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for
Seattle Municipal Code, Chapters 22.800 – 22.808

Stormwater Code

Volume 3

Stormwater Flow Control and Water Quality Treatment Technical Requirements Manual

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Preface

Background

This Directors' Rule, the *Stormwater Flow Control and Water Quality Treatment Technical Requirements Manual*, presents approved methods, requirements, criteria, details, and general guidance for analysis and design of flow control and water quality facilities pursuant to the Seattle Municipal Code (SMC), Chapter 22.800 – 22.808, the *Stormwater Code.*

Purpose of the Stormwater Code

In addition to meeting the specific stormwater needs of the City of Seattle (City), the Stormwater Code also meets certain requirements of the City's National Pollutant Discharge Elimination System (NPDES) Phase I General Permit for Discharges from Municipal Separate Storm Sewers. Issued to the City under the federal Clean Water Act by the Washington State Department of Ecology (Ecology), one of the conditions of this permit requires Seattle to regulate activities that impact the quality and quantity of stormwater runoff. This is accomplished, in large measure, through the Stormwater Code and its associated Directors' Rules, which Ecology has determined to be equivalent to the minimum requirements contained in the City's Phase I NPDES permit as well as the *Stormwater Management Manual for Western Washington* (Ecology 2005).

The Stormwater Code contains regulatory requirements that provide for and promote the health, safety, and welfare of the general public. The provisions of the Stormwater Code are designed to accomplish the following purposes:

1. Protect, to the greatest extent practicable, life, property and the environment from loss, injury, and damage by pollution, erosion, flooding, landslides, strong ground motion, soil liquefaction, accelerated soil creep, settlement and subsidence, and other potential hazards, whether from natural causes or from human activity;
2. Protect the public interest in drainage and related functions of drainage basins, watercourses, and shoreline areas;
3. Protect receiving waters from pollution, mechanical damage, excessive flows and other conditions in their drainage basins which will increase the rate of downcutting, streambank erosion, and/or the degree of turbidity, siltation and other forms of pollution, or which will reduce their low flows or low levels to levels which degrade the environment, reduce recharging of groundwater, or endanger aquatic and benthic life within these receiving waters and receiving waters of the state;
4. Meet the requirements of state and federal law and the City's municipal stormwater NPDES permit;

5. Protect the functions and values of environmentally critical areas as required under the state's Growth Management Act and Shoreline Management Act;

6. Protect the public drainage system from loss, injury, and damage by pollution, erosion, flooding, landslides, strong ground motion, soil liquefaction, accelerated soil creep, settlement and subsidence, and other potential hazards, whether from natural causes or from human activity; and

7. Fulfill the responsibilities of the City as trustee of the environment for future generations.

To support implementation of the Stormwater Code, the Director of Seattle Public Utilities (SPU) and the Director of the Department of Planning and Development (DPD) promulgate rules that provide specific technical requirements, criteria, guidelines, and additional information. There are currently four joint Directors' Rules:

- **Volume 1: the Source Control Technical Requirements Manual** (Directors' Rule 2009-003 [SPU], 15-2009 [DPD]) provides information designed to help individuals, businesses, and public agencies in Seattle implement best management practices for controlling pollutants at their source and preventing contamination of stormwater runoff.

- **Volume 2: the Construction Stormwater Control Technical Requirements Manual** (Directors' Rule 2009-004 [SPU], 16-2009 [DPD]) contains temporary erosion and sediment control technical requirements, which are required to prevent contaminants from leaving projects during construction. It also provides submittal requirements for drainage control review to help ensure stormwater controls are appropriately implemented during construction projects.

- **Volume 3: the Stormwater Flow Control and Water Quality Treatment Technical Requirements Manual** (Directors' Rule 2009-005 [SPU], 17-2009 [DPD]) presents approved methods, criteria, and details for analysis and design of stormwater flow control and water quality treatment best management practices. It also provides information regarding hydrologic modeling for stormwater designs.

- **Volume 4: the Stormwater Code Enforcement Manual** (Directors' Rule 2009-006 [SPU], 18-2009 [DPD]) provides standards, guidelines, and requirements for enforcing the Stormwater Code.

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Chapter 1 - Introduction

1.1 What is the Purpose of this Manual?

The Stormwater Flow Control and Water Quality Treatment Technical Requirements Manual is a Directors' Rule that describes and provides technical requirements for selecting, designing, constructing, and maintaining stormwater flow control and water quality treatment best management practices (BMPs) that are required by the Stormwater Code. The BMPs contained in this manual are designed to reduce the flow rates and/or volumes of stormwater runoff, and reduce the level of pollutants contained in that runoff. This manual contains BMPs for all types of land development and redevelopment – including residential, commercial, industrial, and roadway projects. In accordance with the provisions of the Stormwater Code, additional controls beyond those specified in this manual may be required.

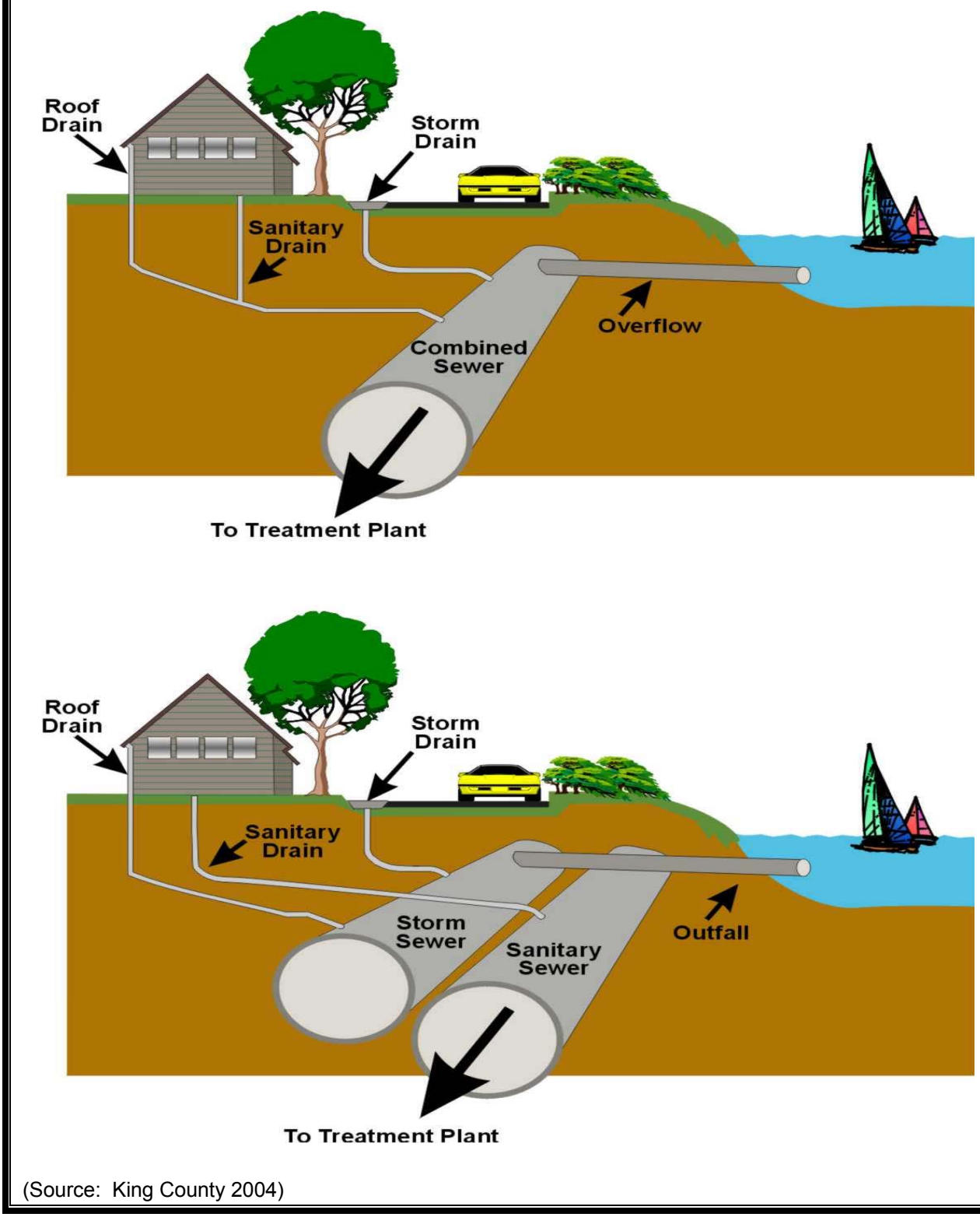
Note that a comprehensive list of definitions is provided in Appendix A, as well as in the Stormwater Code.

1.2 What is Stormwater Runoff?

Stormwater is the water that originates from rainfall and other forms of precipitation, and eventually reaches the ground to become surface water runoff. In undeveloped landscapes, most precipitation is caught by trees and vegetation (called interception) or infiltrates into the soil. In urbanized areas such as Seattle however, a substantial amount of precipitation lands on impervious surfaces such as rooftops and paving and becomes stormwater runoff.

1.3 What is the Difference between a Combined Sewer and a Drainage System?

The Stormwater Code refers to public combined sewers and public drainage systems. Combined sewer systems collect and convey in a single system a combination of sewage and industrial discharges (collectively called "wastewater") and stormwater runoff. Drainage systems collect and convey stormwater in a system that is separated from the wastewater system. In partially separated drainage systems, rooftop drainage is generally directed to the sanitary sewer, while street runoff is directed to the separate storm drainage system. Figure 1.1 provides an illustration of a combined sewer system (upper) and a separated drainage system (lower). The way in which the different types of drainage systems are generally distributed in Seattle is illustrated in Figure 1.2.



(Source: King County 2004)

Figure 1.1. Combined Sewer System (Upper) versus Separated Drainage System (Lower).

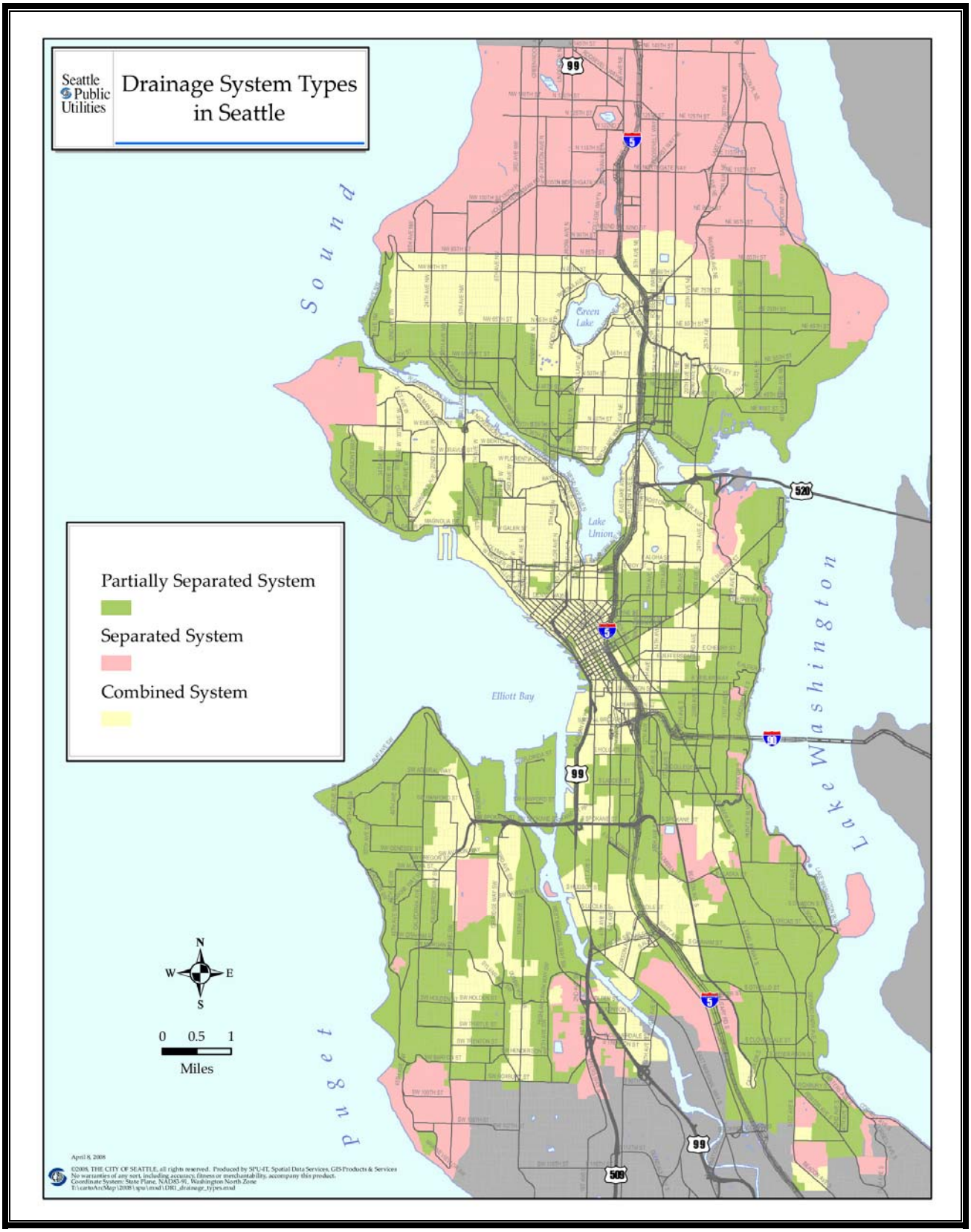


Figure 1.2. Map of Seattle Drainage Systems.

1.4 Why is there a Need for Flow Control and Water Quality Treatment for Stormwater Runoff?

Prior to development, Seattle was a forested landscape that allowed most precipitation to be intercepted by trees and vegetation, where it was eventually evaporated or transpired (i.e., absorbed by the trees and vegetation), or infiltrated into the ground; only a relatively small portion of precipitation became surface runoff. However, the extensive urbanization that has occurred in Seattle has significantly altered the physical landscape, resulting in the introduction of expansive impervious areas and an accompanying loss of vegetation (for evapotranspiration) and natural ground cover (for infiltration). Other less visible changes have also occurred with development, including compaction or removal of native soil. Additionally, the construction of drainage systems caused runoff to be more rapidly conveyed to nearby streams and receiving water bodies. Figure 1.3 illustrates how urban development alters the way in which rainwater runs off the land, infiltrates the ground, and evapo-transpires through vegetation. As development occurs, dramatic increases are seen in both the volume and intensity of runoff compared to natural conditions, including:

- Increased flow rates of runoff (by a factor of 2 to 5)
- Increased volume of runoff (upwards of 50-75 percent increases)
- Decreased time for runoff to reach a receiving water
- Reduced ground water recharge
- Increased frequency and duration of high stream flows and wetland inundation during and after wet weather
- Reduced stream flows and wetland water levels during the dry season
- Greater stream flow velocities.

These changes in watershed hydrology often trigger dramatic physical changes in stream morphology, including channel scour, channel straightening, increased sedimentation in low energy areas, loss of over-bank vegetation, and loss of large woody debris and other habitat-forming features that are normally present in natural channels. In addition, erosion and subsequent deposition of sediment can smother fish spawning and rearing habitats. Habitats are altered when a stream changes its physical configuration and substrate due to increased flows. Natural riffles, pools, gravel bars, and other areas can be altered or destroyed. These and other alterations produce a habitat structure that is very different from the one in which the resident aquatic life evolved. The biological communities in wetlands can also be severely impacted and altered by the hydrological changes. Relatively small changes in the natural water elevation fluctuations can cause dramatic shifts in vegetative and animal species composition. To mitigate these effects of development, typical stormwater management practices include flow

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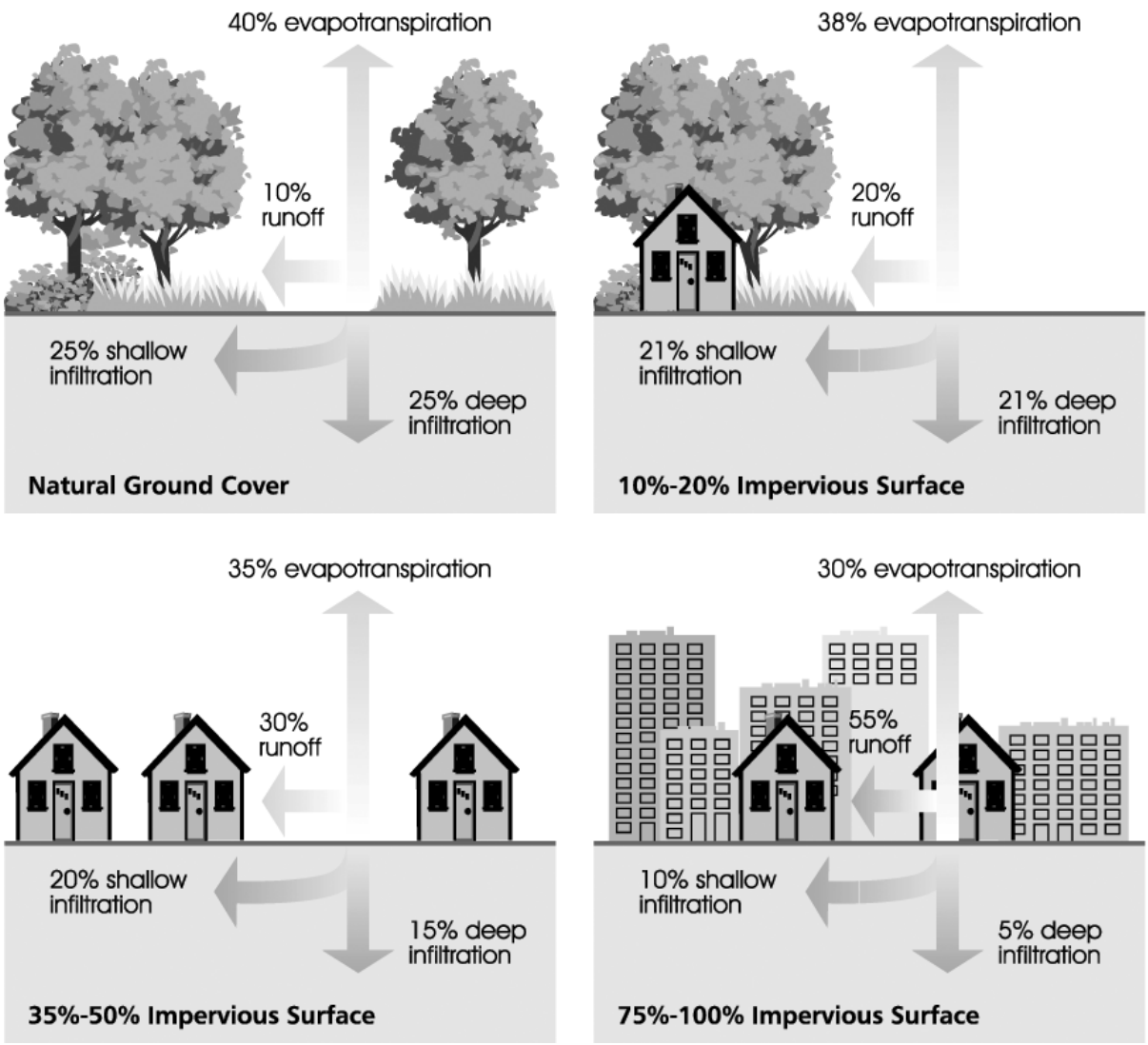
control BMPs. Flow control BMPs are intended to reduce and potentially reverse the negative effects described above.



(Source: Federal Interagency Stream Restoration Working Group 1998)

Figure 1.3. Relation between Amount of Impervious Cover in a Watershed and Changes in Stormwater Runoff.

Urbanization can also cause an increase in the types and quantities of pollutants present in surface and ground waters. Runoff from urban areas has been shown to contain many different types of pollutants, depending on the nature of the activities in those areas. The runoff from roads and highways can be contaminated with pollutants from vehicles. Oil and grease, polycyclic aromatic hydrocarbons (PAHs), lead, zinc, copper, cadmium, and sediments (soil particles) are typical pollutants in road runoff. Runoff from industrial areas can

contain many types of heavy metals, sediments, and a broad range of man-made organic pollutants, including phthalates, PAHs, and other petroleum hydrocarbons. Residential areas can contribute many of the same pollutants as in roadway runoff, as well as herbicides, pesticides, nutrients (from fertilizers), and bacteria and viruses (from animal waste). At high enough concentrations or under continuous loads, these contaminants can be toxic to aquatic organisms and can also limit the recreational use of the receiving waters. To minimize the effects of new and re-development on water quality, typical stormwater management practices include water quality treatment BMPs. Water quality BMPs are intended to reduce and potentially reverse the negative effects described above.

Urbanization also can cause changes in water temperature. Heated stormwater from impervious surfaces and exposed treatment and detention ponds may discharge to streams that are already unnaturally warm due to reduced shade from riparian vegetation. Urbanization can also reduce ground water recharge, which lowers the natural water table levels and reduces sources of cool ground water entering Seattle's streams. Additionally, in the winter, stream temperatures may actually be unnaturally low due to loss of riparian cover. There is also concern that the replacement of warmer ground water inputs with colder surface runoff during colder periods may have biological impacts. To reduce the effects of development on receiving water temperature, the Stormwater Code is requiring and encouraging the use of green stormwater infrastructure (parcel-scale stormwater facilities that utilize infiltration, evapotranspiration, vegetation, or stormwater reuse). The intention is to move the City towards re-establishing lost vegetation in the fully developed areas.

1.5 How can Stormwater Flow Control and Water Treatment Best Management Practices (BMPs) Help?

Stormwater flow control and water quality treatment BMPs are measures implemented to protect the public drainage system, public combined sewer, and receiving surface waters from pollution and the impacts of high flows.

For projects that involve creating or replacing impervious surfaces, flow control BMPs are designed and constructed to:

- Reduce stormwater flow rates from areas draining to the combined sewer system
- Reduce stormwater flow rates and durations from areas draining to Seattle's creeks and other flow-critical receiving waters in order to reduce streambank erosion and other adverse impacts of high flows on aquatic ecosystems
- Reduce stormwater flow rates from areas draining to areas of the city that have capacity-constrained drainage systems.

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For projects that involve creating or replacing pollution-generating surfaces, water quality treatment BMPs are designed and constructed to:

- Reduce the level of pollutants from developed areas entering into Puget Sound or Seattle's creeks, bays, lakes, and other receiving surface waters. Targeted pollutants include: sediments, nutrients, metals, oil, and other contaminants that can be removed by settling, filtration, biofiltration, and separation.

1.6 What are Stormwater Flow Control Best Management Practices (BMPs)?

Flow control BMPs typically detain, retain, or infiltrate stormwater runoff to control the flow rate, frequency, duration, and sometimes the volume of stormwater runoff leaving the site. Many different types of BMPs are available to manage stormwater flow rates. These BMPs range from simple landscaping-based BMPs (e.g., soil amendments, biofiltration swales, and bioretention systems) to underground detention vaults. Determining which BMPs are best suited for a given site will depend on available land surface, infiltration capacity of the soils, and other factors. Although more than one type of flow control facility may be appropriate at a single project site, developers for some projects are required to consider first using green stormwater infrastructure, which is designed to reduce runoff from development using infiltration, evapotranspiration, or stormwater reuse. Details on green stormwater infrastructure are provided in Section 4.4.

The minimum requirements presented in Chapter 2 outline when a flow control facility must be installed, pursuant to the Stormwater Code. Descriptions and design standards for the common Flow Control BMPs are provided in Chapter 4 of this manual. BMPs that can serve a dual purpose of flow control and treatment are identified in Chapter 4, Section 4.2.

1.7 What are Water Quality Treatment Best Management Practices (BMPs)?

Water quality treatment BMPs are facilities that remove pollutants by some combination of gravity settling of particulate pollutants, filtration, biological processes, and adsorption. Examples include biofiltration swales, sand filtration systems, and stormwater wet ponds. Treatment BMPs can provide significant levels of pollutant load reductions if properly designed and maintained. The minimum requirements presented in Chapter 2 outline when a water quality treatment facility must be installed, pursuant to the Stormwater Code. Descriptions and design standards for the common water quality treatment BMPs are provided in Chapter 5 of this manual.

1.8 Contents of this Manual

Chapter 1 of this manual serves as an introduction to stormwater runoff and stormwater flow control and water quality treatment best management practices utilized to meet some of the requirements of the Stormwater Code. The remaining chapters of this manual are organized as follows:

- Chapter 2 describes the minimum requirements contained in the Stormwater Code
- Chapter 3 summarizes site planning, site assessment, and drainage control review
- Chapter 4 details selection and specific design standards for flow control BMPs
- Chapter 5 details selection and specific design standards for runoff treatment BMPs
- Chapter 6 describes the methods of hydrologic analysis.

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Chapter 2 - Minimum Requirements

2.1 Introduction

This Chapter provides guidance regarding how to apply the minimum requirements contained in the Stormwater Code pertaining to flow control and treatment. Minimum requirements represent the minimum set of actions, activities, and prohibitions that must be adhered to in order to comply with the Stormwater Code. In general, the minimum requirements for flow control and treatment depend on:

- The project type (e.g., single family residential, roadway)
- The size of the project (e.g., impervious area, pollution generating surface area)
- The type of receiving water (e.g., creek, public combined sewer)
- The capacity of the downstream drainage system.

Note: this chapter uses many key terms that have a significant impact on the applicability of each minimum requirement. Designers and other responsible parties must reference Appendix A (as well as SMC 22.801) for a comprehensive list of definitions.

2.2 Organization

There are several major categories of minimum requirements contained in the Stormwater Code. The key minimum requirements pertaining to stormwater flow control and water quality treatment are summarized in the following sections. Methods to determine the applicable minimum requirements and details on each Minimum Requirement are also outlined in detail in the subsequent sections as follows:

- Section 2.3 outlines the steps for determining the applicable Minimum Requirements for flow control and water quality treatment that apply to a given project.
- Section 2.4 provides details on the Minimum Requirements for Flow Control and Minimum Requirements for Treatment.
- Section 2.5 provides a summary and quick reference for determining the applicable elements of the Minimum Requirements for Flow Control and Minimum Requirements for Treatment.
- Section 2.6 outlines other Minimum Requirements that may apply to a given project (beyond flow control and water quality treatment).

- Section 2.7 summarizes special circumstances that do not closely fit into the descriptions of the various project types, and therefore may require case-by-case review.
- Section 2.8 summarizes options for alternative means of complying with the Minimum Requirements.
- Section 2.9 outlines the next steps for identifying and planning for stormwater management on a given project.

2.3 Steps for Determining Minimum Requirements for Flow Control and Treatment

There are seven basic steps used to determine which minimum requirements for flow control and water quality treatment apply to a particular project. Per the Stormwater Code (22.801.170), a "project" means “the addition or replacement of impervious surface or the undertaking of land disturbing activity on a site.” Note that the steps are focused on determining applicable minimum requirements for flow control and treatment specifically. **All projects must also review and comply with all other Stormwater Code requirements, in particular the Minimum Requirements for All Discharges and All Real Property (Stormwater Code Section 22.803) and the Minimum Requirements for All Projects (Stormwater Code Section 22.805).**

Step 1 – Define the boundaries of the project site

The boundaries of the project site must contain the discharge point, all land disturbing activities, and all new and replaced impervious surfaces. See Chapter 3 for additional details. Defining the project boundaries will help establish the type of project in Step 2.

Step 2 – Identify the type of project

For the purposes of determining applicable minimum requirements, there are six general classifications of projects:

- Single-family residential project: A single-family residential project is defined in the Stormwater Code (SMC 22.801.200) as a project that constructs one single-family dwelling unti located in land classified as being Single-family Residential 9,600 (SF 9600), Single-family Residential 7,200 (SF 7200), or Single-family Residential 5,000 (SF 5000) per SMC 23.30.010; and the total new plus replaced impervious surface is less than 10,000 square feet; and the total new plus replaced pollution-generating impervious surface is less than 5,000 square feet. Note that projects with 10,000 square feet or more of new plus replaced impervious surface, or over 5,000 square feet of PGIS are considered a parcel-based project. See Figure 2.1 for an illustration of the boundaries of the single family residential project definition.

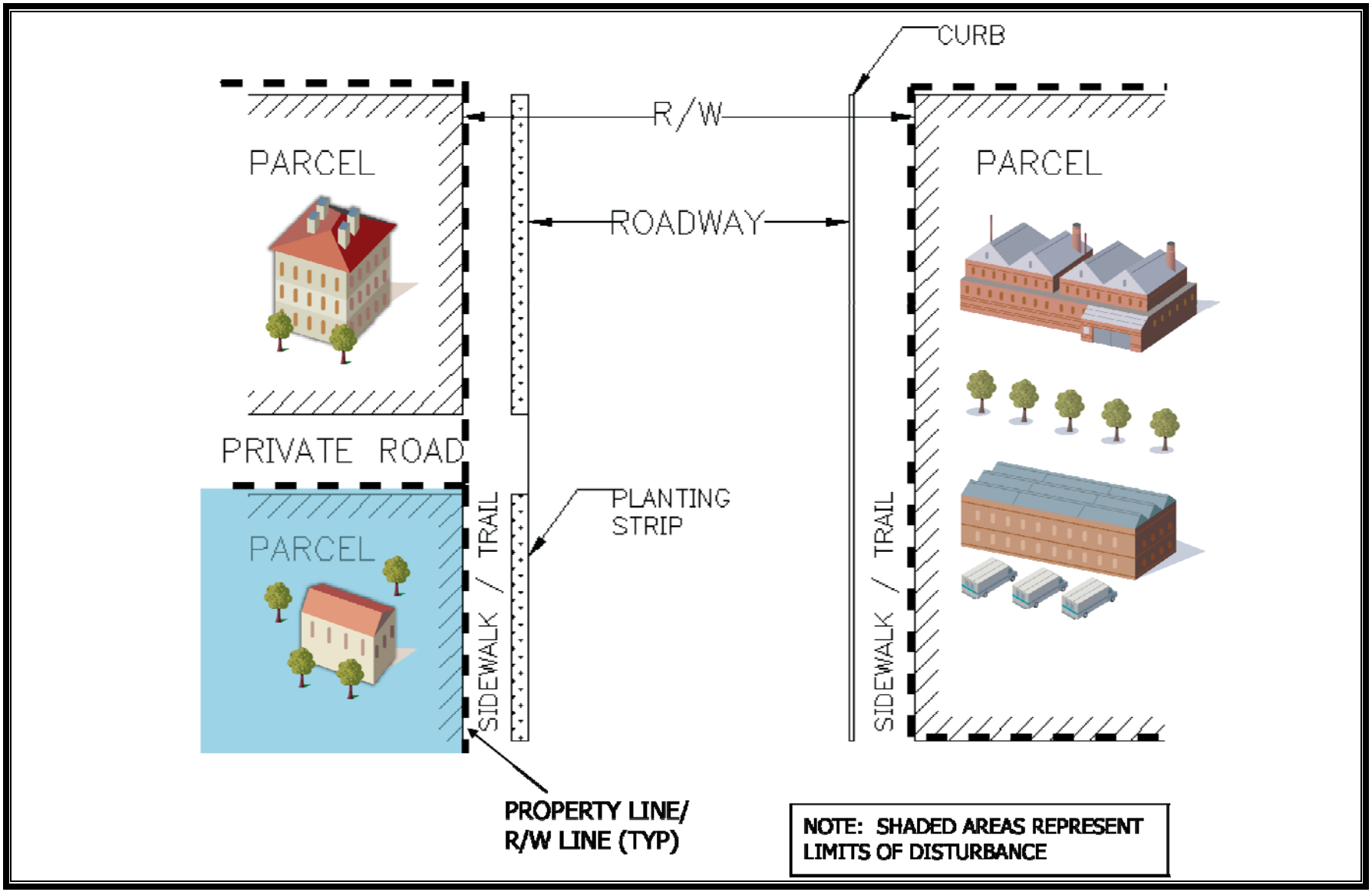

Figure 2.1. Single-family Residential Project Site Definition.

- Parcel-based project: A parcel-based project means any project that is not a roadway project, single-family residential project, sidewalk project, or trail project (SMC 22.801.170). Examples include commercial developments, and multi-family developments. See Figure 2.2 for two example footprints of parcel-based projects.
- Roadway project: A roadway project is a project located in the public right-of-way, that involves the creation of a new roadway or replacement of an existing roadway, or that involves the creation of new or replacement of existing impervious surface (SMC 22.801.190). Although a roadway project, as defined in the Stormwater Code, typically involves creating a new roadway surface or replacing an existing roadway surface, it can include other improvements located in the public right-of-way. See Figure 2.3 for an illustration of the boundaries of the roadway project definition.
- Joint project: A joint project is a project that is both a parcel-based project and a roadway project (SMC 22.801.110). A joint project must evaluate applicable minimum requirements for each type of project, using the right-of-way boundary as the line of demarcation. See Figure 2.4 for an illustration of a joint project including parcel work and roadway work.
- Sidewalk project: A sidewalk project is a project that exclusively involves the creation of a new or replacement of an existing sidewalk, including any associated planting strip, curb, or gutter (SMC 22.801.200). See Figure 2.5 for an illustration of the boundaries of the sidewalk project definition.
- Trail project: A trail project is a project that exclusively involves creating a new or replacement of an existing trail, and which does not contain pollution-generating impervious surfaces (22.801.210).

Step 3 – Identify the type of downstream collection system and receiving water

Runoff leaving the project site is classified for minimum requirement purposes based on the type of receiving surface water and type of drainage system into which the project discharges. **The minimum requirements vary considerably for the different types of receiving systems, therefore it is very important to determine and specify receiving surface water and type of drainage system.** These are described briefly below (see Figures 2.6 and 2.7 for an overview of the types of drainage systems in Seattle).

- Wetlands. Wetlands (designated under SMC 25.09.020) require specific flow control protection.
- Creek Basins. There are multiple stream systems throughout the city, generally referred to as “creeks” (see Figures 2.6 and 2.7). Projects that discharge to creek basins must meet specific flow control and water quality treatment requirements. **Minimum requirements vary among**

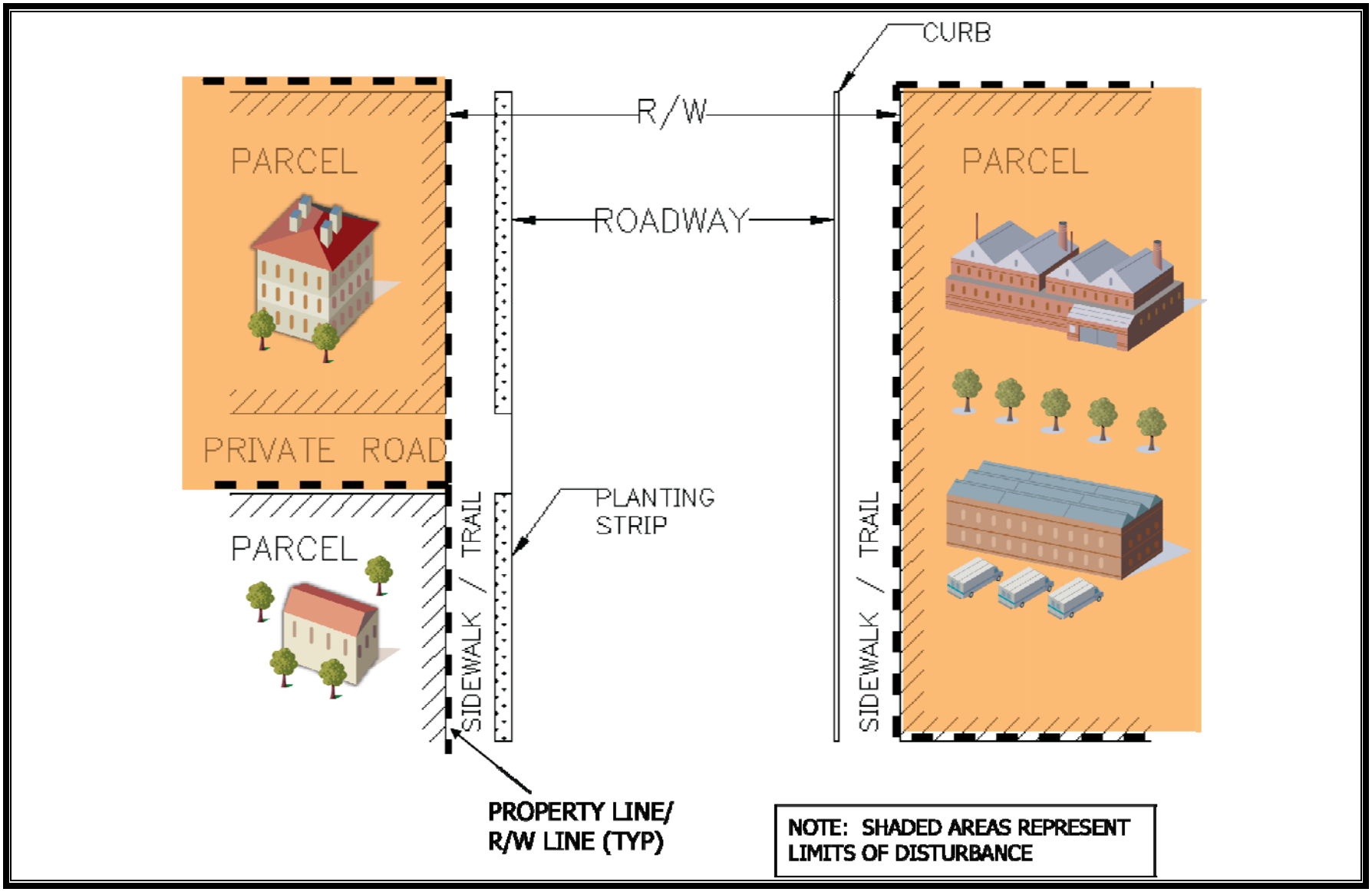

Figure 2.2. Parcel-based Project Site Definition.

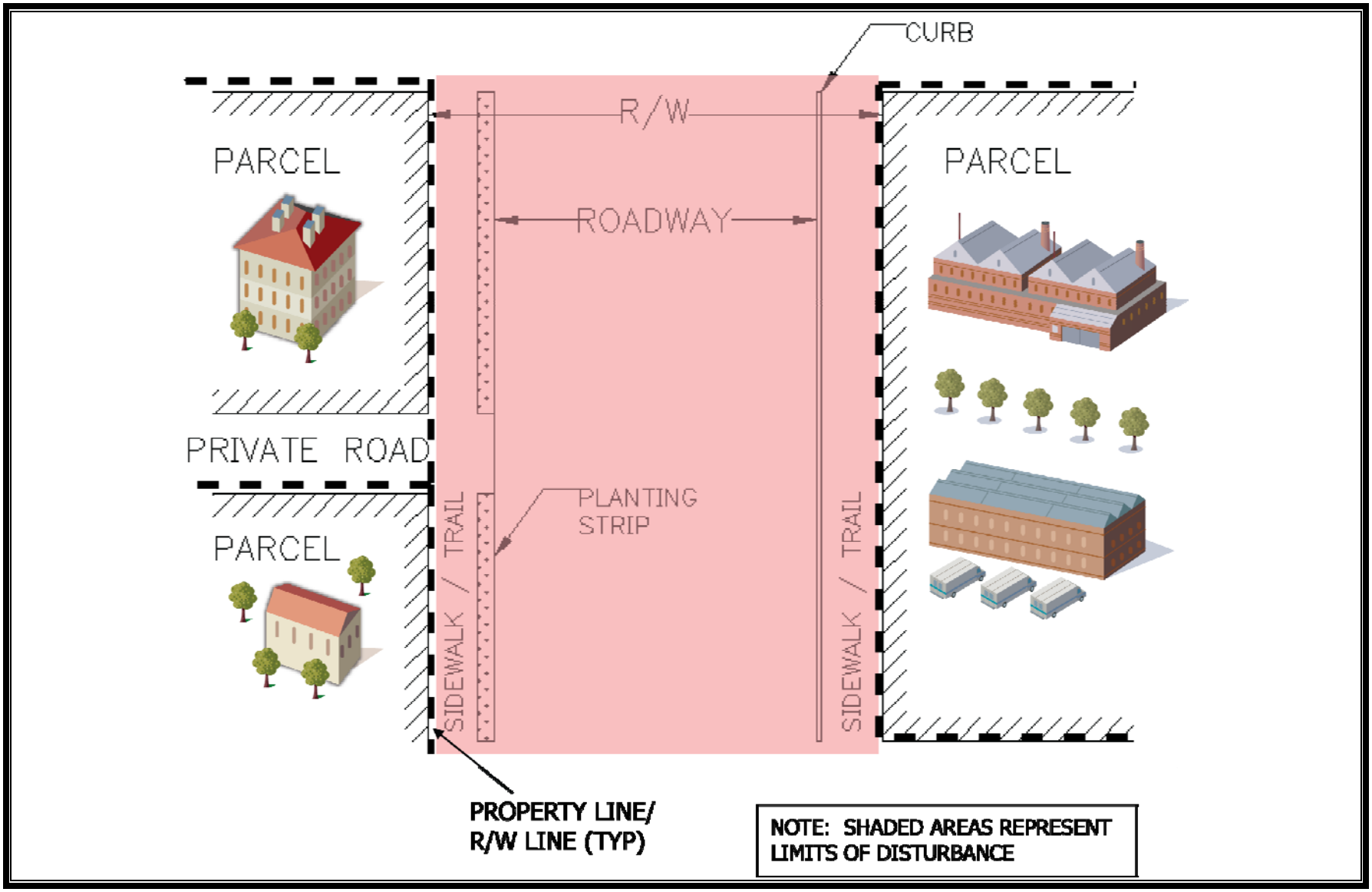

Figure 2.3. Roadway Project Site Definition.

