneous soils should be used for treatment soil suitability determinations (WEF and ASCE, 1998), although a value between 2 and 4 but never less than 2, could be assigned, depending on the soil textural classification (WA DOE, 1991). These correction factors consider an average degree of long-term facility maintenance, TSS reduction through pretreatment, and site variability in the subsurface conditions that affect homogeneity. However, these correction factors could be reduced, subject to the approval of the local jurisdiction, under the following conditions for sites with little soil variability:

- where there will be a high degree of long-term facility maintenance, and
- where specific, reliable pretreatment is employed to reduce TSS entering the infiltration facility.

Correction factors higher than the general recommended values should be considered under the following situations:

- difficulty in implementing long-term maintenance,
- little or no pretreatment, and
- highly variable or uncertain site conditions (WA DOE, 1991).

Hydraulic Gradient

In areas with a shallow water table or impermeable layer, the hydraulic gradient may have an impact on the allowable design depth. The hydraulic gradient is given by equation 3-11 (VA DCR, 1999):

$$I = \frac{(h + L)}{L} \tag{3-11}$$

where
I = hydraulic gradient;
h = height of the water column over the infiltrating surface (ft); and
L = distance from the top surface of the BMP to the water table, bedrock, impermeable layer, or other soil layer of a different infiltration rate (ft).

The hydraulic gradient will be assumed to be equal to one in all infiltration designs since the gradient approaches unity as the facility drains. Therefore,

$I = 1$

Maximum Ponding or Storage Time and Trench Depth

The minimum and maximum time for the trench to empty the stormwater volume into the soil by infiltration is based upon balancing optimum pollutant removal and assuring adequate stormwater management performance. Trenches should be designed in general to provide a detention time of 6 to 72 h. A minimum drainage time of 6 h should be provided to ensure satisfactory pollutant removal in the infiltration trench (Schueler, 1987). Trenches may be designed to provide temporary storage of stormwater, yet it should drain prior to the next storm event. The drainage time will vary by precipitation zone and the maximum drain time for the total design infiltration volume varies from 24 (WA DOE, 2001) to 72 h (Metro Council, 2001). The Northern Virginia Planning District Commission (NVPDC and ESI, 1992) recommends that the infiltration trench be designed with a maximum of 48 h for the water quality volume, 72 h for the total volume, and with a minimum retention time of 24 h for the water quality volume. According to the Commonwealth of Virginia Department of Conservation and Recreation (VA DCR, 1999), following the occurrence of a storm event, all infiltration trenches should be designed with a maximum drain time, $T_{\max}$, of 48 h for the water quality volume. The maximum drain time, along with the minimum design soil infiltration rate, f_d, as verified through a subsurface investigation and analysis, will dictate the maximum allowable design depth, $d_{\max}$, of the structure. The maximum depth for an infiltration trench may be defined as:

$$d_{\max} = \frac{f_d T_{\max}}{V_r} \tag{3-12}$$

where
d_{max} = maximum allowable depth of the trench, in ft;
f_d = design infiltration rate of the trench area soils, in ft/h ($f_d = 0.5f$);
T_{max} = maximum allowable drain time (48 h)
V_r = void ratio of the stone reservoir expressed in terms of the percentage of porosity divided by 100 (0.4 typically)
A void ratio of 0.40 is assumed for stone reservoirs using 1.5 to 3.5 in. stone - VDOT No. 1 Coarse-graded Aggregate (VA DCR, 1999).

The minimum surface area of the facility bottom may be defined by equation 3-13 :

$$SA_{min} = \frac{Vol_{wq}}{f_d T_{max}} \qquad \textbf{(3-13)}$$

where
SA_{min} = minimum trench bottom surface area, in ft^2;
Vol_{wq} = water quality volume requirements, in ft^3;
f_d = design infiltration rate of the trench area soils, in ft/h ($f_d = 0.5f$)
T_{max} = maximum allowable drain time = 48 h

The storage volume of the facility is defined as: $L \times W \times D \times V_r$ **(3-14)**

Determination of the dimensions of the storage reservoir is made by fitting the length, width and depth into a configuration that satisfies drain-time and storage volume requirements while keeping the storage reservoir bottom within the optimum depth for infiltration (NVPDC and ESI, 1992). A long, narrow trench is less affected by water table mounding and is advisable when the depth to seasonal high water table or bedrock is within 5 ft of the trench bottom. In order to keep the trench bottom elevation within the optimum depth in the soil profile, long trenches may need to be curved parallel to the topographic contour. If greater storage is needed than the design storm volume requirement, the trench dimensions could be adjusted by the following recommendations, by order of priority:

- Increase the length of the trench if the seasonal high water table or bedrock is within 5 ft of the trench bottom.
- Increase the width, if the length cannot be increased due to site constraints.
- It is permissible to increase the depth if the seasonal high water table and bedrock are known to be at a depth greater than 5 ft below the bottom of the trench, provided that the new bottom elevation meets the same criteria for optimum depth.

Most infiltration trenches are generally greater than 2 ft in depth, but the frost depth needs to be considered in shallow design trenches. The bottom of the structure should be 18 in. below the surface to avoid freezing of the trench bottom surface.

Backfill Material

Backfill material for the trench should be clean aggregate with a maximum diameter of 3.5 in. and a minimum diameter of 1.5 in., and the aggregate should contain few aggregates smaller than the selected size. An 8 in. bottom sand layer is required for most of the trenches to promote better drainage and reduce the risk of soil compaction when the trench is backfilled with stone (Schueler, 1992b).

Filter Fabric

The sides and bottom of the trench should be lined with geotextile fabric (filter fabric). For an aggregate surface trench, filter fabric should surround all of the aggregate fill material except the top one foot (VA DCR, 1999). A separate piece of fabric should be used for the top layer to act as a failure plan. There can be a layer of non-woven filter fabric 6 to 12 in. below the ground surface to prevent suspended solids from clogging the majority of the storage media. The filter fabric may need frequent replacement, depending on the volume of suspended solids transported to the trench.

The filter fabric material must be compatible with the surrounding soil textures and application purposes, with the cut width of the filter fabric having sufficient material for a minimum 12 in. overlap. When overlaps are required between rolls, the upstream roll must lap a minimum of two ft over the downstream roll to provide a shingled effect. The bottom of the infiltration trench can be covered with a 6 to 12 in. layer of clean sand in place of filter fabric.

Storage Media

The basic infiltration trench design utilizes stone aggregate in the top of the trench to provide adequate void space (at least 40%) (Schueler, 1987) for filtering and removing pollutants. The trench should be filled with clean, washed stone with a diameter of 1.5 to 3 in. Pea gravel could also be substituted for stone aggregate in the top 0.3 meter (1 ft) of the trench, as it improves sediment filtering and maximizes pollutant removal in the top of the trench. When these modified trenches become clogged, they can generally be restored to full performance by removing and replacing only the pea gravel layer, without replacing the lower stone aggregate layers.

Observation Well

An observation well should be installed for every 50 ft of infiltration trench length. The purpose of the well is to show how quickly the trench dewaters following a storm, as well as providing a means of determining when the filter fabric is clogged and requires maintenance. It should be installed in the center of the structure, flush with the ground elevation of the trench. This can be a 4 to 6 in. diameter PVC pipe, anchored vertically to a foot plate at the bottom of the trench, and the well should have a lockable above-ground cap (Metro Council, 2001).

Overflow Channel

Although an emergency spillway is not necessary because of the small drainage areas controlled by an infiltration trench, the overland flow path taken by surface runoff when the trench capacity is exceeded needs to be evaluated. A non-erosive overflow channel leading to a stabilized water course should be provided, as necessary, to insure that uncontrolled, erosive, concentrated flow does not develop.

Pretreatment

Infiltration trenches are susceptible to high failure rates due to clogging from sediments, and therefore require pretreatment of stormwater in order to remove as much of suspended solids as possible from the runoff before it enters the trench. Pretreatment such as grit chambers, swales with check dams, filter strips, or sediment forebays/traps should be a fundamental component of any BMP system relying on infiltration. Pretreatment facilities should be installed off-line in order to reduce both the frequency of turbulent flow-through and the associated scour and/or resuspension of residual material (Schueler, 1992b).

A grass strip or other type of vegetated buffer at least 20 ft wide should be maintained around trenches that accept surface runoff as sheet flow. The slope of the filter strip should be approximately 1% along its entire length and 0% across its width. A minimum filter length of 50 ft is desirable for areas receiving high loads of suspended solids.

All trenches with surface inlets should be engineered to capture sediment from the runoff before it enters the stone reservoir. The design of the trench must include a pretreatment facility design, complete with maintenance and inspection requirements.

Bypass

A bypass system should be implemented for all infiltration trenches. A bypass flow path should be incorporated in the design of an infiltration trench to convey high flows around the trench. The overland flow path of surface runoff exceeding the capacity of the infiltration trench should be evaluated to preclude erosive concentrated flow. If computed flow velocities do not exceed the non-erosive threshold, overflow may be accommodated by natural topography.