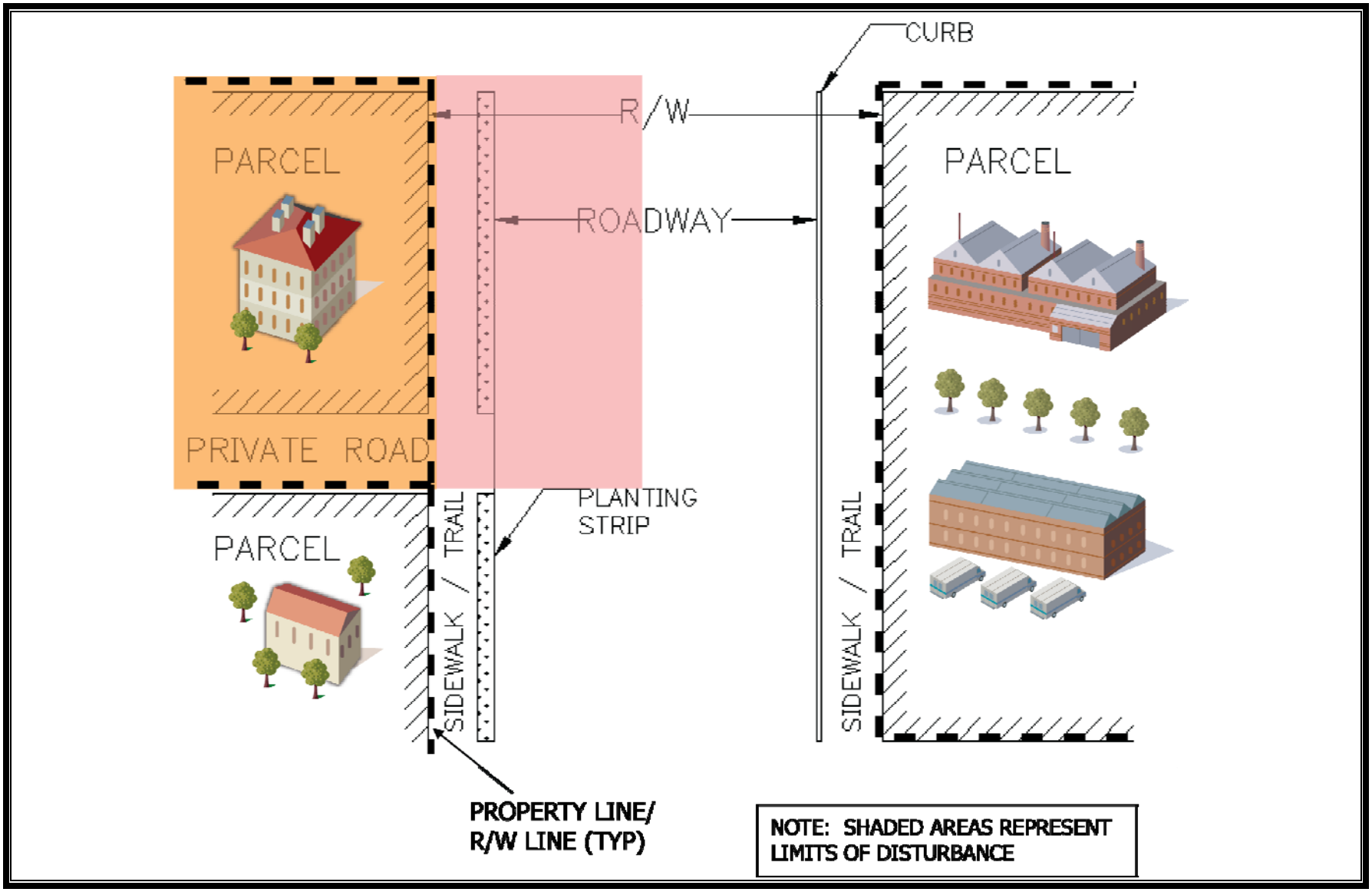

Figure 2.4. Joint Project Site Definition.

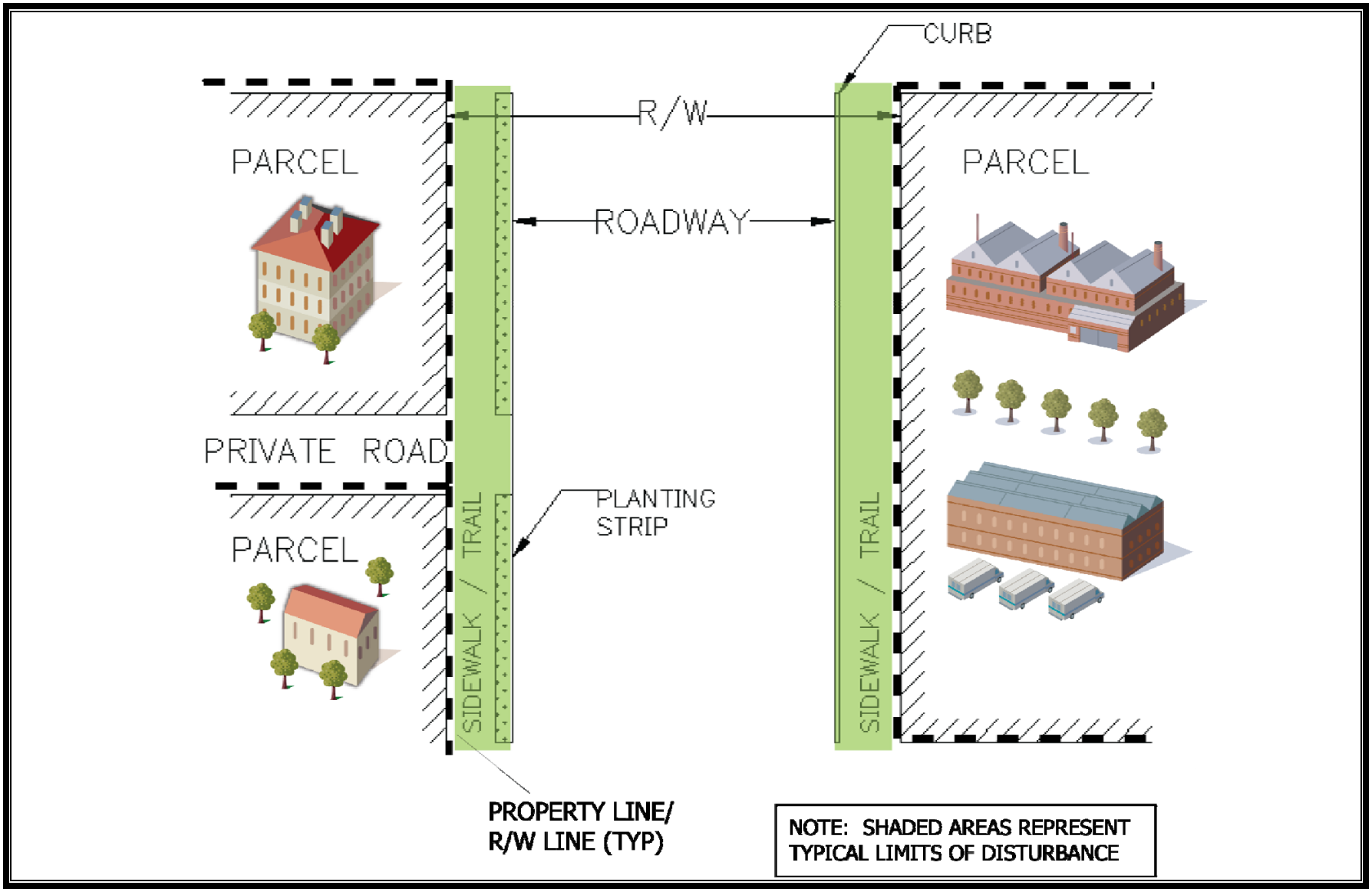

Figure 2.5. Sidewalk-only Site Definition.

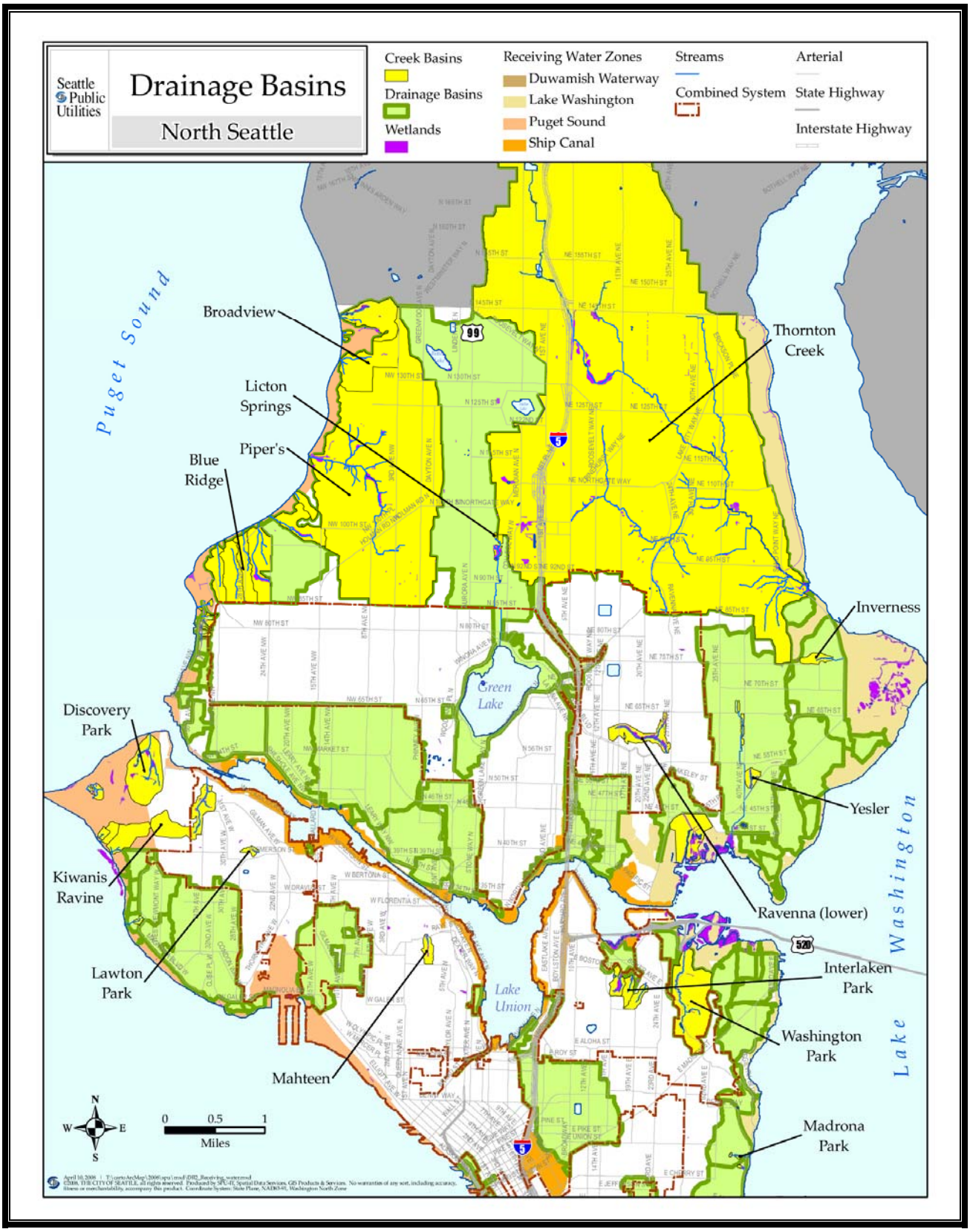

Figure 2.6. North End Drainage Basins.

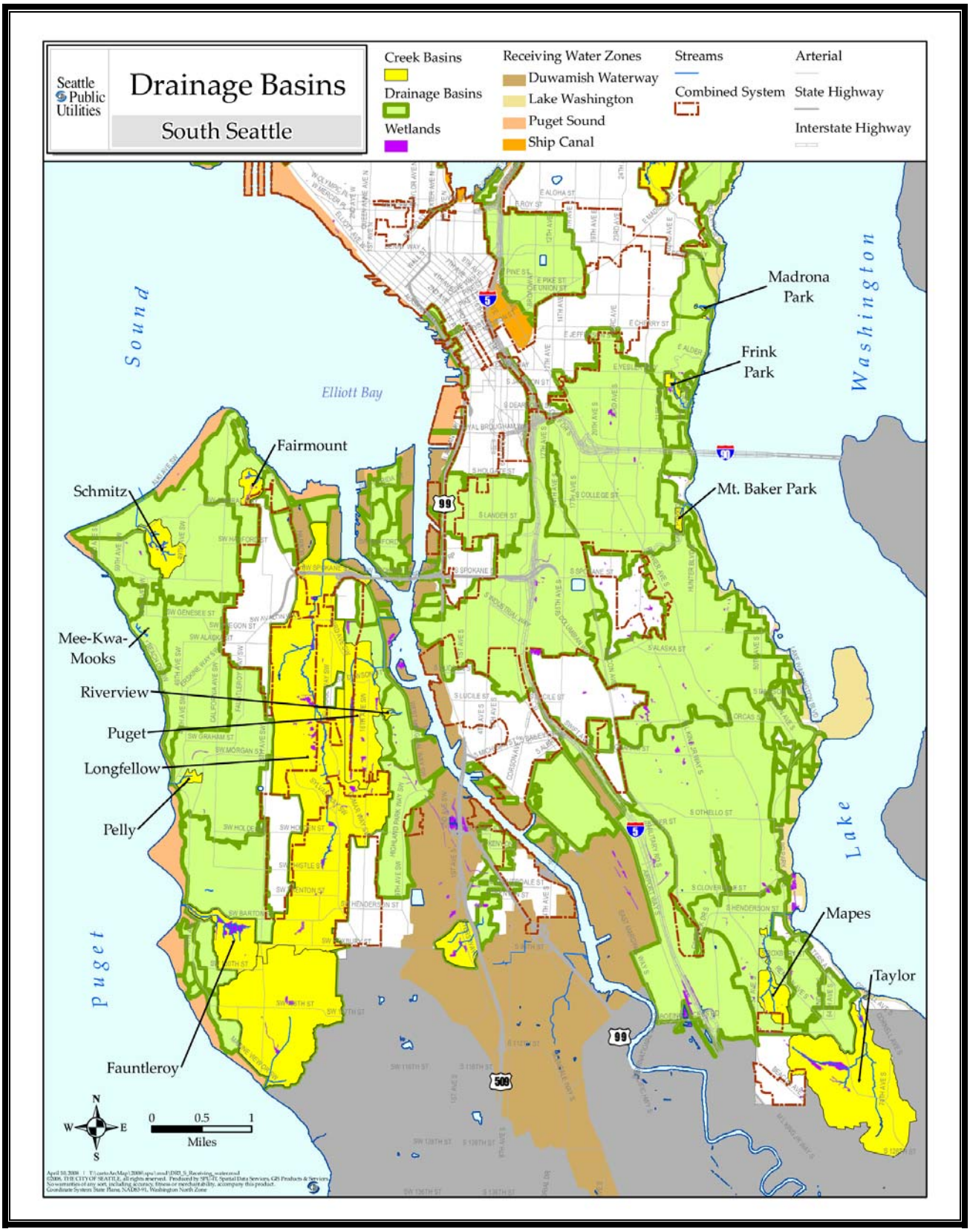

Figure 2.7. South End Drainage Basins.

the different creek basins. The two flow control standards specific to creek basins (see Sections 2.4.6 and 2.5.2), and are commonly referred to as the "Pre-developed Forest Standard" and "Pre-developed Pasture Standard." Note that projects that discharge to a creek basin that then discharge to a designated receiving water must meet the requirements applicable to creek basins.

- Public Combined Sewer System. The public combined sewer is a publicly owned and maintained system that carries drainage water and wastewater and flows to a publicly owned treatment works (SMC 22.801.170).
- Small Lake Basins. Seattle's small lake basins include Bitter Lake, Green Lake, or Haller Lake.
- Designated Receiving Water. This means the Duwamish River, Puget Sound, Lake Washington, Lake Union, Elliott Bay, Portage Bay, Union Bay, the Lake Washington Ship Canal, and other receiving waters determined by the Director of SPU and approved by Ecology as having sufficient capacity to receive discharges of drainage water such that a site discharging to the designated receiving water is not required to implement flow control (SMC 22.801.050).

Note also that capacity constraints in any drainage system can modify the flow control requirements for discharges.

- Capacity-constrained System. A capacity-constrained system is a drainage system that the Director of SPU has determined to have inadequate capacity to carry drainage water.

Step 4 – Calculate how much new plus replaced impervious surface is created, and how much native vegetation is converted

The thresholds triggering specific Minimum Requirements for Flow Control and Treatment are based on the amount of the project's new plus replaced impervious surface and converted native vegetation. The project's total affected impervious surface areas are all summed to determine the total new plus replaced impervious surface area. See Appendix A and SMC 22.801 for detailed definitions of these key terms. Note that open, uncovered retention/detention facilities shall not be considered as impervious surfaces for the purposes of determining whether the thresholds for application of minimum requirements are exceeded. However, these facilities shall be considered impervious surfaces for the purposes of stormwater BMP modeling. **It is important to review SMC 22.801 and Appendix A for complete definitions.**

The amount of native vegetation that is removed and replaced with lawn, landscaping, and pasture ground cover must also be calculated.

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Separate summaries of new plus replaced impervious surface areas and converted native vegetation are required for work within and outside the right-of-way.

Step 5 – Calculate how much new plus replaced pollution generating surface is created

The thresholds triggering specific Minimum Requirements for Treatment are based on the total amount of the project's new plus replaced pollution-generating *impervious* surface (PGIS) and pollution-generating *pervious* surface (PGPS). PGIS and PGPS include areas that are considered to be a significant source of pollutants in stormwater runoff. **It is important to review SMC 22.801 and Appendix A for complete definitions of these key terms.** Examples include areas subject to vehicular use (including permeable pavement); certain industrial activities; and outdoor storage of erode-able or leachable materials, wastes, or chemicals. Metal roofs are also considered to be PGIS unless they are coated with an inert, non-leachable material (e.g., baked-on enamel coating). PGPS includes, for example, lawns and landscaping areas subject to the use of fertilizers and pesticides.

Separate summaries of new plus replaced pollution generating impervious and pervious surface areas are required for work within and outside the right-of-way.

Step 6 – Determine which Minimum Requirements for Flow Control and Which Minimum Requirements for Treatment apply

Based on the information obtained during Step 1 through Step 5, the applicable minimum requirements for flow control and treatment can be determined. See the following section (2.4) for additional details on the minimum requirements for treatment and flow control for the different types of projects. An overview of the other minimum requirements applicable to all projects is included in Section 2.6.

Step 7 – Site Planning, Assessment, and BMP Selection

After the applicable minimum requirements have been identified, each project should evaluate project design considerations and perform a site assessment as outlined in Chapter 3. The goal of the site planning and assessment step is to identify any additional issues that must be addressed in association with stormwater management requirements before choosing flow control and/or treatment BMPs. Site specific factors to consider may include, but are not limited to: site boundaries and structures, soil conditions and infiltration capacity, critical area issues (e.g., flood plains, landslide prone areas, and site contamination), groundwater elevations, etc. Project proponents need to evaluate all the applicable code requirements and conduct a full site assessment to characterize site opportunities and constraints before choosing and designing stormwater strategies (see Chapter 3). Once the site conditions are known and the applicable minimum requirements have been identified, proceed to Chapters 4 and 5 to begin the BMP selection and design processes.

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2.4 Minimum Requirements for Flow Control and Treatment

The Minimum Requirements for Flow Control and Minimum Requirements for Treatment cover the primary stormwater requirements applicable to most projects. Note that all italicized text presented in boxes in this chapter is excerpted from the Stormwater Code. These excerpts are provided for reference to help highlight key code requirements. Project proponents and designers must refer to the Stormwater Code for full requirements and details, but the major components of these minimum requirements are provided in the following sections. **Note that this rule interprets but does not replace or alter Stormwater Code requirements.**

2.4.1 Minimum Requirements for Single-family Residential Projects (SMC 22.805.030)

All single-family residential projects shall implement green stormwater infrastructure to the maximum extent feasible.

Guidance on how to comply with this requirement is provided in a separate Director's Rule (DR-2009-008). See DPD website (www.seattle.gov/dpd/). Note single-family residential projects are not required to install an underground detention flow control facility to meet the Minimum Requirements for Flow Control.

2.4.2 Minimum Requirements for Trail and Sidewalk Projects (SMC 22.805.040)

All trail and sidewalk projects with 2,000 square feet or more of new plus replaced impervious surface or 7,000 square feet or more of land disturbing activity shall implement green stormwater infrastructure to the maximum extent feasible.

Guidance on how to comply with this requirement is provided in a separate Director's Rule (DR-2009-007). See DPD website (www.seattle.gov/dpd/).

2.4.3 Minimum Requirements for Parcel-Based Projects (SMC 22.805.050)

All projects with 7,000 square feet or more of land disturbing activity or 2,000 square feet or more of new plus replaced impervious surface must implement green stormwater infrastructure to the maximum extent feasible per SMC 22.805.020.F (see Section 2.6.6). In addition, parcel-based projects shall meet the Minimum Requirements for Flow Control (SMC 22.805.080) and the Minimum Requirements for Treatment (SMC 22.805.090) as prescribed below:

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2.4.3.1 Parcel-based Projects Discharging to Wetlands – Flow Control (SMC 22.805.050.A.1)

Parcel-based projects discharging into a wetland shall comply with subsection 22.805.080.B.1 (Wetland Protection Standard) if:

a. The total new plus replaced impervious surface is 5,000 square feet or more; or

b. The project converts 3/4-acres or more of native vegetation to lawn or landscaped areas and from which there is a surface discharge into a natural or man-made conveyance system from the site; or

c. The project converts 2.5 acres or more of native vegetation to pasture and from which there is a surface discharge into a natural or man-made conveyance system from the site.

2.4.3.2 Parcel-based Projects Discharging to Listed Creek Basins – Flow Control (SMC 22.805.050.A.2)

Parcel-based projects discharging into Blue Ridge Creek, Broadview Creek, Discovery Park Creek, Durham Creek, Frink Creek, Golden Gardens Creek, Kiwanis Ravine/Wolfe Creek, Licton Springs Creek, Madrona Park Creek, Mee-Kwa-Mooks Creek, Mount Baker Park Creek, Puget Creek, Riverview Creek, Schmitz Creek, Taylor Creek, or Washington Park Creek shall:

a. Comply with subsection 22.805.080.B.2 (Pre-developed Forested Standard) if the existing impervious coverage is less than 35 percent and one or more of the following apply:

1. The project adds 5,000 square feet or more of new impervious surface and the total new plus replaced impervious surface is 10,000 square feet or more; or

2. The project converts 3/4 acres or more of native vegetation to lawn or landscaped areas and from which there is a surface discharge into a natural or man-made conveyance system from the site; or

3. The project converts 2.5 acres or more of native vegetation to pasture and from which there is a surface discharge into a natural or man-made conveyance system from the site; or

4. The project adds 5,000 square feet or more of new impervious surface and, through a combination of effective impervious surfaces and converted pervious surfaces, causes a 0.1 cubic feet per second increase in the 100-year recurrence interval flow frequency as estimated using a continuous model approved by the Director.

b. Comply with subsection 22.805.080.B.3 (Pre-developed Pasture Standard) if the criteria in subsection 22.805.050.A.2.a do not apply and the total new plus replaced impervious surface is 2,000 square feet or more.

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2.4.3.3 Parcel-based Projects Discharging to Non-listed Creek Basins – Flow Control (SMC 22.805.050.A.3)

Parcel-based projects discharging into a creek not listed in subsection 22.805.050.A.2 shall:

a. Comply with subsection 22.805.080.B.2 (Pre-developed Forested Standard) if the existing land cover is forested and one or more of the following apply:

1. The project adds 5,000 square feet or more of new impervious surface and the total new plus replaced impervious surface is 10,000 square feet or more; or

2. The project converts 3/4 acres or more of native vegetation to lawn or landscaped areas and from which there is a surface discharge into a natural or man-made conveyance system from the site; or

3. The project converts 2.5 acres or more of native vegetation to pasture and from which there is a surface discharge into a natural or man-made conveyance system from the site; or

4. The project adds 5,000 square feet or more of new impervious surface and, through a combination of effective impervious surfaces and converted pervious surfaces, causes a 0.1 cubic feet per second increase in the 100-year recurrence interval flow frequency as estimated using a continuous model approved by the Director.

b. Comply with subsection 22.805.080.B.3 (Pre-developed Pasture Standard) if the criteria in subsection 22.805.050.A.3.a do not apply and the total new plus replaced impervious surface is 2,000 square feet or more.

2.4.3.4 Parcel-based Projects Discharging to Small Lake Basins – Flow Control (SMC 22.805.050.A.4)

Parcel-based projects discharging into Bitter Lake, Green Lake, or Haller Lake drainage basins shall comply with subsection 22.805.080.B.4 (Peak Control Standard) if the total new plus replaced impervious surface is 2,000 square feet or more.

2.4.3.5 Parcel-based Projects Discharging to Public Combined Sewer – Flow Control (SMC 22.805.050.A.5)

Parcel-based projects discharging into the public combined sewer shall comply with subsection 22.805.080.B.4 (Peak Control Standard) if the total new plus replaced impervious surface is 10,000 square feet or more.

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2.4.3.6 Parcel-based Projects Discharging to a Capacity-constrained System – Flow Control (SMC 22.805.050.A.6)

In addition to applicable minimum requirements for flow control in subsection 22.805.050.A.1 through subsection 22.805.050.A.5, parcel-based projects discharging into a capacity-constrained system shall also comply with subsection 22.805.080.B.4 (Peak Control Standard) if the total new plus replaced impervious surface is 2,000 square feet or more.

2.4.3.7 Parcel-based Projects – Water Quality Treatment (SMC 22.805.050.B)

Parcel-based projects not discharging to the public combined sewer shall comply with the minimum requirements for treatment contained in Section 22.805.090, to the extent allowed by law, if:

1. The total new plus replaced pollution-generating impervious surface is 5,000 square feet or more; or

2. The total new plus replaced pollution-generating pervious surfaces is three-quarters of an acre or more and from which there is a surface discharge in a natural or man-made conveyance system from the site.

2.4.4 Minimum Requirements for Roadway Projects (SMC 22.805.060)

All projects with 7,000 square feet or more of land disturbing activity or 2,000 square feet or more of new plus replaced impervious surface must implement green stormwater infrastructure to the maximum extent feasible per SMC 22.805.020.F (see Section 2.6.6). In addition, roadway projects shall meet the Minimum Requirements for Flow Control (SMC 22.805.080) and the Minimum Requirements for Treatment (SMC 22.805.090) as prescribed below:

2.4.4.1 Roadway Projects Discharging to Wetlands – Flow Control (SMC 22.805.060.A.1)

Roadway projects discharging into a wetland shall comply with subsection 22.805.080.B.1 (Wetland Protection Standard) if:

a. The total new plus replaced impervious surface is 5,000 square feet or more; or

b. The project converts 3/4 acres or more of native vegetation to lawn or landscaped areas and from which there is a surface discharge into a natural or man-made conveyance system from the site; or

c. The project converts 2.5 acres or more of native vegetation to pasture and from which there is a surface discharge into a natural or man-made conveyance system from the site.

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2.4.4.2 Roadway Projects Discharging to Listed Creek Basins – Flow Control (SMC 22.805.060.A.2)

Roadway projects discharging into Blue Ridge Creek, Broadview Creek, Discovery Park Creek, Durham Creek, Frink Creek, Golden Gardens Creek, Kiwanis Ravine/Wolfe Creek, Licton Springs Creek, Madrona Park Creek, Mee-Kwa-Mooks Creek, Mount Baker Park Creek, Puget Creek, Riverview Creek, Schmitz Creek, Taylor Creek, or Washington Park Creek shall:

a. Comply with subsection 22.805.080.B.2 (Pre-developed Forested Standard) if the existing impervious coverage is less than 35 percent and one or more of the following apply:

1. The project adds 5,000 square feet or more of new impervious surface and the total new plus replaced impervious surface is 10,000 square feet or more; or

2. The project converts 3/4 acres or more of native vegetation to lawn or landscaped areas and from which there is a surface discharge into a natural or man-made conveyance system from the site; or

3. The project converts 2.5 acres or more of native vegetation to pasture and from which there is a surface discharge into a natural or man-made conveyance system from the site; or

4. The project adds 5,000 square feet or more of new impervious surface and, through a combination of effective impervious surfaces and converted pervious surfaces, causes a 0.1 cubic feet per second increase in the 100-year recurrence interval flow frequency as estimated using a continuous model approved by the Director.

b. Comply with subsection 22.805.080.B.3 (Pre-developed Pasture Standard) if the criteria in subsection 22.805.060.A.2.a do not apply and the total new plus replaced impervious surface is 10,000 square feet or more.

2.4.4.3 Roadway Projects Discharging to Non-listed Creek Basins – Flow Control (SMC 22.805.060.A.3)

Roadway projects discharging into a creek not listed in subsection 22.805.060.A.2 shall:

a. Comply with subsection 22.805.080.B.2 (Pre-developed Forested Standard) if the existing land cover is forested and one or more of the following apply:

1. The project adds 5,000 square feet or more of new impervious surface and the total new plus replaced impervious surface is 10,000 square feet or more; or

2. The project converts 3/4 acres or more of native vegetation to lawn or landscaped areas and from which there is a surface discharge into a natural or man-made conveyance system from the site; or

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3. The project converts 2.5 acres or more of native vegetation to pasture and from which there is a surface discharge into a natural or man-made conveyance system from the site; or

4. The project adds 5,000 square feet or more of new impervious surface and, through a combination of effective impervious surfaces and converted pervious surfaces, causes a 0.1 cubic feet per second increase in the 100-year recurrence interval flow frequency as estimated using a continuous model approved by the Director.

b. Comply with subsection 22.805.080.B.3 (Pre-developed Pasture Standard) if the criteria in subsection 22.805.060.A.3.a do not apply and the total new plus replaced impervious surface is 10,000 square feet or more.

2.4.4.4 Roadway Projects Discharging to Small Lake Basins – Flow Control (SMC 22.805.060.A.4)

Projects discharging into Bitter Lake, Green Lake, or Haller Lake drainage basins shall comply with subsection 22.805.080.B.4 (Peak Control Standard) if the total new plus replaced impervious surface is 10,000 square feet or more.

2.4.4.5 Roadway Projects Discharging to Public Combined Sewer – Flow Control (SMC 22.805.060.A.5)

Roadway projects discharging into the public combined sewer shall comply with subsection 22.805.080.B.4 (Peak Control Standard) if the total new plus replaced impervious surface is 10,000 square feet or more.

2.4.4.6 Roadway Projects Discharging to a Capacity-constrained System – Flow Control (SMC 22.805.060.A.6)

In addition to applicable minimum requirements for flow control in subsection 22.805.060.A.1 through subsection 22.805.060.A.5, roadway projects discharging into a capacity-constrained system shall also comply with subsection 22.805.080.B.4 (Peak Control Standard) if the total new plus replaced impervious surface is 10,000 square feet or more.

2.4.4.7 Roadway Projects – Water Quality Treatment (SMC 22.805.060.B)

Roadway projects not discharging to the public combined sewer shall comply with the minimum requirements for treatment contained in Section 22.805.090, to the extent allowed by law, if:

1. The total new plus replaced pollution-generating impervious surface is 5,000 square feet or more; or

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2. The total new plus replaced pollution-generating pervious surfaces is three-quarters of an acre or more and from which there is a surface discharge in a natural or man-made conveyance system from the site.

2.4.5 Minimum Requirements for Joint Roadway and Parcel-Based Projects (SMC 22.805.070)

The parcel-based portion of joint projects shall comply with the minimum requirements for parcel-based projects contained in Section 22.805.050. The roadway portion of joint projects shall comply with the minimum requirements roadway projects contained in Section 22.805.060. The boundary of the public right-of-way shall form the boundary between the parcel and roadway portions of the joint project for purposes of determining applicable thresholds.

2.4.6 Minimum Requirements for Flow Control (SMC 22.805.080)

Projects triggering this minimum requirement must install flow control facilities meeting the applicable design requirements for the given project and discharge location.

2.4.6.1 Flow Control Requirements – General (SMC 22.805.080.B)

Flow control facilities shall be installed to the extent allowed by law and maintained per rules promulgated by the Director to receive flows from that portion of the site being developed. Post-development discharge determination must include flows from dewatering activities. All projects shall use green stormwater infrastructure to the maximum extent feasible to meet the minimum requirements. Flow control facilities that receive flows from less than that portion of the site being developed may be installed if the total new plus replaced impervious surface is less than 10,000 square feet, the project site uses only green stormwater infrastructure to meet the requirement, and the green stormwater infrastructure provides substantially equivalent environmental protection as facilities not using green stormwater infrastructure that receive flows from all of the portion of the site being developed.

2.4.6.2 Wetland Protection Standard (SMC 22.805.080.B1)

All projects discharging to wetlands or their buffers shall protect the hydrologic conditions, vegetative community, and substrate characteristics of the wetlands and their buffers to protect the functions and values of the affected wetlands. The introduction of sediment, heat and other pollutants and contaminants into wetlands shall be minimized through the selection, design, installation, and maintenance of temporary and permanent controls. Discharges shall maintain existing flows to the extent necessary to protect the functions and values of the wetlands. Prior to authorizing new discharges to a wetland, alternative discharge locations shall be evaluated and infiltration options outside the wetland shall be

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maximized unless doing so will adversely impact the functions and values of the affected wetlands. If one or more of the flow control requirements contained in 22.805.080.B.2 through 22.805.080.B.4 also apply to the project, an analysis shall be conducted to ensure that the functions and values of the affected wetland are protected before implementing these flow control requirements

Projects triggering this minimum requirement need to refer to Guide sheets 1B and 2B presented in Appendix I-D of Ecology's *Stormwater Management Manual for Western Washington* (Ecology 2005) for additional guidance.

2.4.6.3 Pre-developed Forested Standard (SMC 22.805.080.B.2)

The post-development discharge peak flow rates and flow durations must be matched to the pre-developed forested condition for the range of pre-developed discharge rates from 50% of the 2-year recurrence interval flow up to the 50-year recurrence interval flow.

2.4.6.4 Pre-developed Pasture Standard (SMC 22.805.080.B.3)

The post-development discharge peak flow rates and flow durations must be matched to the pre-developed pasture condition for the range of pre-developed discharge rates from 50% of the 2-year recurrence interval flow up to the 2-year recurrence interval flow.

2.4.6.5 Peak Flow Control Standard (SMC 22.805.080.B.4)

The post-development peak flow with a 4% annual probability (25-year recurrence flow) shall not exceed 0.4 cubic feet per second per acre. Additionally, the peak flow with a 50% annual probability (2-year recurrence flow) shall not exceed 0.15 cubic feet per second per acre.

2.4.7 Minimum Requirements for Treatment (SMC 22.805.090)

Projects triggering this minimum requirement must install treatment facilities, which typically remove pollutants through a combination of gravity settling, filtration, biological uptake, and soil adsorption.

2.4.7.1 Treatment Requirements – General (SMC 22.805.090.B)

Water quality treatment facilities shall be installed to the extent allowed by law and maintained per rules promulgated by the Director to treat flows from the pollution generating pervious and impervious surfaces on the site being developed. When stormwater flows from other areas, including non-pollution generating surfaces (e.g., roofs), dewatering activities, and offsite areas, cannot be separated or bypassed, treatment BMPs shall be designed for the entire area draining to the treatment facility. All projects shall use green stormwater infrastructure the maximum extent feasible to meet the minimum requirements.

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Stormwater treatment facilities shall be designed based on the stormwater runoff volume from the contributing area or a peak flow rate as outlined in the following sections.

2.4.7.2 Runoff Treatment Volume (SMC 22.805.090.B.1.a)

The water quality design treatment volume is determined as follows:

> *The daily runoff volume at or below which 91 percent of the total runoff volume for the simulation period occurs, as determined using an approved continuous model.*

Additional guidance on how to comply with this requirement is provided in Chapter 5, Section 5.4.1.1.

2.4.7.3 Runoff Treatment Rates (SMC 22.805.090.B.1.b)

> *Different design flow rates are required depending on whether a treatment facility will be located upstream or downstream of a detention facility:*
>
> *1. For facilities located upstream of detention or when detention is not required, the design flow rate is the flow rate at or below which 91 percent of the total runoff volume for the simulation period is treated, as determined using an approved continuous runoff model.*
>
> *2. For facilities located downstream of detention, the design flow rate is the release rate from the detention facility that has a 50 percent annual probability of occurring in any given year (2-year recurrence interval), as determined using an approved continuous runoff model.*

2.4.7.4 Infiltration Treatment Requirements (SMC 22.805.090.B.1.c)

> *Infiltration facilities designed for water quality treatment must infiltrate 91 percent of the total runoff volume as determined using an approved continuous runoff model. To prevent the onset of anaerobic conditions, an infiltration facility designed for water quality treatment purposes must be designed to drain the water quality design treatment volume (the 91st percentile, 24-hour volume) within 48 hours.*

[Note that the “91st percentile, 24-hour volume” referenced above is equivalent to the daily runoff volume at or below which 91 percent of the total runoff volume for the simulation period occurs.]

2.4.7.5 Basic Treatment (SMC 22.805.090.B.2)

> *A basic treatment facility shall be required for all projects. The requirements of subsection 22.805.090.B.3 (Oil Control Treatment), subsection 22.805.090.B.4*

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(Phosphorus Treatment), subsection 22.805.090.B.5 (Enhanced Treatment) are in addition to this basic treatment requirement.

2.4.7.6 Oil Control Treatment (SMC 22.805.090.B.3)

An oil control treatment facility shall be required for high-use sites, as defined in 22.801.090.

2.4.7.7 Phosphorus Treatment (SMC 22.805.090.B.4)

A phosphorus treatment facility shall be required for projects discharging into nutrient-critical receiving waters.

2.4.7.8 Enhanced Treatment (SMC 22.805.090.B.5)

An enhanced treatment facility for reducing concentrations of dissolved metals shall be required for projects discharging to a fish-bearing stream or lake, and to waters or drainage systems that are tributary to fish-bearing streams, creeks, or lakes, if the project meets one of the following criteria:

a. For a parcel-based project, the total of new plus replaced pollution-generating impervious surface is 5,000 square feet or more, and the site is an industrial, commercial, or multi-family project.

b. For a roadway project, the project adds 5,000 square feet or more of pollution-generating impervious surface, and the site is either:

1. A fully controlled or a partially controlled limited access highway with Annual Average Daily Traffic counts of 15,000 or more; or

2. Any other road with an Annual Average Daily Traffic count of 7,500 or greater.

2.4.7.9 Discharges to Groundwater (SMC 22.805.090.B.6)

Direct discharge of untreated drainage water from pollution-generating impervious surfaces to ground water is prohibited.

2.5 Summary of Minimum Requirements for Flow Control and Minimum Requirements for Treatment

The following sections provide a summary of the key Minimum Requirements for Flow Control and Minimum Requirements for Treatment. The intent is for these sections to serve as a quick reference for designers and other responsible parties already familiar with the detailed requirements presented in the Stormwater Code and Section 2.4. **This section is not comprehensive, and designers must refer to the detailed requirements in the Stormwater Code to confirm all applicable requirements are met.**

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2.5.1 Summary of Key Minimum Requirements for Flow Control

The performance standards applicable to the key Minimum Requirements for Flow Control are summarized below. The applicability of each minimum requirement depends upon the project type, size, and receiving water as summarized in Section 2.5.3.