Groundwater Mounding

Groundwater mounding means the local elevation of the water table as a result of infiltrated surface water, and calculations may be necessary in cases where slope stability is a concern, and/or a high water table is encountered. The results from these calculations should be regarded as an indication of the mounding potential rather than as an accurate representation of the actual mounding depth. Figure 3-7 is an example schematic of an infiltration trench (MDE, 2000).

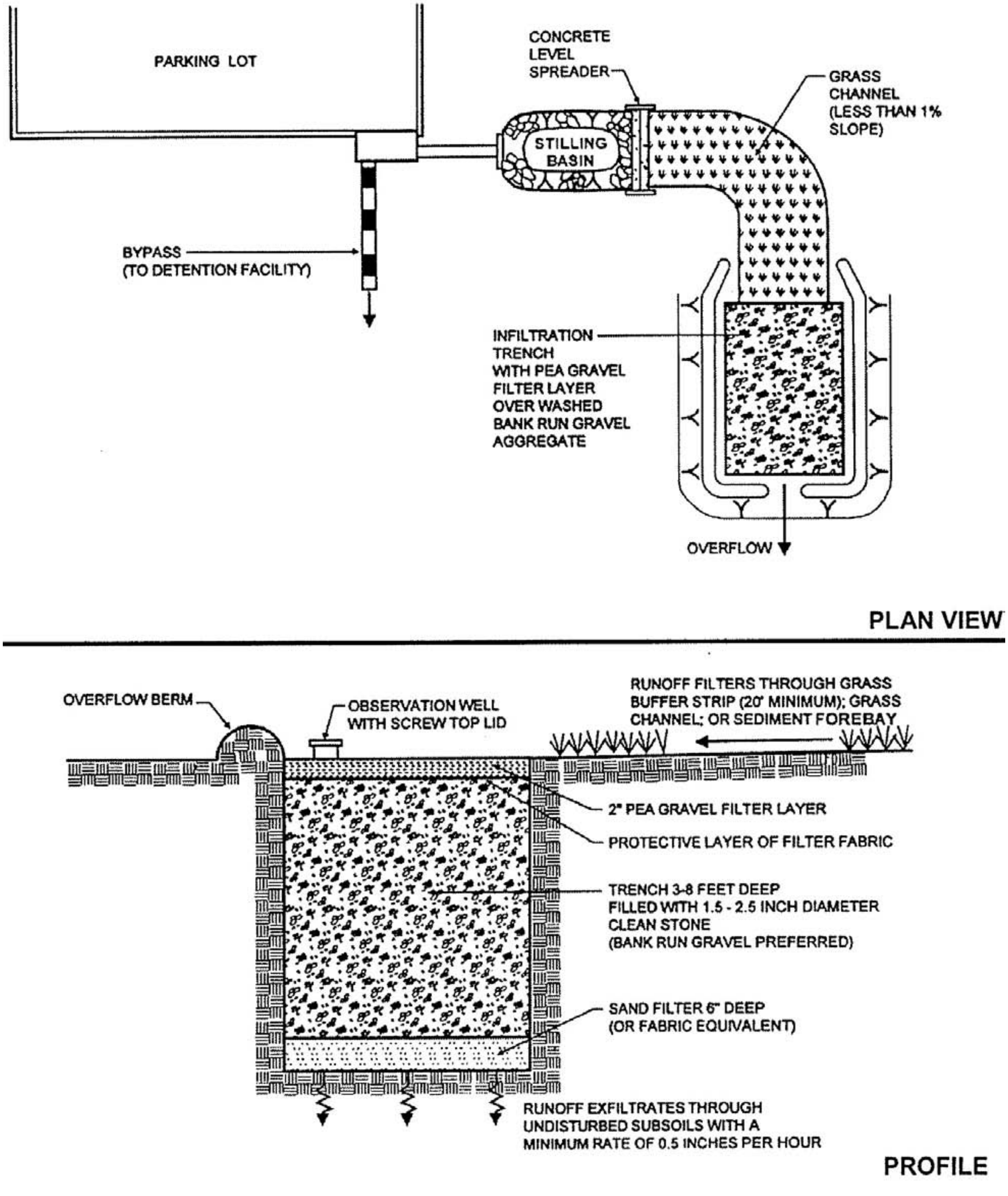

Figure 3-7. **Schematic of an Infiltration Trench** (MDE, 2000)

3.3.7 Porous Pavement

Porous concrete and asphalt pavements are being used as BMPs and replace conventional asphalt pavement or other hard paving surfaces whereby runoff is diverted through a porous asphalt layer and into an underground stone reservoir. The stored runoff then gradually infiltrates into the subsoil. The basic porous pavement system consists of a top layer of porous asphalt concrete covering a layer of gravel that covers a layer of uniformly sized aggregate, which is placed on top of the existing soil sub-base (Schueler, 1987). Stormwater penetrates the porous asphalt and is filtered through the first layer of gravel. The voids in the lower level of large aggregate are filled with runoff. The stored runoff gradually infiltrates into the underlying soil. A sheet of filter fabric below the aggregate prohibits the underlying soil from entering and clogging the facility (NVPDC and ESI, 1992). Provided that the grades, subsoil drainage characteristics, and groundwater table conditions are suitable for its use, porous pavement can be effectively used to recharge groundwater supplies and reduce stormwater runoff as well as water pollution from paved low volume traffic areas (WA DOE, 2001). When properly designed and carefully installed, porous pavement has load bearing strength, longevity, and maintenance requirements similar to conventional pavement.

The surface of the pavement is designed to provide adequate strength to accommodate vehicles while allowing infiltration of surface water and filtration of pollutants. If infiltration into the soil is not practical, the filtered runoff can be discharged through a sub-base drainage system that would outfall into a storm sewer system or a natural drainage path. Pollutant filtration is greatly reduced when the pavement drains into a storm sewer. Several studies have concluded that porous asphalt pavement is sufficiently strong and able to withstand freeze/thaw cycles such that it will last as long as conventional pavement. Porous pavement systems are typically used in low-traffic areas such as the following types of applications:

- parking pads in parking lots,
- overflow parking areas,
- residential street parking lanes,
- recreational trails,
- golf cart and pedestrian paths, and
- emergency vehicle and fire access lanes.

There are three types of porous pavement: porous asphalt pavement; porous concrete pavement; and, modular porous concrete block (WEF and ASCE, 1998).

Porous asphalt pavement is an open-graded coarse aggregate, bound together by asphalt cement into a coherent mass, with sufficient interconnected voids to provide a high rate of permeability to water.

Porous concrete (also referred to as enhanced porosity concrete, porous concrete, Portland cement and pervious pavement) is a subset of a broader family, including porous asphalt, and various kinds of grids and paver systems. Also known as “no fines concrete,” it is a special type of concrete that allows stormwater to pass through it, thereby reducing the runoff from a site. In addition, porous concrete provides runoff treatment through filtration and allows for ground water recharge. Porous concrete or "no fines concrete paving" is a structural, open textured pervious concrete paving surface consisting of standard Portland cement, fly ash, locally available open graded coarse aggregate, admixtures, fibers, and potable water. When properly handled and installed, porous concrete has a high percentage of void space (approximately 17 to 22%) which allows rapid percolation of stormwater through the pavement. Porous concrete is thought to have a greater ability than porous asphalt to maintain its porosity in hot weather and thus is provided as a limited application control. Although, porous concrete has seen growing use in Georgia, there is still very limited practical experience with this measure. Porous concrete is designed primarily for stormwater quality, i.e., the removal of stormwater pollutants. However, they can provide limited runoff quantity control, particularly for smaller storm events. For some smaller sites, trenches can be designed to capture and infiltrate the channel protection volume (Cp_v) in addition to WQ_v. Porous concrete will need to be used in conjunction with another structural control to provide overbank and extreme flood protection, if required (Atlanta Regional Commission, 2001).

Modular porous pavers are structural units, such as concrete blocks, bricks, or reinforced plastic mats, with regularly inter-dispersed void areas used to create a load bearing pavement surface. The void areas are filled with pervious materials (gravel, sand, or grass turf) to create a system that allows for the infiltration of stormwater runoff. Porous paver systems provide water quality benefits in addition to groundwater recharge and a reduction in stormwater volume. The use of porous paver systems results in a reduction of the effective impervious area on a site. There are many different types of modular porous pavers available from different manufacturers, including both pre-cast and mold in-place concrete blocks, concrete grids, interlocking bricks, and plastic mats with hollow rings or hexagonal cells. Modular porous pavers are typically placed on a gravel (stone aggregate) base course. Runoff infiltrates through the porous paver surface into the gravel base course, which acts as a storage reservoir as it exfiltrates to the underlying soil. The infiltration rate of the soils in the subgrade must be adequate to support drawdown of the entire runoff capture volume within 24 to 48 h. Special care must be taken during construction to avoid undue compaction of the underlying soils, which could affect the soils' infiltration capability.

The construction of porous asphalt and concrete are similar to a conventional pavement, except that sand and finer fraction of the aggregate are left out of the pavement mix, and is typically placed on top of a granular base. The modular block pavement is constructed by placing the blocks over a layer of coarse gravel, which in turn is located on a porous geotextile fabric layer. Porous concrete and asphalt pavements have a tendency to clog and seal within 1 to 3 yr (Urbonas and Stahre, 1993), with faster sealing rates reported in areas with excessive winter salting and sanding. Notable exceptions to this were the concrete pavement installations in the state of Florida. Interlocking cellular concrete block pavement seems to seal at a slower rate and has a good record of service under a wide range of climatic conditions (WEF and ASCE, 1998).

3.3.7.1 Stormwater Control

Based on the runoff storage provided by the stone reservoir and the degree of reliance on exfiltration, porous pavement designs fall into three basic categories: complete exfiltration systems; partial exfiltration systems; water quality exfiltration systems (discussed in 3.3.6.).

3.3.7.2 Pollutant Removal Capability

Porous pavement systems in operation show high removal rates for sediment, nutrients, organic matter, and trace metals. The majority of the removal occurs as a result of the exfiltration of runoff into the subsoil, and subsequent adsorption or straining of pollutants within the subsoil (WY DEQ, 1999). Mechanisms of removal include adsorption, straining, and microbial decomposition in the subsoil below the aggregate chamber, and trapping of particulate matter within the aggregate chamber. The first pollutant removal process occurs in the large aggregate reservoir wherein pollutants adsorb to and are absorbed by the aggregate material. Suspended matter will settle out at the bottom of the aggregate layer. The second process for removing pollutants occurs only if the runoff drains into the soil instead of being discharged by a drain. Pollutants that enter the soil sub-base are also adsorbed to and absorbed by the soil particles in addition to aerobic decomposition as well as chemical precipitation of the pollutants within the soil strata (NVPDC and ESI, 1992).

Positive factors influencing pollutant removal include:
- high exfiltration volumes,
- high surface area,
- routine vacuum sweeping,
- maximum drainage time two days,
- highly permeable soils,
- clean-washed aggregate,
- organic matter in subsoils, and
- pre-treatment of off-site runoff.

Negative factors influencing pollutant removal include:
- poor construction practices,
- inadequate surface maintenance,
- use of sand during snow conditions, and
- low exfiltration volumes (WY DEQ, 1999).

3.3.7.3 Design Considerations

Siting

A prerequisite in the construction of porous pavement systems is the evaluation of the site for feasibility to rely on exfiltration to dispose of runoff. The use of porous pavement is highly constrained, requiring deep and permeable soils, restricted traffic, and suitable adjacent land uses

Use may also be restricted in regions with colder climates, arid regions, or regions with high wind erosion rates and in areas of sole-source aquifers (WY DEQ, 1999). Pretreatment using filter strips or vegetated swales for removal of coarse sediments is recommended (Atlanta Regional Commission, 2001).

As porous pavements cannot withstand the passage of heavy trucks due to a lower tensile strength than a conventional pavement, these are typically recommended for lightly used satellite parking areas and access roads.

The design of porous pavement systems should include a seepage analysis. Possible adverse impacts of seepage from infiltration measures to building foundations, basements, roads, parking lots, and sloping areas should be addressed. It is recommended that the porous pavement be located 10 or more ft down gradient of foundation walls, particularly in residential areas (NVPDC and ESI, 1992).

Porous pavement systems should be located at least 100 ft away from a drinking water well to minimize the possibility of groundwater contamination, at least 10 ft down-gradient from nearby building foundations, and at least 100 ft up-gradient.

Porous concrete systems should typically be used in applications where the pavement receives tributary runoff only from impervious areas. If runoff is coming from adjacent pervious areas, it is important that those areas be fully stabilized to reduce sediment loads and prevent clogging of the porous paver surface. Any significant amount of offsite flow should be diverted around the pavement surface. Limited offsite runoff and all onsite runoff should be filtered before it flows over the pavement.

To protect groundwater from potential contamination, runoff from designated hotspot land uses or activities must not be infiltrated. Porous concrete should not be used under the following conditions:

- manufacturing and industrial sites, where there is a potential for high concentrations of soluble pollutants and heavy metals;
- areas with a high pesticide concentration; and
- areas with karst geology without adequate geotechnical testing by qualified individuals and in accordance with local requirements (Atlanta Regional Commission, 2001).

Soils

Porous pavement is not suitable for sites with soil infiltration rates of less than 0.5 in./h (D soils), or any soils with a clay content greater than 30%. C soils (silt loam and sandy clay loams) provide marginal infiltration rates, and should probably only be considered for partial exfiltration systems (Schueler, 1987; Atlanta Regional Commission, 2001). Soils with a combined silt/clay content of over 40% by weight are susceptible for frost heave, and may not be suited for these applications. These systems should never be constructed over fill soils, which often form an unstable upgrade, and are prone to slope failure. Stone subgrade must extend below the frost line irrespective of soil conditions. During construction and preparation of the subgrade, special care must be taken to avoid compaction of the soils.

The most critical factor in determining the applicability of porous pavement as a BMP device is the infiltration capacity of the underlying soil (NVPDC and ESI, 1992). Core samples or trenches at least 2 to 4 ft below the anticipated level of the bottom of the stone reservoir should be examined for any impermeable soil strata that might impede infiltration, such as localized clay lenses, hardpans, or fragipans. Subsurface drainage may be required if the soil does not exhibit adequate infiltration capacity. Subsoils are generally susceptible to frost heave if the soil contains more than 3% of particles smaller than 0.02 mm in diameter. Such soils do not allow the infiltration from the facility and should be avoided (NVPDC and ESI, 1992).

Slope

Porous concrete systems should not be used on slopes greater than 5%; 2% grade is recommended. For slopes greater than 1%, barriers perpendicular to the direction of drainage should be installed in sub-grade material to keep it from washing away, or filter fabric should be placed at the bottom and sides of the aggregate to keep soil from migrating into the aggregate and reducing porosity.

Depth to Bedrock and Seasonally High Water Table

The depth from the bottom of the gravel base course to the level of the seasonally high water table or to bedrock must be sufficient (2 to 4 ft) to allow for adequate infiltration and filtering of water released through the bottom of the structure. A minimum of 3 ft (preferably 4 ft) of clearance is needed between the bottom of the stone reservoir and the bedrock level. This data can be inferred from local soil data maps, but needs to be confirmed by actual soil test bores (WY DEQ, 1999). To insure complete draining of the stone reservoir in 72 h, it may be necessary to limit the depth of the stone reservoir if underlying soils have relatively low exfiltration rates. Soil limitations for porous pavement are shown in Table 3-8.

Table 3-8. Soil Limitations for Porous Pavement

Soil Type	**Minimum Infiltration Rate (*f*) (in./h)**	**SCS Soil Group***	**Maximum Depth of Storage** (in.) 48 h**	**72 h**
Sand	8.27	A	992	595
Loamy Sand	2.41	A	290	174
Sandy Loam	1.02	B	122	183
Loam	0.52	B	62	93

Silt Loam	0.27	C	32	49

* Sandy Clay Loams, Clay Loams, Silty Clay Loams, Sandy Clay, Silty Clay, and Clay soils are not included as these soil types are all not feasible for infiltration basins.
**Maximum Depth of stone reservoir that can drain completely within 48 or 72 h after a storm, given the soil infiltration rate.
(Schueler, 1987)

Watershed Size

The most suitable drainage area for porous pavement sites should be restricted to between 0.25 and 10 acres. This guideline tends to reflect the perceived economic and liability problems associated with larger applications and the cost-effectiveness of other BMPs outside of this range.

Design Parameters

Since the surface area of the porous pavement will typically depend on how large a parking lot will be built, the critical design consideration will be the depth of the large aggregate layer. As with infiltration trenches, the maximum depth of the large aggregate layer is a function of allowable detention time, the porosity of the aggregate, and the soil infiltration rate. The bottom of the facility should be below the frost line and approximately 4 ft above bedrock and the level of the seasonally high water table. The same design steps presented for infiltration facilities earlier in this chapter apply to porous pavement, and include the additional step of determining the thickness of the porous pavement layer. The depth of the asphalt layer and underlying stone reservoir depends on the strength of the sub-base soil and the projected traffic intensities (NVPDC and ESI, 1992).

The following list of general design elements should be considered in any porous pavement design:

- anticipated traffic intensities, defined by the average daily equivalent axle load (EAL);
- California Bearing Ratio (CBR) of the soils; and
- susceptibility of the soils to forest heave.

Methods for conducting the CBR test are described in ASTM D1883 and AASH0 T193 (VA DCR, 1999). The asphalt layer is typically 2.5 to 4 in. thick. The minimum combined thickness of the asphalt layer and stone reservoir can be determined from the Table 3-9.