- **Flow Control** Minimum Requirement #1 (FC#1) – Implement Green Stormwater Infrastructure. Install and maintain green stormwater infrastructure to the maximum extent feasible. Green stormwater infrastructure BMPs are detailed in Section 4.4.
- **Flow Control** Minimum Requirement #2 (FC#2) – Wetland Protection. Protect the functions and values of the wetland.
- **Flow Control** Minimum Requirement #3 (FC#3) – Pre-developed Forest. Match the post-development discharge flow rates and durations to a pre-developed forest condition for the range of predeveloped discharge rates from 50 percent 2-year recurrence interval flow up to the 50-year recurrence interval flow.
- **Flow Control** Minimum Requirement #4 (FC#4) – Pre-developed Pasture. Match the post-development discharge flow rates and durations to a pre-developed pasture condition for the range of predeveloped discharge rates from 50 percent of 2-year recurrence interval flow up to the 2-year recurrence interval flow.
- **Flow Control** Minimum Requirement #5 (FC#5) – Peak Flow Control. The post-development 25-year recurrence interval flow shall not exceed 0.4 cubic feet per second per acre; and the 2-year recurrence interval flows shall not exceed 0.15 cubic feet per second per acre.

When triggered, flow control facilities must be installed to manage flows from the impervious surfaces and converted pervious surfaces on the site being developed. Post development discharge determination must include flows from dewatering activities. When offsite flows cannot feasibly bypass proposed flow control facilities, the flow control facilities shall be sized for the combined total of onsite and offsite flows with the allowable discharge rates determined by the onsite runoff calculations (see Section 4.2.4). All projects shall use green stormwater infrastructure the maximum extent feasible to meet the minimum requirements.

SPECIAL NOTES:

➢➢➢ If a project requires FC#5 and either FC#3 or FC#4, the facility shall be sized to the largest applicable size (i.e., to meet the more stringent of the requirements).

➢➢➢ Stormwater flow control and treatment facilities are not required if the site produces no stormwater runoff discharge as determined by a licensed civil engineer using an approved continuous runoff model.

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2.5.2 Summary of Key Minimum Requirements for Treatment

The performance standards applicable to the key Minimum Requirements for Treatment are summarized below. The applicability of each minimum requirement depends upon the project type, size, and receiving water as summarized in Section 2.5.3.

- **Treatment** Minimum Requirement #1 (WQ#1) – Basic Treatment. Install and maintain a basic water quality treatment facility. The requirements for determining the applicable water quality treatment volume and/or rate are presented in Section 2.4.7, with additional modeling requirements and guidance presented in Chapter 6. If the requirement for basic treatment applies to a project, all other treatment minimum requirements (WQ#2, WQ#3 and WQ#4) must be evaluated to determine if they are applicable.
- **Treatment** Minimum Requirement #2 (WQ#2) – Oil Control. Install and maintain an oil control treatment facility for high-use sites.
- **Treatment** Minimum Requirement #3 (WQ#3) – Phosphorus Treatment. Install and maintain a phosphorus treatment facility for projects discharging into nutrient-critical receiving waters.
- **Treatment** Minimum Requirement #4 (WQ#4) – Enhanced Treatment. Install and maintain an enhanced treatment facility.

When triggered, water quality treatment facilities must be installed to treat flows from the pollution generating pervious and impervious surfaces on the site being developed. When stormwater flows from other areas, including non-pollution generating surfaces (e.g., roofs), dewatering activities, and offsite areas cannot be separated or bypassed, treatment BMPs shall be designed for the entire area draining to the treatment facility. All projects shall use green stormwater infrastructure the maximum extent feasible to meet the minimum requirements.

SPECIAL NOTES:

➢➢➢ Stormwater flow control and treatment facilities are not required if the site produces no stormwater runoff discharge as determined by a licensed civil engineer using an approved continuous runoff model.

2.5.3 Summary of Minimum Requirement Applicability

Figures 2.8 through 2.11 summarize the key minimum requirements for flow control and treatment for five main types of projects:

- Single-Family Residential Projects (Figure 2.8)
- Sidewalk and Trail-Only Projects (Figure 2.8)
- Parcel-Based Projects (Figure 2.8, 2.9, and 2.11)
- Roadway Projects (Figure 2.8, 2.10, and 2.11)
- Joint Projects (Figure 2.8, 2.9, 2.10, and 2.11)

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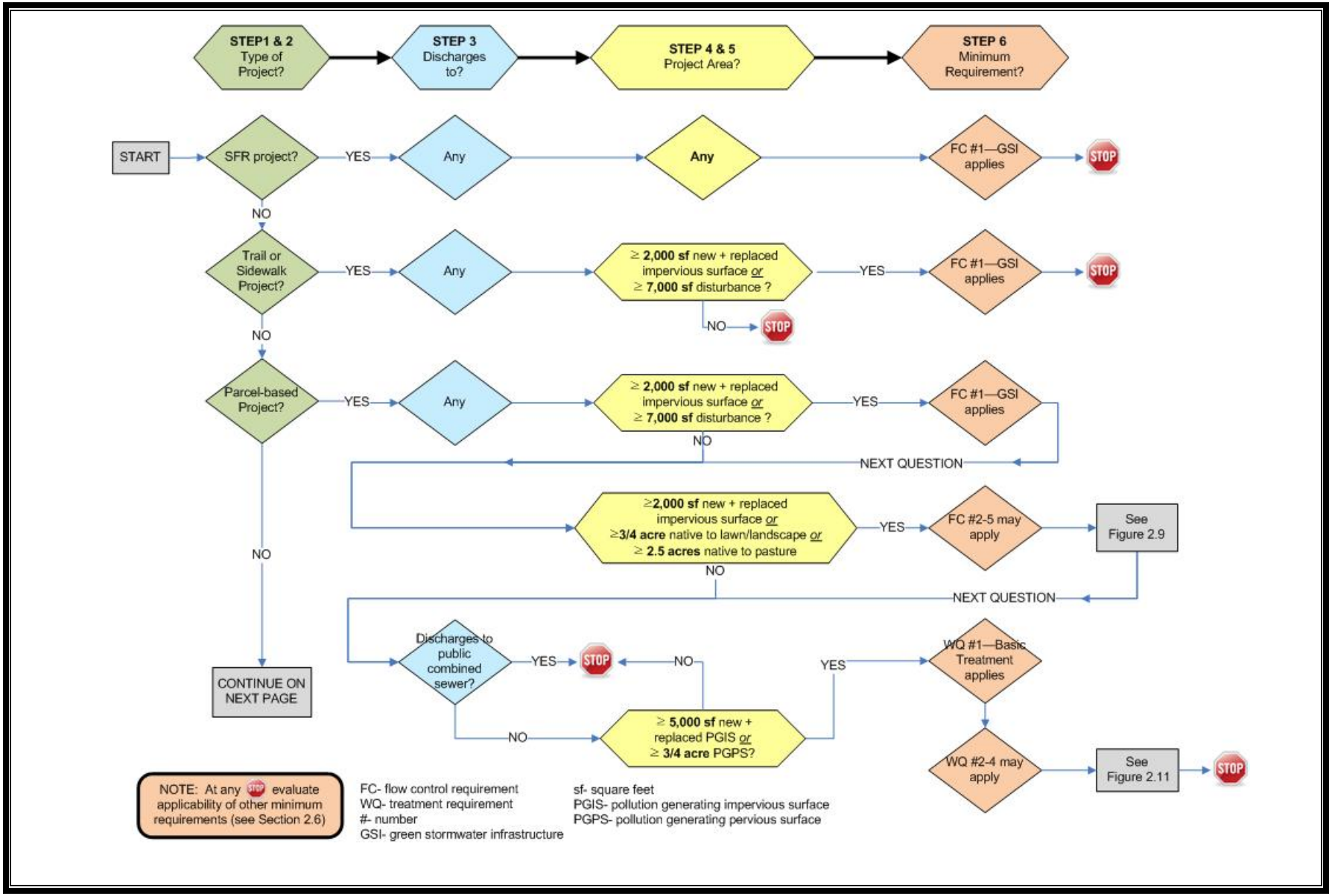

Figure 2.8. Minimum Requirement Determination Overview Flow Chart

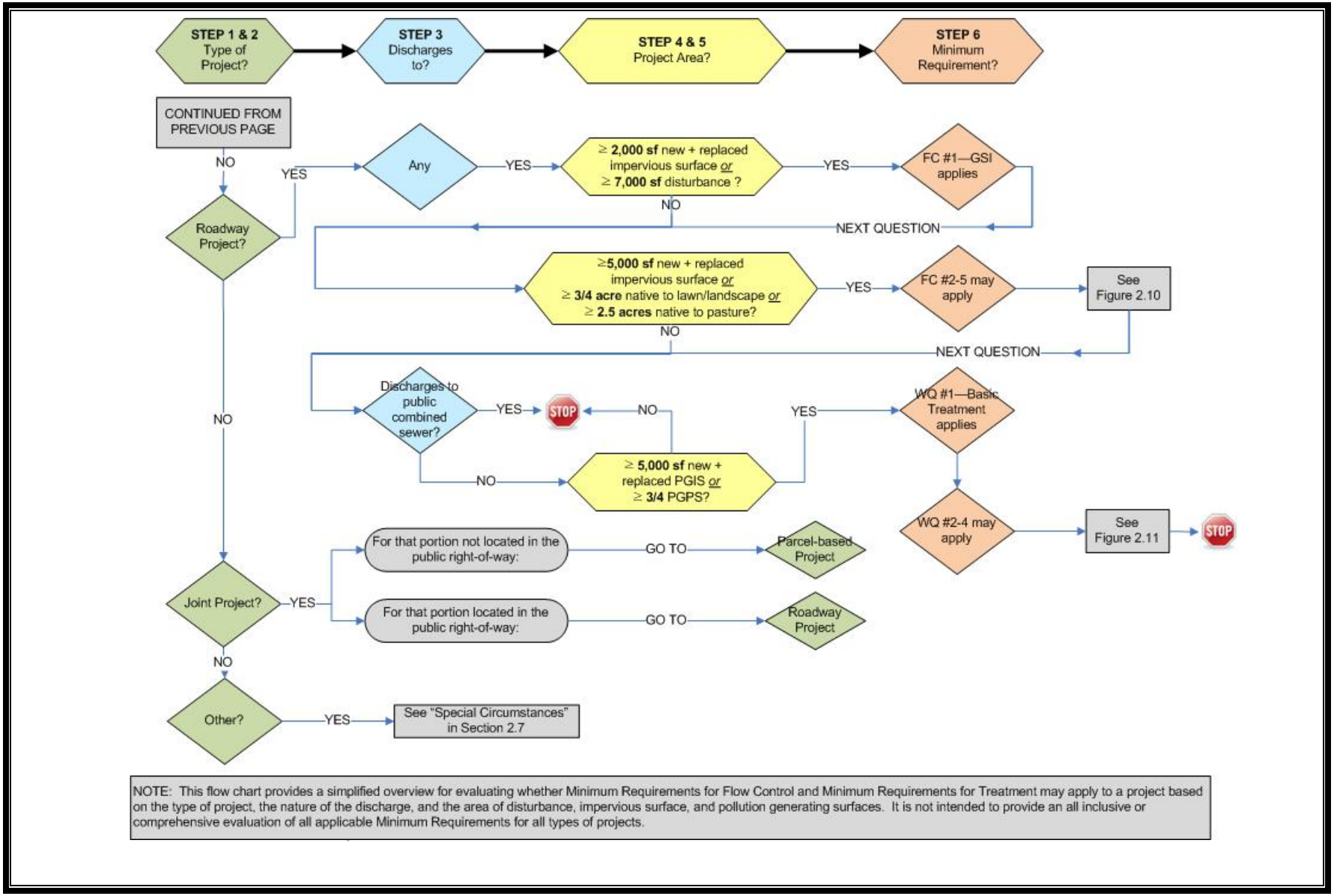

Figure 2.8 (continued). Minimum Requirement Determination Overview Flow Chart.

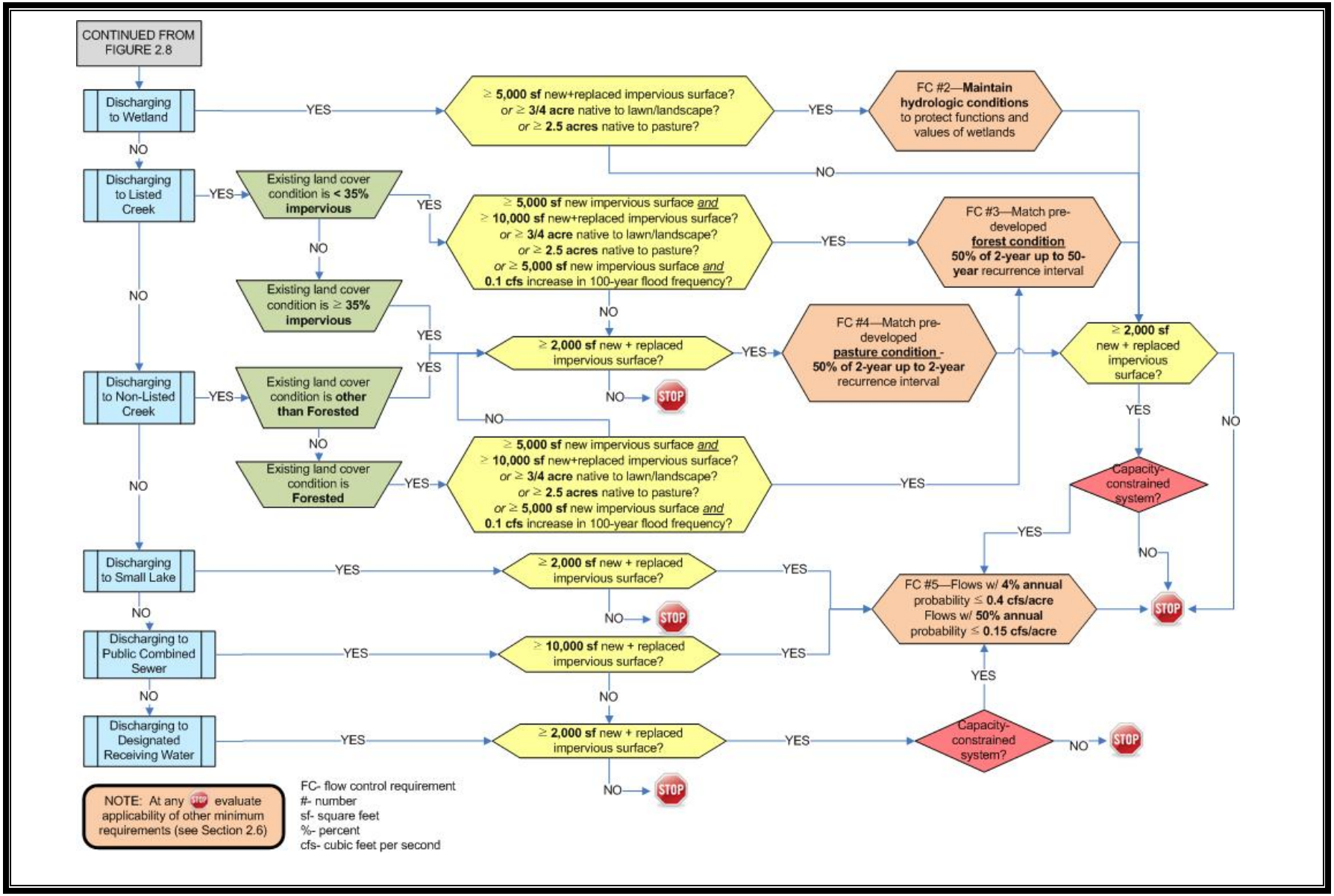

Figure 2.9. Applicability of FC#2-FC#5 to Parcel-Based Projects.

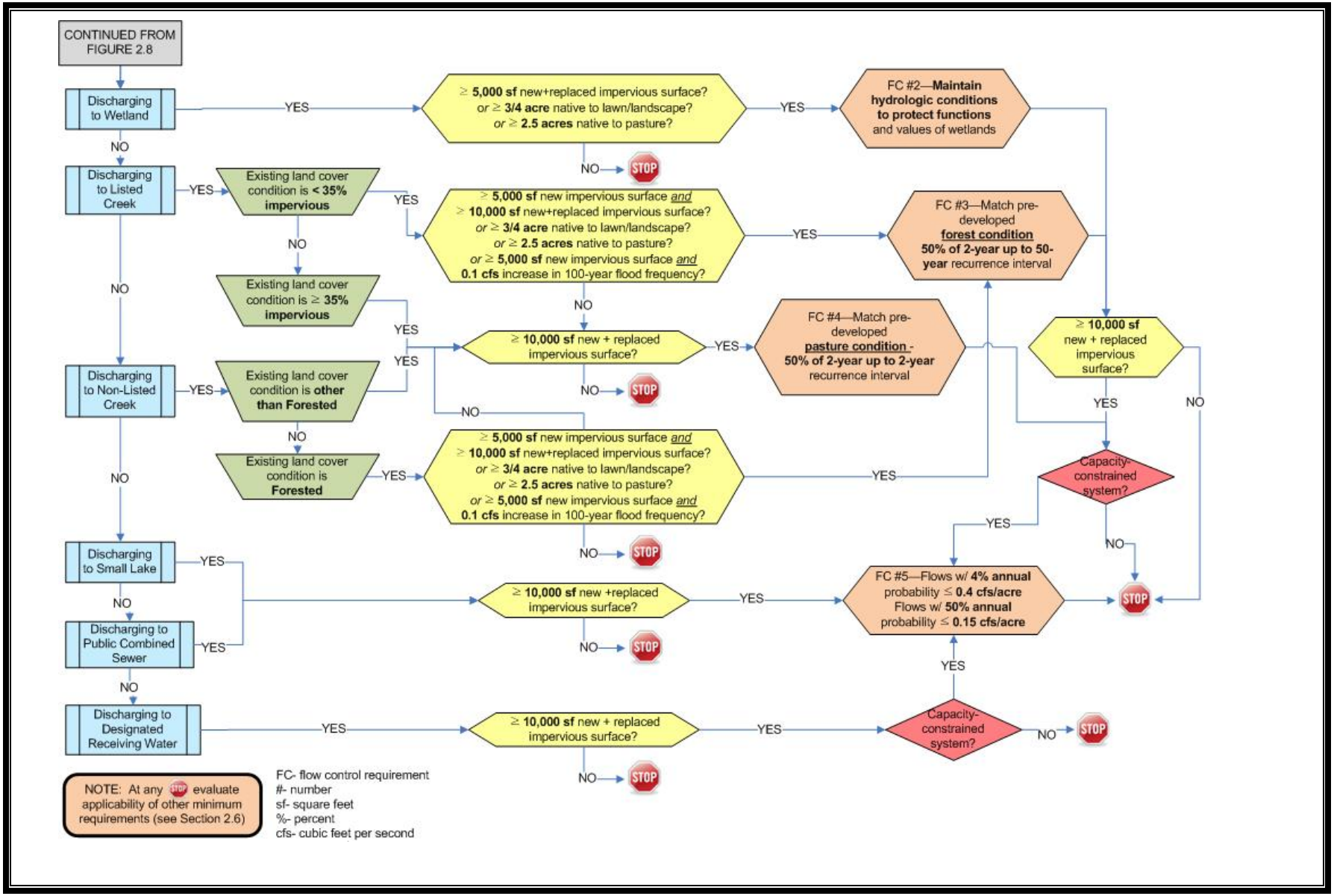

Figure 2.10. Applicability of FC#2-FC#5 to Roadway Projects.

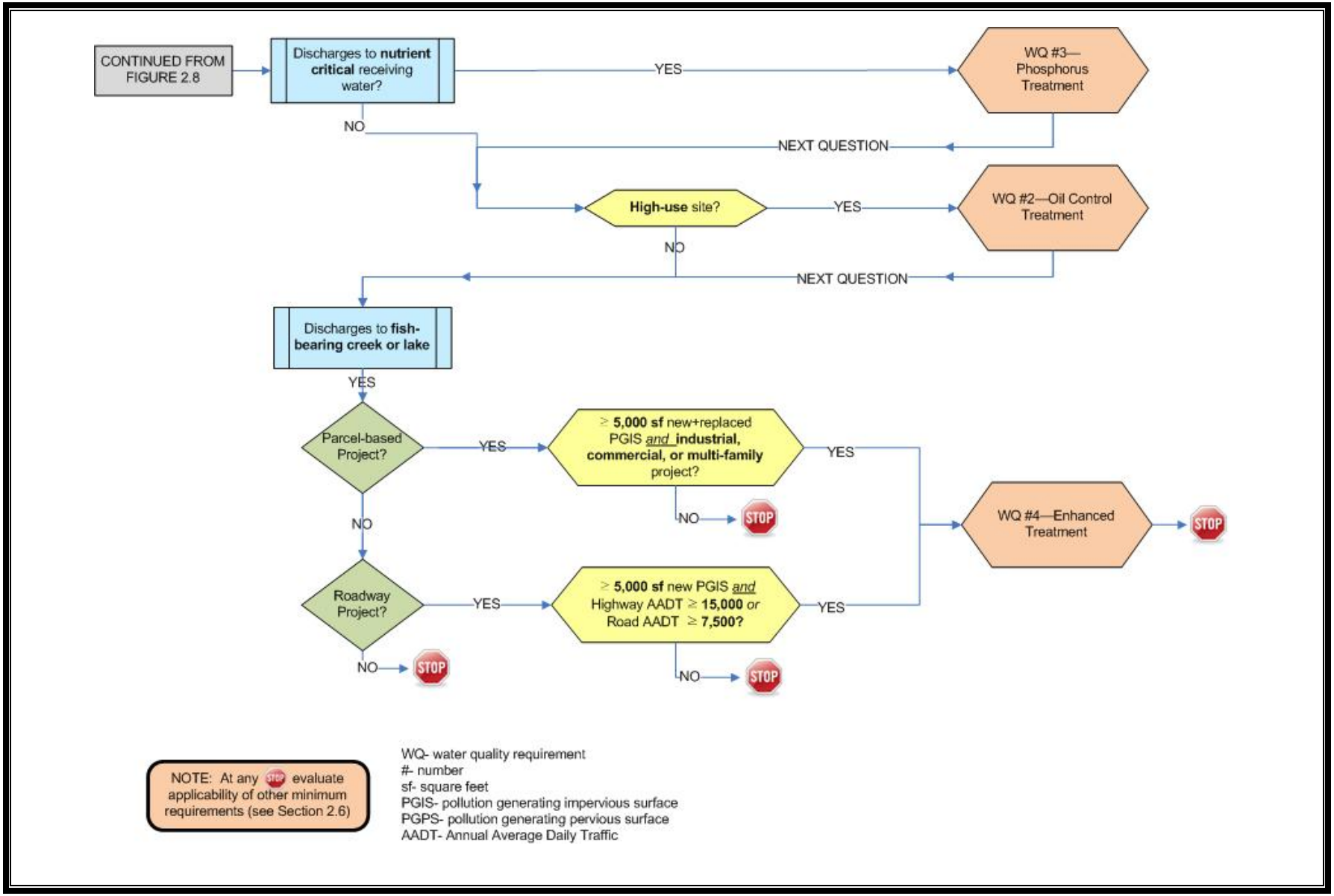

Figure 2.11. Applicability of WQ#2-WQ#4 to Parcel-Based and Roadway Projects.

For each of these types of projects, the minimum requirements for flow control and treatment are a function of the following factors:

- The type of drainage basin and receiving water
- Whether the drainage system is capacity-constrained
- The amount of new plus replaced impervious surface
- The amount of converted pervious surface
- The amount of new plus replaced pollutant-generating impervious surface
- The amount of new plus replaced pollutant-generating pervious surface

2.6 Other Minimum Requirements that May Apply

All projects are required to comply with the minimum requirements listed in Section 22.805, even where drainage control review is not required. The specifics of the minimum requirement applicable to all projects, as per Section 22.805.020 are as follows:

2.6.1 Minimum Requirements for Maintaining Natural Drainage Patterns (SMC 22.805.020.A)

For all projects, natural drainage patterns shall be maintained and discharges shall occur at the natural location to the maximum extent feasible and consistent with subsection 22.805.020.B. Drainage water discharged from the site shall not cause a significant adverse impact to receiving waters or down-gradient properties. Drainage water retained on the site shall not cause significant adverse impact to up-gradient properties.

2.6.2 Minimum Requirements for Discharge Point (SMC 22.805.020.B)

The discharge point for drainage water from each site shall be selected using criteria that shall include, but not be limited to, preservation of natural drainage patterns and whether the capacity of the drainage system is adequate for the flow rate and volume. For those projects meeting the drainage review threshold, the proposed discharge point shall be identified in the drainage control plan required by this subtitle, for review and approval or disapproval by the Director.

2.6.3 Minimum Requirements for Flood-prone Areas (SMC 22.805.020.C)

On sites within flood prone areas, responsible parties are required to employ procedures to minimize the potential for flooding on the site and to minimize the potential for the project to increase the risk of floods on adjacent or nearby properties. Flood control measures shall include those set forth in other titles of the Seattle Municipal Code and rules promulgated thereunder, including, but not limited to, Chapter 23.60 (Shoreline Master Program), Chapter 25.06 (Floodplain

Development) and Chapter 25.09 (Environmentally Critical Areas) of the Seattle Municipal Code.

2.6.4 Minimum Requirements for Construction Site Stormwater Pollution Prevention Control (SMC 22.805.020.D)

Temporary and permanent construction controls shall be used to accomplish [the 18 construction site stormwater pollution prevention control requirements outlined in SMC 22.805.020.D and Volume 2, the Construction Stormwater Control Technical Requirements Manual]. All projects are required to meet each of the elements or document why an element is not applicable. Additional controls may be required by the Director when minimum controls are not sufficient to prevent erosion or transport of sediment or other pollutants from the site.

See SMC 22.805.020.D for the 18 construction site stormwater pollution prevention control requirements and Volume 2, the Construction Stormwater Control Technical Requirements Manual for additional information on construction stormwater control.

2.6.5 Minimum Requirement to Amend Soils (SMC 22.805.020.E)

Prior to completion of the project all new, replaced, and disturbed topsoil shall be amended with organic matter per rules promulgated by the Director to improve onsite management of drainage water flow and water quality.

2.6.6 Implement Green Stormwater Infrastructure (SMC 22.805.020.F)

All Single-family residential projects and all other projects with 7,000 square feet or more of land disturbing activity or 2,000 square feet or more of new plus replaced impervious surface must implement green stormwater infrastructure to infiltrate, disperse, and retain drainage water onsite to the maximum extent feasible without causing flooding, landslide, or erosion impacts.

2.6.7 Protect Wetlands (SMC 22.805.020.G)

All projects discharging into a wetland or its buffer, either directly or indirectly through a conveyance system, shall prevent impacts to wetlands that would result in a net loss of functions or values.

2.6.8 Protect Streams and Creeks (SMC 22.805.020.H)

All projects, including projects discharging directly to a stream or creek, or to a drainage system that discharges to a stream or creek, shall maintain the water quality in any affected stream or creek by selecting, designing, installing, and maintaining temporary and permanent controls.

2.6.9 Protect Shorelines (SMC 22.805.020.I)

All projects discharging directly or indirectly through a drainage system into the shoreline district as defined in Chapter 23.60 shall prevent impacts to water quality and stormwater quantity that would result in a net loss of shoreline ecological functions as defined in WAC 173-26-020 (11).

2.6.10 Ensure Sufficient Capacity (SMC 22.805.020.J)

All large projects, all projects with an excavation depth of 12 feet or more below the existing grade, and all projects with an excavation depth of less than 12 feet located in an area expected to have shallow groundwater depths shall ensure that sufficient capacity exists in the public drainage system and public combined sewer to carry existing and anticipated loads, including any flows from dewatering activities. Capacity analysis shall extend to at least 1/4-mile from the discharge point of the site. Sites at which there is insufficient capacity may be required to install a flow control facility or improve the drainage system or public combined sewer to accommodate flow from the site. Unless approved otherwise by the Director as necessary to meet the purposes of this subtitle:

1. Capacity analysis for discharges to the public drainage system shall be based on peak flows with a 4% annual probability (25-year recurrence interval); and

2. Capacity analysis for discharges to the public combined sewer shall be based on peak flows with a 20% annual probability (5-year recurrence interval).

2.6.11 Install Source Control BMPs (SMC 22.805.020.K)

Source control BMPs shall be installed for specific pollution-generating activities as specified in the joint SPU/DPD Directors' Rule, "Source Control Technical Requirements Manual," to the extent necessary to prevent prohibited discharges as described in Section 22.802.020, and to prevent contaminants from coming in contact with drainage water. This requirement applies to the pollution-generating activities that are stationary or occur in one primary location and to the portion of the site being developed. Examples of installed source controls include, but are not be limited to, the following:

1. A roof, awning, or cover erected over the pollution-generating activity area;

2. Ground surface treatment in the pollution-generating activity area to prevent interaction with, or breakdown of, materials used in conjunction with the pollution-generating activity;

3. Containment of drainage from the pollution-generating activity to a closed sump or tank. Contents of such a sump or tank must be pumped or

hauled by a waste handler, or treated prior to discharge to a public drainage system.

4. Construct a berm or dike to enclose or contain the pollution-generating activities;

5. Direct drainage from containment area of pollution-generating activity to a closed sump or tank for settling and appropriate disposal, or treat prior to discharging to a public drainage system;

6. Pave, treat, or cover the containment area of pollution-generating activities with materials that will not interact with or break down in the presence of other materials used in conjunction with the pollution-generating activity; and

7. Prevent precipitation from flowing or being blown onto containment areas of pollution-generating activities.

2.6.12 Do Not Obstruct Watercourses (SMC 22.805.020.L)

Watercourses shall not be obstructed.

2.6.13 Comply with Side Sewer Code (SMC 22.805.020.M)

1. All privately owned and operated drainage control facilities or systems, whether or not they discharge to a public drainage system, shall be considered side sewers and subject to Chapter 21.16 (Side Sewer Code), SPU Director's Rules promulgated under Title 21, and the design and installation specifications and permit requirements of SPU and DPD for side sewer and drainage systems.

2. Side sewer permits and inspections shall be required for constructing, capping, altering, or repairing privately owned and operated drainage systems as provided for in Chapter 21.16. When the work is ready for inspection, the permittee shall notify the Director of DPD. If the work is not constructed according to the plans approved under this subtitle, Chapter 21.16, the SPU Director's Rules promulgated under Title 21, and SPU and DPD design and installation specifications, then SPU, after consulting with DPD, may issue a stop work order under Chapter 22.808 and require modifications as provided for in this subtitle and Chapter 21.16.

2.7 Special Circumstances

Certain projects do not closely fit into the descriptions of the various project types, and therefore require case-by-case review regarding the applicable minimum requirements. These projects must be routed first through a pre-permit review process established by DPD to assist the proponet in identifying the specific minimum requirements to be applied. The following list is not comprehensive, but gives the proponent an indication of the complexity of the

special circumstances. Examples of projects that involve the elements that trigger special circumstances review include:

- Bridges or tunnels
- Construction over water
- Close-contour basins
- Permanent dewatering
- Draining into more than one basin
- Multiple blocks or a subdivision
- Alleys
- Railroads
- Work in one or more jurisdictions.

2.8 Alternative Compliance

In SMC 22.800.080, the Director of SPU is authorized, to the extent allowed by law:

> To develop, review, or approve an Integrated Drainage Plan as an equivalent means of complying with the requirements of this subtitle, in which the developer of a project voluntarily enters into an agreement with the Director of SPU to implement an Integrated Drainage Plan that is specific to one or more sites where best management practices are employed such that the cumulative effect on the discharge from the site(s) to the same receiving water is the same or better than that which would be achieved by a less integrated, site-by-site implementation of best management practices. (SMC 22.800.080.E)
>
> To enter into an agreement with the developer of a project for the developer to voluntarily contribute funds toward the construction of one or more drainage control facilities that mitigate the impacts to the same receiving water that have been identified as a consequence of the proposed development. (SMC 22.800.080.F)
>
> To enter into an agreement with the developer of a project for the developer to voluntarily construct one or more drainage control facilities at an alternative location, determined by the Director, to mitigate the impacts to the same receiving water that have been identified as a consequence of the proposed development. (SMC 22.800.080.G)

When the consequences of the proposed development are from new impervious surfaces, the mitigation should be provided at the same time as completion of the new surfaces. When the consequences of the proposed development are from replaced impervious surfaces, there should be a plan and a schedule for

construction such that the drainage control facility or facilities mitigating the impacts are constructed in a timely manner, and in all cases within 5 years of the original development.

2.9 Next Steps

After completing the steps outlined in this chapter, the project proponent will have determined which minimum requirements for flow control and treatment apply to the project. The next step is to review **Chapter 3** to ensure a detailed site assessment has been performed. Site specific conditions will affect the selection and design of stormwater BMPs. Chapter 3 also provides information on submittal requirements, which should be kept in mind throughout the site assessment and facility design process.

After completing the site assessment, **Chapters 4 and 5** should be reviewed for details on stormwater flow control and water quality treatment facility selection and design, respectively. Section 4.4 in particular provides design requirements specific to green stormwater infrastructure. **Chapter 6** provides guidelines and background on stormwater modeling applicable to site assessment and facility design.

2.10 References

Ecology. 2005. Stormwater Management Manual for Western Washington. 5 vols. Ecology Publications 05-10-029 through 05-10-033. Washington State Department of Ecology, Water Quality Program. February 2005.

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Chapter 3 - Site Planning, Site Assessment, and Drainage Control Review

3.1 Design Considerations

The first step in addressing stormwater management begins with the site planning and design process. In order to achieve effective and efficient stormwater management, site design for both the construction phase and post-development condition should be done in unison with the design and layout of the stormwater infrastructure. When carefully planned efforts are made to conserve natural areas, reduce impervious surfaces, and better integrate stormwater controls into the existing site layout, impacts from development projects as well as water quality and flow control costs can be reduced. Prior to designing the site, the following should be considered.

Stormwater considerations include:

- Manage stormwater runoff (quantity and quality) as close to the point of origin as possible
- Minimize the quantity of stormwater collection and conveyance systems required
- Utilize simple, nonstructural methods for stormwater management, which often reduce cost and maintenance relative to structural controls.

Green stormwater infrastructure considerations include:

- Utilize green stormwater infrastructure (e.g., infiltration, evapotranspiration, and stormwater reuse-based facilities) to the maximum extent feasible (see Section 4.4).

Natural landscape considerations include:

- Maintain and utilize natural drainage patterns
- Preserve natural features and resources, including trees
- Create a multifunctional landscape using hydrology as a framework for site design
- Design the project to accommodate and utilize existing terrain and minimize land disturbance
- Confine and phase construction activities to minimize disturbed areas, and minimize impacts to environmentally critical areas and their associated buffers
- Minimize clearing activities in areas with existing tree canopy or established vegetation.

Impervious and pervious surfaces considerations include:

- For sites with varied soil types, locate impervious areas over less permeable soil (e.g., till), and try to restrict development where porous soils exist (e.g., outwash)
- Minimize impervious surfaces (e.g., buildings, sidewalks)
- Minimize pollution generating impervious surfaces (areas subject to vehicular use, e.g., driveways and parking strips)
- Minimize pollution generating pervious surfaces (e.g., fertilized lawns).

3.2 Site Assessment and Planning

3.2.1 Overview

Each project must assess and evaluate existing and post-development site conditions to ensure the project will comply with all applicable code requirements. This chapter provides additional guidance on collecting the information needed to effectively evaluate the minimum requirements, reduce the project's drainage costs and impacts, and start the process for selecting flow control and water quality treatment BMPs, as needed for the post-development project conditions. Minimum requirements contained in the Stormwater Code pertaining to flow control and water quality treatment are presented in Chapter 2.

3.2.2 Identifying Key Project Components

The following project components must be addressed to identify applicable stormwater flow control and water quality treatment requirements from the Stormwater Code, and to appropriately plan for the related aspects of the project.

3.2.2.1 Project Description

Project Type: Determine the type of project proposed for the development. Refer to Chapter 2 for additional information.

- Single-family residence (SFR)
- Parcel-Based
- Roadway Project
- Joint Roadway and Parcel-Based
- Sidewalk or Trail only.

Existing and Proposed Uses: The following lists existing and proposed areas of the site that must be identified in order to determine applicable water quality treatment and source control requirements.

For the following items, refer to the glossary (Appendix A) and Chapter 5 for more information:

- Surfaces subject to vehicular use
- Parking areas and the associated Average Daily Traffic (ADT) count
- Parking for vehicles over 10 tons gross weight
- Areas of petroleum storage and transfer
- Roadway areas and the associated Average Annual Daily Traffic (AADT) count.

For the following items, refer to Volume 1, the Source Control Technical Requirements Manual for more information and required BMPs:

- Pollution Generating Activities
- Areas of Outdoor Storage.

3.2.2.2 Impervious Surfaces

New and Replaced Impervious Surface

- The total amount of new and replaced impervious surface must be identified, evaluated, and potentially mitigated.

New and Replaced Pollution Generating Impervious Surface

- The total amount of new and replaced pollution generating impervious surface must be identified, evaluated, and potentially mitigated.

3.2.2.3 Locations of Boundaries and Structures

Project Boundaries

- For most SFR and parcel-based projects, the project boundaries coincide with the right-of-way and property line.

Setbacks

- In order to evaluate the impacts the project may have on adjacent properties; property lines, existing and proposed structures, and adjacent right-of-way boundaries must be identified and considered.

Location of Buildings

- All existing and proposed buildings must be identified, including all existing and proposed temporary and permanent structures (such as retaining walls) and impervious surfaces (driveways, patios, etc.).

Proposed Foundation Type

- The type of foundations, location, and extent, proposed for the project must be determined.
  - Conventional spread footings
  - Pile shaft
  - Basement
  - Footing drains and their associated point of discharge, if applicable
  - Water-tight foundation without footing drains.

3.2.2.4 Soil Condition Assessment

The soil type and land cover types on the project must be considered in order to assess the infiltration capacity of the site and the applicability of various BMPs. General requirements for infiltration facilities, including site characterization and infiltration rate determination are presented in Section 4.3.

3.2.2.5 Soil Management Plan

A Soil Management Plan is required to show how the post construction soil quality and depth requirements will be met (see Section 4.4.1).

3.2.2.6 Environmentally Critical Areas (ECA)

Additional regulatory requirements are placed upon projects that are within or near ECAs, pursuant to SMC 25.09. Depending upon the type of ECA, additional requirements or limitations regarding stormwater management may apply.

3.2.2.7 Dewatering

Temporary and permanent dewatering needs need to be identified early on in the project in order to prevent delays in construction, and to determine the approved stormwater point of discharge.

3.2.2.8 Topography

Because topography will influence how and where stormwater facilities are incorporated onto the site, the existing and proposed topography must be considered. Important features to assess include the following:

- Key terrain features, such as closed depressions and grade breaks
- Natural drainage courses, such as swales, ditches, rills, and gullies
- Off-site flow entering and exiting the property
- Roadway grades and elevations.

3.2.2.9 Point of Discharge

The drainage discharge location for the project site will determine applicable minimum requirements for flow control and treatment based upon the basin in which the project is located. Flow control and water quality treatment requirements differ for the five basic types of drainage basins identified below. Additionally, flow control criteria vary when the downstream system is capacity-constrained. See Chapter 2 for additional details.

- Wetlands
- Creek Basins
- Combined Sewer
- Small Lakes
- Designated Receiving Waters.

3.3 Drainage Control Review for SFR and Parcel-Based Projects

There are two levels of Drainage Control Review required when applying for construction permits from DPD: Standard Drainage Review, and Comprehensive Drainage Review. The type of Drainage Control Review required is based on the size of the development project and the total amount of new plus replaced impervious surface. For joint projects, the total impervious surface must be counted.

3.3.1 Standard Drainage Review

Standard Drainage Review generally applies to small projects. These projects involve 750 square feet or more of land disturbing activity and less than 5,000 square feet of new plus replaced impervious surface. Additional criteria for Standard Drainage Review are contained in the Stormwater Code (see SMC 22.807).

All submittals that require Standard Drainage Review shall include the following:

- Stormwater Control Plan (see Volume 2)
- DPD Screening Standards Checklists for Grading, Drainage, and Stormwater
- Technical Information Report (TIR) to provide a standard format for presenting the information required
- Site Plan
- Soil Amendment Plan for Landscaped Areas
- Flow Control documentation, if triggered. Required documentation may include:

- Pre-Sized Approach Worksheet or modeling documentation including Hydrostats summary (see below).
- Geotechnical report and/or Pilot Infiltration Test (PIT) for infiltration facilities, if required.
- BMP operations and maintenance plan.

3.3.2 Comprehensive Drainage Review

Comprehensive Drainage Review is required for large projects. These projects involve 5,000 square feet or more of new plus replaced impervious surface or over 1 acre of land disturbing activity. In addition to the requirements of the Standard Drainage Review, the following information is required for Comprehensive Drainage Review:

- A Drainage Report: including, at a minimum:
  - A narrative detailing the proposed project, and how stormwater will be mitigated
  - HydroStats summary of continuous model results for flow control and treatment, if required. (Note: HydroStats is a statistical post-processor that reads output from approved continuous simulation hydrologic models and creates a standardized project report for design review submittal. See Section 6.2.3)
  - Calculations for sizing water quality and flow control facilities
  - Drainage basin map.
  - Insprection and maintenance schedule.
  - Memorandum of Drainage Control.