Table 3-9. Minimum Thickness of Porous Paving

Traffic Group	**General Character**	**California Bearing Ratio**				**EAL**
		>15	**10-14**	**6-9**	**<5***	
1	Light Traffic	5"	7"	9"		<5

2	Medium Light Traffic (Max. 1,000 VPD)	6"	8"	11"		6-20
3	Medium Traffic (Max. 3,000 VPD)	7"	9"	12"		21-75

*Studies indicate that for all traffic groups with CBR of 5 or less, the subgrade was improved to CBR 6 with crushed stone 2 in. size.
VPD = Vehicles Per Day
EAL = Equivalent Axle Load (18 Kips) average daily
Note: Thicknesses refer to the minimum combined depth of asphalt layer and stone reservoir necessary to carry appropriate load.
(NVPDC and ESI, 1992)

The following design procedure represents a generic list of the steps typically required for the design of porous pavement:

- Determine if the anticipated development conditions and drainage area are appropriate for a porous pavement application.
- Determine if the soils (permeability, bedrock, water table, Karst, etc.) and site topographic conditions (slopes, etc.) are appropriate for a porous pavement application.
- Locate the porous pavement section on a site within topographic constraints.
- Determine the drainage area for the porous pavement and calculate the required water quality volume.
- Evaluate the hydrology of the contributing drainage area to determine peak rates of runoff.
- Design the porous pavement stone reservoir; e.g., as shown in the Virginia BMP manual.
  - Design infiltration rate, $f_d = 0.5\,f$
  - Max. storage time $T_{\max} = 48$ h
  - Max. storage depth, $d_{\max}$
  - Stone backfill of clean aggregate (1.5 to 3.5 in.) VDOT No. 1 open-graded coarse aggregate
  - Filter gravel layer - 2 in. of clean aggregate (0.5 in.) VDOT No. 57 open-graded coarse aggregate
  - Sand layer on trench bottom (8 in.) or filter fabric, per geotechnical and pavement design recommendations
  - Filter fabric on trench sides and top (not on trench bottom) keyed into trench
- Overflow channel or large storm bypass.
- Observation well.
- Provide pavement section design and material specifications.
- Provide sequence of construction.
- Provide maintenance and inspection requirements (VA DCR, 1999).

A schematic representation of a porous pavement is presented in Figure 3-8 (Schueler, 1987).

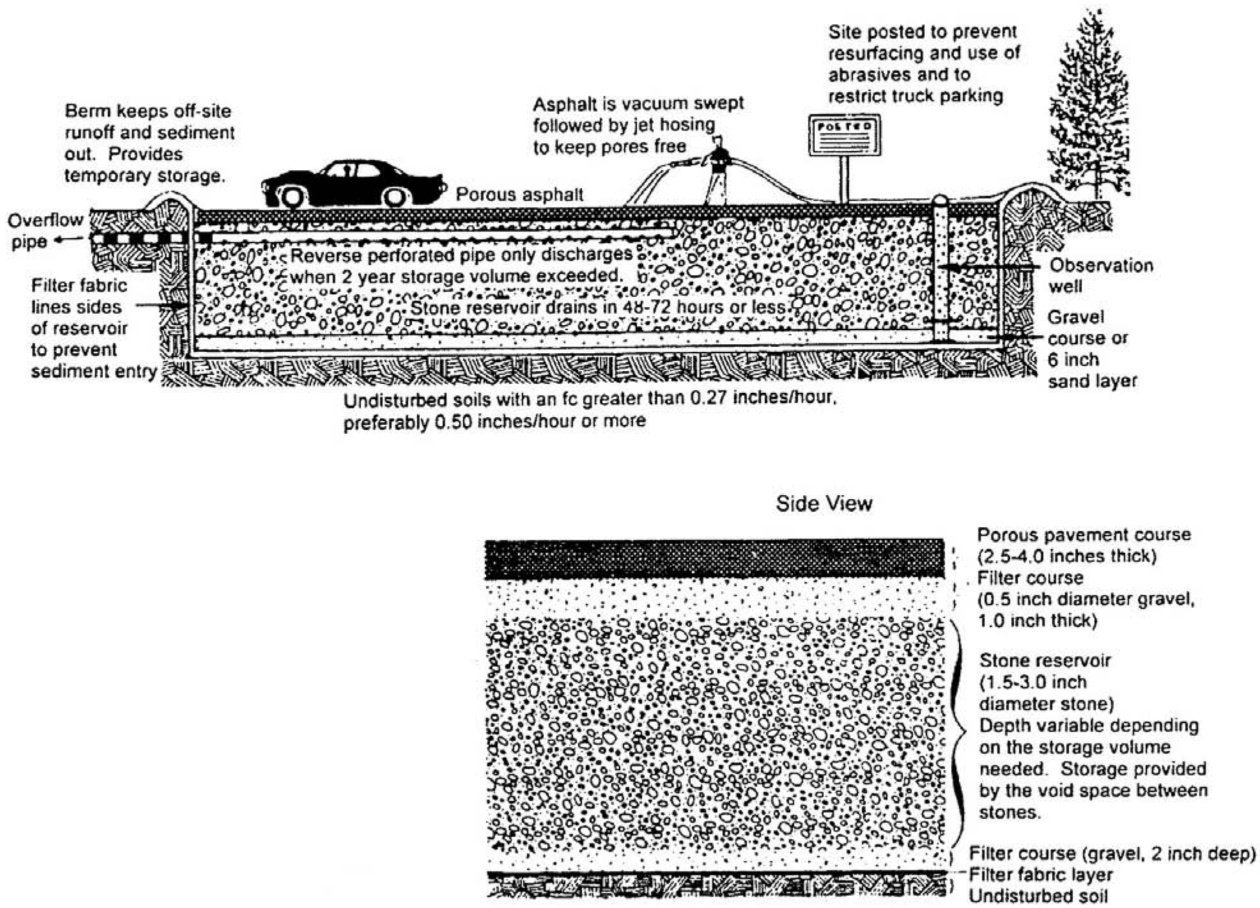

Figure 3-8. **Schematic of a Porous Pavement** (Schueler, 1987)

3.3.8 Sand and Organic Filters

Sand filters are structural stormwater controls that capture and temporarily store stormwater runoff and pass it through a filter bed of sand. They have been successfully used in Austin, TX, the District of Columbia, the state of Delaware, and in Alexandria, VA over the last two decades (VA DCR, 1999). Most sand filter systems consist of two-chamber structures. The first chamber is a sediment forebay or sedimentation chamber, which removes floatables and heavy sediments. The second, which is a filtration chamber, removes additional pollutants by filtering the runoff through a sand bed. The filtered runoff is typically collected and returned to the conveyance system, though it can also be partially or fully exfiltrated into the surrounding soil in areas with porous soils.

Sand filters may be "unconfined" sand-filled trenches with perforated underdrains or "confined" systems where the filter medium is contained in a concrete vault with a drain at the bottom of the vault. Depending on the specific design, these types of filters are often referred to as "Delaware Filters" or "Austin Filters" after the localities where they were originally designed and installed. Large sand filters are installed above ground and are self-contained sand beds that can treat stormwater from drainage areas as much as 5 acres in size. Enhanced sand filters utilize layers of peat, limestone, leaf compost, and/or topsoil, and may also have a grass cover crop. The adsorptive media of enhanced sand filters is expected to improve removal rates (WY DEQ, 1999). Sand filters can fall under two basic designs: (i) surface sand filter; and, (ii) perimeter sand filter.

The surface sand filter is a ground-level open air structure that consists of a pretreatment sediment forebay and a filter bed chamber. This system can treat drainage areas up to 10 acres in size and is typically located offline. Surface sand filters can be designed as an excavation with earthen embankments or as a concrete or block structure.

The perimeter sand filter is an enclosed filter system typically constructed just below grade in a vault along the edge of an impervious area such as a parking lot. The system consists of a sedimentation chamber and a sand bed filter. Runoff flows into the structure through a series of inlet grates located along the top of the system.

Yet another design variant, the underground sand filter, is intended primarily for extremely space-limited and high density areas, and is considered a limited structural application control (Atlanta Regional Commission, 2001).

3.3.8.1 Stormwater Control

Sand filter systems are designed primarily as offline systems for stormwater quality and typically need to be used in conjunction with another structural control to provide downstream channel protection, overbank flood protection, and extreme flood protection, if required. However, under certain circumstances, filters can provide limited runoff quantity control, particularly for smaller storm events.

3.3.8.2 Pollutant Removal Capability

Pollutant removal is primarily achieved by straining pollutants through the filtering medium (sand or peat) and settling on top of the sand bed and/or pretreatment pool. A grass cover crop on the filter helps in the additional removal of nutrients by plant uptake. Sand filter removal rates are high for sediment and trace metals, and moderate for nutrients, biochemical oxygen demand (BOD) and fecal coliform (FC) (City of Austin, 1991).

Positive factors influencing pollutant removal include:

- offline systems,
- peat and/or limestone layer,
- grass cover,
- longer draw down times ranging from 24 to 40 h,
- pretreatment pool,
- minimum depth of 18 in.,
- regular maintenance, and
- no direct connection to groundwater.

Negative factors influencing pollutant removal include:

- online systems, and
- freezing weather.

3.3.8.3 Design Considerations

General Considerations

Several types of intermittent sand filter facilities are recognized for stormwater quality management purposes, and the general design criteria presented below apply to the design of these facilities for water quality control. This implies that the volume of runoff to be treated is determined by the water quality volume and the desired pollutant removal efficiency (VA DCR, 1999).

The Austin, Texas Filter

The concept and use of surface filters initially originated in Austin, Texas, where these filters have been extensively used in catchments of up to 20 ha (50 acres). Austin filter has two design variants, one with full sedimentation and the other with partial sedimentation. The full sedimentation configuration includes a sedimentation basin designed to hold the entire water quality volume (i.e., equivalent to the 40 h drain time maximized volume) and to release this volume to the filter over a 40 h drawdown period. This system should be used unless topographical constraints make this design unfeasible. The partial sedimentation configuration requires less depth than the full sedimentation system and may be applicable where topographical constraints exist. In this system, a smaller sedimentation chamber is located upstream of the filtration basin, is designed to remove the heavier sediment and trash litter only, and requires more intensive maintenance than the full sedimentation system. The volume of the sediment chamber should be no less than 20% of the water quality volume used for the full sedimentation design. The design must ensure that the sediment chamber discharges the flow evenly(WEF and ASCE, 1998).

Linear Filter - Delaware

The Delaware Filter is an underground system that uses a vault with a permanent pool of water as the pretreatment device. Recommended for catchments of up to 2 ha (5 acres), the volume of both the sedimentation and filter chambers are approximated to 38 m^3/ha (540 ft^3 per contributing acre) and the surface area of each chamber should be 25 m^3/ha. Pavement and inlet design and construction are critical in a Delaware filter (the filter should be positioned relative to the pavement to evenly distributed the flow as it enters the sedimentation chamber) (WEF and ASCE, 1998).

Underground Vault - Washington D. C

The initial settling chamber is undersized for effective sedimentation, causing the filter to clog quickly. When the filter clogs, the flow simply overtops the overflow weir and flows directly to the outlet, with no indication that the filter is plugged. This filter type when used, should be sized using the Delaware linear filter criteria, including the pre-settlement chamber. It is also strongly recommended that the overflow weir and de-watering drain in the filter chamber be blocked and that the entrance manhole covers over the sedimentation chamber and the outflow chamber be replaced with grates. If the filter clogs, the water will back up in the vault, overflow out of the inlet grate over the sedimentation compartment, and back into the outfall chamber, giving a clear visual indication that the filter is plugged(WEF and ASCE, 1998).

Location and Siting

Surface sand filters should have a contributing drainage area of 10 acres or less. The maximum drainage area for a perimeter sand filter is 2 acres.

Sand filter systems are generally applied to land uses with a high percentage of impervious surfaces. Sites with less than 50% imperviousness or high clay/silt sediment loads must not use a sand filter without adequate pretreatment due to potential clogging and failure of the filter bed. Any disturbed areas within the sand filter facility drainage area should be identified and stabilized.

Surface sand filters are generally used in an offline configuration where the water quality volume (WQ_v) is diverted to the filter facility through the use of a flow diversion structure and flow splitter. The diversion structure or flow splitter is used to divert stormwater flows greater than the WQ_v to other controls or downstream. Perimeter filters are typically sited along the edge or perimeter of an impervious area such as a parking lot.

Sand filter systems are designed for intermittent flow and must be allowed to drain and re-aerate between rainfall events and should not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

Physical Specifications/Geometry

An access ramp with a slope not exceeding 7:1 or the equivalent should be included at the inlet and outlet of a surface filter for maintenance purposes. Side slopes for earthen or grass embankments should not exceed 3:1 (H:V) to facilitate mowing and site slope should be no more than 6% across filter location (Metro Council, 2001).

A major drawback for a media filtration inlet is the need for elevation differences in the storm drainage system in order to accommodate live pool storage and sand filter thickness. The minimum elevation difference needed at a site from the inflow to the outflow is 5 ft for surface sand filters and 2 to 3 ft for perimeter sand filters (Knoxville, 2003).

A minimum depth of 2 ft is required between the bottom of the sand filter and the elevation of the seasonally high water table for surface sand filters with exfiltration (i.e,. earthen structure). While there are no restrictions on the type of soils, Group “A” soils are generally required to allow exfiltration (Atlanta Regional Commission, 2001).

Sizing

Many guidelines recommend sizing the filter bed using Darcy’s Law, which relates the velocity of fluids to the hydraulic head and coefficient of permeability of a medium. Hydraulic calculations based on Darcy’s Law used to establish the filter area of a sand filter allow flow-through of the treatment volume within the desired time frame, typically 40 to 48 h (City of Austin, 1988; VA DCR, 1999). The State of Florida uses more complex falling-head computations and allows a drawdown time of up to 72 h (Florida DER, 1988). However, creating storage for the full WQ_v in shallow configuration systems may result in a larger filter than the hydraulic calculations would indicate (VA DCR, 1999).

The Austin Sand Filter Formula (City of Austin, 1988) derived from Darcy’s Law to size sand filters is given as:

$$A_f = \frac{I_a H d_f}{k(h + d_f)t_f} \qquad \textbf{(3-15)}$$

where
A_f = surface area of sand bed (acres or ft^2);
I_a = impervious drainage area contributing runoff to the basin (acres or ft^2);
H = runoff depth to be treated (ft);
d_f = sand bed depth (ft);

k = coefficient of permeability for sand filter (ft/h);
h = average depth (ft) of water above surface of sand media between full and empty basin conditions (½ max. depth) (City of Austin, 1996); and
t_f = time required for runoff volume to pass through filter media (h)

A BMP drawdown time (t_f) of 40 h allows the filter to fully drain down and dry out to maintain an aerobic environment between storms. Typical values for k are shown in Table 3-10.

Table 3-10. Coefficient of Permeability *k* Values for Stormwater Filtering Practices

Filter Medium	Coefficient of Permeability (ft/d)
Sand	3.5
Peat/Sand	2.75
Compost	8.7

(CWP, 1996a)

The permeability of sand shown in Table 3-10 is extremely conservative, but is widely used since it is incorporated in the design guidelines of the City of Austin(City of Austin, 1988; City of Austin, 1996). When the sand is initially installed, the permeability is so high (over 100 ft/d) that generally only a portion of the filter area is required to infiltrate the entire volume, especially in a "full sedimentation" Austin design where the capture volume is released to the filter basin over 24 h. This methodology results in a filter bed area that is oversized when new and the entire water quality volume is filtered in less than a day with no significant height of water on top of the sand bed. The Austin design variations are still preferred where there is sufficient space, because they lack a permanent pool, which eliminates vector concerns. Consequently, the simple rule of thumb is adequate for sizing the filter area.

For filters with full sedimentation protection (sedimentation basin containing full WQ_v with 24 h drawdown to filter), k = 3.5 ft/d (0.146 ft/h), and t_f = 40 h, the sand filter formula reduces to:

$$A_{f(FS)} = \frac{310 I_a d_f}{(h + d_f)} \qquad \textbf{(3-16)}$$

where
A_f is in ft^2 and I_a is in acres.

For filters with partial sedimentation protection (sediment chamber containing 20% of WQ_V with free hydraulic flow to filter), k = 2.0 ft/day (0.0833 ft/h) and t_f = 40 h, the formula reduces to:

$$A_{f(PS)} = \frac{545 I_a d_f}{(h + d_f)} \quad \textbf{(3-17)}$$

Where
A_f is in ft^2 and I_a is in acres.

Capture volume

The facility should be sized to capture the required water quality volume, preferably in a separate pretreatment sedimentation basin.

Geometry

The water depth in the sedimentation basin when full should be at least 2 ft and no greater than 10 ft. A fixed vertical sediment depth marker should be installed in the sedimentation basin to indicate when 20% of the basin volume has been lost because of sediment accumulation.

Basic Components

Surface sand filters generally employ the following layers, from top to bottom: sand, geotextile and an underdrain system. Runoff discharging to the sand filter must be pretreated (e.g., a presettling basin) to remove debris and other gross solids and any oil from high-use sites. The type of pretreatment device will depend on the type of pollutants present. The length-to-width ratio of the presettling basin should be 3:1 and the recommended depth varies from 3 to 6 ft.

Inlet structures such as flow spreaders, weirs or multiple orifice openings should be designed to minimize turbulence and spread the flow uniformly across the surface of the filter media. Stone riprap or other dissipation devices should be installed to prevent gouging of the sand media and promote uniform flow. Offline outlet structures are typically sized for the 15-min peak flow of a 2-yr, 24-h storm.

An impermeable liner (clay, geomembrane or concrete) may be required under the filter to protect groundwater or where underflow could damage structures. If the impermeable liner is not required, a geotextile liner should be installed, unless the bed has been excavated to bedrock (Metro Council, 2001).

The sand filter is typically constructed with 18 in. of sand overlying 6 in. of gravel. The sand and gravel media are separated by permeable geotextile fabric and the gravel layer is situated on geotextile fabric. Four-in.perforated PVC pipe is used to drain captured flows from the gravel layer. A minimum of 2 in. of gravel must cover the top surface of the PVC pipe.

Sand Specification

The sand in a filter must consist of medium-sized sand that meets the size gradation (Table 3-11). A laboratory analysis to determine the sand's hydraulic conductivity K is also highly recommended. The designer should then adjust this number to account for conditioning of the sand during operation.

Table 3-11. Sand Medium Specification

U. S. Sieve Number	**Percent Passing**
4	95-100
8	70-100
16	40-90
30	25-75
50	2-25
100	<4
200	<2

(King County, 1998)

Underdrain Systems

Several types of underdrains may be used: a central collector pipe (with lateral feeder pipes or a geotextile drain strip in an 8 in. gravel backfill or drain rock bed) or a longitudinal pipe in an 8 in. gravel backfill or drain rock with a collector pipe at the outlet end.