The following additional information *may be* required for all Drainage Review if deemed necessary to fully evaluate a project:

- Soils Analysis / Geotechnical Report
- Topographic / Boundary Survey
- Environmental Assessment for potentially contaminated sites
- Off-site or Downstream Analysis.

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Chapter 4 - Flow Control Design

4.1 Introduction

The focus of this chapter is on permanent flow control best management practices (BMPs).

4.1.1 Organization

This chapter provides specific design criteria for flow control BMPs and is divided into six major sections:

- Section 4.1 serves as an introduction and summarizes available options for stormwater flow control
- Section 4.2 outlines a step-by-step process for selecting flow control facilities for new development and redevelopment projects
- Section 4.3 discusses general requirements for infiltration facilities
- Section 4.4 includes detailed information regarding the following green infrastructure BMPs:
  - Post Construction Soil Quality and Depth
  - Tree Planting and Retention
  - Downspout Dispersion
  - Sheet Flow Dispersion
  - Bioretention
  - Rainwater Harvesting
  - Permeable Pavement
  - Green Roofs.
- Section 4.5 includes detailed information regarding the following infiltration flow control BMPs (note that although infiltration basins, trenches, and drywells utilize infiltration for flow control, they are not considered green stormwater infrastructure in this manual):
  - Infiltration basins
  - Infiltration trenches
  - Drywells.
- Section 4.6 includes detailed information regarding the following detention flow control BMPs:
  - Detention ponds
  - Detention pipes

- Detention vaults
- Detention cisterns
- Other detention options (surface detention on roofs and parking areas)
- Control structures for most non-infiltrating flow control facilities.

For each flow control BMP outlined above, detailed technical information is organized as follows:

- ***Site Considerations*** identifies the limitations associated with siting each flow control BMP. The application of a BMP may be constrained by factors such as approximate footprint, groundwater elevation, soil characteristics, and other site conditions. The intention is to allow the designer to quickly evaluate whether a proposed BMP might be suitable for a particular site or project.
- ***Design Criteria*** provides descriptions and specifications for BMP components and materials such as geotextile fabric underdrains, and soil amendments.
- ***BMP Sizing*** presents sizing requirements and modeling procedures for each BMP. General modeling guidance is provided in Chapter 6, Hydrologic Analysis.
- ***Construction Specifications and Criteria*** describe additional design requirements to be considered during construction of the BMP, such as landscape stabilization, phasing and flow bypass.

Note that per SMC 22.803.020 Sections C and E and 22.805.080.C, all flow control facilities shall be maintained per the requirements outlined in this manual. Detailed maintenance requirements for each BMP are provided in Appendix D.

4.1.2 Relevant Information from Other Chapters

Chapter 2 describes the minimum requirements contained in the Stormwater Code, including flow control standards and where they apply.

Chapter 3 summarizes site planning, site assessment and drainage control review procedures.

Chapter 5 details selection procedures and design requirements for runoff treatment BMPs.

Chapter 6 provides detailed guidance for hydrologic analysis.

The appendices referenced in this chapter and volume contain more detailed information on selected topics described in the various sections. Appendices are located at the end of the volume.

4.1.3 Summary of Minimum Requirements for Flow Control

The Minimum Requirements for Flow Control are presented in Chapter 2 of this manual. This section provides a brief summary of the requirements. Designers must refer to the detailed requirements in the Stormwater Code and Chapter 2 to confirm all applicable requirements are met.

All single-family residential projects and all other projects with 7,000 square feet or more of land disturbing activity or 2,000 square feet or more of new plus replaced impervious surface must implement green stormwater infrastructure (SMC 22.801.080) to infiltrate, disperse, and retain drainage water onsite to the maximum extent feasible (Implement Green Stormwater Infrastructure FC#1). This means that green stormwater infrastructure BMPs must be incorporated throughout the project site wherever feasible, constrained only by the physical limitations of the site, practical considerations of engineering design and reasonable considerations of financial costs and environmental impacts (SMC 22.801.140).

For projects that are subject to this requirement, but not to the numerical performance standards listed below, guidance is provided in seperate Director's Rules. Guidance for single family residential projects, sidewalk, trail and roadway projects, and parcel-based projects, are provided in Director's Rules DR-2009-008, DR-2009-007, and DR-2009-00X, respectively.

In addition to FC#1, many scenarios also require that numerical flow control standards (i.e., peak and/or duration standard) be met:

- Wetland Protection (FC#2): functions and values of the affected wetlands shall be protected.
- Pre-developed Forest (FC#3): post-development discharges shall match peak flow rates and durations from half of the 50 percent annual probability (2-year recurrence) flow to the 2 percent annual probability (50-year recurrence) flow to a predeveloped forest condition.
- Pre-developed Pasture (FC#4): post-development discharges shall match peak flow rates and durations from half of the 50 percent annual probability (2-year recurrence) flow to the 50 percent annual probability (2-year recurrence) flow to a predeveloped pasture condition.
- Peak Flow Control (FC#5): the post-development peak discharge with a 4 percent annual probability (25-year recurrence) flow shall not exceed 0.4 cubic feet per second per acre; and peak discharge with a 50 percent annual probability (2-year recurrence) flow shall not exceed 0.15 cubic feet per second per acre.

When numerical performance standards are triggered, green stormwater infrastructure must be implemented to the maximum extent feasible to meet the applicable performance standards. If two standards apply (e.g., the

Pre-developed Pasture standard and the Peak Control standard might apply for a capacity constrained system in a non-listed creek basin) both standards must be met. Additional information about the flow control standards and where they apply can be found in Chapter 2.

In addition to FC#1 through FC#5, post construction soil quality and depth requirements must be achieved for all projects (see Section 4.4.1).

4.1.4 Flow Control Facility Categories

All flow control BMPs fall within at least one of three categories, summarized in the subsequent sections:

- Infiltration
- Detention
- Dispersion.

In addition, several flow control BMPs are classified as "green stormwater infrastructure."

4.1.4.1 Infiltration

Infiltration facilities are designed to facilitate percolation of surface water into the ground. Examples of infiltration facilities include: infiltration basins, infiltration trenches, and drywells. Many green stormwater infrastructure BMPs are also infiltration facilities, including unlined bioretention cells and permeable pavement facilities. Infiltration is feasible only where sufficiently porous soils are available, and where other site constraints are not limiting (e.g., steep slopes, high groundwater). When properly sited and designed, infiltration facilities can help decrease stormwater runoff, recharge groundwater, and protect downstream receiving waters.

4.1.4.2 Detention

The concept of detention is to collect and temporarily store runoff, and then release it over a period of time at a reduced rate. In Seattle, detention facilities generally store the runoff in an underground pipe, vault, or tank. These facilities have an outlet control structure designed to release flows at an attenuated rate that meets flow control performance standards. The green stormwater infrastructure practice of rainwater harvesting, which involves storage of runoff in cisterns for later use onsite, may be combined with stormwater detention.

Of particular note for detention facilities is that the minimum allowable orifice diameter is 0.5 inches for belowground detention systems and 0.25 inches for aboveground detention systems. In some instances, particularly for smaller sites, the minimum bottom orifice diameter will be too large to meet standard

release rates. In these cases, the live storage depth need not be reduced to less than 3 feet in an attempt to meet the flow control standards. Additional guidance for this scenario is provided in Section 4.6.1.

4.1.4.3 Dispersion

Dispersion is a simple method of flow control that can be used for impervious or pervious surfaces that are graded to avoid concentrating flows. Dispersion uses adjacent vegetation, soils, and topography to distribute stormwater runoff to promote infiltration and to generally slow the flows by preventing concentration of stormwater runoff. Site selection is very important to the success of dispersion BMPs, and the site constraints in urban settings often make it infeasible. Dispersion methods are considered green stormwater infrastructure BMPs.

4.1.4.4 Green Stormwater Infrastructure

Green stormwater infrastructure BMPs include facilities that utilize infiltration, dispersion, and/or detention. In order for a BMP to be considered green stormwater infrastructure, it must provide a function in addition to stormwater management such as water reuse or providing greenspace and/or habitat in the City. Green stormwater infrastructure can be used to comply with the Minimum Requirements for Flow Control, Minimum Requirements for Treatment, or both, depending on how they are designed and constructed. Table 4.1 provides a list of BMPs that are considered green stormwater infrastructure for the purposes of compliance with the Minimum Requirements for Flow Control and Minimum Requirements for Treatment. Note that although infiltration basins, trenches, and drywells use infiltration for flow control, they are not considered green stormwater infrastructure with respect to stormwater code compliance.

Table 4.1. Types of Green Stormwater Infrastructure for flow control.

Flow Control	Water Quality Treatment
Trees Planting and Retention	Bioretention
Bioretention	Permeable Pavement
Permeable Pavement	
Water Harvesting	
Green Roofs	
Dispersion	

Note that while some green stormwater infrastructure BMPs can be used for both flow control and water quality treatment purposes, their most common application is for flow control. Therefore green stormwater infrastructure facilities are described in this chapter.

4.2 BMP Selection and Design Process for Flow Control Facilities

4.2.1 Selection of Flow Control Facilities

Designers should follow the steps presented below and in Figure 4.1 to select the appropriate flow control facility or facilities for a given project to meet the Stormwater Code. Note that water quality treatment requirements may also apply and need to be addressed separately.

Step 1 – Based on the project type, discharge location / drainage basin, and existing and proposed site conditions (e.g., the amount of new and replaced impervious area and land disturbing activity), determine the applicable Minimum Requirements for Flow Control (see Chapter 2 and Section 4.1.4). Most project require that green stormwater infrastructure be implemented to the maximum extent feasible (SMC 22.805.020.F). Many scenarios also require that numerical flow control standards (i.e., peak and/or duration standards) be met.

Step 2 –Review preliminary design concepts and evaluate whether there are opportunities to reduce areas of new and replaced impervious surface (e.g., low impact development site design) and minimize impacts to existing vegetation. Such design changes could significantly reduce or eliminate flow control requirements.

Step 3 – Based on the revised preliminary site design, determine whether the Minimum Requirements for Flow Control are triggered. If so, determine which if any numerical flow control standards must be met (in addition to implementing green stormwater infrastructure to the maximum extent feasible).

Step 4 – Determine whether infiltration is feasible (see Section 4.3). If infiltration is feasible, determine the design infiltration rate for the native soil. Infiltration testing is encouraged and may be required.

Step 5 – Develop a Soil Amendment Plan to show that site post construction soil quality and depth requirements (see Section 4.4.1) will be achieved.

Step 6 – Analyze BMPs in the required order for flow control BMPs presented in Table 4.2. Green stormwater infrastructure must be implemented to the maximum extent feasible. If the Minimum Requirements for Treatment also apply to a project, look for opportunities to use facilities which can meet both flow control and water quality treatment requirements (see Table 4.3).

Step 7 – For projects with less than 10,000 square feet of new and replaced impervious area, proceed to Step 7a. For sites with 10,000 square feet or more new and replaced impervious area, proceed to Step 7b.

Step 7a – The Pre-Sized Approach for flow control facility design (Section 4.2.2) may be applied. The designer may also choose to use the Modeling Approach (Step 7b).

Step 7b – The Modeling Approach for flow control facility design must be applied (see Section 4.2.3). Perform continuous modeling as specified for each BMP. See Chapter 6 for additional modeling guidelines.

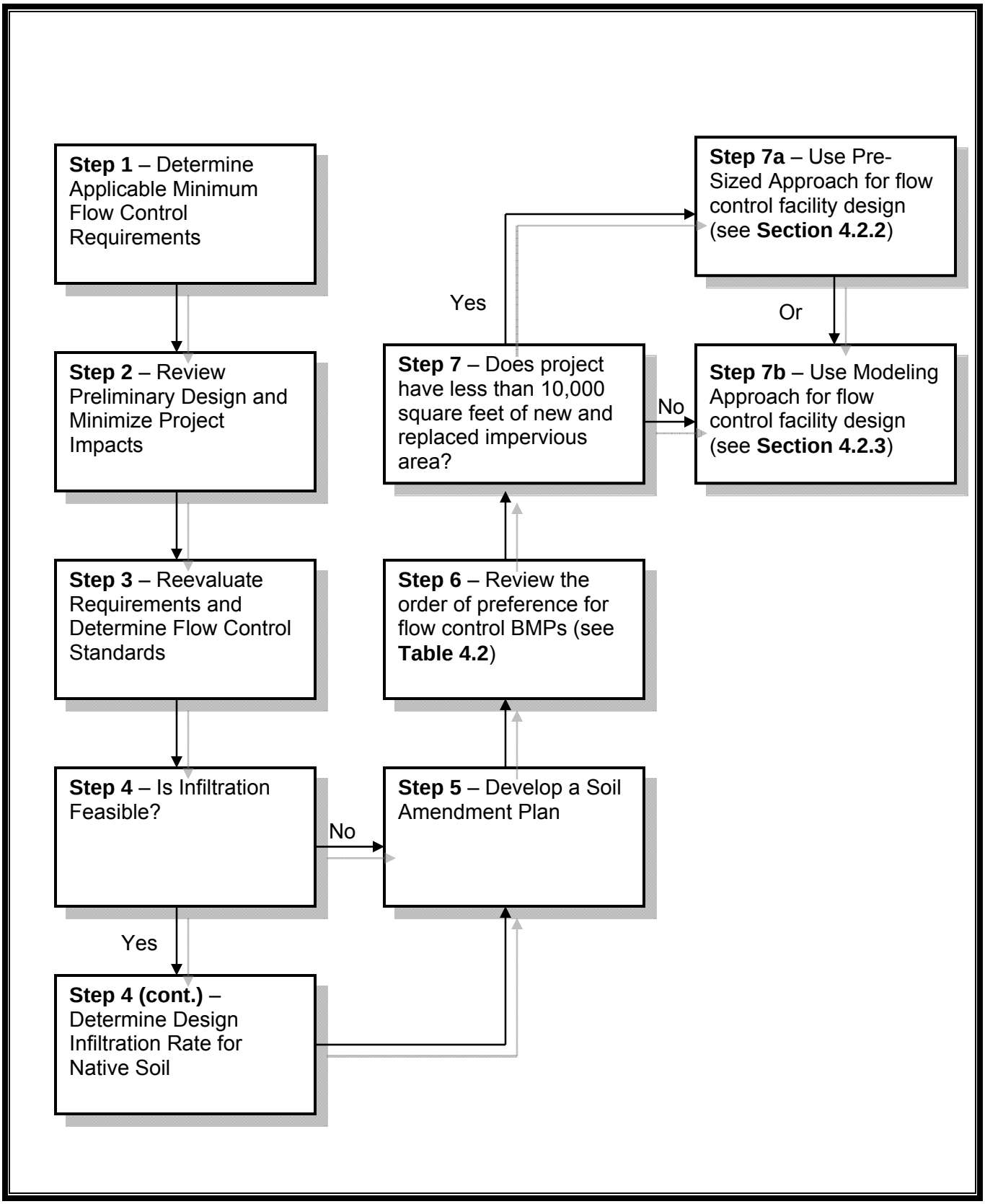


Figure 4.1. Flow Control Facility Selection Flow Chart.

Table 4.2. Required Order of Analysis for Flow Control BMPs.

Required Order of Analysis	BMP Type	BMPs	BMP Section
Required	Green Stormwater Infrastructure – Soil Quality and Depth	Post Construction Soil Quality and Depth	4.4.1
1	Green Stormwater Infrastructure – Runoff reduction methods	Maintain Existing Trees	4.4.2
		Dispersion (downspout or sheet flow) [a]	4.4.3 / 4.4.4
		Plant New Trees	4.4.2
2	Green Stormwater Infrastructure – Infiltrating and reuse facilities	Bioretention Cells (without underdrain)	4.4.5
		Rainwater Harvesting	4.4.6
		Permeable Pavement Facilities (with storage reservoir and overflow)	4.4.7
3	Green Stormwater Infrastructure – Impervious surface reduction methods	Green Roof	4.4.8
		Permeable Pavement Surfaces	4.4.7
4	Green Stormwater Infrastructure – Non infiltrating facilities	Bioretention Cells (with underdrain)	4.4.5
		Detention Cisterns, aboveground with harvesting capacity [b]	4.6.6
5	Traditional Stormwater Infrastructure – Small-scale / distributed infiltrating facilities	Infiltration Trenches	4.5.2
		Dry wells	4.5.3
6	Traditional Stormwater Infrastructure – Large-scale infiltrating facilities	Infiltration Basins	4.5.1
7	Traditional Stormwater Infrastructure – Non infiltrating facilities	Detention Pipes	4.6.4
		Detention Vaults	4.6.5
		Detention Pond	4.6.3
		Detention Cisterns	4.6.6
		Other Detention Options	4.6.7

[a] Dispersion is typically not feasible in Seattle due to required flowpath lengths. Therefore, the applicant is not required to evaluate dispersion as an option to meet the requirement to implement green stormwater infrastructure to the maximum extent feasible.

[b] Detention cisterns with harvesting capacity are considered green stormwater infrastructure for single family residential projects only.

Table 4.3. Flow Control BMPs Capable of Providing Water Quality Treatment

Order of Preference	BMP Type	BMPs	Flow Control Section	Treatment Section	Notes
1	Green Stormwater Infrastructure – Infiltrating Facilities	Bioretention Cells (with or without underdrain)	4.4.5	5.8.5	May be designed to meet the requirements for basic, phosphorus, and enhanced treatment. At least 91 percent of the total runoff volume for the simulation period must be infiltrated through soils meeting the minimum criteria for infiltration treatment (Section 5.4.1.2 and 5.8.4).
		Permeable Pavement Facilities	4.4.7	5.8.5	
2	Traditional Stormwater Infrastructure – Small-scale / distributed infiltrating facilities	Infiltration Trenches	4.5.2	5.8.5	
		Dry Wells	4.5.3	5.8.5	
3	Traditional Stormwater Infrastructure – Large-scale infiltrating facilities	Infiltration Basins	4.5.1	5.8.5	
4	Traditional Stormwater Infrastructure – Non infiltrating facilities	Detention/Wetpond	4.6.3	5.10.6	May be designed by adding dead storage to meet the requirements for basic treatment. Enhanced treatment may be met when in treatment train with another treatment facility (see Section 5.3.4)
		Detention/Wetvault	4.6.5	5.10.6	
		Detention/Stormwater Wetland	4.6.3	5.10.6	

4.2.2 Pre-sized Approach for Flow Control Design

Designers must review steps 1 through 6 in Section 4.2.1 before beginning the Pre-Sized Approach. The Pre-Sized Approach for flow control design may be used when the new and replaced impervious surface associated with a project does not exceed 10,000 square feet and the project is subject to the Pre-developed Pasture and/or Peak Control standards. Under the Pre-Sized Approach, flow control BMPs can be selected and sized without performing continuous flow modeling. The sections below provide requirements and guidance on applying the pre-sized facilities to meet the Minimum Requirements for Flow Control.

The flow control benefits of each of the pre-sized facilities are provided as either flow control credit, facility sizing factor or facility sizing equation. Credits and sizing factors were developed for the City's Pre-developed Pasture and Peak Control standards. If both standards apply to a project (such as a site in a non-listed creek basin with a capacity constrained drainage system), the larger facility size or conservative flow control credit must be used. The Pre-Sized Approach was not developed for the Pre-developed Forest Standard because it will not be triggered as often.

Generalized assumptions were used to design the pre-sized facilities that may result in conservative sizing or may underestimate flow control credits for some sites. Designers have the option to use the pre-sized facilities provided in this section, or to follow the Modeling Approach (see Section 4.2.3) and submit an alternative facility size with supporting engineering calculations for SPU review and consideration.

The pre-sized facilities, flow control credits, sizing factors, sizing equations, and procedures are discussed below. Additional design requirements for the pre-sized facilities (e.g., side slopes, freeboard, aggregate thickness, soil depth) are provided in the "BMP Sizing" or "Flow Control Credit" section for each facility in Sections 4.4 through 4.6.

While the credits, factors and equations presented in this chapter are applicable to flow control requirements, some infiltrating BMPs are capable of achieving water quality treatment standards as well (see Table 4.3). If bioretention (without detention or underdrain), permeable pavement, infiltration trenches or drywells are sized using the flow control sizing factors in this chapter, they will also achieve the water quality requirement for infiltration of 91 percent of the total runoff volume. Therefore, if designed to infiltrate through soils meeting the minimum criteria for infiltration treatment (Section 5.8.4), these facilities will meet the basic, phosphorus, and enhanced treatment requirements in addition to flow control requirements. To use these infiltration facilities for water quality treatment only, see the pre-sized tables in Section 5.8.5.

4.2.2.1 Pre-Sized Facilities

The flow control BMPs included in the Pre-Sized Approach consist of all green stormwater infrastructure BMPs and selected traditional infiltration and detention facilities.

The green stormwater infrastructure runoff reduction and impervious surface reduction methods included in the Pre-Sized Approach are listed below:

- Retaining Trees
- Planting New Trees
- Dispersion
- Permeable Pavement Surfaces (may not receive run-on)
- Green Roofs

The green stormwater infrastructure facilities included in the Pre-Sized Approach are listed below:

- Bioretention Cells (without underdrain)
- Bioretention Cells with Detention (without underdrain)
- Permeable Pavement Facilities (may receive run-on)
- Aboveground Detention Cisterns with Harvesting Capacity (single family residential projects only)
- Bioretention Planters (with underdrain)

The traditional infiltration and detention facilities included in the Pre-Sized Approach are listed below:

- Infiltration Trenches
- Dry Wells
- Detention Pipes
- Detention Vaults
- Aboveground Detention Cisterns

For each BMP, flow control credits or sizing factors were developed for typical design variations (e.g., ponding depths, aggregate thickness, slopes, etc.). To use these BMPs with a different design configuration or BMPs not listed above, the designer must use the Modeling Approach (see Section 4.2.3).

4.2.2.2 Flow Control Credits

Under the Pre-Sized Approach, flow control credit may be achieved by retaining trees, planting new trees, dispersing runoff, using permeable pavement surfaces, and installing green roofs. Flow control credits for each method are provided in Table 4.4. The credit values are based on the degree to which these methods

achieve the flow control standards. A 100 percent credit means that the flow control goal is achieved and no further control is required for the impervious area managed by the BMP. If partial credit is received, the standard is not achieved and additional downstream flow control measures are required to meet the numerical flow control standards.

Table 4.4. Flow Control Credits for Pre-Sized Approach (for sites with less than 10,000 sf impervious surface)

Facility Type	Design Variable	Flow Control Credit[1]		Section Providing Design Requirements
		Pre-developed Pasture Standard	Peak Control Standard	
Retained Tree[2]	Evergreen	20% canopy area (min 100 sf/tree)		4.4.2
	Deciduous	10% canopy area (min 50 sf/tree)		
New Tree[2]	Evergreen	50 sf / tree		4.4.2
	Deciduous	20 sf / tree		
Downspout or Sheet Flow Dispersion[3]	Dispersion to compost amended lawn or landscape	90%	100%	4.4.3, 4.4.4
Permeable Pavement Surface (may not receive run-on)	Subgrade slope less than or equal to 2%	100%	100%	4.4.7
	Subgrade slope 2 - 5%	45%	70%	
Green Roof	Single-course 4 inch depth growth medium	46%	71%	4.4.8
	Multi-course 4 inch depth growth medium	46%	71%	
	Multi-course 8 inch depth growth medium	54%	79%	

sf – square feet; % - percent

[1] Impervious area mitigated by a BMP is calculated as: [Flow Control Credit (%)/100] x [Existing Tree Canopy Area, Number New Trees Planted, Green Roof Area, Dispersed Area or Pavement Area]. Note that some of the BMPs do not achieve full credit (i.e., 100 percent credit) and the site design would require additional flow control measures to meet City standards.

[2] Trees must be within 20 feet of ground-level impervious surface. The total tree credit shall not exceed 25% of the impervious surface requiring mitigation.

[3] Dispersion is typically not feasible for sites in Seattle due to required flowpath lengths.

The impervious area mitigated by a BMP is calculated as:

> Impervious Area Mitigated = [Flow Control Credit (%)/100] x [Existing Tree Canopy Area, Number New Trees Planted, Area Directed to Dispersion, Pavement Area, or Green Roof Area]

As an example, for a site subject to the Pre-developed Pasture Standard, a permeable pavement surface overlying subgrade with a slope between 2 and 5 percent would receive a 45 percent credit. Therefore, 45 percent of the

permeable pavement surface can be excluded from drainage calculations. The effective impervious area (area used to size downstream flow control facility) would be calculated as 55 percent of the permeable pavement surface area.

To use these credits, the BMPs must meet the specific design requirements specified for pre-sized facilities in the "Flow Control Credit" Section for each BMP in Sections 4.4 through 4.6.

4.2.2.3 Design Sizing Factors

Under the Pre-Sized Approach, green stormwater infrastructure facilities and selected traditional flow control facilities may be sized using the sizing factors provided in Table 4.5. Sizing factors were developed using a continuous runoff hydrologic model to achieve applicable flow control standards.

For facilities with variable allowable depths, sizing factors are provided for at least two typical depths. Designers may linearly interpolate BMP size for intermediate design depths, but may not extrapolate.

For infiltrating facilities, sizing factors are provided for native soil design infiltration rates of 0.25, 0.5, and 1.0 inch per hour. Site design infiltration rates must be rounded down to the nearest rate specified in Table 4.5.

The impervious area mitigated by a given facility size is calculated as:

Impervious Area Mitigated = BMP Area ÷ Factor (%)/100

Alternatively, the facility size required to mitigate a given impervious area is calculated as:

BMP Area = Contributing Impervious Area x Factor (%)/100

Because the sizing factors are used to calculate the facility *bottom* area (unless otherwise noted), the total footprint area (top area) is larger for facilities with sloping sides (e.g., bioretention cells). The footprint area is calculated as a function of the bottom area, the side slopes and the total depth (ponding and freeboard depth).

To use these sizing factors, the facility must meet all the specific design requirements specified for pre-sized facilities in the "BMP Sizing" Section for each BMP in Sections 4.4 through 4.6.

Note that the maximum area that may be routed to a pre-sized facility shall be twice the area for which it is sized. No flow control credit is given for runoff from areas beyond the design area. If additional runoff is routed to a facility, it must be clearly noted on submitted plans and the overflow infrastructure requires engineering design (see Section 4.2.5).

4.2.2.4 Design Sizing Equations

Under the Pre-Sized Approach, detention facilities may be sized using the sizing equations provided in Table 4.6.

Table 4.5. Flow Control Sizing Factors for Pre-Sized Approach (for sites with less than 10,000 sf impervious surface).

Facility Type	Facility Overflow Depth	Native Soil Design Infiltration Rate (inch/hour)	Sizing Factor [a] (% of contributing impervious area)		Section Providing Design Requirements
			Pre-developed Pasture Standard	Peak Flow Control Standard	
Green Stormwater Infrastructure Facilities					
Bioretention Cell [b]	2 inch ponding depth	0.25	23.0%	--	4.4.5
		0.5	15.8%	--	
		1.0	9.3%	--	
	6 inch ponding depth	0.25	14.6%	33.1%	
		0.5	9.9%	20.5%	
		1.0	6.4%	10.6%	
	12 inch ponding depth	0.25	NA	NA	
		0.5	6.5%	13.4%	
		1.0	4.1%	6.7%	
Bioretention Cell with Detention	12 inch ponding depth	0.25	See sizing equations in Table 4.6		4.4.5
		0.5			
		1.0			
Permeable Pavement Facility (may receive run-on)	6 inch ponding depth in storage reservoir	0.25	60.6%	131.4%	4.4.7
		0.5	34.5%	52.4%	
		1.0	33.3% [c]	33.3% [c]	
	12 inch ponding depth in storage reservoir	0.25	38.2%	75.8%	
		0.5	33.3% [c]	38.7%	
		1.0	33.3% [c]	33.3% [c]	
Bioretention Planter with Underdrain	12 inch ponding depth	NA	NA	6.5%	4.4.5

Table 4.5 (continued). Flow Control Sizing Factors for Pre-Sized Approach (for sites with less than 10,000 sf impervious surface).

Facility Type	Facility Overflow Depth	Native Soil Design Infiltration Rate (inch/hour)	Sizing Factor [a] (% of contributing impervious area)		Section Providing Design Requirements
			Pre-developed Pasture Standard	Peak Flow Control Standard	
Traditional Infiltration Facilities					
Infiltration Trench	1.5 foot depth	0.25	27.3%	55.1%	4.5.2
		0.5	16.8%	30.0%	
		1.0	10.7%	17.4%	
	3.0 foot depth	0.25	17.0%	31.9%	4.5.2
		0.5	12.0%	20.1%	
		1.0	7.8%	13.1%	
Drywell	4.0 foot depth	0.25	13.6%	24.9%	4.5.3
		0.5	10.1%	17.6%	
		1.0	6.7%	11.1%	
	6.0 foot depth	0.25	9.7%	17.5%	4.5.3
		0.5	7.9%	13.2%	
		1.0	5.7%	8.9%	

-- sizing factor not provided; NA-not applicable; %-percent.
[a] BMP area is calculated as a function of impervious area draining to it: BMP Area = Contributing Impervious Area x Factor (%)/100
[b] Sizing factors are for bioretention facility bottom area. Total footprint area may be calculated based on side slopes (3H:1V), ponding depth, and freeboard.
[c] The City requires that the contributing impervious area for permeable pavement facilities be no larger than 3 times the area of the permeable pavement facility receiving runoff, corresponding to a minimum sizing factor of 33.3%.

Table 4.6. Flow Control Sizing Equations for Pre-Sized Approach (for sites with less than 10,000 sf impervious surface).

Facility Type	Facility Overflow Depth [b]	Native Soil Design Infiltration Rate / Contributing Area	Sizing Equation [a] (function of contributing impervious area)		Section Providing Design Requirements
			Pre-developed Pasture Standard	Peak Control Standard	
Green Stormwater Infrastructure Facilities					
Bioretention Cell with Detention [c,d]	12 inch ponding depth	0.25 inch/hour	NA	0.0382A +199	4.4.5
		0.5 inch/hour	NA	0.0297A +129	
		1.0 inch/hour	NA	0.0208A + 97	
Traditional Detention Facilities					
Detention Pipe [e,f]	24 inch diameter	2,000 – 5,000 sf	0.0000263A^1.94	0.00000508A^2.10	4.6.4
		5,000 – 10,000 sf	0.120A – 235	0.123A - 314	
	36 inch diameter	2,000 – 5,000 sf	0.000000219A^2.40	0.00000277A^2.05	
		5,000 – 10,000 sf	0.0515A – 110	0.0546A- 167	
Detention Vault [f]	3 foot depth	2,000 – 5,000 sf	0.0000428A^1.87	0.0000616A^1.77	4.6.5
		5,000 – 10,000 sf	0.121A – 255	0.0700A - 130	
	4 foot depth	2,000 – 5,000 sf	NA	0.00000441A^2.04	
		5,000 – 10,000 sf	NA	0.0508A – 98.0	
Aboveground Detention Cistern	3 foot depth	< 6,140 sf	0.00599A^1.35	0.000531A^1.59	4.6.6
		6,140 – 8,000 sf	760 sf		
		> 8,000 sf	0.131A – 289		
	4 foot depth	NA	NA	0.000417A^1.58	

A- contributing impervious area; NA-not applicable; sf-square feet.
[a] BMP area is calculated as a function of impervious area draining to it (A) using one of the following equations:
BMP Area (square feet) = [Factor x A (square feet)] + Integer.
BMP Area (square feet) = Factor x [A (square feet)^Integer].
[b] Facility overflow depth is vertical distance from low flow orifice invert elevation to overflow elevation.
[c] Sizing equations are for bioretention facility bottom area. Total footprint area may be calculated based on side slopes (3H:1V), ponding depth, and freeboard.
[d] Applicable for contributing areas of at least 1,500 square feet (for smaller areas, bioretention cells without detention requires less space).
[e] Sizing equations are for pipe length in feet (not area).
[f] Applicable for contributing areas between 2,000 and 10,000 square feet.

The impervious area mitigated by a given facility size is calculated as:

Impervious Area Mitigated = (BMP Size – Integer) ÷ Factor, or

Impervious Area Mitigated = [BMP Area/Factor]^(1/Integer)

(all area units must be square feet and all length units must be feet)

Alternatively, the facility size required to mitigate a given impervious area is calculated as:

BMP Size = (Contributing Impervious Area x Factor) + Integer, or

BMP Size = Factor x [(Contributing impervious Area) ^Integer].

(all area units must be square feet and all length units must be feet)

4.2.2.5 Pre-Sized Approach Worksheet

Pre-Sized Approach worksheets are available to guide the designer through selecting and sizing flow control facilities. If used, the Pre-Sized Approach worksheet must be included in the project submittal (see Chapter 3). Electronic worksheets (with automatic calculations) are available by project type on the DPD website (www.seattle.gov/dpd/). Additional guidelines are provided below.

Projects Requiring GSI to the Maximum Extent Feasible

Single-family residential projects, sidewalk or trail projects, and smaller parcel-based and roadway projects, are required to manage new and replaced impervious surfaces to the maximum extent feasible using green stormwater infrastructure (FC#1), but are not subject to numerical performance standards. Guidance for these projects and pre-sized approach worksheets are provided in separtrate Director's Rules (see Section 4.1.3).

Projects Subject to Numerical Performance Standards

Some parcel-based and roadway projects must meet numerical flow control standards (i.e., peak and/or duration standards) using green stormwater infrastructure to the maximum extent feasible. Submittal for drainage review shall demonstrate evaluation of BMPs per Table 4.2.

Under the Pre-Sized Approach, parcel-based and roadway projects may mitigate a smaller portion of the project's new and replaced impervious area if only green stormwater infrastructure BMPs are employed. If only green stormwater infrastructure is used for flow control, the impervious area requiring management is 70 percent of the total impervious area. If any traditional infiltration or detention facilities are used, all of the new and or replaced impervious area must be mitigated. Note that the minimum impervious area tributary to an orifice controlled detention facility shall be 2,000 square feet.

A sample Pre-Sized Approach worksheet for parcel-based and roadway projects is presented as Table 4.7. The designer should follow the steps presented below.

Step A – Review the Pre-Sized Approach worksheet for parcel-based and roadway projects (Table 4.7) to identify initial BMP options for the project site.

Step B – Divide the project area into distinct drainage basin(s) (e.g., creek basin, combined sewer basin, etc) and determine applicable flow control standard(s) (see Chapter 2 and Section 4.1.4).

Step C – Divide the project area into distinct drainage areas (e.g., roof tops, parking lots, roadway, lawn, etc).

Step D – Calculate and report total new and replaced impervious surface area.

Step E – Calculate and report seventy percent of the new and replaced impervious surface (Step C). This is the impervious area requiring mitigation if only green stormwater infrastructure is implemented.

Step F – Identify opportunities for retaining trees, planting new trees, and dispersion.

Find flow control credits for selected BMPs (Table 4.4).

For each BMP, calculate and report the impervious area mitigated as a product of the flow control credit and the existing tree canopy area, number of new trees planted, or area directed to dispersion.

Step G – For the remaining unmitigated impervious surfaces, identify opportunities and available space for infiltrating green stormwater infrastructure (bioretention cells without underdrains with or without detention and permeable pavement facilities).

Find sizing factors or equations for selected BMPs (Table 4.5).

For each BMP, calculate and report the impervious area mitigated using the sizing equation.

Step H – For the remaining unmitigated impervious surfaces, identify opportunities and available space for greenroofs and permeable pavement surfaces.

Find flow control credits for selected BMPs (Table 4.4).

For each BMP, calculate and report the impervious area mitigated as a product of the flow control credit and the greenroof area or permeable pavement areas.

Table 4.7. SAMPLE Pre-Sized Flow Control Worksheet – Parcel-based and Roadway Projects [1] (for sites with less than 10,000 sf impervious surface).

Flow Control Standard(s) →		
New and Replaced Impervious Area →		sf
Area Requiring Mitigation if Only GSI Used [2] →		sf
Area Requiring Mitigation if Traditional Facilities Used [3] →		sf

GSI Runoff Reduction Methods	Facility Size				Credit (Table 4.4)		Area Mitigated	
Retained Trees								
Existing Evergreen	Canopy Area		sf	x		=		sf
	# Trees		trees					
Existing Deciduous	Canopy Area		sf	x		=		sf
	# Trees		trees					
New Trees								
New Evergreen	# Trees		trees	x		=		sf
New Deciduous	# Trees		trees	x		=		sf
Dispersion								
Downspout or Sheet Flow Dispersion	Impervious Area		sf	x		=		sf

GSI Facilities			Facility Size				Factor / Eqn (Table 4.5)		Area Mitigated	
Infiltrating Facilities										
Bioretention Cell (without underdrain)										
Ponding Depth		in	Bottom Area		sf	÷		=		sf
Design Infiltration Rate		in/hr								
Permeable Pavement Facility (may receive run-on)										
Reservoir Ponding Depth		in	Pavement Area		sf	÷		=		sf
Design Infiltration Rate		in/hr								
Bioretention Cell with Detention (without underdrain)										
Ponding Depth		in	Bottom Area	(	sf -	) ÷		=		sf
Design Infiltration Rate		in/hr								

GSI Impervious Surface Reduction Methods	Facility Size				Credit (Table 4.4)		Area Mitigated	
Alternative Pavement Surfaces								
Permeable Pavement Surface with slope ≤2%	Permeable Pvmnt Area		sf	x		=		sf
Permeable Pavement Surface with slope 2-5%	Permeable Pvmnt Area		sf	x		=		sf
Alternative Roof Surfaces								
Green Roof (Single-Course/ 4" Growth Medium)	Green Roof Area		sf	x		=		sf
Green Roof (Multi-Course/ 4" Growth Medium)	Green Roof Area		sf	x		=		sf
Green Roof (Multi-Course/ 8" Growth Medium)	Green Roof Area		sf	x		=		sf

Table 4.7 (continued). SAMPLE Pre-Sized Flow Control Worksheet – Parcel-based and Roadway Projects. [1]

GSI Facilities			Facility Size				Factor / Eqn (Table 4.5)		Area Mitigated	
Non Infiltrating Facilities										
Bioretention Planter (with underdrain)										
Ponding Depth	_____	in	Bottom Area	_____	sf	÷	_____	=		sf
Total Area Mitigated by GSI →									_____	sf
Percent of Impervious Area Mitigated by GSI [4]									_____	%

Traditional Facilities			Facility Size				Factor / Eqn (Table 4.5 /4.6)		Area Mitigated	
Infiltrating Facilities										
Infiltration Trench										
Trench Depth	_____	in	Trench Area	_____	sf	÷	_____	=	_____	sf
Design Infiltration Rate	_____	in/hr								
Drywell										
Well Depth	_____	in	Well Area	_____	sf	÷	_____	=	_____	sf
Design Infiltration Rate		in/hr								
Detention Facilities[5]										
Detention Pipe										
Max. Head Above Orifice	_____	in	Pipe Length	(_____	ft	-	_____)	÷ _____	= _____	sf
			Pipe Length	(_____	ft	÷	_____)^	(1/ _____)	= _____	sf
Detention Vault										
Max. Head Above Orifice	_____	in	Vault Area	(_____	sf	-	_____)	÷ _____	= _____	sf
			Vault Area	(_____	sf	÷	_____)^	(1/ _____)	= _____	sf
Detention Cistern (aboveground)										
Max. Head Above Orifice	_____	in	Cistern Area	(_____	sf	-	_____)	÷ _____	= _____	sf
			Cistern Area	(_____	sf	÷	_____)^	(1/ _____)	= _____	sf
Area Mitigated by Traditional Facilities →									_____	sf
Total Area Mitigated →									_____	sf

GSI – Green Stormwater Infrastructure; sf – square feet; ft – feet; in – inch; in/hr – inch per hour; eqn – equation

1 Approved electronic worksheet available on DPD website. Refer to latest DPD CAM for more information and guidance.

2 70 percent of the new and replaced impervious area.

3 The total new and replaced impervious area.

4 If area mitigated by GSI is at least 70 percent of the new and replaced impervious area, no more mitigation is required.

5 Equation used to calculate detention facility size varies depending upon standard and contributing area (see Table 4.6).

Step I – For the remaining unmitigated impervious surfaces, identify opportunities and available space for non-infiltrating green stormwater infrastructure (bioretention cells with underdrain).

Find sizing factors for selected BMPs (Table 4.5).

For each BMP, calculate and report the impervious area mitigated using the sizing equation.