Hydraulically, the system is typically sized for the 15 min peak flow from a 2-yr, 24-h storm, with 1 ft of head above the invert of the upstream end of the collector pipe. Yet, local sizing requirements should be used when available.

A geotextile fabric must be used between the sand layer and drain rock or gravel and placed so that 1.0 in. of drain rock or gravel is above the fabric. Drain rock should be 1.5 to 0.75 in. rock or gravel backfill, washed free of clay and organic material.

Cleanout wyes with caps or junction boxes must be provided at both ends of the collector pipes. Cleanouts must extend to the surface of the filter. A valve box must be provided for access to the cleanouts.

Impermeable Layers

Impermeable liners such as clay, concrete, or geomembrane should be used when non-conventional soluble pollutants such as metals or organics are present, and where the underflow could cause problems with structures or groundwater. Clay liners should have a minimum thickness of 12 in. and meet the specifications in Table 3-12.

A geomembrane liner should be at least 30 mils thick and ultraviolet resistant. It should be protected from puncture, tearing and abrasion by installing geotextile fabric on the top and bottom of the geomembrane.

Concrete liners may also be used for basins less than 1,000 ft^2 in area. Concrete should be 5 in. thick (Class A or better) and reinforced by steel wire mesh. The concrete should have a minimum 6 in. compacted aggregate base consisting of either coarse sand and river stone or crushed stone or its equivalent with diameter of 0.75 to 1 in., when the underlying soil is clay or has an unconfined compressive strength of 0.25 ton/ft^2 (Metro Council, 2001).

If an impermeable liner is not provided, an analysis should be made of possible adverse impacts of seepage zones on groundwater and nearby built areas. Sand filters without impermeable liners should not be built on fill sites, and should be located at least 20 ft downslope and 100 ft upslope from building foundations (Metro Council, 2001).

Table 3-12. Clay Liner Specifications

Property	**Test Method**	**Unit**	**Specification**
Permeability	ASTM D-2434	cm/sec	1×10^{-6} max
Plasticity index of clay	ASTM D-423 & D-424	percent	Not less than 15
Liquid limit of clay	ASTM D-2216	percent	Not less than 30
Clay particles passing	ASTM D-422	percent	Not less than 30
Clay compaction	ASTM D-2216	percent	95% of Standard Proctor Density

(WA DOE, 2001)

Underground Filters

Although, in general, sand filter design criteria apply to underground filters as well, additional specific recommendations for underground filters are as follows:

- One ft of sediment storage in the presettling cell should be provided.

- The retaining baffle for oil/floatables in the pre-settling cell must extend at least 1 ft above to 1 ft below the design flow water level, and be spaced a minimum of 5 ft horizontally from the inlet. Provision for the passage of flows in the event of plugging must be provided. Access opening and ladder must be provided on both sides of the baffle.

- The inlet flow distribution should be optimized with minimal sand bed disturbance. One recommendation is to provide a maximum of 8 in. of distance between the top of the spreader and the top of the sand bed. Flows may enter the sand bed by spilling over the top of the wall into a flow spreader pad. Alternatively a pipe and manifold system may be used. Any pipe and manifold system must retain the required dead storage volume in the first cell, minimize turbulence, and be readily maintainable. Multiple inlets are recommended to minimize turbulence and reduce local flow velocities.

- Erosion protection must be provided along the first foot of the sand bed adjacent to the spreader. Geotextile fabric secured on the surface of the sand bed, or an equivalent method may be used. A dewatering gate valve should be constructed just above the sand bed and removable sand panels must be provided over the entire sand bed.

- To prevent anoxic conditions, a minimum of 24 ft^2. of ventilation grate must be provided for each 250 ft^2 of sand bed surface area. For sufficient distribution of air flow across the sand bed, grates may be located in one area if the sand filter is small, but placement at each end is preferred. Small grates may also be dispersed over the entire sand bed area.

Organic Filters

The organic filter is a design variant of the surface sand filter, which uses organic materials such as leaf compost or a peat/sand mixture as the filter media. The organic material enhances pollutant removal by providing adsorption of contaminants such as soluble metals, hydrocarbons, and other organic chemicals (Atlanta Regional Commission, 2001). Additional specific recommendations for organic filters are as follows: The type of peat used is critically important. Fibric peat, in which undecomposed fibrous organic material is readily identifiable, is preferred. Hemic peat containing more decomposed material may also be used. Sapric peat, made up largely of decomposed matter, is not recommended. They are typically used on relatively small sites (up to 10 acres), to minimize potential clogging. The minimum head requirement (the elevation difference needed at a site from the inflow to the outflow), 5 to 8 ft, is higher than the surface sand filter.

Two typical media bed configurations are the peat/sand and the compost filter. Both variants utilize a gravel underdrain system. The peat filter includes an 18 in. 50/50 peat/sand mix over a 6 in. sand layer and can be optionally covered by 3 in. of topsoil and vegetation. The compost filter has an 18 in. compost layer.

Figures 3-9 to 3-12 show the schematic of a surface sand filter, perimeter sand filter, underground sand filter, and organic filter, respectively.

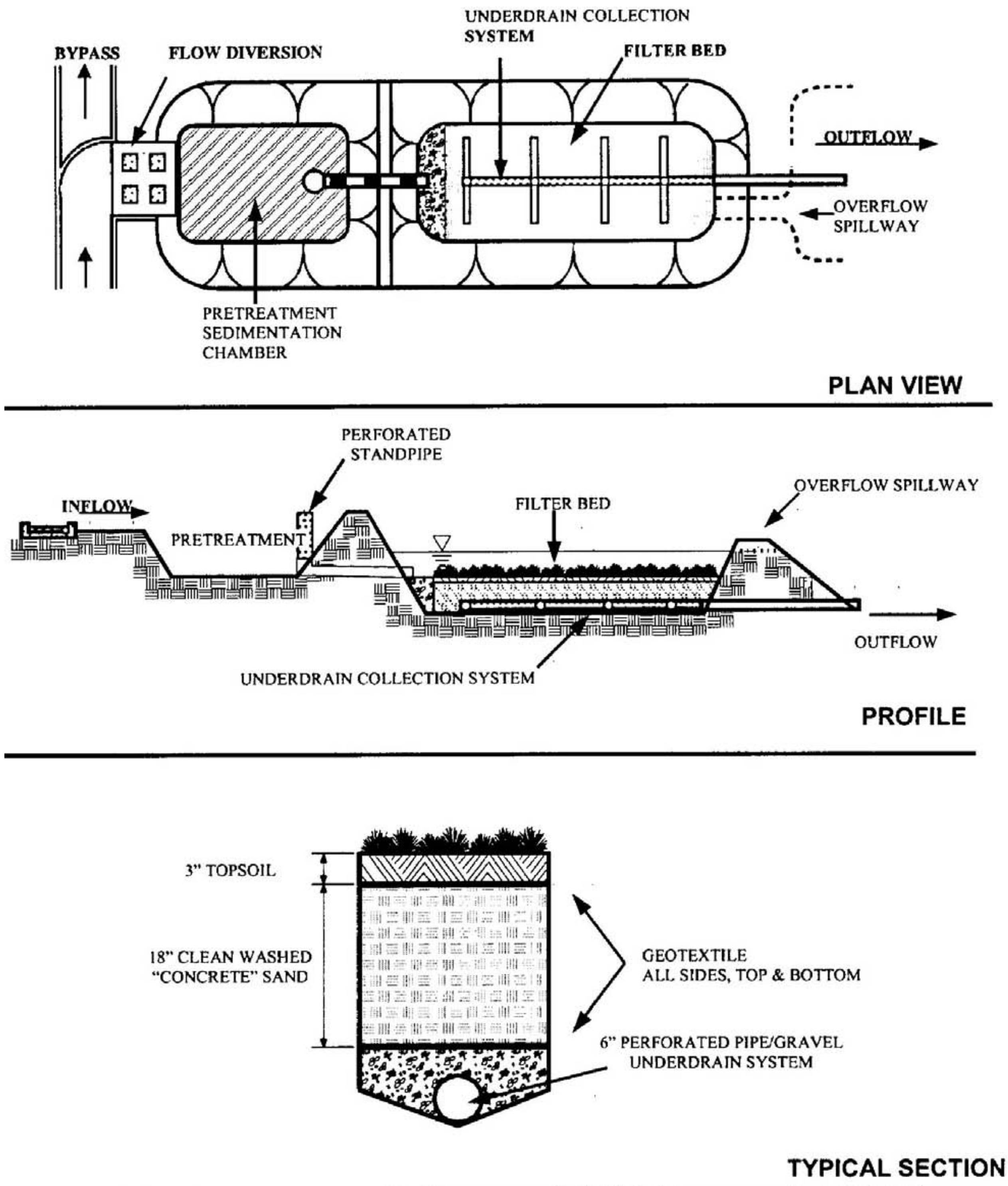

Figure 3-9. **Schematic of a Surface Sand Filter** (MDE, 2000)