Step J – Calculate and report the total impervious area mitigated by summing the area mitigated by green stormwater infrastructure flow control practices (Steps F through I).

Step K – If the total impervious area mitigated (Step J) is greater than or equal to the area requiring mitigation with only green stormwater infrastructure (Step E), no further mitigation us required.

If the total impervious area mitigated (Step J) is less than the area requiring mitigation with only green stormwater infrastructure (Step E), evaluate additional BMPs, and/or alternative BMPs, BMP sizes, or BMP locations. If it is not feasible to achieve flow control standards using only green stormwater infrastructure, proceed to Step L.

Step L – For the remaining unmitigated impervious surfaces, select traditional stormwater facilities (infiltration trench, dry wells, detention pipe, detention vault and detention cistern). See order of preference for flow control facilities presented in Table 4.2.

Find sizing factors for selected BMPs (Tables 4.5 and 4.6).

For each BMP, calculate and report the impervious area mitigated using the sizing equation.

Step M – Calculate and report the total impervious area mitigated by traditional stormwater facilities.

Step N Calculate and report the total impervious area mitigated by summing the area mitigated by green stormwater infrastructure (Step J) and traditional stormwater facilities (Step M).

Step O – If the total impervious area mitigated (Step N) is less than the total new and replaced impervious surface reported (Step D), evaluate additional BMPs, and/or alternative BMPs, BMP sizes, or BMP locations.

Iterate steps D through O to attain a design which mitigates impervious area to the applicable flow control standards.

4.2.3 Modeling Approach for Flow Control Design

The Modeling Approach may be used to design flow control facilities. Projects with new and replaced impervious surface equal to or exceeding 10,000 square

feet are required to use this approach. Projects triggering the Pre-Developed Forest standard must also use the Modeling Approach.

Under the Modeling Approach, flow control facilities are designed to achieve flow control standards using a continuous rainfall-runoff model. Continuous simulation methods and a list of approved continuous simulation models are provided in Chapter 6. Specific modeling requirements are presented in the "BMP Sizing" or "Flow Control Credit" section for each BMP.

Unless otherwise specified, all continuous modeling must be performed using the City of Seattle Design Time Series consisting of a 158-year precipitation and evaporation time series that are representative of the climatic conditions in the City of Seattle. See Section 6.5.2.1.

Retained and newly planted trees receive flow control credits to reduce the impervious surface area requiring flow control (see Section 4.4.2). The impervious area mitigated by trees is subtracted from the drainage area entered into the continuous model.

4.2.4 Requirements for Bypassing Offsite or Onsite flows

For flows subject to the Minimum Requirements for Flow Control, the following three flow bypass-related scenarios require that additional considerations be taken into account when designing flow control facilities (see also Section 5.4.5 for design guidelines for flow splitters):

1. Offsite flow currently enters the project site, but can be bypassed as part of the proposed project improvements
2. Offsite flow currently enters the project site, but cannot be bypassed as part of the proposed project improvements
3. Onsite flow that is within the project limits cannot feasibly be routed to the project flow control facility.

The requirements and guidelines applicable to each scenario are outlined below.

4.2.4.1 Scenario 1 – Offsite Flow Management

Offsite flows may bypass flow control and treatment facilities under the following conditions:

- Natural drainage courses are maintained.
- Existing flows to wetlands are maintained.
- Peak flows are not increased by bypassing. For instance, flows through existing vegetation cannot be bypassed over pavement, or in a pipe, without providing flow control.
- The discharge point does not create significant adverse impacts to down gradient properties.

4.2.4.2 Scenario 2 – Offsite Flow Management

When offsite flows cannot feasibly bypass proposed flow control facilities, the flow control facilities shall be sized for the combined total of onsite and offsite flows with the allowable discharge rates determined by the onsite runoff calculations.

When offsite flows cannot feasibly bypass proposed treatment facilities, the treatment facilities shall be sized for the combined total of onsite and offsite flows.

4.2.4.3 Scenario 3 – Uncontrolled Onsite Flow Management

Runoff from a project that cannot feasibly be routed to the proposed flow control facilities or treatment facilities may be bypassed under the following conditions:

- When the proposed flow control and treatment facilities are designed to compensate for uncontrolled flow and meet the applicable minimum requirements for the project.
- Or, when the bypass area is due to incidental grading to match surrounding roadways, or properties and is less than 1,000 square feet and will not create significant adverse impacts to down gradient properties. For instance, driveways.

4.2.5 Requirements for Overflow Design

All projects must convey stormwater flow to an approved discharge point(s) and include overflows for all stormwater facilities. Overflows are critical to minimize flooding and on-site and downstream property damage.

Approved discharge points, in order of preference, include:

- Surface waters
- Public storm drain pipes
- Public combined sewer pipes

Conveyance to approved discharge points, in order of preference, include:

- Direct pipe connections
- Ditch and culvert system
- Gutter or street flow line
- Surface dispersal

Overflows may be to the approved discharge point(s), or through interflow to the surface, or to surface discharge, or in combination. Overflows may be piped, or daylighted through a storage reservoir or dispersion BMP, or through overtopping of the facility. Plans shall indicate all flow paths.

At a minimum, overflows shall be designed to convey peak flows with a 4 percent annual probability (25-year recurrence interval flows).

Designers are recommended to consider flooding and nuisance ponding risks associated with larger peak flows. During large storm events, capacity will be limited at the approved discharge point(s) and backwater calculations may be required.

If a facility is designed for full infiltration of the 158-year simulation period, a constructed overflow is not required. Plans should indicate surface flow paths in case of failure of the facility.

For Pre-Sized Approach facilities, the maximum area that may be routed to the facility shall be twice the area for which it is sized. No flow control credit is given for runoff from areas beyond the design area. If additional runoff is routed to a facility, it must be clearly noted on submitted plans and the overflow infrastructure requires engineering design.

4.3 General Requirements for Infiltration Facilities

This section addresses general requirements for infiltration facilities, including infiltrating green stormwater infrastructure. When locating, designing, and constructing infiltration facilities, there are several factors that must be considered to ensure effective performance. This section outlines these general considerations and requirements. Additional requirements are outlined for each individual infiltration BMP presented in Sections 4.4 and 4.5.

4.3.1 Description

Infiltration is the percolation of surface water into the ground. The infiltration facilities included in this manual include:

- Bioretention facilities without impermeable reservoir or liner (Section 4.4.5)
- Permeable pavement facilities (Section 4.4.7.2)
- Infiltration basins (Section 4.5.1)
- Infiltration trenches (Section 4.5.2)
- Drywells (Section 4.5.3).

Permeable pavement surfaces (Section 4.4.7.3) are not considered infiltration facilities because they do not receive runoff from other areas and mitigate only the rain falling on the surface. Although stormwater will infiltrate to the underlying soil, the volumes infiltrated are not considered substantial enough to necessitate the restrictions set forth in this section.

Similarly, dispersion BMPs (Sections 4.4.3, 4.4.4, and 4.4.9) are not considered infiltration facilities for the purposes of this manual. Although stormwater will

infiltrate to the underlying soil, the spatial requirements and setbacks specifically established for these BMPs are sufficient.

4.3.2 General Considerations

Infiltration is the City's preferred method for stormwater management because it is generally the best method for restoring the predevelopment flow regime. Where site conditions allow infiltration, costs for these facilities tend to be lower than traditional subsurface detention facilities.

Due to the geologic and topographic conditions in Seattle, not all sites are suitable for stormwater infiltration. Infiltration is not permitted in some areas due to land slopes and potential landslide hazards. In addition, many locations in Seattle have soils that are underlain by a compacted layer of soil (i.e., glacial till or hardpan) that limits infiltration capacity and causes water to perch on the relatively impervious layer during the wet season. While reduced infiltration rates will make infiltration less cost effective, smaller scale infiltration is often practicable in these areas.

Infiltration practices are also susceptible to clogging by suspended solids in stormwater runoff. Catchment areas contributing runoff to infiltration facilities must be permanently stabilized. If catchment area exceeds 2,000 square feet and flow is concentrated, infiltration facilities must be preceded by a presettling technique (e.g., variation on filter strip, presettling catch basin, or vault. The presettling is intended to remove larger solids, but not expected to meet water quality treatment goals or sizing guidelines for pretreatment facilities.

When water quality treatment requirements are triggered, runoff from pollution generating surfaces must be preceded by a basic treatment BMP or the infiltration facility must be designed to meet treatment requirements. Infiltration facilities meet basic, phosphorus and enhanced water quality treatment requirements when it is shown that at least 91 percent of the total runoff volume for the simulation period is infiltrated through a media which meets the treatment soil requirements outlined in Section 5.8.4. Applicable drawdown requirements must also be met (see Section 5.4.1.2).

Infiltration practices are not appropriate in areas that contribute uncontrolled concentrations of oil, grease, floatable organic or floatable inorganic material without adequate pretreatment.

Because infiltration practices recharge stormwater directly to groundwater, they can potentially contaminate groundwater supplies with dissolved pollutants contained in stormwater runoff or mobilized from subsurface contamination. Runoff sources that cause particular problems for infiltration structures include sites with high pesticide levels; and manufacturing and industrial sites, due to potentially high concentrations of soluble toxicants and heavy metals. Infiltration practices should be carefully sited and designed to minimize the risk of groundwater contamination.

A flow chart to lead the designer through the process of determining whether or not an infiltration facility is allowable on a particular site is presented as Figure 4.2.

4.3.3 Infiltration Rates

Infiltration rates may be measured using the modified Pilot Infiltration Test (PIT) presented in Appendix E. The measured, or "short-term", infiltration rate must be reduced using a correction factor to account for site variability and number of tests conducted, degree of long-term maintenance and influent pretreatment/ control, and potential for long-term clogging due to siltation and bio-buildup. The corrected infiltration rate is considered the "long-term" or "design" infiltration rate and is used for all BMP sizing calculations. Testing methods and correction factors are presented in detail in Appendix E.

In no case shall a correction factor less than 2.0 be used. In addition, the design infiltration rate (after correction factor) shall not exceed 10 inches per hour.

4.3.4 General Requirements for Site Assessment

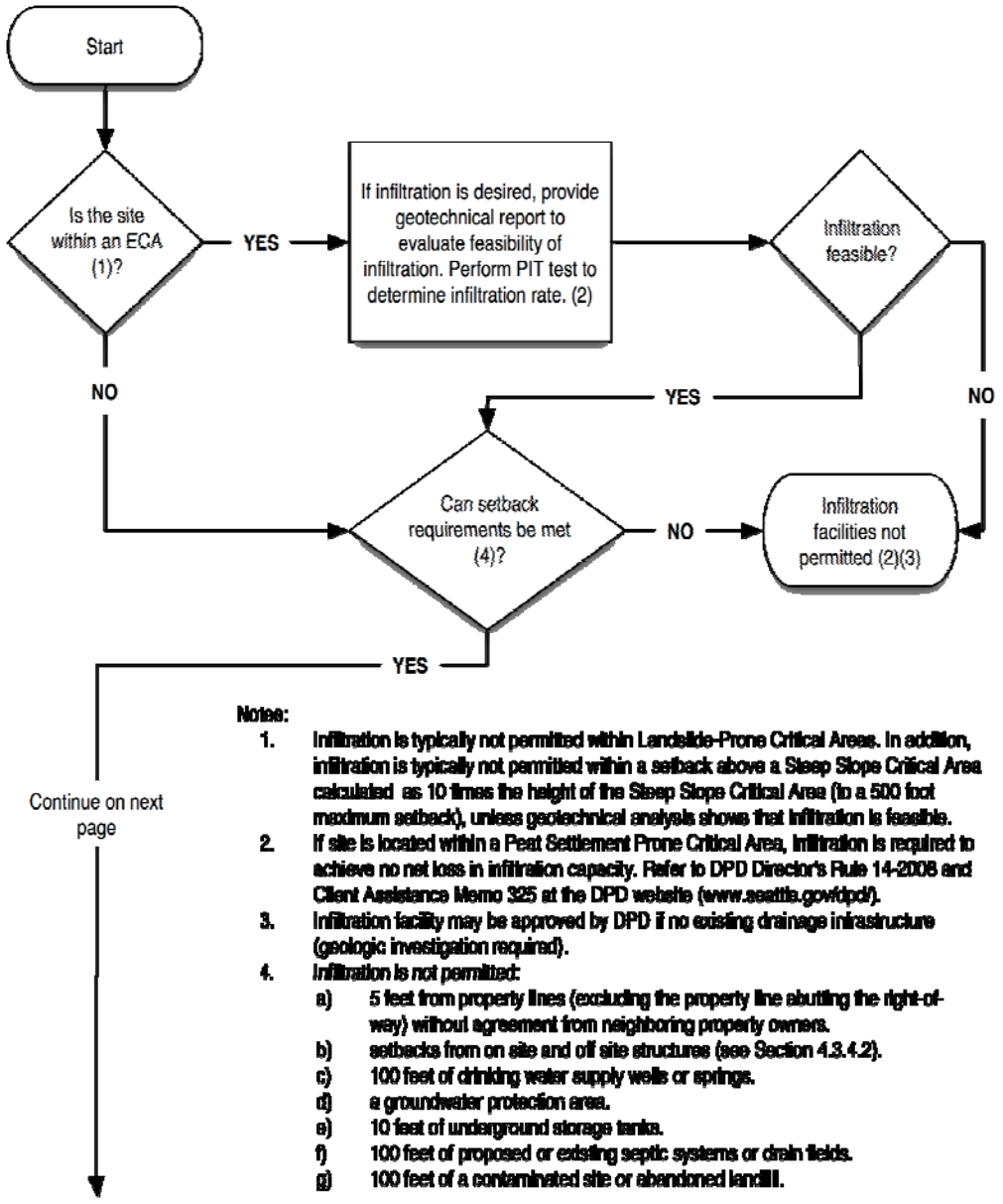
The following procedures must be followed when considering and designing an infiltration facility. Each step is outlined in more detail in the subsequent sections.

- **Step 1** – Conduct general site reconnaissance and review survey and other information to identify existing drinking water wells or aquifers, existing and proposed buildings, contaminated sites, sloped areas, and septic systems in the vicinity of the proposed facility.
- **Step 2** – Evaluate setback and site restrictions for infiltration facilities to determine whether infiltration is feasible for the site.
- **Step 3** – Based on the size of the project, determine the level of subsurface characterization that is required.
- **Step 4** – Complete Step 4a, Step 4b, or Step 4c, as applicable.
  - **Step 4a** – Conduct basic subsurface evaluation
  - **Step 4b** – Conduct detailed subsurface evaluation
  - **Step 4c** – Conduct in-depth subsurface evaluation.

4.3.4.1 Step 1: General Surface Characterization

The first step in designing an infiltration facility is to select a location, and assess the site suitability. The information to be reviewed as part of this initial site characterization may include:

- Topography within 500 feet of the proposed facility (GIS topographic data may be used)



Notes:

1. Infiltration is typically not permitted within Landslide-Prone Critical Areas. In addition, infiltration is typically not permitted within a setback above a Steep Slope Critical Area calculated as 10 times the height of the Steep Slope Critical Area (to a 500 foot maximum setback), unless geotechnical analysis shows that infiltration is feasible.
2. If site is located within a Peat Settlement Prone Critical Area, infiltration is required to achieve no net loss in infiltration capacity. Refer to DPD Director's Rule 14-2008 and Client Assistance Memo 325 at the DPD website (www.seattle.gov/dpd/).
3. Infiltration facility may be approved by DPD if no existing drainage infrastructure (geologic investigation required).
4. Infiltration is not permitted:
   a) 5 feet from property lines (excluding the property line abutting the right-of-way) without agreement from neighboring property owners.
   b) setbacks from on site and off site structures (see Section 4.3.4.2).
   c) 100 feet of drinking water supply wells or springs.
   d) a groundwater protection area.
   e) 10 feet of underground storage tanks.
   f) 100 feet of proposed or existing septic systems or drain fields.
   g) 100 feet of a contaminated site or abandoned landfill.

Figure 4.2. Infiltration Feasibility Flow Chart.

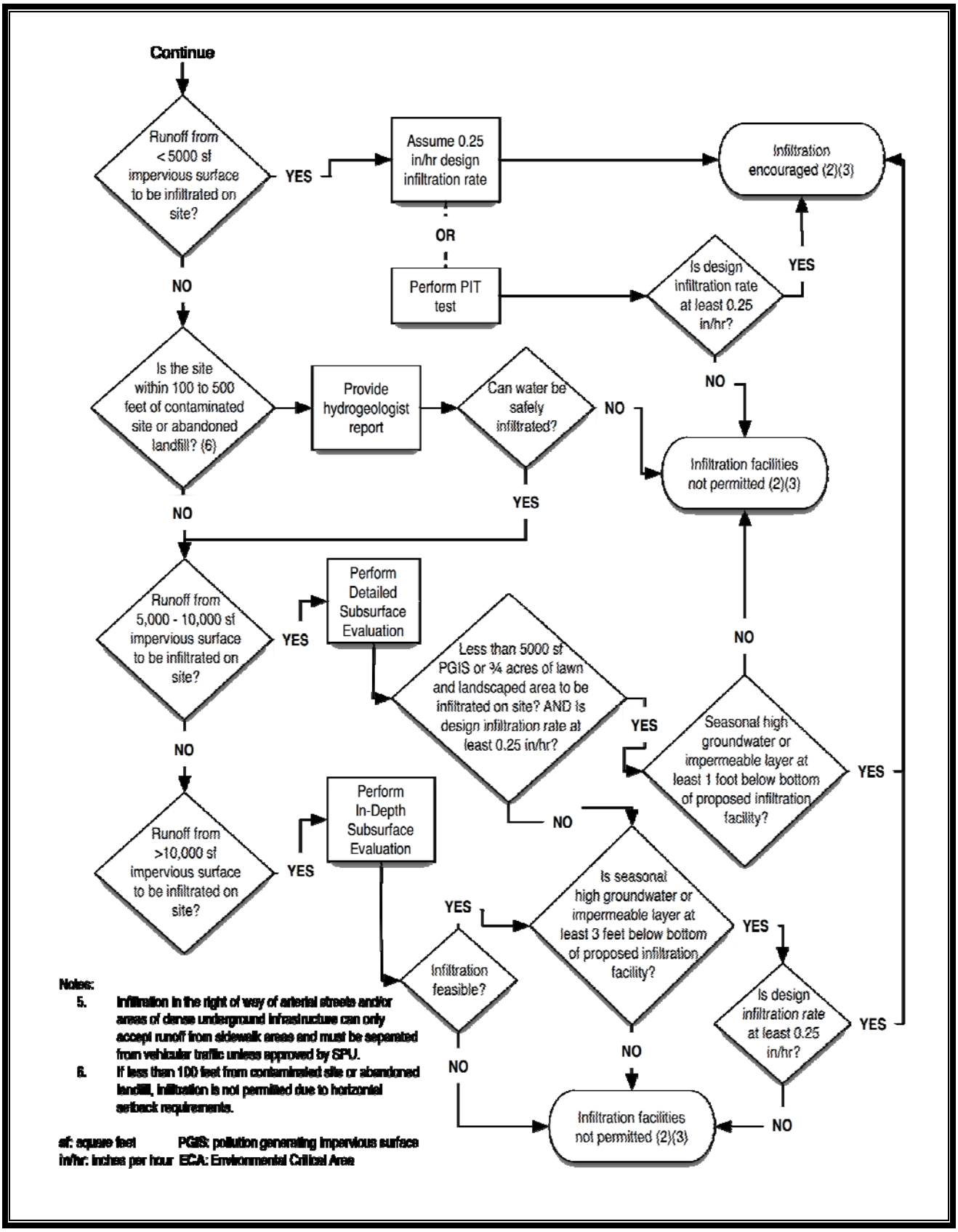


Figure 4.2 (continued). Infiltration Feasibility Flow Chart.

- Anticipated site use (street/highway, residential, commercial, high-use site)
- Location of water supply wells within 500 feet of proposed facility
- Location of ground water protection areas and/or 1-, 5-, and 10-year time of travel zones for municipal well protection areas
- Location of sloped areas or landslide hazard areas within 500 feet of the site
- Location of contaminated sites and abandoned landfills within 500 feet of the site
- Location of septic systems and drain fields in the vicinity of the proposed facility
- A description of local site geology, including soil or rock units likely to be encountered, the groundwater regime, and geologic history of the site.

4.3.4.2 Step 2: Setbacks and Site Restrictions

Prohibitions

The following list identifies conditions that limit or prohibit infiltration facilities.

- If design infiltration rate (after application of correction factor) is less than 0.25 inches per hour, infiltration facilities are typically not approved as a means to meet flow control or water quality treatment requirements.
- Infiltration is typically not permitted within Landslide–Prone Critical Areas as defined by the Regulations for Environmental Critical Areas (SMC 25.09.020).
- Infiltration is typically not permitted within a setback above a Steep Slope Critical Area (SMC 25.09.020). For slopes higher than 10 feet, the setback is calculated as 10 times the height of the Steep Slope Critical Area (to a 500 foot maximum setback). Infiltration within this setback may be feasible provided a detailed slope stability analysis is completed by a geotechnical engineer. The analysis shall determine the effects that infiltration would have on the Steep Slope Critical Area and adjacent properties.
- Other Environmental Critical Areas (ECA) may have infiltration restrictions. Infiltration within an ECA must be evaluated as part of an ECA application submittal (SMC 25.09).
- Unless approved by the Director, infiltration in the right of way of arterial streets and/or areas of dense underground infrastructure can only accept runoff from sidewalk areas and must be separated from vehicular traffic.

Horizontal Setbacks

The following list provides setbacks for infiltration facilities. Note that these setbacks do not apply to permeable pavement surfaces (that do not manage runoff from other areas). Setbacks are measured from the maximum ponding elevation before overflow.

- Infiltration is not permitted within 5 feet from property lines (excluding the property line abutting the right-of-way) without agreement from neighboring property owners.
- Infiltration is not permitted within the following setbacks from on site and off site structures:
  - When runoff from <5,000 square feet of new/replaced impervious area is infiltrated on site, the facility shall be not be within 5 feet from structure without basement and 10 feet from structure with basement.
  - When runoff from ≥5,000 square feet of new/replaced impervious area is infiltrated on site, a building structure shall not intersect with a 1H:1V slope from the bottom edge of an infiltration facility. The resulting setback shall be no less than 5 feet from a structure without basement and 10 feet from a structure with basement. For setbacks from structures on adjacent lots, potential structures should be considered for future build-out conditions.
  - Note: No setback from site structure is required if open draining crawl space or system is approved by geotechnical engineer.
- Infiltration is not permitted within 100 feet of drinking water supply wells or springs used for drinking water.
- Infiltration is not permitted within a groundwater protection area unless approved by the Director. If approved, water quality treatment may be required.
- Infiltration is not permitted within 10 feet of underground storage tank(s).
- Infiltration is not permitted within 100 feet of proposed or existing septic systems or drain fields.

Restrictions Specific to Contaminated Soil or Groundwater

Infiltration has the potential to mobilize contaminants present in soil and groundwater. Therefore, stormwater infiltration systems may not be constructed on sites where soil and/or groundwater contamination problems have been identified. In addition, infiltration is not permitted within 100 feet of a contaminated site or abandoned landfill. For projects where runoff from 5,000 square feet or more of impervious area will be infiltrated on site, infiltration within 500 feet of contaminated sites or abandoned landfills requires analysis and approval by a licensed hydrogeologist to determine whether stormwater can be safely infiltrated. Setbacks and exclusion areas shall be measured from the site

of the proposed infiltration facility to the nearest extent of contamination, or if not known, the edge of the parcel where contamination has been found.

Contaminated sites are regulated by the U.S. Environmental Protection Agency (EPA) and the Washington State Department of Ecology (Ecology). Construction near abandoned landfills is regulated by the City of Seattle Department of Planning and Development (unless known to be contaminated). EPA regulates contaminated sites under Superfund (CERCLA) and Resource Conservation and Recovery Act (RCRA) federal laws. The Superfund program regulates sites where contaminants have been released onto the ground, or into streams, rivers, lakes, and other water bodies as a result of current or historical practices. RCRA regulates the management and disposal of solid and hazardous waste at active and future facilities to protect human health and the environment by requiring that wastes are managed in an environmentally sound manner. EPA maintains a mapping tool that plots the locations of Superfund and RCRA-regulated sites (http://www.epa.gov/environ/emef/). To obtain information about whether specific sites have known groundwater contamination problems, go to the websites for the individual programs:

- EPA Superfund Program site list (http://www.epa.gov/superfund/sites/index.htm)
- EPA RCRA Program site list (http://www.epa.gov/epawaste/hazard/correctiveaction/facility/index.htm).

Ecology regulates contaminated sites (e.g., sediment, industrial sites, hazardous waste sites, and leaking underground storage tanks) under its Toxics Cleanup Program. Site locations can be found at Ecology's website: http://www.ecy.wa.gov/fs/. To obtain information about whether specific sites have groundwater contamination problems, go to the toxics Cleanup Program website (http://www.ecy.wa.gov/cleanup.html).

Vertical Setbacks

Infiltration facilities require a minimum vertical separation from the bottom of the facility to the underlying water table, bedrock or other impermeable layer. Separation requirements depend upon the facility's tributary area and characteristics as follows:

- A minimum of 3 feet of clearance is necessary between the facility bottom and the seasonal high groundwater elevation, bedrock ,or other impermeable layer if the area tributary to the facility meets or exceeds any of the following limitations:
  - 5,000 square feet of pollution-generating impervious surface
  - 10,000 square feet of impervious area
  - ¾ acres of lawn and landscaped area.

- If the tributary area to an individual facility does not exceed the area limitations above, a minimum of 1 foot of clearance is adequate between the lowest elevation of the facility and the seasonal high groundwater elevation or other impermeable layer.

4.3.4.3 Step 3: Level of Subsurface Characterization

The City of Seattle encourages consideration of infiltration facilities for sites where conditions are appropriate. However, to ensure proper performance of the infiltration facility, the City requires a greater level of analysis and site characterization for sites proposing to infiltrate large volumes of water. Therefore, the following thresholds apply to infiltration facility evaluations:

- For projects where runoff from <5,000 square feet of impervious area will be infiltrated on site, see Step 4a below for the basic subsurface evaluation.
- For projects where runoff from ≥5,000 but <10,000 square feet of impervious area will be infiltrated on site, see Step 4b below for the detailed subsurface evaluation.
- For projects where runoff from ≥10,000 square feet of impervious area will be infiltrated on site, see Step 4c below for the in-depth subsurface evaluation.

Note that the thresholds listed above are for the total area for which runoff is infiltrated on site, regardless of the number of infiltration facilities.

4.3.4.4 Step 4a: Basic Subsurface Evaluation

The following analyses are required for all projects where runoff from <5,000 square feet of impervious area will be infiltrated on site.

Design Infiltration Rate

For sidewalk/trail projects and single family residential projects the design infiltration rate shall be determined by one of the following methods:

- A 0.25 inches per hour design infiltration rate may be assumed.
- A design infiltration rate greater than 0.25 inches per hour may be determined by performing the modified PIT method (with correction factors applied) as described in Section 4.3.3 and Appendix E.

For all other projects the design infiltration rate shall be determined by performing the modified PIT method and applying correction factors as described in Section 4.3.3 and Appendix E.

PIT test reports shall be stamped by a Professional Engineer or prepared by an on-site wastewater treatment designer licensed with the State of Washington.

4.3.4.5 Step 4b: Detailed Subsurface Evaluation

The following analyses are required for all infiltration projects where 5,000 to 10,000 square feet of impervious area will be infiltrated on site.

Design Infiltration Rate

The design infiltration rate shall be determined by performing the modified PIT method and applying correction factors as described in Section 4.3.3 and Appendix E.

Seasonal High Groundwater Levels

Test hole or test pit explorations shall be conducted or peizometer data shall be collected during mid to late in the wet season (mid January through April) to provide accurate groundwater elevation information.

4.3.4.6 Step 4c: In-Depth Subsurface Evaluation

For all projects where ≥10,000 square feet of impervious area will be infiltrated on site, a detailed subsurface evaluation shall be performed as described in Section 4.3.4.5. In addition, the infiltration receptor shall be characterized and groundwater monitoring shall be performed as described below.

Characterize Infiltration Receptors

Characterization of the infiltration receptor shall include assessment and documentation of the following elements:

- Depth to groundwater and to bedrock/impermeable layers
- Seasonal variation of groundwater table based on well water levels and observed mottling of soils
- Existing groundwater flow direction and gradient
- Volumetric water holding capacity of the infiltration receptor soils. The volumetric water holding capacity is the storage volume in the soil layer directly below the infiltration facility and above the seasonal high groundwater mark, bedrock, hardpan, or other low permeability layer.
- Horizontal hydraulic conductivity of the saturated zone to assess the aquifer's ability to laterally transport the infiltrated water
- Approximation of the lateral extent of infiltration receptor
- Impact of the infiltration rate and proposed added volume from the project site on local groundwater mounding, flow direction, and water table determined by hydrogeologic methods.

Monitor Groundwater Levels

A minimum of three groundwater monitoring wells shall be installed per infiltration facility to establish a three-dimensional relationship for the groundwater table,

unless the highest groundwater level is known to be at least 50 feet below the proposed base of the infiltration facility. Seasonal groundwater levels must be monitored at the site during at least one wet season (mid January through April).

4.3.5 Verification Testing

The City may require verification testing for infiltration facilities serving greater than 5,000 square feet of contributing area and where the City determines there may be a risk of infiltration system failure. Site conditions that justify infiltration facility verification testing include but are not limited to: low infiltration capacity soils, history of infiltration failure in the project area, high groundwater levels, risk of flooding in the event of system failure, indications of sediment loads to the facility during construction, indications of soil compaction during construction, new information gained during construction with regards to infiltration facility design and performance (e.g., better soils data, groundwater data, etc.).

4.3.6 General Requirements for Infiltration Facility Sizing

The Pre-Sized Approach for sizing infiltration facilities may be used when the new and replaced impervious surface associated with a project does not exceed 10,000 square feet. Sizing factors for selected infiltration facilities are provided for flow control and water quality treatment in sections 4.2.2 and 5.8.5, respectively.

For larger sites, or to use BMPs or design configurations not included in the pre-sized approach, infiltration facilities must be sized to meet flow control and/or water quality treatment requirements using an approved continuous hydrologic model (see Chapter 6, Section 6.2.3). Below are the procedures for sizing an infiltration facility to (A) to infiltrate 100 percent of runoff; (B) to meet the water quality treatment requirements, and (C) to meet flow peak and/or duration control standard.

(A) For 100 percent infiltration:

- Input dimensions of your infiltration facility
- Input design infiltration rate (measured infiltration rate with rate correction factor applied)
- Input a riser height and diameter to represent the facility overflow conditions (any flow through the riser indicates that you have less than 100 percent infiltration and must increase your infiltration pond dimensions)
- Run the model and review the model-reported percentage of runoff infiltrated. If less than 100 percent infiltrated, increase pond dimension until you achieve 100 percent infiltration. There is no need to check duration when infiltrating 100 percent of the full continuous record runoff file.

(B) For 91 percent infiltration (water quality treatment requirement):

The procedure is the same as above, except that your target is to infiltrate 91 percent of the influent runoff volume. In addition, to prevent the onset of anaerobic conditions, an infiltration facility designed for water quality treatment purposes must be designed to drain the water quality design treatment volume within 48 hours. This can be calculated by using a horizontal projection of the infiltration basin mid-depth dimension and the estimated long-term infiltration rate. See also the infiltration soil requirements for treatment outlined in Section 5.8.4.

Infiltration facilities for treatment can be located upstream or downstream of detention and can be off-line or on-line (see Section 5.4.1). See Section 5.4.5 for flow splitter design details.

(C) To meet flow peak and/or duration standard with infiltration:

This design will allow something less than 100 percent infiltration as long as any facility overflows meet the numerical peak and/or duration standards outlined in Chapter 2 and Section 4.1.4. Set up the model as explained for 100 percent infiltration. Run the model and review the flow duration and flow frequency results to determine if the standard is achieved.

4.3.7 Infiltration Facilities and Underground Injection Control

In addition to the requirements presented in this section, infiltration is regulated by the Department of Ecology and the Underground Injection Control (UIC) Program (WAC 173-218). The following information on Underground Injection Control (UIC) is primarily excerpted from the 2006 Department of Ecology document titled *Guidance for UIC Wells that Manage Stormwater* (Ecology 2006). Refer to the Department of Ecology website for updates and revisions. Project proponents are responsible for all regulatory compliance and permitting for each element they choose to implement.

"The UIC program in the state of Washington is administered by the Department of Ecology. In 1984, the Department of Ecology adopted Chapter 173-218 WAC - Underground Injection Control to implement the program… A UIC well is a manmade subsurface fluid distribution system designed to discharge fluids into the ground and consists of an assemblage of perforated pipes, drain tiles, or other similar mechanisms, or a dug hole that is deeper than the largest surface dimension (WAC 173-218-030)." The person responsible for the infiltration facility (i.e., not the City for private systems) must determine whether the facility is a regulated well and what requirements apply. The City does not advise on the application of Ecology's UIC program or its requirements.

UIC systems include drywells, pipe or french drains, drain fields, and other similar devices that are used to discharge stormwater directly into the ground… Infiltration trenches with perforated or slotted pipe used to disperse and inject flows (as opposed to collect and route to surface drainage, as in an underdrain)

are considered to be UIC wells. This type of infiltration trench must be registered with Ecology.

The following may not be UIC wells; therefore, Ecology's requirements may not apply:

- Buried pipe and/or tile networks that serve to collect water and discharge that water to a conveyance system or to surface water
- Surface infiltration basins and flow dispersion stormwater infiltration facilities
- Infiltration trenches designed without perforated/slotted pipe or a similar mechanism
- A system receiving roof runoff from a single family home.

"The two basic requirements of the UIC Program are:

1. Register UIC wells with the Washington State Department of Ecology unless the wells are located on tribal land. (Those wells should be registered with the Environmental Protection Agency.)
2. Make sure that current and future underground sources of ground water are not endangered by pollutants in the discharge (non-endangerment standard).

"UIC wells must either be rule-authorized or covered by a state waste discharge permit to operate. If a UIC well is rule-authorized, a permit is not required. Rule-authorization can be rescinded if a UIC well no longer meets the non-endangerment standard. Ecology can also require corrective action or closure of a UIC well that is not in compliance."

4.4 Green Stormwater Infrastructure

Green stormwater infrastructure (GSI) includes stormwater best management practices designed to reduce runoff from development using infiltration, evapotranspiration, or stormwater reuse. Examples of green stormwater infrastructure include permeable pavement, bioretention facilities, and green roofs (SMC 22.801.080). Other examples of GSI include trees, rainwater harvesting, and bioretention planters with underdrains.

Note that although infiltration basins, trenches, and drywells utilize infiltration for flow control, they are not considered green stormwater infrastructure in this manual. Because green stormwater infrastructure includes some stormwater management techniques that are relatively new, updates and supplemental information will be posted on SPU green stormwater infrastructure website (http://www.seattle.gov/util/greeninfrastructure).

The following sections outline design guidelines and criteria for these green stormwater infrastructure BMPs. These facilities may be designed using either the Pre-Sized Approach or the Modeling Approach (Section 4.2). Note that per SMC 22.803.020 Sections C and E and 22.805.080.C, all facilities shall be maintained per the requirements outlined in this manual. Detailed maintenance requirements for each Green Stormwater Infrastructure BMP are provided in Appendix D.

Information presented in this section builds upon guidance provided in the *Low Impact Development Technical Guidance Manual* for Puget Sound prepared by the Washington State University Pierce County Extension and the Puget Sound Action Team (WSU 2005), and the City of Seattle would like to express its thanks for this contribution.

4.4.1 Post Construction Soil Quality and Depth

Undisturbed soil, soil organisms, and vegetation provide important stormwater management functions including water infiltration; nutrient, sediment, and pollutant adsorption; sediment and pollutant biofiltration; water interflow storage and transmission; and pollutant decomposition. These functions are largely lost when development strips away native soil and vegetation and replaces it with minimal soil and sod. Not only are these important stormwater management functions lost, but such landscapes themselves can become pollution-generating pervious surfaces due to compaction; increased use of pesticides, fertilizers, and other landscaping and household/industrial chemicals; the concentration of pet wastes; and pollutants that accompany roadside litter.

While establishing a minimum soil quality and depth is not the same as preserving undisturbed soil and vegetation, it will provide improved onsite management of stormwater flow and water quality. Flow control standards can be met when post construction quality and depth requirements are integrated into a stormwater dispersion design. Amending disturbed soils with compost in the post development landscape also re-establishes a healthy soil ecosystem which provides increased treatment of pollutants and sediments that result from development and habitation and minimizes the need for some landscaping chemicals, thus reducing pollution through prevention.

Typical cross sections of compost-amended soil in planting bed and turf applications are shown in Figures 4.3 and 4.4, respectively.

Additional guidance for this BMP can be found in *Building Soil: Guidelines and Resources for Implementing Soil Quality and Depth BMP T5.13 in WDOE Stormwater Management Manual for Western Washington*, which is available at www.soilsforsalmon.org under the “How-To” section, or at www.buildingsoil.org.

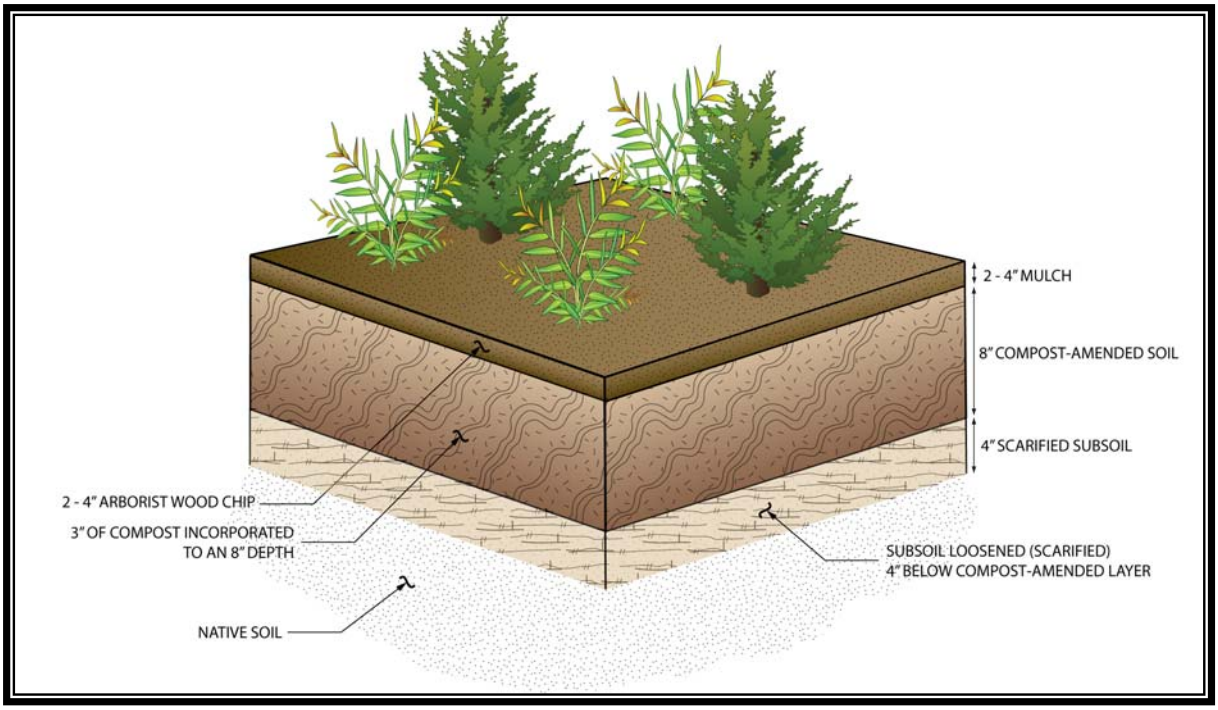


Figure 4.3. Cross Section of Planting Bed Soil Amendment.

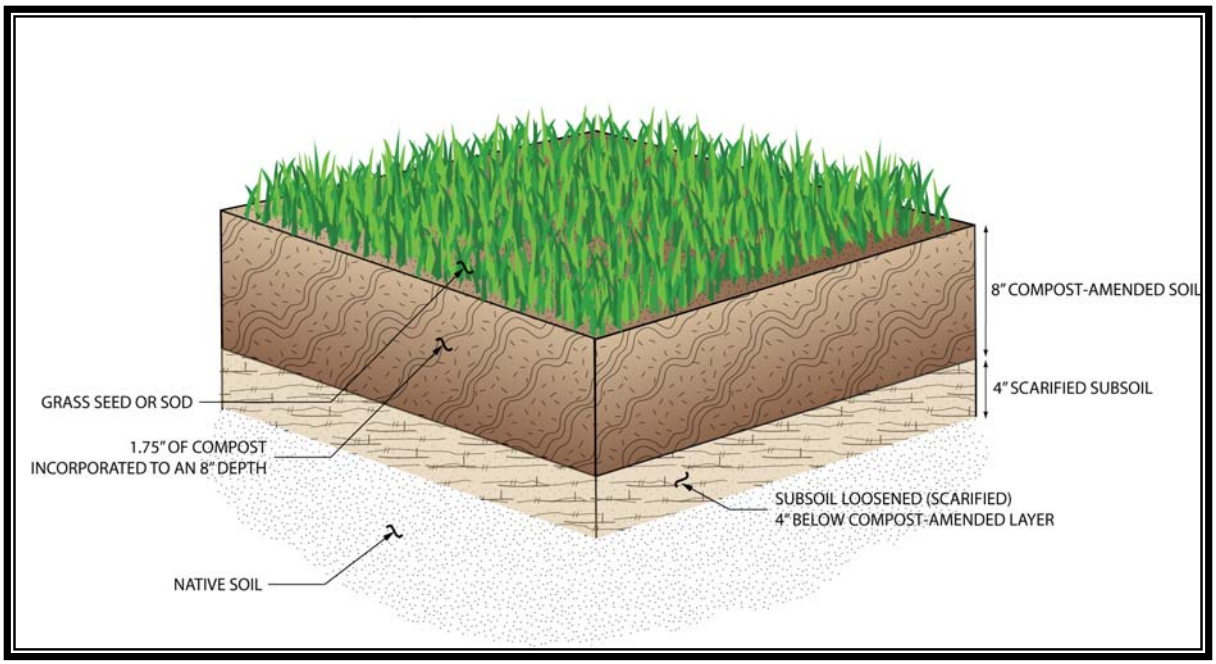


Figure 4.4. Cross Section of Turf Soil Amendment.

4.4.1.1 Applications and Limitations

All areas subject to clearing, grading, or compaction that have not been covered by impervious surface, incorporated into a drainage facility, or engineered as structural fill or slope shall, at project completion, meet post construction soil

quality and depth requirements. Only the areas of the sites where existing vegetation and/or soil are disturbed or compacted must be restored.

4.4.1.2 Design Criteria

Implementation Options

The soil quality design requirements can be met by using one of the four options listed below:

1) Leave undisturbed vegetation and soil, and protect from compaction by fencing and keeping materials storage and equipment off these areas during construction

For all areas where soil or vegetation are disturbed, use option 2, 3 or 4.

2) Amend existing site topsoil or subsoil either at default "pre-approved" rates, or at custom calculated rates to meet the soil quality guidelines based on engineers' tests of the soil and amendment. The default pre-approved rates are:
   - In planting beds, place 3 inches of compost and till in to an 8 inch depth
   - In turf areas, place 1.75 inches of compost and till in to an 8 inch depth
   - Subsoil shall be scarified (loosened) 4 inches below amended layer, to produce 12-inch depth of un-compacted soil.
   - After planting, 2 to 4 inches of arborist wood chip or compost mulch shall be applied to planting beds. Coarse bark mulch may be used but has lower benefits to plants and soil. Fine bark should not be used because it can seal the soil surface.

3) Stockpile existing topsoil during grading, and replace it prior to planting. Stockpiled topsoil must also be amended if needed to meet the organic matter or depth requirements, either at the default "pre-approved" rate or at a custom calculated rate (see Building Soil manual or website, above, for custom calculation method). Scarify subsoil, and mulch planting beds, as described in option (2) above.

4) Import topsoil mix of sufficient organic content and depth to meet the requirements. Imported soils should not contain excessive clay or silt fines (more than 5 percent passing the No. 200 sieve) because that could restrict stormwater infiltration. The default pre-approved rates for imported topsoils are:
   - For planting beds, a mix by volume of 35 percent compost with 65 percent mineral soil is pre-approved to achieve the requirement of a minimum 8 percent (target 10 percent) organic matter by loss-on-ignition test

- For turf areas, a mix by volume of 20 percent compost with 80 percent mineral soil is pre-approved to achieve the requirement of a minimum 4 percent (target 5 percent) organic matter by loss-on-ignition test.
- Scarify subsoil, and mulch planting beds, as described in option (2) above.

More than one method may be used on different portions of the same site.

Soil Retention

The duff layer and native topsoil should be retained in an undisturbed state and protected from compaction to the maximum extent feasible (SMC 22.805.020. D.2). In any areas requiring grading, remove and stockpile the duff layer and topsoil on site in a designated, controlled area, not adjacent to public resources and critical areas, to be reapplied to other portions of the site where feasible.

Soil Quality

Soil organic matter is often missing from disturbed soils. Organic matter may be replenished by amending with compost. It is important that the materials used to meet the soil quality and depth BMP be appropriate and beneficial to the plant cover to be established. Likewise, it is important that imported topsoils improve soil conditions and do not have an excessive percent of clay or silt fines.

Standardized "pre-approved" soil amendment rates have been established for planting beds and turf areas. Alternatively, custom amendment rates may be calculated. Both options are described in further detail in the subsequent section.

All areas subject to clearing and grading that have not been covered by impervious surface, incorporated into a drainage facility, or engineered as structural fill or slope shall, at project completion, demonstrate the following:

- A topsoil layer meeting these requirements:
  - Topsoil shall have a an organic matter content by the loss-on-ignition test of a minimum 8 percent (target 10 percent) dry weight in planting beds, and a minimum 4 percent (target 5 percent) organic matter content in turf areas, and a pH from 6.0 to 8.0 or matching the pH of the original undisturbed soil[1]
  - The topsoil layer shall have a minimum depth of 8 inches
  - Where tree roots limit the depth of incorporation of amendments, those root zones are exempted from this requirement only if they are fenced and protected from stripping of soil, grading, or compaction to the maximum extent practical

[1] Acceptable test methods for determining loss-on-ignition soil organic matter include the most current version of ASTM D2974 "Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils" and TMECC 05.07A "Loss-On-Ignition Organic Matter Method"

  - Subsoils below the topsoil layer shall be scarified at least 4 inches, for a finished minimum depth of 12 inches of uncompacted soil, with some incorporation of the upper material to avoid stratified layers, where feasible.

- Planting beds must be mulched after planting with 2 to 4 inches of organic material such as arborist wood chips, bark, shredded leaves, compost, etc.
- Quality of compost and other materials used shall meet the organic content requirements:
  - The organic content for "pre-approved" amendment rates can be met only using compost that meets the definition of "composted materials" in WAC 173-350 section 220. This code is available at the Dept. of Ecology's website (http://www.ecy.wa.gov/). The compost must also have an organic matter content of 40 percent to 65 percent, and a carbon to nitrogen ratio below 25:1. The carbon to nitrogen ratio may be as high as 35:1 for plantings composed entirely of plants native to the Puget Sound Lowlands region.
  - Calculated amendment rates may be met through use of composted materials as defined above; or other organic materials amended to meet the carbon to nitrogen ratio requirements, and meeting the contaminant standards of specified in WAC 173-350 section 220.
  - The method for calculating custom amendment rates is established in the *"Building Soil"* manual referenced above and below.

The resulting soil should be conducive to the type of vegetation to be established.

Soil Management Plan

A "Soil Management Plan" is required, including:

- A site map showing areas to be fenced and left undisturbed during construction, and areas that will be amended at the turf or planting bed rates.
- Calculations of the amounts of compost, compost amended topsoil, and mulch to be used on the site.
- Sample forms for the Soil Management Plan, and more guidance on these procedures, can be found in the *Building Soil* manual, available on the www.soilsforsalmon.org website.

4.4.1.3 Flow Control Credits

Flow control credit can be taken in runoff modeling when Post Construction Soil Quality and Depth requirements are met and used as part of a dispersion design under the conditions described in:

- Downspout Dispersion via Splash Blocks (Section 4.4.3)
- Downspout Dispersion via Dispersion Trenches (Section 4.4.3)
- Sheet Flow Dispersion (Section 4.4.4).

4.4.1.4 Construction Specifications and Criteria

Minimum construction requirements for disturbed areas include the following:

- Soil quality and depth must be established toward the end of construction and once established, must be protected from compaction and erosion.
- Soil must be planted and planting beds must be mulched after installation.

For inspection and verification procedures used for facility approval by City staff, see GSI web site (http://www.seattle.gov/util/greeninfrastructure). These procedures are also summarized in the *Building Soil* manual referenced above.

4.4.1.5 Operations and Maintenance Requirements

Operations and maintenance requirements for compost amended soil are provided in Appendix D. Most important is that organic matter be replenished by leaving leaf litter and grass clippings on-site (or by adding compost and mulch regularly. This BMP is designed to reduce use of irrigation, fertilizers, herbicides, and pesticides. These activities should be adjusted where possible, rather than continuing to implement formerly established practices. In particular, regular use of soluble fertilizers, herbicides, and insecticides degrades soil life and compacts soils. Instead, fertilization can be eliminated by retaining leaf litter in landscape beds and mulch mowing. If soils are nutrient deficient, fertilizations can be reduced by using slow-release or organic products. Integrated pest management techniques will minimize the need for pesticides.

4.4.2 Tree Planting and Retention

Trees provide flow control via interception, transpiration, and increased infiltration. Additional environmental benefits include improved air quality, carbon sequestration, reduced heat island effect, pollutant removal, and habitat preservation or formation.

When implemented in accordance with the criteria outlined below, retained and newly planted trees receive credits toward meeting flow control requirements. The degree of flow control provided by a tree depends on the tree type (i.e., evergreen or deciduous), canopy area, and whether or not the tree canopy overhangs impervious surfaces. Flow control credits may be applied to project sites of all sizes.

4.4.2.1 Applications and Limitations

Trees are a landscape amenity with flow control benefits that can be applied in most settings. Flow control credit is given for retaining or planting trees within 20 feet of ground level impervious surfaces such as driveways, patios, and parking lots.

Trees planted or retained to meet flow control requirements may also count toward Green Factor, landscaping, and/or tree protection requirements.

4.4.2.2 Retained Trees

Site Considerations

Setbacks of proposed infrastructure from existing trees are critical considerations. Tree protection requirements limit grading and other disturbances in proximity to the tree.

Design Criteria

The following provides requirements and recommendations associated with tree retention for flow control credit. Submittal for review shall include the existing tree species, trunk diameter, canopy area (and dripline delineation), and location (relative to ground level impervious surfaces) on the plan set.

Tree Species and Condition

Existing tree species and location must be clearly shown on submittal drawings. Trees must be viable for long-term retention (i.e., in good health and compatible with proposed construction).

Tree Size

To receive flow control credit, retained trees shall have a minimum 6 inches diameter at breast height (DBH). DBH is defined as the outside bark diameter at 4.5 feet above the ground on the uphill side of a tree. For existing trees smaller than this, the newly planted tree credit may be applied as presented in Section 4.4.2.3.

The retained tree canopy area shall be measured as the area within the tree drip line. A drip line is the line encircling the base of a tree, which is delineated by a vertical line extending from the outer limit of a tree's branch tips down to the ground (see City of Seattle Standard Plan 133). If trees are clustered, overlapping canopies are not double counted.

Tree Location

Flow control credit for retained trees depends upon proximity to ground level impervious surfaces. To receive a credit, the existing tree must be on the development site and within 20 feet of new and/or replaced ground level

impervious surfaces (e.g., driveway or patio) on the development site. For single family residential projects, the higher credit is also given for trees that are 20 feet or less from existing ground level impervious surfaces and in the right-of-way (e.g., sidewalk). Distance from impervious surfaces is measured from the tree trunk center.

An arborist report may be required if impervious surface is proposed within the critical root zone of the existing tree. The critical root zone is defined as the line encircling the base of the tree with half the diameter of the dripline (see City of Seattle Standard Plan 133). If the arborist report concludes that impervious surface should not be placed within 20 feet of the tree and canopy overlap with impervious surface is still anticipated given a longer setback, the higher tree flow control credit may be approved.

Trees planted in planter boxes are not eligible for flow control credit.

Tree Protection Measures during Construction

The existing tree roots, trunk, and canopy shall be fenced and protected during construction activities per Seattle Municipal Code Tree Protection Chapter 25.11, City of Seattle Standard Specification 1-07.16(2) and City of Seattle Standard Plans 132, 133, and 134.

Long-term Tree Retention and Protection

Trees shall be retained, maintained and protected on the site after construction and for the life of the development or until any approved redevelopment occurs in the future. Trees that are removed or die shall be replaced with like species during the next planting season (typically in fall). Trees shall be pruned according to industry standards (ANSI A 300 standards).

Flow Control Credit

Flow control credits for retained trees are provided in Table 4.8 by tree type. These credits can be applied to reduce impervious surface area requiring flow control. Credits are given as a percentage of the existing tree canopy area. The minimum credit for existing trees ranges from 50 to 100 square feet. Credits are the same for all flow control standards and may be applied to project sites of all sizes.

Table 4.8. Flow Control Credits for Retained Trees.

Tree Type	Credit
Evergreen	20% of canopy area (minimum of 100 square feet/tree)
Deciduous	10% of canopy area (minimum of 50 square feet/tree)

Impervious Area Mitigated = Σ Canopy Area x Credit (%)/100.
%- percent

To use these credits, the retained tree and protection measures must meet the requirements outlined in this section. The tree trunk center must be within 20 feet from a ground level impervious surface as explained in "Tree Location" section above.

Tree credits are not applicable to trees in native vegetation areas used for flow dispersion or other flow control credit. The total tree credit for retained and newly planted trees shall not exceed 25 percent of impervious surface requiring mitigation.

Operations and Maintenance Requirements

Operations and maintenance requirements for trees are provided in Appendix D.

4.4.2.3 Newly Planted Trees

Site Considerations

Mature tree height, size, and rooting depth must be considered to ensure that the tree location is appropriate given adjacent and above- and below-ground infrastructure. Although setbacks will vary by species, some general recommendations are presented below.

- Minimum 5 foot setback from structures
- Minimum 5 foot setback from underground utility lines
- Minimum 2 foot setback from edge of any paved surface.

Design Criteria

The following provides requirements and recommendations associated with tree planting for flow control credit. Submittal for review shall include the tree species, tree size (diameter or height), and tree location (with setbacks from ground level impervious surfaces structures and belowground utilities) on the plan set.

Tree Species

Approved tree species are listed in the City of Seattle Tree List available via link from the SPU GSI web site (http://www.seattle.gov/util/greeninfrastructure). Trees in the "small" category are not eligible for flow control credit. Tree species not included on the City of Seattle Tree List may be given credit with prior approval by the Director.

Tree Size

To receive flow control credit, new deciduous trees shall be at least 1.5 inches in diameter measured 6 inches above the ground. New evergreen trees shall be at least 4 feet tall.

Tree Location

Trees shall be sited according to sun, soil, and moisture requirements. Planting locations shall be selected to ensure that sight distances and appropriate setbacks are maintained given mature height, size, and rooting depths.

Similar to retained trees, flow control credit for newly planted trees depends upon proximity to ground level impervious surfaces. To receive a credit, the tree must be planted on the development site and within 20 feet of new and/or replaced ground level impervious surfaces (e.g., driveway, patio, or parking lot). For single family residential projects, the credit is also given for trees that are 20 feet or less from existing ground level impervious surfaces in the right-of-way (e.g., sidewalk). Distance from impervious surfaces is measured from the edge of the surface to the center of the tree at ground level.

To help ensure tree survival and canopy coverage, the minimum tree spacing for newly planted trees shall accommodate mature tree spread (see City of Seattle Tree List). In no circumstance shall flow control credit be given for new tree density exceeding 10 feet on center spacing.

Trees planted in planter boxes are not eligible for flow control credit.

Plant Material and Planting Specifications

Recommended guidelines for planting materials and methods are provided in City of Seattle Standard Specifications 8-02 and 9-14, and Standard Plans 100a, 100b, and 101.

Irrigation

Provisions shall be made for supplemental irrigation during the first three growing seasons after installation to help ensure tree survival.

Long-term Tree Retention and Protection

Trees shall be retained, maintained and protected on the site after construction and for the life of the development as required for retained trees (Section 4.4.2.2).

Flow Control Credit

Flow control credits for newly planted trees are provided in Table 4.9 by tree type. These credits can be applied to reduce the impervious surface area requiring flow control. Credits range from 20 to 50 square feet per tree. Credits are the same for all flow control standards and may be applied to project sites of all sizes.

Table 4.9. Flow Control Credits for Newly Planted Trees.

Tree Type	Credit
Evergreen	50 square feet/tree
Deciduous	20 square feet/tree

Impervious Area Mitigated = Σ Number of Trees x Credit (%)/100.
%- percent

To use these credits, the newly planted tree and planting methods must meet the requirements outlined in this section. The tree trunk center must be planted within 20 feet from a ground level impervious surface as explained in "Tree Location" section above.

Tree credits are not applicable to trees in native vegetation areas used for flow dispersion or other flow control credit. The total tree credit for retained and newly planted trees shall not exceed 25 percent of impervious surface requiring mitigation.

Operations and Maintenance Requirements

Operations and maintenance requirements for trees are provided in Appendix D.

4.4.3 Downspout Dispersion

Downspout dispersion BMPs are splashblocks or gravel-filled trenches that serve to spread roof runoff over vegetated pervious areas. Dispersion attenuates peak flows by slowing entry of the runoff into the conveyance system, allows for some infiltration, and provides some water quality benefits.

4.4.3.1 Applications and Limitations

Downspout dispersion BMPs generally require large areas of vegetated ground cover and are not feasible in most urban settings. Downspout disconnection or dispersion is not allowed in situations where the disconnection might cause erosion or flooding problems, either on site or on adjacent lots. While downspout dispersion is recommended for sites where downspout disconnection is feasible, credit for the roof area draining to the dispersion area (see below) is only obtained when the dispersion area meets the specific BMP sizing and design criteria outlined below.

If the designer wishes to use the dispersion BMP for water quality treatment in addition to flow control, the facility must meet any additional design requirements for filter strips per Section 5.7.

Two options for downspout dispersion are outlined below: splashblocks and gravel-filled dispersion trenches.

4.4.3.2 Design Criteria for Splashblocks

Downspout Discharge Point

This BMP is comprised by a splashblock, a downslope flow path of at least 50 feet and overflow conveyance. There are two approved methods for dispersion at the downspout discharge point: splashblocks and splashblocks with downspout extensions (Figure 4.5). In general, if the ground is sloped away from the foundation, and there is adequate vegetation and area for effective dispersion, splashblocks will adequately disperse storm runoff. If the ground is fairly level, if the structure includes a basement, or if foundation drains are proposed, splashblocks with downspout extensions may be a better choice because the discharge point is moved away from the foundation. Downspout extensions can include piping to a splashblock/discharge point a considerable distance from the downspout.

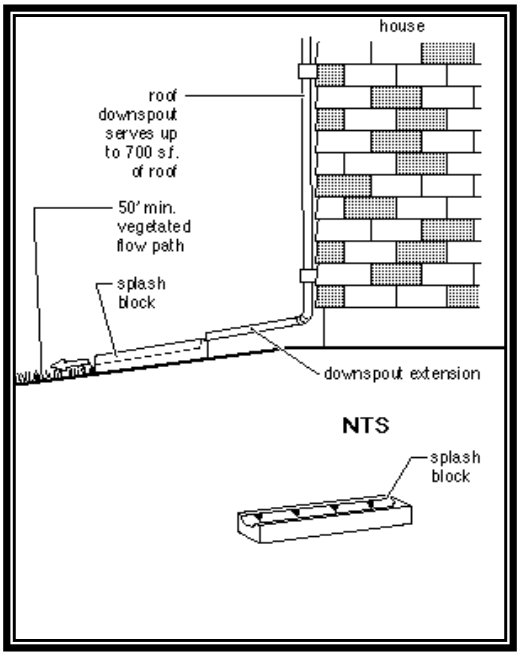


Figure 4.5. Typical Downspout Splashblock Dispersion.

Minimum requirements associated with the downspout discharge point include the following:

- A maximum of 700 square feet of roof area may drain to each splashblock
- A splashblock or a pad of crushed rock (2 feet wide by 3 feet long by 6 inches deep) shall be placed at each downspout discharge point.

Dispersion Area

Minimum requirements associated with dispersion area design include the following:

- A vegetated flowpath of at least 50 feet must be maintained between the discharge point and any property line, structure, slope, stream, wetland, lake, or other impervious surface.
- To achieve flow control credit, each downspout discharge point shall have a separate flowpath. To maintain adequate separation of flows from adjacent dispersion points, each flow path shall be at least 50 feet from an adjacent flow path segment at the downstream end of which ever segment is shorter.
- The vegetated flowpath must be covered with well-established lawn or landscape area (in accordance with the amended soil requirements outlined in Section 4.4.1), landscaping with well-established groundcover, or native vegetation with natural groundcover. The groundcover shall be dense enough to help disperse and infiltrate flows and to prevent erosion.
- Dispersion is typically not permitted within Landslide–Prone Critical Areas as defined by the Regulations for Environmental Critical Areas (SMC 25.09.020).
- Dispersion is typically not permitted within a setback above a Steep Slope Critical Area (SMC 25.09.020). The setback is calculated as 10 times the height of the Steep Slope Critical Area (to a 500 foot maximum setback). Infiltration within this setback may be feasible provided a detailed slope stability analysis is completed by a geotechnical engineer. The analysis shall determine the effects that dispersion would have on the Steep Slope Critical Area and adjacent properties.
- Dispersion is typically not permitted over contaminated sites or abandoned landfills.
- For sites with septic systems, the discharge point must be downgradient of the drainfield primary and reserve areas. This requirement may be waived if site topography clearly prohibits flows from intersecting the drainfield.

Overflow Conveyance

Minimum requirements associated with overflow conveyance design include the following:

- Dispersion area shall convey excess flow to an approved discharge point per Section 4.2.5. Conveyance of large storms shall be considered.
- No erosion or flooding of downstream properties may result.

4.4.3.3 Design Criteria for Dispersion Trenches

As an alternative to splashblocks, dispersion trenches may be used for downspout dispersion applications. This BMP is comprised by a dispersion trench, a downslope flow path of at least 25 feet and overflow conveyance.

Dispersion Trench

Minimum requirements associated with dispersion trench design include the following:

- Trench shall be a minimum of 18 inches deep and 2 feet wide.
- Trench shall be level and aligned parallel to site contours to disperse water to downslope flow path. Trench shall be constructed to prevent point discharge and erosion.
- Water shall be delivered to the trench via a perforated or slotted pipe with a minimum diameter of 4 inches. Pipe cover shall be a minimum of 6 inches.
- Trenches serving up to 700 square feet of roof area may be 10-foot-long by 2-foot-wide gravel filled trenches. For roof areas larger than 700 square feet, a dispersion trench with a dispersion device, such as a notched grade board, is recommended. The total length of this design must provide at least 10 feet of trench per 700 square feet of roof area and not exceed 50 feet.
- A setback of at least 5 feet must be maintained between any edge of the trench and any structure or property line

Dispersion Area

Minimum requirements associated with dispersion area design include the following:

- A vegetated flowpath of at least 25 feet in length must be maintained between the outlet of the trench and any property line, structure, stream, wetland, or impervious surface. A vegetated flowpath of at least 50 feet in length must be maintained between the outlet of the trench and any steep slope.
- The vegetated flowpath must be covered with well-established lawn or landscape area (in accordance with the amended soil requirements outlined in Section 4.4.1), landscaping with well-established groundcover, or native vegetation with natural groundcover. The groundcover shall be dense enough to help disperse and infiltrate flows and to prevent erosion.
- Dispersion is typically not permitted within Landslide–Prone Critical Areas as defined by the Regulations for Environmental Critical Areas (SMC 25.09.020).
- Dispersion is typically not permitted within a setback above a Steep Slope Critical Area (SMC 25.09.020). The setback is calculated as 10 times the height of the Steep Slope Critical Area (to a 500 foot maximum setback). Infiltration within this setback may be feasible provided a detailed slope stability analysis is completed by a geotechnical engineer. The analysis

shall determine the effects that dispersion would have on the Steep Slope Critical Area and adjacent properties.

- Dispersion is typically not permitted over contaminated sites or abandoned landfills.
- For sites with septic systems, the discharge point must be downgradient of the drainfield primary and reserve areas. This requirement may be waived if site topography clearly prohibits flows from intersecting the drainfield.

Overflow Conveyance

Minimum requirements associated with overflow conveyance design include the following:

- Dispersion area shall convey excess flow to an approved discharge point per Section 4.2.5. Conveyance of large storms shall be considered.
- No erosion or flooding of downstream properties may result.

4.4.3.4 Flow Control Credit

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), flow control credits may be achieved by using downspout dispersion. Credits are provided in Table 4.10, organized by flow control standard. These credits can be applied to reduce the effective impervious surface area used in drainage calculations. A 100 percent credit means that the flow control goal is achieved and no further control is required for the area from which runoff is dispersed. If partial credit is received, the standard is not achieved and additional flow control measures may be required. As an example, for a site subject to the Pre-developed Pasture Standard, a dispersed impervious area would receive a 90 percent credit. Therefore, 90 percent of the impervious area dispersed can be excluded from drainage calculations. The effective impervious area (area used to size downstream flow control facility) would be calculated as 10 percent of the impervious area dispersed.

Table 4.10. Flow Control Credits for Downspout Dispersion.

	Credit (%)	
Dispersion Type	**Pre-developed Pasture Standard**	**Peak Control Standard**
Splashblock	90%	100%
Dispersion Trench	90%	100%

%- percent
Impervious Area Mitigated = Impervious Area Dispersed x Credit (%)/100.

The flow control credits outlined above are applicable only if downspout dispersion meets the minimum design requirements outlined in this section.

Alternatively, dispersion can be evaluated using a continuous model as described below.

Facility Modeling

When using continuous modeling to quantify the flow control benefits of downspout dispersion, the assumptions listed in Table 4.11 shall be applied. If roof runoff is dispersed according to the requirements of this section and the vegetative flowpath is 50 feet or larger through undisturbed native landscape or lawn/landscape area that meets the amended soil requirements outlined in Section 4.4.1, the designer may model the contributing roof area as a lawn surface over native soil type.

Table 4.11. Continuous Modeling Assumptions for Downspout Dispersion.

Variable	Assumption
Precipitation Series	Seattle 158-year, 5-minute series.
Computational Time Step	5-minutes
Basin	Impervious area dispersed modeled as grass over the underlying soil type (e.g., till). Existing slope condition of dispersion area should be used.

4.4.4 Sheet Flow Dispersion

Sheet flow dispersion is one of the simplest methods of runoff control. This BMP can be used for any impervious or pervious surface that is graded so as to avoid concentrating flows. Because flows are already dispersed as they leave the surface (i.e., not concentrated), they need only traverse a narrow band of adjacent vegetation for effective flow attenuation and treatment.

4.4.4.1 Applications and Limitations

Sheet flow dispersion is applicable for impervious surfaces with slopes less than 15 percent, such as driveways, sport courts, patios, and roofs without gutters, or other situations where concentration of flows can be avoided.

If the designer wishes to use the dispersion BMP for water quality treatment in addition to flow control, the facility must meet any additional design requirements for filter strips per Section 5.7.

4.4.4.2 Design Criteria

Dispersion Area

Minimum requirements associated with dispersion area design include the following:

- A 2-foot-wide transition zone to discourage channeling shall be provided between the edge of the contributing impervious area and the downslope vegetation. This may be an extension of subgrade material (crushed

rock), modular pavement, drain rock, or other material approved by the Director.

- A vegetated buffer width of 10 feet of vegetation must be provided for up to 20 feet of width of contributing impervious surface. An additional 5 feet of width must be added for each additional 20 feet of width of contributing area or fraction thereof.
- The vegetated flowpath must be covered with well-established lawn or landscape area (in accordance with the amended soil requirements outlined in Section 4.4.1), landscaping with well-established groundcover, or native vegetation with natural groundcover. The groundcover shall be dense enough to help disperse and infiltrate flows and to prevent erosion.
- Dispersion is typically not permitted within Landslide–Prone Critical Areas as defined by the Regulations for Environmental Critical Areas (SMC 25.09.020).
- Dispersion is typically not permitted within a setback above a Steep Slope Critical Area (SMC 25.09.020). The setback is calculated as 10 times the height of the Steep Slope Critical Area (to a 500 foot maximum setback). Infiltration within this setback may be feasible provided a detailed slope stability analysis is completed by a geotechnical engineer. The analysis shall determine the effects that dispersion would have on the Steep Slope Critical Area and adjacent properties.
- Dispersion is typically not permitted over contaminated sites or abandoned landfills.
- For sites with septic systems, the discharge point must be downgradient of the drainfield primary and reserve areas. This requirement may be waived if site topography clearly prohibits flows from intersecting the drainfield.

Overflow Conveyance

Minimum requirements associated with overflow conveyance design include the following:

- Dispersion area shall convey excess flow to an approved discharge point per Section 4.2.5. Conveyance of large storms shall be considered.
- No erosion or flooding of downstream properties may result

4.4.4.3 Flow Control Credit

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), flow control credits may be achieved by using sheet flow dispersion. Credits are provided in Table 4.12, organized by flow control standard (i.e., Pre-developed Pasture and Peak Control Standards). These credits can be applied to reduce the effective impervious

surface area used in drainage calculations as explained for downspout dispersion.

Table 4.12. Flow Control Credits for Sheet Flow Dispersion.

Dispersion Type	Credit (%)	
	Pre-developed Pasture Standard	Peak Control Standard
Sheet Flow	90%	100%

Impervious Area Mitigated = Impervious Area Dispersed x Credit (%)/100.
%- percent

The flow control credits outlined above are applicable only if sheet flow dispersion meets the minimum design requirements outlined in this section.

Alternatively, dispersion can be evaluated using a continuous model as described below.

Facility Modeling

When using continuous modeling to quantify the flow control benefits of sheet flow dispersion, the assumptions listed in Table 4.11 shall be applied. Where sheet flow dispersion is used to disperse runoff into an undisturbed native landscape area or an area that the amended soil requirements outlined in Section 4.4.1, the designer may model the contributing impervious area as a lawn over native soil type.

4.4.5 Bioretention

The term bioretention is used to describe various designs using soil and plant complexes to manage stormwater. The healthy soil biology, soil structure and vegetation associated with bioretention facilities promote infiltration, storage, and slow release of stormwater flows to more closely mimic predeveloped conditions. Bioretention facilities are also known as rain gardens.

Bioretention can provide flow control via detention, attenuation, and losses due to infiltration, interception, evaporation, and transpiration. When designed in accordance with the criteria in this section, bioretention facilities without underdrains can meet Pre-developed Forest, Pre-developed Pasture and Peak Flow Control standards (Chapter 2). Bioretention facilities with underdrains may be designed to meet the Peak Control Standard, but cannot achieve the pre-developed pasture standard.

Water quality treatment can be accomplished through sedimentation, filtration, biodegradation of pollutants by soil organisms, adsorption, and phytoremediation (remediation by vegetation). Basic, phosphorus and enhanced water quality treatment requirements (Chapter 2) may also be met when the bioretention soil is designed in accordance with the treatment soil requirements outlined in Section 5.8.4, and it is shown that at least 91 percent of the total runoff volume

for the simulation period is infiltrated. Applicable drawdown requirements must also be met (see Section 5.4.1.2).

Two types of bioretention facilities are included in this manual:

- *Bioretention cell*: Bioretention cells are shallow depressions with a designed soil mix and plants adapted to the local climate and soil moisture conditions. A typical bioretention cell is shown in Figure 4.6. Bioretention cells may or may not have an underdrain. Bioretention cells may be connected in series, with the overflows of upstream cells directed to downstream cells to provide both flow control and conveyance. See Section 4.4.5.2.

Figure 4.6. Bioretention Cell, SEA Street Project, Seattle, WA.

- *Bioretention planter*: A bioretention planter is similar to a bioretention cell except that it is designed with an impervious bottom preventing infiltration to surrounding soil. Planters are most commonly configured as concrete reservoirs adjacent to building structures. After percolating through the bioretention soil, the water is discharged via an underdrain. See Section 4.4.5.3.

Infiltration facilities must meet the requirements in Section 4.3. Bioretention cells are considered infiltration facilities when water infiltrates into underlying native soils. Bioretention facilities are not considered infiltration facilities when they

include an underdrain and are underlain by an impermeable liner or soil with a negligible infiltration rate as determined by a geotechnical engineering evaluation. Bioretention planters are not considered infiltration facilities.

4.4.5.1 Applications and Limitations

As with other green stormwater infrastructure, bioretention facilities are intended to be small-scale, with no single bioretention cell larger than 800 square feet of bottom area.

Bioretention facilities are a stormwater management practice and a landscape amenity that can be applied in various settings, including:

- Individual lots for infiltration of rooftop and driveway runoff
- Common landscaped areas in apartment complexes or other multifamily housing designs
- Landscaped parking lot islands
- Areas within right-of-ways along roads (see the City of Seattle Right of Way Improvement Manual)
- Planters may also be used on building roofs, patios, and as part of streetscapes.

Due to the geologic and topographic conditions in Seattle, not all sites are suitable for stormwater infiltration. General requirements and limitations related to infiltrating facilities are provided in Section 4.3. Where infiltration is allowable and the surrounding native soils have adequate infiltration rates, bioretention may be used as a retention facility (i.e., provide full infiltration). If full infiltration cannot be achieved, the facility can detain and infiltrate stormwater and direct excess flows via an overflow to an approved discharge point. With lower infiltration rates, underdrain systems can be installed in the base of the facility to accommodate water that exceeds the infiltration capacity of the surrounding soil. Where infiltration is not permitted, bioretention planters or lined bioretention cells with underdrains may be used.

4.4.5.2 Bioretention Cell

Typical components of bioretention cells without underdrains and with underdrains are shown in Figures 4.7 and 4.8, respectively.

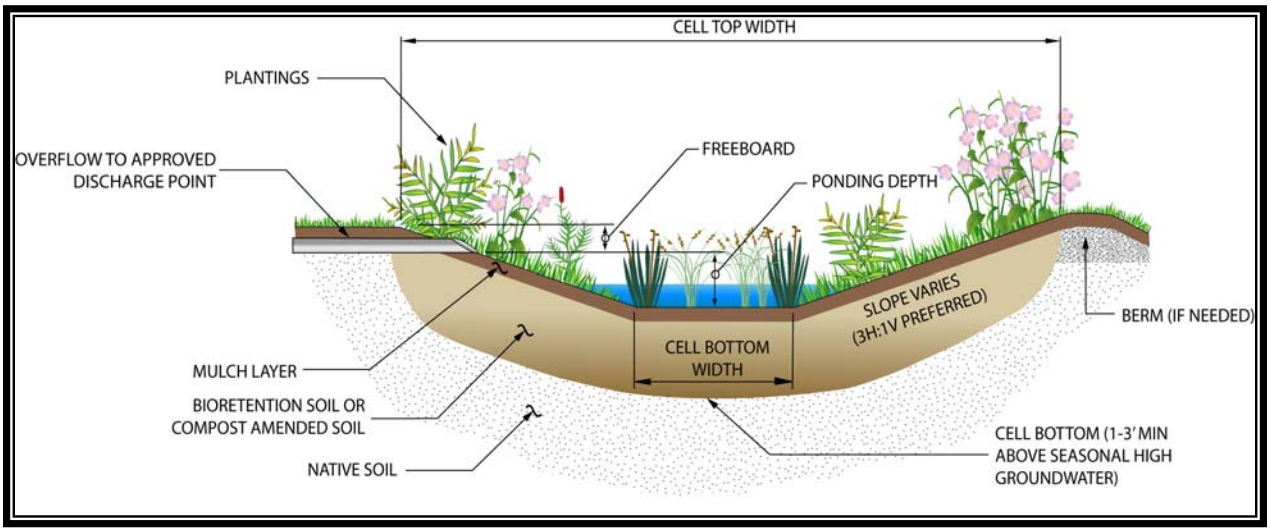


Figure 4.7. Bioretention Cell.

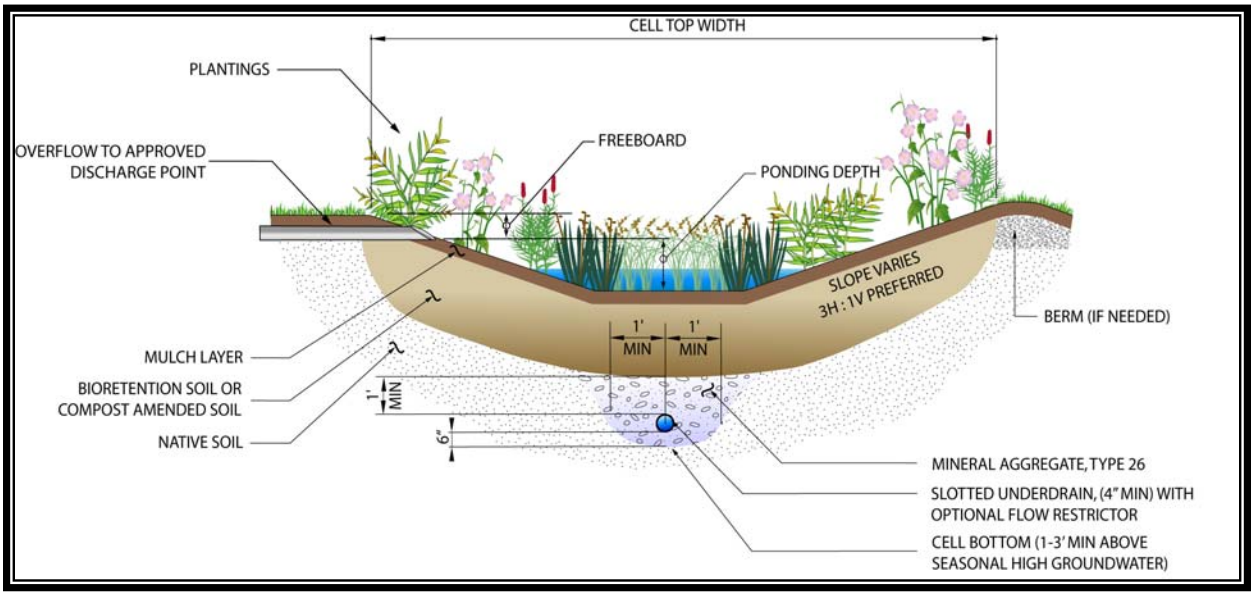


Figure 4.8. Bioretention Cell with Underdrain.

Site Considerations

Site considerations for the applicability of bioretention cells include:

- *Infiltration permitted:* Bioretention facilities with infiltration to underlying soils must meet the requirements for infiltration facilities presented in Section 4.3. Requirements include horizontal setbacks and a minimum vertical separation from the bottom of the facility to the underlying water table, bedrock, or other impermeable layer. For areas where infiltration is not permitted, lined bioretention cells with underdrains may be used (see Section 4.4.5.3).

- *Native Soil Infiltration Rate:* The native soils underlying and surrounding bioretention facilities are a principal factor for determining infiltration

capacity, facility sizing, and facility type. Lower infiltration rates will necessitate a larger facility to achieve flow control goals and may require an underdrain.

- *Site topography*: On sloped sites, bioretention cells require a larger facility footprint area. In addition, infiltration is prohibited in and near certain sloped areas (see Section 4.3).
- *Site growing characteristics and plant selection*: Appropriate plants should be selected for sun exposure, soil moisture, and adjacent plant communities.
- *Impacts of surrounding activities*: Human activity influences the location of the facility in the development. Where possible, bioretention facilities should be located away from traveled areas on individual lots to prevent soil compaction and damage to vegetation, and provide barriers to restrict vehicle access in roadside applications.

Design Criteria

The following provides a description, recommendations, and requirements for the components of bioretention cells. Some or all of the components may be used for a given application depending on the site characteristics and restrictions, pollutant loading, and design objectives. Submittal for bioretention facility review must include the following elements:

- Flow entrance / presettling
- Cell ponding area
- Bioretention soil
- Underdrain (if any)
- Flow restrictor (if any)
- Overflow (including invert elevation)
- Hydraulic restriction layers (if any)
- Plant material
- Mulch layer.

Flow Entrance/Presettling

Flow entrance design will depend upon topography, flow velocities, flow volume, and site constraints. Flows entering a bioretention facility should be less than 1.0 foot per second to minimize erosion potential. Vegetated buffer strips are the preferred entrance type because they slow incoming flows and provide initial settling of particulates.

Four primary types of flow entrances can be used for bioretention cells:

- *Dispersed, low velocity flow across a grass or landscape area*: This is the preferred method of delivering flows to the bioretention cell. This method can provide initial settling of particulates.
- *Sheet flow across pavement or gravel and past wheel stops for parking areas*
- *Drainage curb cuts for driveway or parking lot areas*: Curb cuts shall include rock or other erosion protection material in the channel entrance to dissipate energy.
- *Pipe flow entrance*: Piped entrances shall include rock or other erosion protection material in the channel entrance to dissipate energy and/or provide flow dispersion.