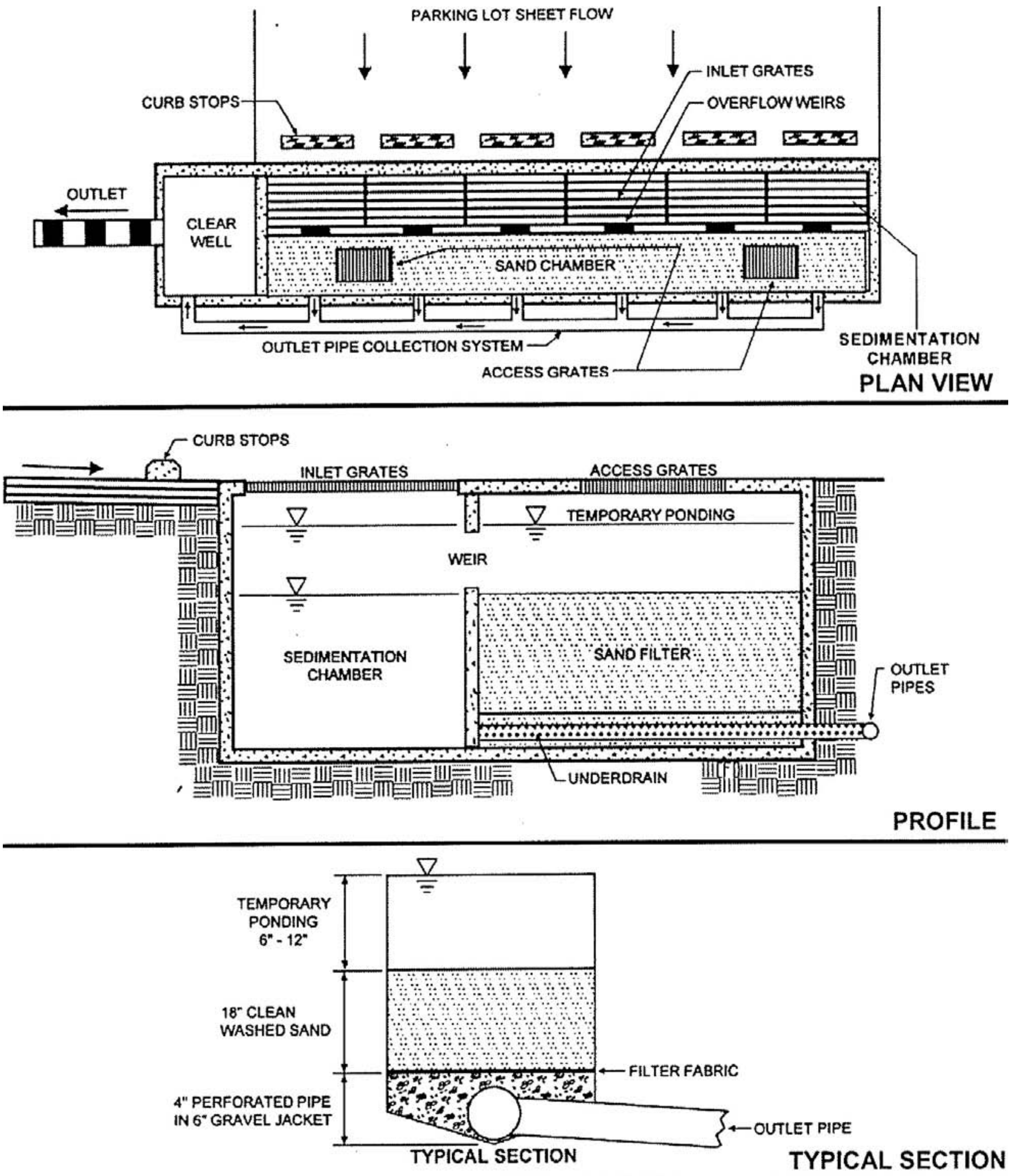

Figure 3-10. **Schematic of a Perimeter Sand Filter** (MDE, 2000)

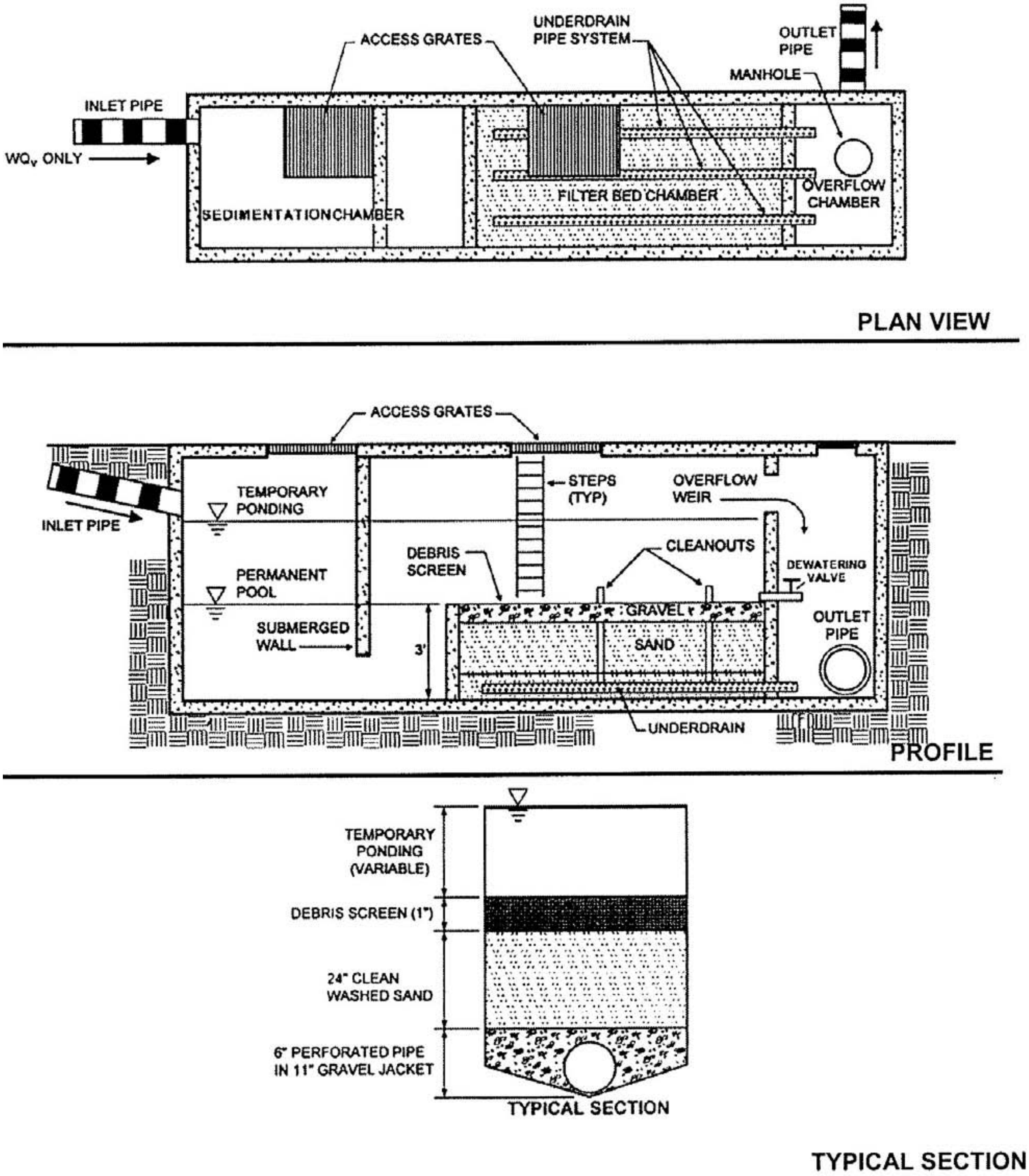

Figure 3-11. **Schematic of an Underground Sand Filter** (MDE, 2000)