Woody plants should not be placed directly in the entrance flow path because they can restrict or concentrate flows and can be damaged by erosion around the root ball.

Minimum requirements associated with the flow entrance/presettling design include the following:

- If concentrated flows are entering the cell, engineered flow energy dissipation (e.g., rock pad or flow dispersion weir) must be incorporated
- A minimum 1-inch grade change between the edge of a contributing impervious surface and the vegetated flow entrance is required
- Until the upstream catchment area is thoroughly stabilized, flow diversion and erosion control measures must be installed to protect the bioretention area from sedimentation
- If the catchment area exceeds 2,000 square feet and flow is concentrated, bioretention shall be preceded by a presettling technique (e.g., variation on filter strip, presettling catch basin, or vault). The presettling is intended to remove larger solids, but not expected to meet water quality treatment goals or sizing guidelines for pretreatment facilities.

Cell Ponding Area

The ponding area provides surface storage for storm flows, particulate settling, and the first stages of pollutant treatment within the cell. Ponding depth and draw-down rate requirements are to provide surface storage, adequate infiltration capability, and soil moisture conditions that allow for a range of appropriate plant species. Soils must be allowed to dry out periodically in order to 1) restore hydraulic capacity of system, 2) maintain infiltration rates, 3) maintain adequate soil oxygen levels for healthy soil biota and vegetation, 4) provide proper soil conditions for biodegradation and retention of pollutants, and 5) prevent conditions supportive of mosquito breeding.

Minimum requirements associated with the cell ponding area design include the following:

- The ponding depth shall be a maximum of 12 inches. In high density right-of-way applications, the ponding depth shall be no greater than 6 inches, unless approved by the Director.
- The surface pool drawdown time shall be a maximum of 24 hours[2].
- The maximum planted side slope shall be 2.5H:1V. If total cell depth exceeds 3 feet, the maximum planted side slope is 3H:1V. If steeper sides are necessary, rockery, concrete walls or steeper soil wraps may be used.
- The bottom width shall be no less than 2 feet.
- No single cell shall receive runoff from more than 5,000[3] square feet of impervious area.
- Runoff from more than 5,000 square feet of impervious area may be directed to an interconnected series of cells, but the bottom area of each individual cell shall be no larger than 800 square feet.
- The minimum freeboard measured from the invert of the overflow pipe or earthen channel to cell overtopping elevation shall be 2 inches for drainage areas less than 1,000 square feet and 6 inches for drainage areas 1,000 square feet of greater.
- If berming is used to achieve the minimum top cell elevation needed to meet ponding depth and freeboard needs, maximum slope on berm shall be 2.5H:1V, and minimum top width of design berm shall be 1 foot. Soil used for berming shall be imported bioretention soil or amended native soil and compacted to a minimum of 90 percent dry density.

For compliance with flow control requirements, the maximum area that may be routed to a bioretention facility shall be twice the area for which it is sized. No flow control credit is given for runoff from areas beyond the design area. If additional runoff is routed to a facility, it must be clearly noted on submitted plans and the overflow infrastructure requires engineering design (see Section 4.2.5). Appropriate presettling measures must be included and the designer must provide calculations demonstrating that the water velocities in the vegetated areas of the facility do not exceed 3 feet per second during the 100-year recurrence interval flow.

[2] Drain time should be calculated as a function of ponding depth and native soil design infiltration rate.
[3] Area limitation is to ensure that bioretention facilities are small-scale and distributed. Also, the infiltration rate correction factor applied to City of Seattle standard bioretention soil mixes is based on a contributing area that does not exceed 5,000 square feet of impervious surface.

Bioretention Soil

Proper soil specification, preparation, installation, and maintenance are critical factors for bioretention performance. There are two options available to meet bioretention soil requirements. The native soil may be amended with compost according to requirements in Section 4.4.1.2 or the native soil may be over excavated and imported bioretention soil may be placed. The minimum bioretention soil depth to achieve flow control credit is 12 inches.

Minimum requirements associated with the bioretention soil design include the following:

- Imported bioretention soil must meet City of Seattle Bioretention Soil Standard Specification 7-21 (see SPU GSI website http://www.seattle.gov/util/greeninfrastructure) and shall have a minimum depth of 12 inches for flow control credit. The bioretention soil porosity used for facility evaluation shall be 40 percent and the design infiltration rate shall be as outlined in Table 4.13 below.

Table 4.13. Infiltration Rates for City of Seattle Turf and Landscape Bioretention Soil (Standard Specification 7-21)[a].

Bioretention Soil Type	Compaction (%)	Short-term Infiltration Rate, (inch/hour)	Design Infiltration Rate (inch/hour)	
			Drainage Area<5,000 sf PGIS; <10,000 sf impervious; or <¾ acre lawn and landscape	Drainage Area >5,000 sf PGIS; >10,000 sf impervious; or >¾ acre lawn and landscape
Long-term Infiltration Rate Correction Factor			2	4
Landscape without underdrain	85	6	3	1.5
Turf without underdrain	85	6	3	1.5
Landscape with underdrain	85	6	3	NA – exceeds permissible limits for cells with underdrains
Turf with underdrain	85	6	3	NA – exceeds permissible limits for cells with underdrains

PGIS – pollution generating impervious surface
NA – Not Applicable
[a] City of Seattle Bioretention Soil Standard Specification 7-21 is provided on the SPU GSI website (http://www.seattle.gov/util/greeninfrastructure).

- If native soil meets the aggregate specification in the City of Seattle Bioretention Soil Standard Specification 7-21, it may be amended with compost per the specification rather than importing materials. Amended soil depth shall be a minimum depth of 12 inches for flow control credit. The design bioretention soil infiltration rate used for facility evaluation shall be as outlined in Table 4.13. The soil porosity is 40 percent.

- For projects where runoff from less than 5,000 square feet of impervious areas will be infiltrated on-site and the ponding depth in the bioretention cell does not exceed 3 inches, the native soil may be amended per Section 4.4.1.2. The design infiltration rate of the amended soil shall be either 0.25 inches per hour or the tested infiltration rate of the underlying soil corrected by a factor of 2.
- If the bioretention cell is to provide water quality treatment, bioretention soil must be at least 18 inches deep and meet the requirements for treatment soil provided in Section 5.8.4.2 (e.g., soil meeting City of Seattle Bioretention Soil Standard Specification 7-21).
- When bioretention cells are located near areas of high foot traffic, the designer should evaluate mechanisms to restrict foot traffic in the cell and the use of turf (rather than landscape) bioretention soil.

Design infiltration rates for City of Seattle specified imported soil mixes are presented in Table 4.13. The soil porosity is 40 percent. Note that when imported bioretention soil is used, no correction factor is required for the infiltration rate of the underlying native soil.

Underdrain (Optional)

The area above an underdrain pipe in a bioretention area provides attenuation and pollutant filtering. Underdrain systems (see Figure 4.9) should be installed only if the bioretention area is:

- Located where infiltration is not permitted and liner is used
- In soils with infiltration rates that are not adequate to meet maximum pool drawdown time.

An underdrain is only allowed for water quality treatment facilities when soil used meets the City of Seattle Bioretention Soil Standard Specification 7-21 (see Section 5.8.5.4). When underdrains are used, it is not possible to meet the Pre-developed Forest or Pre-developed Pasture flow control standards and it is difficult to achieve the Peak Control Standard. The underdrain pipe diameter will depend on hydraulic capacity required. The underdrain can be connected to a downstream BMP such as another bioretention cell as part of a connected system or to an approved discharge point.

If included, minimum requirements associated with the underdrain design include the following:

- Flows shall be conveyed to an approved discharge point.
- Slotted, thick-walled plastic pipe shall be used. For specification see SPU GSI website (http://www.seattle.gov/util/greeninfrastructure).
- Pipe shall have a minimum diameter of 4 inches.
- Pipe shall be placed in the filter material as shown in Figure 4.8.

- Filter material shall meet specifications of City of Seattle Mineral Aggregate Type 26 (gravel backfill for drains, City of Seattle Standard Specifications).

Flow Restrictor (Optional)

A flow restrictor assembly can be installed at the outlet of an underdrain system to further detain outflow. If included, minimum requirements associated with the flow restrictor design include the following:

- An inspection chamber (catch basin or maintenance hole with clearances per City of Seattle Standard Plans 270 and 272A) shall be installed at the flow control assembly to allow for access and maintenance.
- The minimum orifice diameter shall be 0.25 inches.

Overflow

Unless designed to provide full infiltration into native soil (Section 4.3.4), bioretention cells must have an overflow (Section 4.2.5). Facility overflow can be provided by a drain pipe or earthen channel installed at the designed maximum ponding elevation and connected to a downstream BMP or an approved discharge point.

Minimum requirements associated with the overflow design include the following:

- Overflows shall be designed to convey excess flow to approved discharge point per Section 4.2.5.
- Pipe, if used, shall have a minimum diameter of 4 inches.

Hydraulic Restriction Layers (Optional)

Adjacent roads, foundations, slopes, utilities, or other infrastructure may require that infiltration pathways are restricted to prevent excessive hydrologic loading. Where clay or geomembrane liners are used underdrain systems are required. Two types of restricting layers can be incorporated into bioretention designs:

- Clay (bentonite) liners are low permeability liners
- Geomembrane liners completely block flow and are used for groundwater protection when bioretention facilities are used for filtering storm flows from pollutant hotspots. The liner shall have a minimum thickness of 30 mils and be ultraviolet (UV) resistant.

Plant Materials

The design intent for bioretention plantings is to replicate, to the maximum extent feasible, the hydrologic function of a mature forest including succession plants and ground cover. Plant roots aid in the physical, biological, and chemical bonding of soil particles that is necessary to form stable aggregates, improve soil

structure, and increase infiltration capacity. During the wet months in the Pacific Northwest (October through April) interception is the predominant aboveground mechanism for attenuating precipitation in the native forest setting. Deciduous plant material is leafless during our wettest months, and has little interception capacity compared to evergreen plant material. Transpiration occurs in evergreens year round, while deciduous plant material is dormant during our wettest months and is unable to transpire. As vegetation increases in size, the interception, and transpiration potential also increases. As vegetation matures it contributes greater amounts of leaf litter and debris to feeds soil organisms capable of increasing infiltration and the water holding capacity of soil. The primary and significant benefits of small trees, shrubs, and ground cover in bioretention areas during the wet season are the presence of root activity and contribution of organic matter that aids in the development of soil structure and infiltration capacity. Organic matter in the form of leaf litter or mulch helps retain stormwater runoff in the root zone where it can be infiltrated or transpired.

Critical to creating a self sustaining landscape is a healthy soil ecosystem where organisms in the soil can form symbiotic relationships with plant root systems improving water holding capacity, drought tolerance and nutrient absorption. Minimizing pesticide and synthetic fertilizer use, coupled with retaining leaf litter and plant debris and/or the annual addition of organic matter in the form of compost, leaf litter or arborist woodchip mulch will help foster a robust soil biota and a self sustaining landscape.

The primary design considerations for plant selection include:

- *Soil moisture conditions*: Plants should be tolerant of summer drought, ponding fluctuations, and saturated soil conditions for the lengths of time anticipated by the facility design.
- *Above and belowground infrastructure in and near the facility*: Plant size and structural stability should be considered within the context of the surrounding infrastructure. Species with aggressive rooting tendencies should not be selected if underground utilities are present. Slotted pipe shall be 5 feet minimum from tree locations and all side sewer pipes.
- *Adjacent plant communities and potential invasive species control:* If adjacent to invasive species, consider planting fast growing, hearty species. Shrubs may be used to help shade-out invasive weeds.
- *Aesthetics*: Visually pleasing plant designs add value to the property and encourage community and homeowner acceptance.

In general, the predominant plant material utilized in bioretention areas are species adapted to stresses associated with wet and dry conditions. Soil moisture conditions will vary within the facility from saturated (bottom of cell) to relatively dry (rim of cell). Accordingly, wetland plants may be used in the lower areas, if saturated soil conditions exist for appropriate periods, and drought-tolerant species planted on the perimeter of the facility or on mounded areas.

Native plant species when chosen to match the site's sun, soil, and moisture conditions, tolerate local climate and biological stresses, provide habitat benefits, and usually require no nutrient or pesticide application in properly designed and maintained soils. Natives can be used as the exclusive material in a bioretention cell or in combination with hardy cultivars that are not invasive and do not require chemical inputs. To increase survival rates and ensure quality of plant material, the following guidelines are suggested:

- As a general guideline, a minimum of three types of shrubs and three types of herbaceous species should be incorporated to protect against facility failure due to disease and insect infestations of a single species.
- Turf forming grasses requiring mowing are generally not allowed in bioretention cells. Exception may be granted by the Director.
- Plants should conform to the standards of the current edition of *American Standard for Nursery Stock* as approved by the American Standards Institute, Inc. All plant grades shall be those established in the current edition of *American Standards for Nursery Stock* (current edition: ANSI Z60.1-2004).
- All plant materials should have normal, well-developed branches and root systems, and be free from physical defects, plant diseases, and insect pests.
- Plant size: In the excellent soil conditions provided by bioretention areas, small plant material provides several advantages and is recommended. Specifically, small plant material requires less careful handling, less initial irrigation, experiences less transplant shock, is less expensive, adapts more quickly to a site, and transplants more successfully than larger material. Small trees and shrubs are generally supplied in pots of 3 gallons or less.
- All plants should be tagged for identification when delivered.
- Optimum planting time is typically fall (beginning mid October). Winter planting is acceptable; however, extended freezing temperatures shortly after installation can increase plant mortality. Spring is also acceptable, but requires more summer watering than fall plantings. Summer planting is the least desirable and requires regular watering for the dry months immediately following installation.

A permanent irrigation system using potable water may be used, but an alternative means of irrigation, such as air conditioning condensate or another readily available nonpotable source should be considered to maximize efficient use of resources. Any nonpotable sources should be analyzed to ensure that they do not contain chemicals that might harm or kill the vegetation. Any permanent irrigation system that relies on potable water should be designed to apply no more than 0.2 inches of water every 14 days from June through September, after the 2-year establishment period.

Minimum requirements associated with the vegetation design include the following:

- Plans shall specify that vegetation coverage of selected plants will achieve 90 percent coverage within 2 years or additional plantings will be provided until this coverage requirement is met.
- For sites with 5,000 square feet or more new and replaced impervious surface, plant spacing and plant size shall be designed to achieve specified coverage by certified landscape architect
- Plants shall be sited according to sun, soil, wind and moisture requirements
- At a minimum, provisions must be made for supplemental irrigation during the first two growing seasons following installation.

For more guidance, refer to the SPU GSI website (http://www.seattle.gov/util/greeninfrastructure).

Mulch Layer

Bioretention areas shall be designed with a mulch layer or a dense groundcover (except for turf-vegetated cells, which should be mulch-mowed). Research indicates that most attenuation of heavy metals in bioretention cells occurs in the first 1 to 2 inches of the mulch layer. That layer can be easily removed or added to as part of a standard and periodic landscape maintenance procedure. No indications of special disposal needs are indicated at this time from older bioretention facilities in the eastern U.S. (WSU, 2005). Properly selected mulch material also reduces weed establishment, regulates soil temperatures and moisture, and adds organic matter to soil. Compost mulch is an excellent slow-release source of plant nutrients, but does not suppress weed growth as well as bulkier, higher carbon mulches like wood chips. Arborist wood chips are superior to bark mulch in promoting plant growth, feeding beneficial soil organisms, reducing plant water stress, and maintaining surface soil porosity.

Mulch shall be:

- Compost in the bottom of the facilities (compost is less likely to float than wood chip mulch and is a better source for organic materials)
- Arborist wood chip mulch composed of shredded or chipped hardwood or softwood on cell slopes. Arborist mulch is chipped, mostly woody trimmings from trees and shrubs, and is available from tree services.
- Free of weed seeds, soil, roots and other material that is *not* from the aboveground components of a tree
- A minimum of 2 inches and a maximum of 3 inches of compost or 4 inches of wood chips (thicker applications can inhibit proper oxygen and carbon dioxide cycling between the soil and atmosphere).

Mulch shall not include mineral aggregate, pure bark (bark is essentially sterile and inhibits plant establishment), or grass clippings (decomposing grass clippings are a source of nitrogen and are not recommended for mulch in bioretention areas).

As an alternative to mulch, a dense groundcover may be used. Mulch is required in conjunction with the groundcover until groundcover is established.

For more information on compost and woody mulch, see the Post Construction Soil Quality and Depth section (Section 4.4.1).

BMP Sizing

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), pre-sized bioretention cells and bioretention cells with detention may be used to achieve Pre-developed Pasture and Peak Control Standards.

Sizing factors for bioretention cells without underdrains receiving runoff from an impervious surface are provided in Table 4.14. Factors are organized by flow control standard, facility ponding depth, and native soil design infiltration rate. A 2-, 6- or 12-inch facility ponding depth may be selected. The design rate for the native soils must be rounded down to the nearest infiltration rate in the pre-sized table (i.e., 0.25, 0.5, or 1.0 inch per hour). To use these pre-sized facilities to meet flow control standards, the bioretention cell must meet the general requirements for bioretention facilities outlined in this section plus the following specific requirements:

- Bottom area shall be sized using the applicable sizing factor
- Bottom area shall be flat (0 percent slope)
- No underdrain or impermeable liner shall be used
- Side slopes within ponded area shall be no steeper than 3H:1V
- Imported bioretention soil per City of Seattle Bioretention Soil Standard Specification 7-21 shall be used.
- Bioretention soil depth shall be a minimum of 1.0 foot
- Minimum ponding depth shall be set at the designated height (2, 6, or 12 inches). For intermediate ponding depths (between 2 and 12 inches), the sizing factor may be linearly interpolated.

Table 4.14. Sizing Factors for Bioretention Cell (without underdrain).

Ponding Depth	Native Soil Design Infiltration Rate	Sizing Factor for Cell Bottom Area	
		Pre-developed Pasture Standard	Peak Control Standard
2-inches	0.25 inch/hour	23.0%	--
	0.5 inch/hour	15.8%	--
	1.0 inch/hour	9.3%	--
6-inches	0.25 inch/hour	14.6%	33.1%
	0.5 inch/hour	9.9%	20.5%
	1.0 inch/hour	6.4%	10.6%
12-inches	0.25 inch/hour	NA	NA
	0.5 inch/hour	6.5%	13.4%
	1.0 inch/hour	4.1%	6.7%

-- sizing factors not provided; %- percent; NA- not applicable

Bioretention Cell Bottom Area = Contributing Impervious Area x Factor (%)/100.

Impervious Area Mitigated = Bioretention Cell Bottom Area ÷ Factor (%)/100.

Bioretention cells with 12 inches of ponding must have a design infiltration rate of at least 0.5 inches per hour to achieve drawdown requirements

If ponding depth is no greater than 3 inches and contributing area is small, existing soil may be amended rather than importing bioretention soil (see "Bioretention Soil" section). In this case, the sizing factor must be multiplied by a factor of 1.2. As an example, the sizing factor for a bioretention cell with 2 inches of ponding when the native soil infiltration rate is 1.0 inch per hour would be:

9.3 percent x 1.2 = 11.2 percent

To use these pre-sized bioretention facilities to meet water quality treatment standards in addition to flow control standards, the bioretention soil must be at least 18 inches deep and meet requirements for treatment soil provided in Section 5.8.4 (e.g., soil meeting City of Seattle Bioretention Soil Standard Specification 7-21). To use bioretention for water quality treatment only, see the pre-sized tables in Section 5.8.5.4.

Similarly, pre-sized bioretention cells with detention may be used to achieve flow control standards under the Pre-Sized Approach. Sizing factors for bioretention cells with detention and without underdrains receiving runoff from an impervious surface are provided in Table 4.15. Factors are provided for a 12-inch facility ponding depth to meet the Peak Control Standard. The design rate for the native soils must be rounded down to the nearest infiltration rate in the pre-sized table (i.e., 0.25, 0.5, or 1.0 inch per hour). To use these pre-sized facilities to meet flow control standards, the bioretention cell with detention must meet the requirements for pre-sized bioretention facilities listed above plus the following additional requirements:

- This BMP is only applicable for contributing areas of at least 1,500 square feet (for smaller contributing areas, bioretention cells without detention requires less area)
- Bottom area shall be sized using the applicable sizing equation
- The low flow orifice diameter shall be 0.5 inches
- Invert of overflow shall be set at the designated height (12 inches) above the invert of the low flow orifice
- The sizing equations do not achieve the infiltration requirements for water quality treatment facilities.
- Impervious Area Mitigated = [Bioretention Cell Bottom Area (square feet) - Integer] ÷ Factor. If native soil meets the aggregate specification in the City of Seattle Bioretention Soil Standard Specification 7-21, it may be amended with compost per the specification rather than importing materials. Amended soil depth shall be a minimum depth of 12 inches for flow control credit. The design bioretention soil infiltration rate and porosity used for facility evaluation shall be as outlined in Table 4.13 below.
- For projects where runoff from less than 5,000 square feet of impervious areas will be infiltrated on-site and the ponding depth in the bioretention cell does not exceed 3 inches, the native soil may be amended per Section 4.4.1.2. The design infiltration rate of the amended soil shall be either 0.25 inches per hour or the tested infiltration rate of the underlying soil corrected by a factor of 2.
- If the bioretention cell is to provide water quality treatment, bioretention soil must be at least 18 inches deep and meet the requirements for treatment soil provided in Section 5.8.4.2 (e.g., soil meeting City of Seattle Bioretention Soil Standard Specification 7-21).
- When bioretention cells are located near areas of high foot traffic, the designer should evaluate mechanisms to restrict foot traffic in the cell and the use of turf (rather than landscape) bioretention soil.

Table 4.15. Sizing Equations for Bioretention Cell with Detention (without underdrain).

Ponding Depth	Native Soil Design Infiltration Rate	Sizing Equation for Cell Bottom Area	
		Pre-developed Pasture Standard	Peak Control Standard
12-inches	0.25 inch/hour	NA	0.0382A +199
	0.5 inch/hour	NA	0.0297A +129
	1.0 inch/hour	NA	0.0208A + 97

A–contributing impervious area; NA-not applicable.
Bioretention Bottom Area (square feet) = [Factor x A (square feet)] + Integer.

Design infiltration rates for City of Seattle specified imported soil mixes are presented in Table 4.13. The soil porosity is 40 percent. Note that when imported bioretention soil is used, no correction factor is required for the infiltration rate of the underlying native soil.

For both pre-sized configurations, the *bottom* area for the bioretention cell area is calculated as a function of the impervious area routed to it. As an example, to meet the Pre-developed Pasture Standard, the bottom of the bioretention cell area would be equal to 6.4 percent of the impervious area routed to it when the native infiltration rate is equal to or greater than 1.0 inch per hour and the ponding depth is 6 inches. The total cell footprint area (top area) can be calculated as a function of the cell bottom area, the side slopes and the total facility depth (i.e., ponding and freeboard depth).

Sizing factors for bioretention cells with underdrains have not been developed. However, the sizing factors developed for bioretention planters (see Section 4.4.5.3) may be used. When an underlying impermeable liner or reservoir is not included, these sizing factors will result in conservative facility sizes because they do not account for the fraction of water that will be infiltrated to underlying soil.

Alternatively, bioretention cells with or without detention or underdrains can be sized using a continuous model as described in the subsequent section.

Facility Modeling

When using continuous modeling to size bioretention cells, the assumptions listed in Table 4.16 shall be applied. It is recommended that bioretention cells be modeled as a layer of soil (with specified infiltration rate and porosity) with ponding, detention (if applicable), infiltration to underlying soil and overflow. The tributary areas, cell bottom area, and ponding depth should be iteratively sized until the Minimum Requirements for Flow Control are met (see Chapter 2). General sizing procedures for infiltration facilities are presented in Section 4.3.6.

Construction Specifications and Criteria

During construction, it is critical to prevent clogging and over-compaction of the subgrade and bioretention soils. Specific construction requirements are outlined below. For inspection and verification procedures used for facility approval by City staff, see GSI web site (http://www.seattle.gov/util/greeninfrastructure).

Soil Installation and Amendment

Bioretention soil shall be placed per the requirements of City of Seattle Standard Specifications and native soil shall be amended per Section 4.4.1.2.

Table 4.16. Continuous Modeling Assumptions for Bioretention Cells.

Variable	Assumption
Precipitation Series	Seattle 158-year, 5-minute series.
Computational Time Step	5-minutes.
Inflows to Facility	Surface flow and interflow from drainage area routed to facility.
Precipitation and Evaporation Applied to Facility	Yes. If model does not apply precipitation and evaporation to facility, include the facility area in the basin area (note that this will significantly underestimate the evaporation of ponded water).
Bioretention Soil Infiltration Rate	For imported City of Seattle bioretention soil per specification 7-21 see Table 4.13. For compost amended native soil, rate shall be equal to the native soil design infiltration rate.
Bioretention Soil Porosity	For imported City of Seattle bioretention soil per specification 7-21, porosity is 40 percent. For compost amended native soil, porosity can be assumed to be 30 percent.
Bioretention Soil Depth	Minimum of 12 inches for flow control. Minimum of 18 inches for water quality treatment
Native Soil Design Infiltration Rate	Measured infiltration rate with correction factor applied, if applicable (Section 4.3.3, Appendix E). If imported bioretention soil is used, a correction factor for plugging is not required.
Infiltration Across Wetted Surface Area	Yes if side slopes are 3H:1V or flatter. For steeper side slopes, only infiltration across the bottom area is modeled.
Underdrain (optional)	If underdrain is placed at bottom extent of the bioretention soil layer, all water which enters the facility must be routed through the underdrain. If there is no liner or impermeable layer and the underdrain is elevated within the bioretention soil, water stored in the bioretention soil below the underdrain may be allowed to infiltrate.
Outlet Structure	Overflow elevation set at maximum ponding elevation (excluding freeboard). May be modeled as weir flow over riser edge or notch. Note that total facility depth (including freeboard) must be sufficient to allow water surface elevation to rise above the overflow elevation to provide head for discharge.

Sediment Control

Minimizing site disturbance is the most effective sediment control method. Bioretention facilities shall not be used as sediment control facilities and all drainage shall be directed away from bioretention facilities after initial rough grading. Flow can be directed away from the facility with temporary diversion swales or other approved protection. Bioretention facilities shall not be constructed until all contributing drainage areas are stabilized according to erosion and sediment control BMPs and to the satisfaction of the engineer. Erosion and sediment control practices must be inspected and maintained on a regular basis. If deposition of fines occurs in the bioretention area, material shall be removed and the surface scarified to the satisfaction of the engineer.

Compaction Prevention

Soil compaction can lead to facility failure; accordingly, minimizing compaction of the base and sidewalls of the bioretention area is critical. Excavation, soil

placement, or soil amendment shall not be allowed during wet or saturated conditions. Excavation should be performed by machinery operating adjacent to the bioretention facility and no heavy equipment with narrow tracks, narrow tires, or large lugged, high pressure tires should be allowed on the bottom of the bioretention facility. The base shall be scarified at completion of common excavation to a minimum of 6 inches. The sidewalls and bottom shall be roughened where scraped and sealed by excavation equipment.

Operations and Maintenance Requirements

Operations and maintenance requirements for bioretention facilities are provided in Appendix D. For non-single family residential projects, a landscape management plan shall be submitted demonstrating how the maintenance requirements will be met.

4.4.5.3 Bioretention Planter

Bioretention planters differ from bioretention cells because they are contained within an impervious reservoir (e.g., concrete reservoir or lined pond). Bioretention planters consist of a ponding area underlain by a minimum of 18 inches of bioretention soil and a 12-inch layer of uniformly graded washed gravel. The planter must include an underdrain and overflow directed to an approved discharge point. See Figure 4.9.

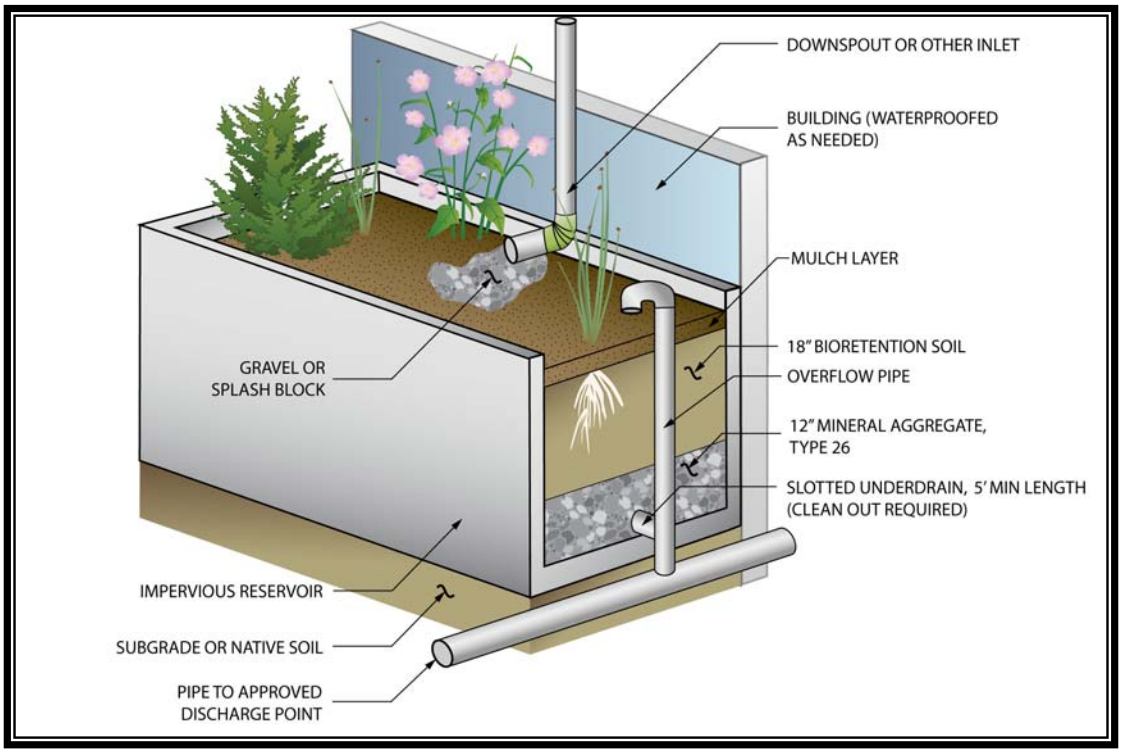


Figure 4.9. Bioretention Planter.

Site Considerations

Because bioretention planters do not infiltrate water to surrounding soils (water discharges from the planter via an underdrain and surface overflow), most of the site considerations associated with bioretention cells are not applicable. Bioretention planters may be employed regardless of native soil infiltration rate, depth to water table, and topography.

Design Criteria

The following provides unique descriptions, recommendations, and requirements for the design of bioretention planters. The design criteria for the following facility components are the same as presented in Section 4.4.5.2 for bioretention cells:

- Flow entrance
- Overflow
- Plant materials
- Mulch layer.

The requirements for the planter ponding area, bioretention soil, underdrain and impermeable reservoir are different than presented for bioretention cells are presented below.

Planter Ponding Area

Like the bioretention cell ponding area, the planter ponding area provides surface storage for storm flows, particulate settling, and the first stages of pollutant treatment within the cell. The planter ponding area shall meet the following criteria:

- No single planter shall receive runoff from more than 5,000 square feet of impervious area
- The ponding depth shall be a maximum of 12 inches
- The surface pool drawdown time shall be a maximum of 24 hours[4]
- Vertical or nearly vertical side slopes are acceptable for rockery and concrete walls. The maximum planted side slope shall be 2.5H:1V. If total cell depth exceeds 3 feet, the maximum planted side slope is 3.0H:1V.
- The planter width shall be no less than 2 feet
- The minimum freeboard measured from the invert of the overflow pipe to the top of the planter wall shall be 2 inches for drainage areas less than 1,000 square feet and 6 inches for drainage areas 1,000 square feet or greater.

[4] Drain time should be calculated as a function of ponding depth and native soil design infiltration rate.

Bioretention Soil

A minimum of 18 inches of bioretention soil is required. Imported soil shall conform to City of Seattle Bioretention Soil Standard Specification 7-21.

Underdrain

The underdrain pipe diameter will depend on hydraulic capacity required (4 to 8 inches is common). The underdrain can be connected to a downstream BMP such as another bioretention cell as part of a connected system or to an approved discharge point.

Minimum requirements associated with the underdrain design include the following:

- Flows shall be conveyed to an approved discharge point.
- Slotted, thick-walled plastic pipe shall be used. For specification see SPU GSI website (http://www.seattle.gov/util/greeninfrastructure).
- Pipe shall have a minimum diameter of 4 inches.
- Pipe shall be placed in 12-inch layer of filter material with minimum dimensions as shown for bioretention cells in Figure 4.8.
- Filter material shall meet specifications of City of Seattle Mineral Aggregate Type 26 (gravel backfill for drains, City of Seattle Standard Specifications).

Impermeable Reservoir/Liner

The planter shall not be hydraulically connected to the surrounding native soils. An above ground planter may be composed of concrete reservoir or other low permeability material. Below ground planters may be lined with either clay (bentonite) or a geomembrane liner with a minimum thickness of 30 mils that is ultraviolet (UV) resistant.

An additional consideration is whether the reservoir is adjacent to a building. If so, waterproofing on the building structure is often necessary.

BMP Sizing

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), pre-sized bioretention planters may used to achieve the Peak Control Standard. Sizing factors for bioretention planters receiving runoff from an impervious surface are provided in Table 4.17. Flow control credit is currently only approved to meet the Peak Control Standard. To use these pre-sized facilities to meet flow control standards, the planter must meet the general requirements for bioretention planters outlined in this section plus the following specific requirements:

- Bottom area shall be sized using the applicable sizing factor (Note that the planter size shall be no smaller or larger)
- Bottom area shall be flat (0 percent slope)
- Side slopes must be vertical
- Imported bioretention soil per City of Seattle Bioretention Soil Standard Specification 7-21 shall be used
- Bioretention soil depth shall be a minimum of 18 inches
- The ponding depth shall be 12 inches.

Table 4.17. Sizing Factors for Bioretention Planter

Ponding Depth	Sizing Factor Bioretention Planter Area	
	Pre-developed Pasture Standard	Peak Control Standard
12-inches	NA	6.5%

%- percent; NA-not applicable
Bioretention Planter Area = Contributing Impervious Area x Factor (%)/100.
Impervious Area Mitigated = Bioretention Planter Area ÷ Factor (%)/100.

The *bottom* area for the bioretention planter area is calculated as a function of the impervious area routed to it as explained for the bioretention cell.

To use these pre-sized facilities to meet water quality treatment standards in addition to flow control standards, the imported soil must meet treatment soil requirements (Section 5.8.4) in addition to City of Seattle Standard Specification 7-21. To use a bioretention planter for water quality treatment only, see the pre-sized tables in Section 5.8.5.4.

Alternatively, bioretention planters can be sized using a continuous model as described in the subsequent section.

Facility Modeling

When using continuous modeling to size bioretention planters, the assumptions listed in Table 4.17A shall be applied. It is recommended that bioretention planters be modeled as a layer of soil (with specified infiltration rate and porosity) with ponding, detention (if applicable), and overflow. The tributary areas, cell bottom area, and ponding depth should be iteratively sized until the Minimum Requirements for Flow Control are met (see Chapter 2). Note that using currently available modeling methods, planters are not capable of meeting the Pre-developed Forest or Pre-developed Pasture Standard. Planters may be sized to achieve the Peak Control Standard with an optimized ratio of planter area and contributing surface area, but performance may diminish with larger and smaller ratios.

Table 4.17A. Continuous Modeling Assumptions for Bioretention Planters.

Variable	Assumption
Precipitation Series	Seattle 158-year, 5-minute series.
Computational Time Step	5-minutes.
Inflows to Facility	Surface flow and interflow from drainage area routed to facility.
Precipitation and Evaporation Applied to Facility	Yes. If model does not apply precipitation and evaporation to facility, include the facility area in the basin area (note that this will significantly underestimate the evaporation of ponded water).
Bioretention Soil Infiltration Rate	For imported City of Seattle bioretention soil per specification 7-21 see Table 4.13. For compost amended native soil, rate shall be equal to the native soil design infiltration rate.
Bioretention Soil Porosity	For imported City of Seattle bioretention soil per specification 7-21, porosity is 40 percent. For compost amended native soil, porosity is can be assumed to be 30 percent.
Bioretention Soil Depth	Minimum of 18 inches for flow control or water quality treatment.
Aggregate Porosity	In-place aggregate porosity.
Aggregate Depth	Aggregate thickness above underdrain invert elevation.
Underdrain	All water which enters the facility must be routed through the underdrain.
Outlet Structure	Overflow elevation set at maximum ponding elevation (excluding freeboard). May be modeled as weir flow over riser edge or notch. Note that total facility depth (including freeboard) must be sufficient to allow water surface elevation to rise above the overflow elevation to provide head for discharge.

Construction Specifications and Criteria

Bioretention soil shall be placed per the requirements of City of Seattle Standard Specifications. Bioretention planters must be protected from sediment during construction and shall not be used as sediment control facilities. For inspection and verification procedures used for facility approval by City staff, see SPU GSI web site (http://www.seattle.gov/util/greeninfrastructure).

Operations and Maintenance Requirements

Operations and maintenance requirements for bioretention facilities are provided in Appendix D. For non-single family residential projects, a landscape management plan shall be submitted demonstrating how the maintenance requirements will be met.

4.4.6 Rainwater Harvesting

Rainwater harvesting is the capture and storage of rainwater for beneficial use. Roof runoff may be routed to cisterns for storage and nonpotable uses such as irrigation, toilet flushing and cold water laundry. Rainwater harvesting can be used to achieve reductions in peak flows, flow durations and runoff volumes. The flow control performance of cisterns is a function of contributing area, storage volume and rainwater use rate. When designed in accordance with the criteria outlined below, cistern harvesting systems can meet Pre-developed Forest, Pre-developed Pasture and Peak Control Standards (Chapter 2).

Rainwater harvesting is an effective form of green infrastructure where infiltration is not permitted or desired. For single family residential projects, the City considers the use of detention cisterns (Section 4.6.6) with harvesting capacity (e.g., faucet for irrigation) green stormwater infrastructure as well.

See SPU GSI website for additional guidance (http://www.seattle.gov/util/greeninfrastructure).

4.4.6.1 Applications and Limitations

Rainwater harvesting can be used in residential, commercial, or industrial development for new or retrofit projects. Unless otherwise approved by the Director, only runoff from roof surfaces is allowed for rainwater harvesting collection. To protect the water quality of the rainwater harvested, avoid roofing materials such as copper or zinc that may release contaminants into your system, as well as roofing materials treated with fungicides or herbicides.

Depending upon site constraints, cisterns may be installed at grade (see Figure 4.10), below ground (see Figure 4.11), under a deck (see Figure 4.12), or in a basement or crawl space. Cisterns may be used individually or connected to each other in series for greater detention and storage capacity.

Figure 4.10. Cistern with Planted Trellis at Chicago Center for Green Technology.

(Courtesy of Tim Pope)

Figure 4.11. Underground Cisterns Being Installed on San Juan Island.

Figure 4.12. Cisterns Installed Under the Deck of a Home in Fremont.

The indoor use of harvested water is regulated by the Washington State Department of Health. Links to resources on water harvesting, including permit requirements, are available at the SPU GSI website (http://www.seattle.gov/util/greeninfrastructure).

4.4.6.2 Design Criteria

The following provides recommendations and requirements for the common components of rainwater harvesting systems. Submittal for drainage review shall include the following elements, described further in the subsequent sections:

- Collection system
- Screen/debris excluder
- Cistern/storage system
- Delivery and distribution system
- Overflow
- Rainwater treatment and use equipment.

Note that all components of a harvesting system must be designed and constructed in accordance with the Uniform Plumbing Code and all other applicable laws.

Collection System

Collection systems include gutters and downspouts, as well as the piping and any other conveyance needed to route harvested water from harvest sources to the cistern.

Screens/Debris Excluder

Pre-storage treatment must be used to divert debris and/or “first flows” prior to entering the storage system, and to keep leaves and other larger debris from entering and clogging the system. “First flows” are defined as the initial rain that falls during a typical rain shower. These waters convey any sediment that has built up on the roof surface. They typically contain the greatest concentration of pollutants in harvested rainwater. It is recommended that the screen be self-cleaning and capable of excluding insects.

Cistern/Storage System

Cisterns can be constructed from a variety of materials and placed in various locations. They can include tanks, pipes, and enclosed portions of buildings — above or below ground level. Construction materials include underground concrete and fiberglass, partial and above ground plastic, and enclosed basement structures. All cistern systems must meet the following criteria:

Minimum requirements associated with cistern design include the following:

- All cisterns must be installed in accordance with manufacturer’s installation instructions and the City of Seattle Building Code and all applicable laws.
- If a foundation is required for installation, then the foundation must be flat and capable of supporting the cistern weight when full with water (in accordance with the City of Seattle Building Code).