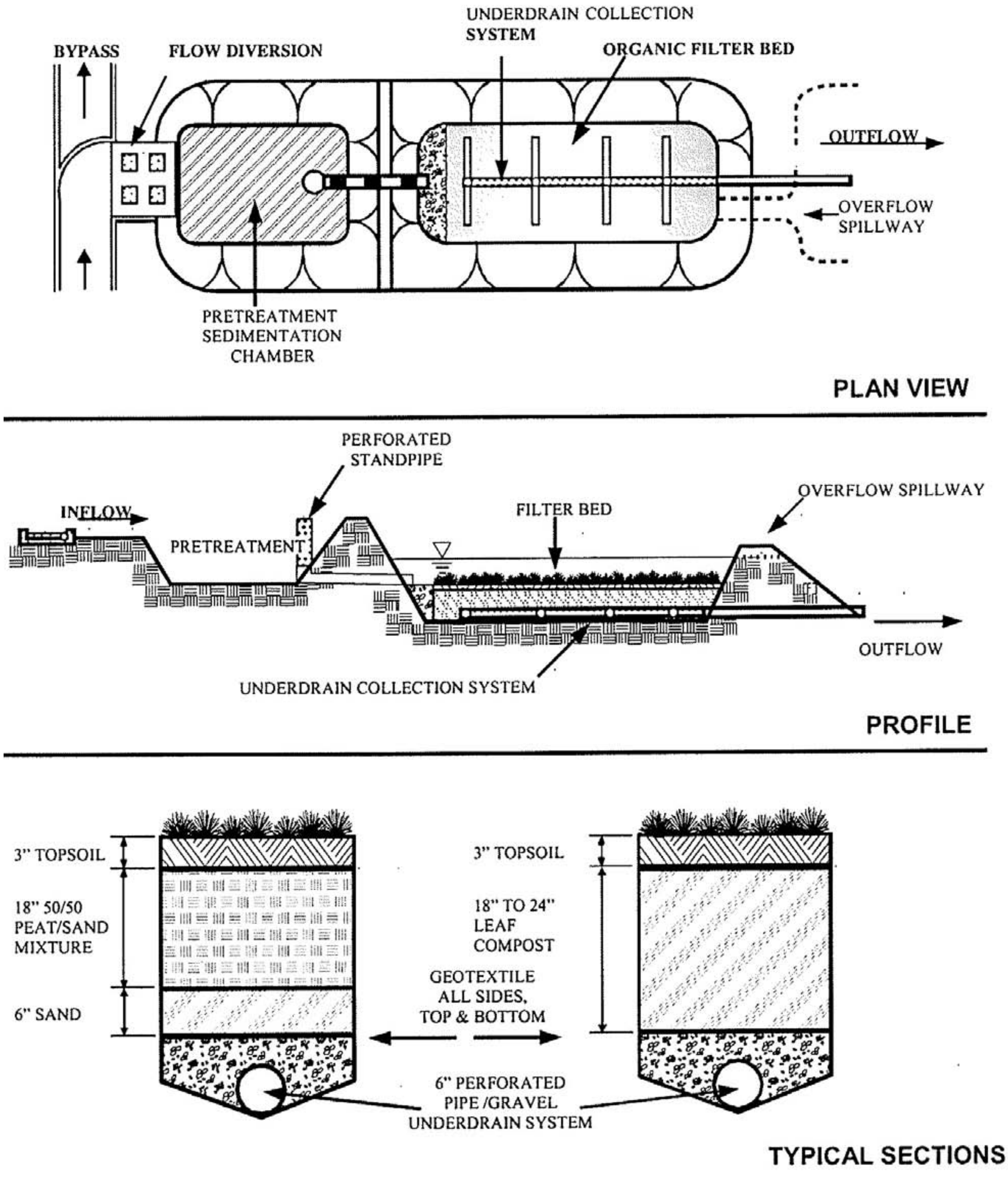

Figure 3-12. **Schematic of an Organic Filter** (MDE, 2000)

REFERENCES

Atlanta Regional Commission. (2001). *Georgia Stormwater Management Manual Volume 2: Technical Handbook*. Atlanta Regional Commission, www.georgiastormwater.com

California Department of Transportation (Caltrans). (2002). *Proposed Final Report: BMP Retrofit Pilot Program*. CTSW-RT-01-050.

California Stormwater Quality Association (CASQA). (2003). *Stormwater Best Management Practice Handbook: New Development and Redevelopment.* www.cabmphandbooks.com.

Chow, V.T. (1959). *Open-Channel Hydraulics*. McGraw Hill, New York, NY.

(City of Austin). (1988). *Environmental Criteria Manual*. Austin, TX.

(City of Austin). (1991). *Water Quality Management. Environmental Criteria Manual*. Austin, TX.

(City of Austin). (1996). *Design of Water Quality Controls*. Austin, TX.

Clar, M., B.J. Barfield, and S. Yu. (2003). *Considerations in the design of treatment best management practices (BMPs) to improve water quality*. EPA 600/R-03/103. U.S. Environmental Protection Agency, Office of Research and Development, Cincinnati, OH. www.epa.gov/ORD/NRMRL/Pubs/600r03103/600r03103.htm October 2003.

Claytor, R.A. and T.R. Schueler. (1996). *Design of Stormwater Filtering Systems*. Center for Watershed Protection and Chesapeake Research Consortium, Ellicott City and Solomons, MD.

Center for Watershed Protection (CWP). (1996a). *Design of Stormwater Filtering Systems*. Ellicott City, MD.

Center for Watershed Protection (CWP). (1996b). *Ditches or Biological Filters? Classifying the pollutant removal performance of open channels. In Watershed Protection Techniques, Volume 2 No. 2*. Ellicott City, MD.

Florida DER. (1988). *The Florida Development Manual: A Guide to Sound Land and Water Management*. Florida Department of Environmental Regulation, Tallahassee, FL.

Guo, C.Y. and B.R. Urbonas. (1995). *Special Report to the Urban Drainage and Flood Control District on Stormwater BMP Capture Volume Probabilities in United States,*. Denver, CO.

Horner, R.R. (1988). *Biofiltration Systems for Storm Runoff Water Quality Control*. Washington State Department of Ecology, Seattle, WA.

King County Department of Natural Resources (King County). (1998). *King County Washington Surface Water Design Manual*. Seattle, WA.

City of Knoxville (Knoxville). (2003). *Best Management Practices (BMP) Manual*. Knoxville, TN.

Maryland Department of the Environment (MDE). (2000). *Maryland Stormwater Design Manual Volumes I & II*. Baltimore, MD. http://www.mde.state.md.us.

(Metro Council). (2001). *Minnesota Urban Small Sites BMP Manual. Stormwater Best Management Practices for Cold Climates*. St. Paul, MN.

Metropolitan Washington Council of Governments (MWCOG). (1979). *Controlling Stormwater Runoff in Developing Areas: Selected Best Management Practices*. Washington, DC.

Northern Virginia Planning District Commission (NVPDC). (1979). *Guidebook for Screening Urban Nonpoint Pollution Management Strategies*. Annandale, VA.

Northern Virginia Planning District Commission (NVPDC). (1987). *BMP Handbook for the Occoquan Basin (Draft).* Annandale, VA.

Northern Virginia Planning District Commission and Engineers and Surveyors Institute (NVPDC and ESI). (1992). *Northern Virginia BMP Handbook: A guide to planning and designing best management practices in northern Virginia*. Annandale, VA.

Schueler, T.R. (1987). *Controlling Urban Runoff: A practical manual for planning and designing urban BMPs*. Metropolitan Washington Council of Governments, Washington, DC.

Schueler, T.R. (1992a). *Design of Stormwater Wetland Systems: guidelines for creating diverse and effective stormwater wetlands in the mid-Atlantic Region*. Metropolitan Washington Council of Governments, Washington, DC.

Schueler, T.R. (1992b). *Design of Stormwater Pond Systems*. Metropolitan Washington Council of Governments,, Washington, DC.

Schueler, T.R., P.A. Kumble, and M.A. Hearty. (1992). *A Current Assessment of Urban Best Management Practices: Techniques for reducing nonpoint source pollution in the coastal zone*. Metropolitan Washington Council of Governments, Washington, DC.

Strecker, W.E., J.M. Kersnar, and E.D. Driscoll. (1990). *The use of wetlands for controlling stormwater pollution*. Woodward-Clyde Consultants. Prepared for the United States Environmental Agency.

United States Environmental Protection Agency (U.S. EPA). (1977). *Process Design Manual for Land Treatment of Municipal Wastewater. EPA-625-1-77-008*. Washington, DC.

United States Environmental Protection Agency (U.S. EPA). (1983). *Results of the Nationwide Urban Runoff Program. Volume I - Final Report. EPA-832-R-83-112*. Washington, DC.

United States Environmental Protection Agency (U.S. EPA). (1996). *Overview of the Stormwater Program. EPA-833-R-96-008*. Washington, DC.

Urban Drainage and Flood Control District (UDFCD). (2002). *Urban Storm Drainage Criteria Manual: volume 3 - best management practices*. Denver, CO. www.udfcd.org.

Urbonas, B.R. and P. Stahre. (1993). *Stormwater - Best Management Practices Including Detention*. Prentice Hall, Englewood Cliffs, NJ.

Commonwealth of Virginia Department of Conservation and Recreation (VA DCR). (1999). *Virginia Stormwater Management Handbook, Volume I,*. Richmond, VA.

Washington State Department of Ecology (WA DOE). (1991). *Stormwater Management Manual for the Puget Sound Basin - Public Review Draft*. Olympia, WA.

Washington State Department of Ecology (WA DOE). (2001). *Stormwater Management Manual for Western Washington. Volume V - Runoff Treatment BMPs*. http://www.ecy.wa.gov/programs/wq/stormwater/index.html.

Water Environment Federation and American Society of Civil Engineers (WEF and ASCE). (1998). *Urban runoff quality management*. 1-57278-039-8 and 0-7844-0174-8. Alexandria, VA.

Wyoming Department of Environmental Quality (WY DEQ). (1999). *Urban Best Management Practices for Nonpoint source Pollution*.

Yousef, Y., M. Wanielista, and H. Harper. (1985). *Best Management Practices: Removal of highway contaminants by roadside swales*. Prepared for Florida Department of Highway Transportation, Tallahassee, FL.