- Cistern/storage systems must have both access points and drains to allow inspection and cleaning. Openings shall be designed to restrict entry from unauthorized personnel and/or appropriate signage shall be provided.
- Cistern/storage systems that are buried below ground level must have a manhole riser that sticks out a minimum of eight inches above the surrounding ground. Manhole covers must be secured and locked to prevent tampering.
- Any cistern/storage system opening that could allow the entry of personnel must be marked: “danger — confined space."
- Cleaning of any accumulated sediment on the bottom of the cistern must be possible by flushing through a drain, vacuuming or another approved method.
- The cistern must have a designated overflow when the volume of the tank meets capacity. The cross section of the overflow must have an area equal to or greater than all of the areas of the devices delivering water to the cistern. The minimum overflow is 4 inches in diameter.
- Overflows shall be designed to convey excess flow to approved discharge point per Section 4.2.5.
- Cisterns must be designed to prevent mosquitoes and other life forms from entering the cistern system. This can be done with appropriate screening at all openings to the cistern.
- Opaque containers must be used for aboveground cisterns to minimize algae growth.

Delivery and Distribution System

Delivery may be accomplished by a gravity system or include the pumps and pipes needed to move water from the storage system to the end use area. Consider designing a potable water back-up that can operate without electricity in emergency conditions.

Minimum requirements associated with delivery and distribution system design include the following:

- Water must be drawn from at least 4 inches above the bottom of the tank.

Overflow

Cisterns or other water storage system must have an overflow to convey water exceeding the capacity of the system to an approved discharge point or another BMP (e.g., bioretention area, vegetated cell, or infiltration trench). Conveyance may be provided by gravity flow or by pumps, but gravity flow is preferred. Minimum requirements associated with overflow design include the following:

- Overflows shall be designed to convey excess flow to approved discharge point per Section 4.2.5.

Water Treatment System

Water quality treatment is typically required to protect the delivery and distribution system and to improve the quality of the collected water for the reuse. The extent of water treatment is dependent on both the quality of the water entering the storage system and the intended water use. Systems must protect the functions of delivery valves and fixtures and range from simple screens to cartridge filters, UV light, and chlorination. Screen systems and/or basic mechanical filtration are typically adequate for irrigation and toilet flushing reuse.

Approval is required by the Seattle-King County Department of Health for any project routing harvesting water to an indoor plumbing system.

4.4.6.3 BMP Sizing

Facility Modeling

If it is documented that the rooftop runoff is fully utilized (i.e., no discharge), the designer may subtract the roof area from the site-wide model.

To receive flow control credit for rainwater harvesting, runoff reduction must be demonstrated by a water balance of rooftop runoff, use of the harvested water, and overflows from the rainwater harvest cistern. The minimum time step to be used for the modeling is 1 day so that the timing, magnitude, and duration of overflows can be considered in sizing residual detention if needed to meet flow control requirements for the site. The methods to be used are as follows.

- Obtain the Seattle 10-year precipitation series at a daily time step via link from the SPU GSI web site (http://www.seattle.gov/util/greeninfrastructure).
- Use a spreadsheet-based water balance model that calculates roof runoff (equal to rainfall volume), water usage (e.g., irrigation, outdoor cleaning, and indoor plumbing), and overflow from the cistern. See SPU GSI website for additional guidance (http://www.seattle.gov/util/greeninfrastructure).
- If overflows do not occur, the designer may subtract the roof area from the site-wide model.
- If overflow occurs, design a downstream BMP to meet residual flow control needs using an approved continuous hydrologic model of the site. Include the cisterns in the site-wide model so that the timing and volume of the overflows can be explicitly considered in the sizing of the downstream BMP.
    - Repeat steps 1 and 2 using the 10-year precipitation series at a 15-minute time step.

- Import the 15-minute overflow timeseries into the site-wide model.
- Import the 10-year precipitation series (15 minute time step) into the site-wide model.
- Subtract the roof are routed to the cistern from the site-wide model.
- Size the downstream BMP to meet flow control requirements.
- Model report must be stamped by PE.

4.4.6.4 Construction Specifications and Criteria

Rainwater Harvesting systems shall be constructed according to the manufacturer's recommendations and the City of Seattle Building Code.

4.4.6.5 Operations and Maintenance Requirements

Operations and maintenance requirements for cisterns are provided in Appendix D. Distribution and treatment systems shall be operated and maintained per manufacturer's recommendations and Washington State Department of Health requirements. A plan shall be submitted demonstrating how the operations and maintenance requirements will be met.

4.4.7 Permeable Pavement

Permeable pavement is a paving system which allows rainfall to percolate into an underlying soil or aggregate storage reservoir, where stormwater is stored and infiltrated to underlying subgrade, or removed by an overflow drainage system.

Permeable pavement systems can be designed to provide differing degrees of flow control. Two categories of permeable pavement systems are included in this manual: permeable pavement *facilities* and permeable pavement *surfaces*. *Facilities* typically have a thicker aggregate subbase than *surfaces* and may be designed to receive runoff from other areas ("run-on"). When designed in accordance with the design criteria outlined below, permeable pavement *facilities* can meet Pre-developed Forest, Pre-developed Pasture and Peak Control Standards (Chapter 2). Permeable pavement *surfaces* help to maintain site perviousness and are assigned flow control credits. *Surfaces* may not receive run-on from other areas.

The flow control performance of a system is a function of the infiltration rate of the underlying native soil and the depth of the aggregate storage reservoir that stores stormwater until it is infiltrated. Because water is infiltrated across a large area, permeable pavement systems can provide effective flow control even where native soil infiltration rates are low.

Basic, phosphorus and enhanced water quality treatment requirements (Chapter 2) may also be met with permeable pavement facilities when the underlying soil meets the treatment soil requirements outlined in Section 5.8.4, and it is shown that at least 91 percent of the total runoff volume for the simulation period is infiltrated. Applicable drawdown requirements must also be met (see Section 5.4.1.2).

The surface layer of any permeable pavement system is the wearing course. Categories of wearing courses include:

- *Porous Asphalt Concrete:* Porous asphalt concrete is open-graded asphalt with reduced fines and stable air pockets encased within it that allow water to drain to the base below. Aggregate binders and additives can be added to increase durability. Like conventional asphalt it is laid with traditional asphalt paving equipment. See Figure 4.13.

Figure 4.13. Porous Asphalt Driveway, Pierce County, WA.

- Porous *Cement Concrete:* Porous cement concrete is similar to porous asphalt in that the mixture omits the fines to create stable air pockets encased within it. Depending upon the mix design, porous cement concrete can have a rougher surface than conventional concrete. See Figure 4.14.

Figure 4.14. Porous Cement Concrete Street, High Point, Seattle, WA.

- *Porous Interlocking Concrete Pavers:* Interlocking concrete paver blocks themselves are not typically permeable, but they are installed with gaps between them to allow stormwater to infiltrate into the subsurface. The gaps, typically 10 percent of the surface area, are filled with a permeable material, usually small clean stone. See Figure 4.15.

Figure 4.15. Interlocking Concrete Pavers, High Point, Seattle, WA.

- *Open-Celled Paving Grid with Vegetation:* Open-celled paving grids consist of a rigid grid composed of concrete or a durable plastic that is filled with a mix of sand, gravel, and topsoil for planting vegetation. The cells can be planted with a variety of non-turf forming grasses or low-growing groundcovers. The support base and the ring walls prevent soil compaction and reduce rutting and erosion by supporting the weight of traffic and concentrated loads. Vegetation in the spaces between the paver blocks provides habitat for beneficial microbes, nutrient cycling, pollutant removal through root uptake, and stormwater volume reduction through evapotranspiration. See Figure 4.16.

Figure 4.16. Open-Celled Paving Grid with Vegetation, Carkeek Park, Seattle, WA.

- *Open-Celled Paving Grid with Gravel:* The same open-celled grid structure is employed but the spaces in the rings are filled with a mix of gravel. Manufacturers provide recommended specifications on the sieve analysis that should be used for the clean gravel fill for the cells. See Figure 4.17.

Figure 4.17. Open-Celled Paving Grid with Gravel, High Point Development, Seattle, WA.

Permeable Pavement Facility

A permeable pavement "facility" consists of the pervious wearing course, and a storage reservoir installed over native soil. The storage reservoir is designed to support expected loads and provide a reservoir for stormwater to allow time for the water to infiltrate into the underlying soil. The facility may also include an underdrain with a flow restrictor.

Permeable pavement facilities may be designed to take run-on from adjacent impervious areas. To reduce the potential for freeze/thaw damage, a permeable pavement facility must be designed to prevent the upper 6 inches of the pavement section from becoming saturated. To meet this requirement, the maximum subsurface water surface elevation may be controlled by subsurface ponding controls (e.g., check dams) or an overflow pipe. Alternatively, the storage reservoir may be designed to provide full infiltration (see Section 4.3) while maintaining a minimum of 6 inches between the maximum ponding depth and surface of pavement.

These types of facilities can be designed to meet flow control objectives. Two typical facility configurations are described below.

- *Facility with overflow:* The storage reservoir is designed with sufficient void volume and ponding depth to store water and provide enough infiltration into the underlying subsoil to achieve the flow control goal. Depending upon the characteristics of the underlying native soil, these systems may be designed to meet water quality treatment requirements. See Figure 4.18.

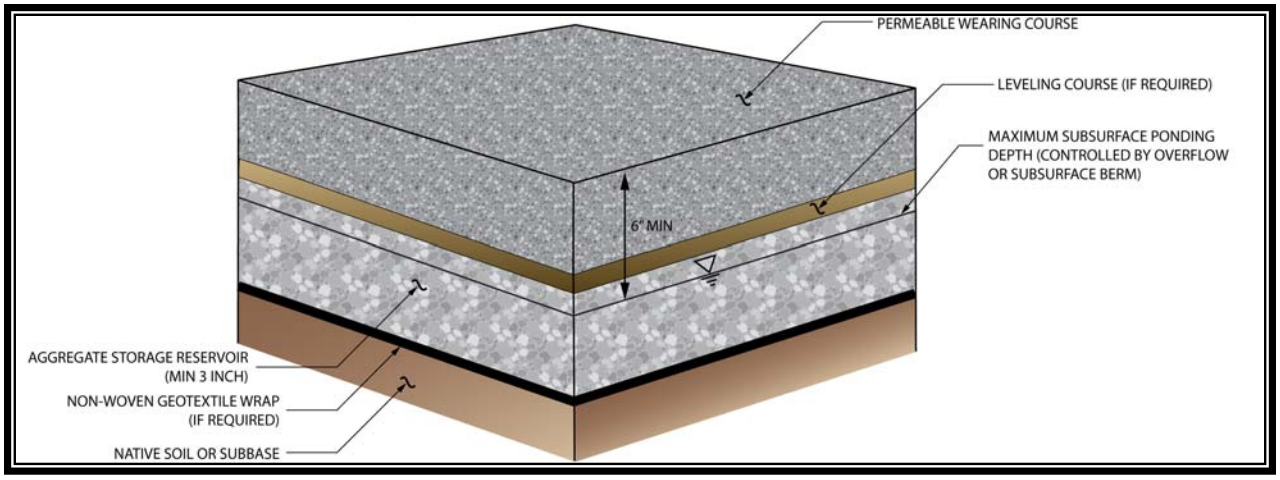


Figure 4.18. Permeable Pavement Facility.

- *Facility with overflow, underdrain and flow restrictor:* The storage reservoir is designed with sufficient void volume to detain water. A slotted drainpipe and flow restrictor are installed at the *bottom* of the reservoir. An orifice in the flow restrictor allows the gradual decanting of water above the slotted pipe, with infiltration occurring as much as possible. These systems operate as underground detention systems with some infiltration. At the time of publication of this manual, these systems are not accepted for water quality treatment. See Figure 4.19.

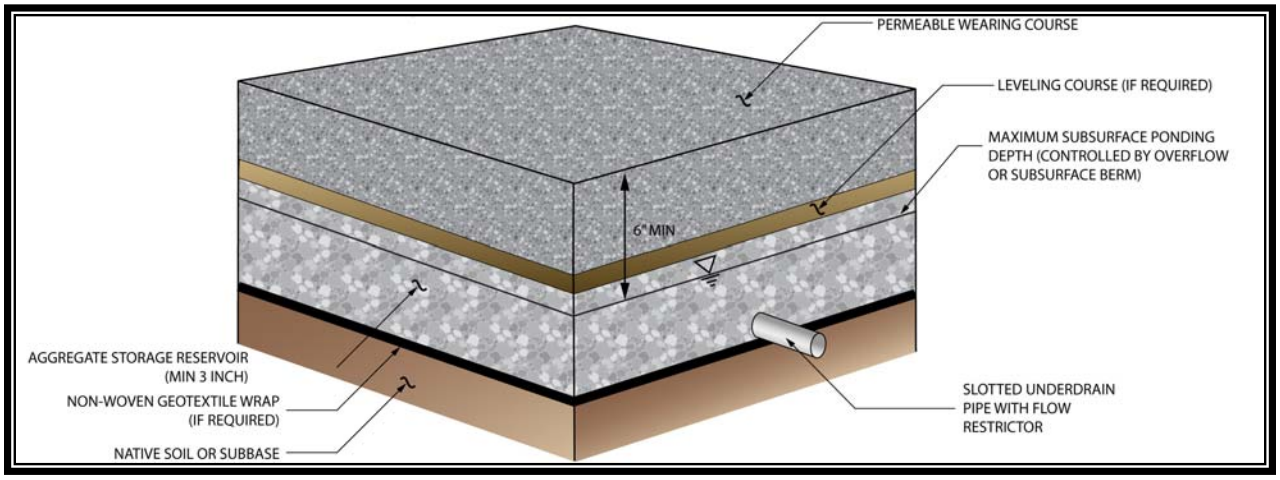


Figure 4.19. Permeable Pavement and Detention Facility with Underdrain and Flow Restrictor.

Permeable Pavement Surfaces

As with the permeable pavement "facility", a permeable pavement "surface" consists of the pervious wearing course and an aggregate subbase course installed over native soil (Figure 4.20). However, the aggregate subbase course is designed to manage only the water which falls upon it and should not be designed to take significant run-off from other areas. Some design features

required for permeable pavement facilities are not required for surface installations (e.g., subsurface berms, overflow pipes).

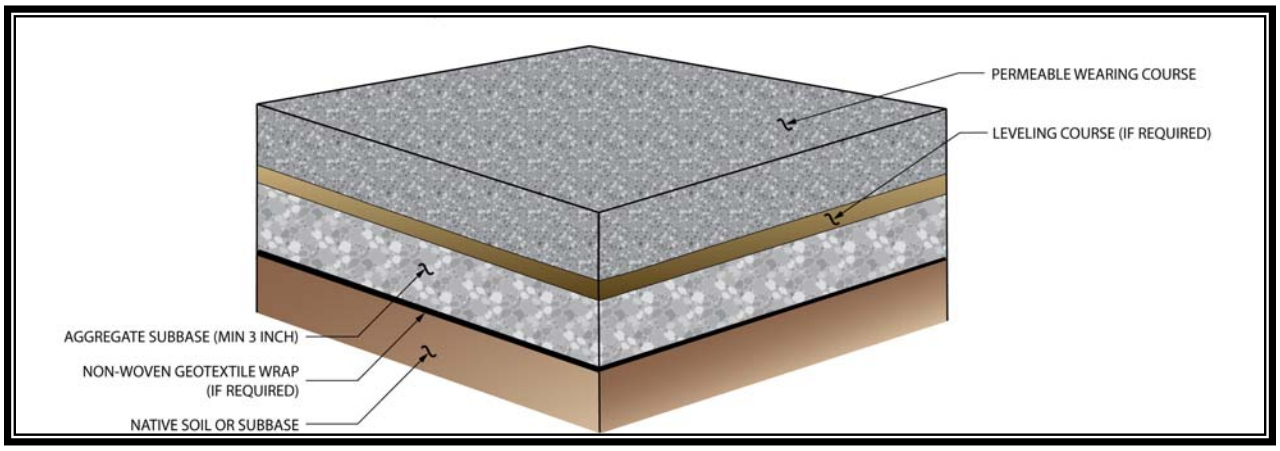


Figure 4.20. Permeable Pavement Surface.

While permeable pavement surfaces will not always achieve stormwater flow control requirements, they will maintain permeability in the pavement surface and some flow control credit is applied.

4.4.7.1 Applications and Limitations

Depending upon site conditions and system design, permeable pavement surfaces and facilities can achieve reduction in stormwater peak flows, durations, and volumes; increase groundwater recharge; and provide stormwater treatment. Unlike many flow control facilities that require dedicated space on a site, pavement systems (facilities and surfaces) are part of the usable lot area and can be used to replace conventional pavements, including:

- Sidewalks and pedestrian plazas
- Pedestrian and bike trails
- Driveways
- Most parking lots
- Low volume private roads

For information on use within the street right-of-way, see the *City of Seattle Right-of-Way Improvement Manual*. A link to this document is available on the SPU GSI website (http://www.seattle.gov/util/greeninfrastructure).

Permeable pavement systems (facilities and surfaces) are generally applicable to flat sites (typically less than 5 percent gradient). A primary consideration in the design and performance of permeable pavement systems is the infiltrative capacity of the native soil and how the soil is expected to infiltrate after any necessary compaction of the subbase. Permeable pavement facilities are considered infiltration facilities; they must be sited where infiltration is permitted.

Because permeable pavement can clog with sediment, these surface treatments should not be used at locations where a significant sediment load is expected (e.g., sediment generation or handling activities performed on the paved surface or run-on from unstabilized erodable areas is expected). To maintain infiltration capacity of existing soils and facility performance, protecting the native soil/subbase from becoming overcompacted during construction is critical.

4.4.7.2 Permeable Pavement Facility (with underlying storage reservoir)

Site Considerations

Site considerations for the applicability of permeable pavement *facilities* include:

- *Infiltration permitted:* Permeable pavement facilities with infiltration to underlying soils must meet the requirements for infiltration facilities presented in Section 4.3. Requirements include horizontal setbacks and a minimum vertical separation from the bottom of the facility to the underlying water table, bedrock, or other impermeable layer. For areas where infiltration *facilities* are not permitted, permeable pavement *surfaces* may be used because they are not considered infiltration facilities (see Section 4.3.1).
- *Native Soil Infiltration Rate*: The native soils underlying the permeable pavement are a principal factor for determining infiltration capacity, facility sizing, and facility type. Lower infiltration rates will necessitate a deeper storage reservoir to achieve flow control goals and may require an underdrain.
- *Site topography:* Permeable pavement should typically not be used on slopes greater than 5 percent. Excessive gradient increases surface and subsurface flow velocities and reduces storage and infiltration capacity of the pavement system.
- *Land use*: Permeable paving systems are *not* recommended where:
  - Excessive sediment contamination is likely on the pavement surface (e.g., construction areas, landscaping material yards)
  - Stormwater run-on to the permeable pavement may occur from unstabilized erodable areas without presettling
  - Regular, heavy application of sand is anticipated for maintaining traction during winter, or in close proximity to areas that will be sanded
  - Concentrated pollutant spills are possible (e.g., gas stations, truck stops, car washes, vehicle maintenance areas, industrial chemical storage sites)
  - Pavement sealant application or other uncontrolled use is likely
- *Setbacks*: Section 4.3 provides setback for infiltration facilities.

- *Accessibility*: ADA accessibility issues must be met when designing a permeable pavement system.

Design Criteria

The following provides recommendations and requirements for the common components of permeable pavement facilities. Some or all of the components may be used for a given application depending on the permeable pavement type (e.g., porous asphalt, gravel pavers, etc.), site characteristics and restrictions, and design objectives.

Guidance and requirements for permeable pavement and underlying aggregate related to structural loads are not included in this document.

Submittal for facility review shall include the following elements:

- Catchment area delineation
- Flow entrance / presettling
- Wearing course
- Leveling course (if required)
- Storage reservoir
- Overflow (if any)
- Underdrain and flow restrictor (if any)
- Non-woven geotextile (if required)
- Water quality treatment layer (if any)
- Native soil or subbase.

Catchment Area

Permeable pavement facilities may receive run-on from other surfaces. Contributing catchment areas shall be permanently stabilized. The catchment area tributary to a facility shall be no larger than 3 times the permeable pavement facility area, unless approved by the Director.

Flow Entrance/Presettling

When designed to take runoff from other catchment areas (“run-on”), permeable pavement areas must be protected from sedimentation which can cause clogging and diminished facility performance.

Minimum requirements associated with the flow entrance/presettling design include the following:

- Until any upstream catchment area is thoroughly stabilized, flow diversion and erosion control measures shall protect the permeable pavement area from sedimentation.

- If the catchment area exceeds 2,000 square feet and flow is concentrated, permeable pavement shall be preceded by a presettling technique (e.g., variation on filter strip, presettling catch basin or vault). The presettling is intended to remove larger solids, but not expected to meet water quality treatment goals or sizing guidelines for pretreatment facilities.
- If the catchment area for a flow entrance exceeds 1,000 square feet and flow is concentrated, run-on shall be dispersed to permeable pavement. Acceptable methods include sheet flow or subsurface delivery to storage reservoir. If subsurface delivery is used, primary settling is required (e.g., via catch basin) followed by distribution to storage reservoir (e.g., via perforated pipe).

Wearing Course

The wearing course or surface layer of the permeable pavement facility may consist of porous asphalt, porous cement concrete, interlocking concrete pavers, or open-celled paving grid with vegetation or gravel. The wearing course must provide adequate porosity for stormwater infiltration.

Minimum requirements associated with the wearing course design include the following:

- Wearing course material shall be on the permeable pavement approved technology list on the SPU GSI website (http://www.seattle.gov/util/greeninfrastructure)
- Positive surface drainage shall be provided to eliminate risk of ponding on pavement surface (minimum surface slope of 1 percent)
- For a vegetated open-celled paving grid, topsoil shall have a minimum 4 percent organic matter by dry weight. If receiving run-on from other surfaces, calculations must be provided to show that run-on is infiltrated through the wearing course to the storage reservoir without ponding.
- For porous cement concrete, City of Seattle standard specifications shall be used (www.seattle.gov\util\greeninfrastructure).

Leveling Course

Depending upon the type of wearing course, a leveling course (also called a bedding or choker course) may be required. A leveling course is often required for open-celled paving grids, interlocking concrete pavers and porous asphalt concrete. This course is a layer of aggregate that provides a more uniform surface for laying pavement or pavers and consists of crushed aggregate smaller in size than the underlying storage reservoir. Course thickness will vary with permeable pavement type.

Minimum requirements associated with the leveling course design include the following:

- Leveling course for proprietary products shall be included as required per manufacturer recommendations.

Storage Reservoir

Stormwater passes through the wearing and leveling courses to underlying aggregate storage reservoir where it is filtered and stored prior to infiltration into the underlying soil. This aggregate also serves as the road, parking, or pedestrian area's support base and must be sufficiently thick to support the expected loads. Design of the subbase for loading is outside of the scope of this manual. A licensed engineer is needed to determine minimum aggregate base thickness and analyze subsoil load bearing for the traffic loading.

Sloped facilities have an increased potential for lateral flows through the storage reservoir aggregate along the top of the relatively impermeable subgrade soil. This reduces the storage and infiltration capacity of the pavement system. For subgrade longitudinal slopes greater than 2 percent, the subbase must be designed to create subsurface ponding to detain subsurface flow and increase infiltration. Ponding may be provided using design features such as terracing berms (check dams) or intermittent infiltration trenches. When the subgrade soil slope is less than 2 percent, intermittent ponding measures are not required, but at least one low permeability check dam should be installed at the downslope end to contain water in the facility. For example design details of permeable pavement on slopes see the *City of Seattle Right-of-Way Improvement Manual* (link available on the SPU GSI website, http://www.seattle.gov/util/greeninfrastructure).

To reduce the potential for freeze/thaw damage, the system must be designed to prevent the upper 6 inches of the pavement section from becoming saturated. To meet this requirement, the maximum subsurface water surface elevation may be controlled by subsurface ponding controls (e.g., check dams) or an overflow pipe. Alternatively, the storage reservoir may be designed to provide full infiltration (see Section 4.3) while maintaining a minimum of 6 inches between the maximum ponding depth and surface of pavement.

When evaluating the flow control performance of a permeable pavement facility, the average water depth in the storage reservoir before berm overtopping or overflow is modeled. Therefore, the design goal is to maximize the average ponding depth while maintaining a 6 inch separation from the pavement surface.

Minimum requirements associated with the storage reservoir design include the following:

- A 3-inch minimum depth of storage reservoir aggregate is required under the wearing course and leveling course (if any) for water storage

- The storage reservoir aggregate composition shall be per manufacturer recommendations. At a minimum, aggregate shall consist of clean, washed, crushed angular material with less than 0.8 percent of material passing the #200 sieve (by weight).
- The storage reservoir shall have a minimum total void volume of 20 percent after compacted in place
- When subgrade slope exceeds 2 percent, subbase must be designed to provide subsurface ponding (e.g., check dams).
- Maximum ponding elevation shall be 6 inches below the top of the wearing course to minimize risk of damage due to freeze/thaw.

Additional guidance can be found in the details and specifications located in the *City of Seattle Right-of-Way Improvement Manual* (http://www.seattle.gov/transportation/rowmanual/manual/).

Overflow

Unless designed to provide full infiltration (Section 4.3.4), permeable pavement facilities must have an overflow (Section 4.2.5). Facility overflow can be provided by subsurface slotted drain pipe(s) or by lateral flow through the storage reservoir to a daylighted conveyance. Flows must be routed to an approved discharge point.

If the overflow is to set the maximum ponding depth, the pipe or daylight invert shall be a minimum of 6 inches below the surface of the pavement. If site constraints necessitate pipe in area subject to traffic or other loading, appropriate cover depth and pipe material must be designed.

The pipe diameter and spacing for slotted overflow pipes will depend on the hydraulic capacity required. For a sloped subgrade, at least one overflow pipe should be installed at the downslope end of facility. Optional observation and cleanout wells can be used to determine whether an overflow is dewatering properly and allows access for back flushing.

Minimum requirements associated with the overflow design include the following:

- Overflows shall be designed to convey excess flow to approved discharge point per Section 4.2.5.
- If a pipe is used to collect water in pavement section, it shall have sufficient strength, have an appropriate cover depth and be easily maintained. For recommended specification see SPU GSI website (http://www.seattle.gov/util/greeninfrastructure).
- If site constraints necessitate overflow pipe in area subject to traffic or other loading, appropriate cover depth and pipe material must be designed.

- If no overflow is proposed it must be shown that the maximum subsurface water elevation is maintained at least 6 inches below the pavement surface by other means.

Underdrain with Flow Restrictor (Optional)

A slotted drain pipe with flow restrictor assembly may be installed at the *bottom* of the storage reservoir. Permeable pavement facilities with underdrains and flow restrictors operate as underground detention systems with some infiltration. Except that the underdrain is installed at the bottom of the storage reservoir, the minimum requirements associated with the underdrain are the same as those for the overflow.

If included, minimum requirements associated with the underdrain and flow restrictor design include the following:

- Flows shall be conveyed to an approved discharge point
- Slotted or perforated, thick-walled plastic pipe with a minimum diameter of 4 inches and appropriate cover depth shall be used.
- An inspection chamber (catch basin or maintenance hole with clearances per City of Seattle Standard Plans 270 and 272A) shall be installed at the flow control assembly to allow for access and maintenance. This requirement may be waived by DPD for small single family residential projects.
- The minimum orifice diameter shall be 0.25 inches.

Non-Woven Geotextile

In order to prevent fine particles from contaminating and migrating into the storage reservoir, a non-woven geotextile may be required. As part of the pavement section design, the designer shall review the existing native soil or subbase characteristics and determine if non-woven geotextile is needed. Additional guidance on geotextile design is provided in Appendix F.

Minimum requirements associated with the geotextile design include the following:

- Non-woven geotextile shall be placed between the storage reservoir and the prepared subgrade (or water quality treatment layer)
- Non-woven geotextile shall be wrapped up and to the sides of the storage reservoir and secured
- The non-woven geotextile shall pass water at a greater rate than the design infiltration rate for the existing subgrade soils while at the same time prevent the migration of fines into the storage reservoir.

Water Quality Treatment Layer (Optional)

If the permeable pavement is being designed to provide water quality treatment, native underlying soils must meet the specific requirements for treatment soil provided in Section 5.8.4.

Native Soil / Subbase

Because the permeability of wearing, leveling, and storage reservoir courses most often exceed the design peak rainfall intensity, the native subgrade soil infiltration rate is usually the limiting factor. Minimum requirements associated with the native soil or subbase design include the following:

- A correction factor used to calculate design infiltration rate shall consider compaction of the native soil or subbase during construction and be determined by a geotechnical engineer
- The design plans and specifications shall include measures to protect the native soil or subbase from unnecessary compaction and clogging from sediment during construction.

BMP Sizing

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), pre-sized permeable pavement facilities may be used to achieve Pre-developed Pasture and Peak Control Standards. Sizing factors for permeable pavement facilities receiving runoff from an impervious surface are provided in Table 4.18. Factors are organized by flow control standard and native soil design infiltration rate. The design rate for the native soils must be rounded down to the nearest infiltration rate in the pre-sized table (i.e., 0.25, 0.5, or 1.0 inch per hour). To use these sizing factors to meet flow control standards, the facility must meet the general requirements for permeable pavement facilities outlined in this section plus the following specific requirements:

- The permeable pavement area shall be sized using the applicable sizing factor
- On average across the facility, the maximum subsurface water ponding depth in the storage reservoir before berm overtopping or overflow shall be at least 6 or 12 inches (requires design measures to provide this subsurface ponding). For intermediate ponding depths (between 6 and 12 inches), the sizing factor may be linearly interpolated.
- The storage reservoir shall be composed of aggregate with a minimum void volume of 20 percent
- No underdrain or impermeable liner shall be used.

Table 4.18. Sizing Factors for Permeable Pavement Facility.

Ponding Depth in Storage Reservoir	Native Soil Design Infiltration Rate	Sizing Factor for Permeable Pavement Facility Area (without underdrain)	
		Pre-developed Pasture Standard	Peak Control Standard
6-inches	0.25 inch/hour	60.6%	131.4%
	0.5 inch/hour	34.5%	52.4%
	1.0 inch/hour	33.3%	33.3%
12-inches	0.25 inch/hour	38.2%	75.8%
	0.5 inch/hour	33.3%	38.7%
	1.0 inch/hour	33.3%	33.3%

%- percent

Permeable Pavement Facility Area = Contributing Impervious Area x Factor (%)/100.
Impervious Area Mitigated = Permeable Pavement Facility Area ÷ Factor (%)/100.

The required permeable pavement facility area is calculated as a function of the impervious area routed to it. As an example, to meet the Pre-developed Pasture Standard, the permeable pavement area would be equal to 60.6 percent of the impervious area routed to it when the average water depth in the storage reservoir is 6 inches and the native infiltration rate is between 0.25 and 0.5 inches per hour (Table 4.18).

The sizing factors for permeable pavement facilities are conservative. For infiltration rates of 0.5 and 1.0 inches per hour, modeling would likely yield ponding depth requirements less than 6 inches. Note that the minimum sizing factor is 33.3 percent because the catchment area tributary to a facility is limited to 3 times the permeable pavement facility area.

To use these pre-sized facilities to meet water quality treatment standards, the underlying soil must meet treatment soil requirements (Section 5.8.4).

Alternatively, permeable pavement facilities can be sized using a continuous model as described below.

Facility Modeling

When using continuous modeling to size permeable pavement, the assumptions listed in Table 4.19 shall be applied. It is recommended that permeable pavement be modeled as an impervious area with runoff routed to a gravel-filled infiltration trench (of the same area). Runoff from other areas draining to the permeable pavement surface can also be routed to the trench. The tributary areas, pavement area, and average water surface depth in the aggregate storage reservoir should be iteratively sized until the Minimum Requirements for Flow Control are met (see Chapter 2). General sizing procedures for infiltration facilities are presented in Section 4.3.4. Specific modeling guidelines are outlined below:

- Only the average depth of the storage reservoir occupied by ponded water before berm overtopping or overflow is modeled. The storage reservoir aggregate above this depth, and the overlying leveling and wearing courses are not modeled. Infiltration through the wearing course and leveling course is neglected (this assumption is reasonable given the high infiltration rate of these materials).
- The area of subbase covered by check dams should be excluded from gravel trench bottom area when these areas are significant relative to the pavement area.
- The tributary areas, pavement area, and ponding depth in the storage reservoir should be iteratively sized until the Minimum Requirements for Flow Control are met. As stated previously, the runoff area tributary to a permeable pavement facility is limited (see "Catchment Area" in this section).

Table 4.19. Continuous Modeling Assumptions for Permeable Pavement Facility.

Variable	Assumption
Precipitation Series	Seattle 158-year, 5-minute series.
Computational Time Step	5-minutes.
Inflows to Facility	Model pavement area as impervious basin routed to a gravel-filled trench with infiltration to underlying soil. Additional areas draining to the pavement (surface flow and interflow), if any, are also routed to the gravel trench.
Precipitation Applied to Facility	No (applied to basin before routing to trench).
Evaporation Applied to Facility	Yes. While evaporation is applied to the impervious basin before routing to the trench, additional evaporation occurs when water is stored in the storage reservoir.
Storage Reservoir Depth	Average maximum subsurface water ponding depth in the storage reservoir (average across the facility) before berm overtopping or overflow. Note: The maximum ponding depth in the storage reservoir should be a minimum of 6 inches below the surface of pavement.
Storage Reservoir Porosity	Assume maximum 20 percent unless test is provided showing higher porosity for aggregate compacted and in place.
Native Soil Design Infiltration Rate	Measured infiltration rate with correction factor applied (Section 4.3.3, Appendix E).
Infiltration Across Wetted Surface Area	No (bottom area only).
Underdrain (optional)	If underdrain is placed at bottom extent of the storage reservoir, all water which enters the facility must be routed through the underdrain. If there is no liner or impermeable layer and the underdrain is elevated within the storage reservoir, water stored in the reservoir below the underdrain may be allowed to infiltrate.
Outlet Structure	Overflow elevation set at average maximum subsurface ponding depth. May be modeled as weir flow over riser edge or notch. Note that freeboard must be sufficient to allow water surface elevation to rise above the overflow elevation to provide head for discharge.

Construction Specifications and Criteria

Proper construction methods and pre-planning are essential for the successful application of any permeable paving system. Over compaction of the underlying soil or excessive fine sediment contamination onto the existing subgrade and pavement section during construction will significantly degrade or effectively eliminate the infiltration capability of the system.

Minimum requirements associated with construction include the following:

Permeable pavement site shall be isolated from sedimentation during construction, either by use of effective erosion and sediment control measures upstream, or by delaying the excavation of the lowest 1 foot of material above the final subgrade elevation for the entire pavement area until after all sediment-producing construction activities have been completed and upstream areas have been permanently stabilized. Once the site is stabilized, the lowest 1 foot of material may be removed. For more information on site stabilization, see the Construction Stormwater Control Technical Requirements Manual.

- Traffic control measures shall protect permeable pavement subgrade areas from heavy equipment operation or truck/vehicular traffic
- Excavation, grading, and compaction equipment shall be selected to minimize the potential for over-compaction
- Contaminating the various base courses and pavement section with fines and sediment shall be prevented. All contaminated material must be removed and replaced.
- Final subgrade excavation shall be completed during dry weather on the same day storage reservoir is placed when practicable
- Field infiltration test of the permeable surface shall be conducted after complete pavement section is installed. See SPU GSI website (http://www.seattle.gov/util/greeninfrastructure) for testing methodology.

In addition, it is recommended that infiltration testing be performed after subgrade preparation to verify that it meets the design infiltration rate (this is particularly important for facilities).

For inspection and verification procedures used for facility approval by City staff, see GSI web site (http://www.seattle.gov/util/greeninfrastructure).

Operations and Maintenance Requirements

Operations and maintenance requirements for permeable pavement facilities are provided in Appendix D. A plan shall be submitted demonstrating how the operations and maintenance requirements will be met.

The most common concern regarding the long-term effectiveness of permeable pavement is clogging. Because the initial infiltration rates of permeable pavement are so high (hundreds of inches per hour), the long-term infiltration

capacity often remains high (relative to the underlying native soil) even when some clogging occurs. However, great care shall be taken to protect permeable pavement areas from high sediment loads, particularly fine sediment. Pavement permeability can be restored with vacuum sweeping or pressure washing or, in the case of permeable pavers, replacing the aggregate between pavers.

4.4.7.3 Permeable Pavement Surface (without underlying storage reservoir)

Site Considerations

Because permeable pavement *surfaces* are not designed to receive runoff from other surfaces and are designed to function as a permeable land surface, they are not considered infiltration facilities. Therefore, the restrictions related to infiltration facilities (e.g., separation from groundwater, setbacks) are not applicable. However, permeable pavement surfaces shall not be installed over contaminated sites or abandoned landfills. The considerations regarding topography, landuse, and accessibility are the same as those presented for permeable pavement facilities (see Section 4.4.7.2).

Design Criteria

The subsections below provide unique recommendations and requirements specific to permeable pavement surfaces. The design criteria for the following facility components are the same as presented for permeable pavement facilities (see Section 4.4.7.2):

- Wearing course
- Leveling course (if required)
- Non-woven geotextile (if required)
- Native soil / subbase.

Unlike permeable pavement facilities, a maximum subsurface ponding depth is not applied and overflow infrastructure is not required.

Permeable pavement surfaces should not be designed to take significant runoff from other areas (“run-on”). In no case shall the surface receive run-on from an impervious area greater than 10 percent of the pavement area. Any run-on must be dispersed.

Aggregate Subbase

The aggregate subbase in a permeable pavement surface serves as the road, parking, or pedestrian area’s support base and must be sufficiently thick to support the expected loads and be free draining. Unlike the permeable pavement facility, no supplemental aggregate base is included to provide a storage reservoir for additional stormwater detention or attenuation. Minimum requirements associated with the aggregate base course design include the following:

- A 3-inch minimum depth of aggregate subbase is required under the wearing course and leveling course (if any)
- The storage reservoir aggregate composition shall be per manufacturer recommendations. At a minimum, aggregate shall consist of clean, washed material with less than 0.8 percent of material passing the #200 sieve (by weight). All installations shall use angular and crushed material.
- The storage reservoir shall have a minimum total void volume of 20 percent after compacted in place.

BMP Sizing

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), pre-sized permeable pavement surfaces may be used to gain flow control credits toward meeting the Pre-developed Pasture and Peak Control Standards. Credits for permeable pavement surfaces are provided in Table 4.20, organized by flow control standard and subgrade slope. These credits can be applied to reduce the effective impervious surface area used in drainage calculations. A 100 percent credit means that the flow control goal is achieved for the surface and no further control is required for the permeable pavement area. If partial credit is received, the standard is not achieved and additional flow control measures will be required. As an example, for a site subject to the Pre-developed Pasture Standard, a permeable pavement surface on subgrade with a slope between 2 and 5 percent would receive a 45 percent credit. Therefore, 45 percent of the permeable pavement surface can be excluded from drainage calculations. The effective impervious area (area used to size downstream flow control facility) would be calculated as 55 percent of the permeable pavement surface area.

Table 4.20. Flow Control Credits for Permeable Pavement Surfaces.

Subgrade Slope	Credit (%)	
	Pre-developed Pasture Standard	Peak Control Standard
Up to 2%	100%	100%
2 to 5%	45%	70%

%- percent
Impervious Area Mitigated = Permeable Pavement Surface Area x Credit (%)/100.

To use these flow control credits to meet flow control standards, the BMP must meet the general requirements for permeable pavement surfaces outlined in this section plus the following specific requirements:

- The permeable pavement area shall be sized using factor in Table 4.20

- The storage reservoir shall be composed of aggregate with a minimum void volume of 20 percent
- Subgrade slope shall be as specified
- The permeable pavement system shall not have an underdrain or impermeable liner.

For subgrade slopes between 2 and 5 percent flow control performance is lower. The surface may be designed as a facility with subsurface ponding and/or increased aggregate subbase depth to improve performance. In this case, the surface must be evaluated as a facility (see Section 4.4.7.2).

Alternatively, permeable pavement surfaces can be sized using a continuous model as described below.

Facility Modeling

The approved continuous modeling methods for permeable pavement surfaces vary depending upon the slope of the underlying subgrade. For flat and low slope installations of permeable pavement surface (0 to 2 percent), both explicit and implicit methods are appropriate. For higher slope installations (2 to 5 percent), only implicit methods are approved (because currently available explicit modeling methodologies do not take the slope of the subgrade into account).

Explicit Method (for slopes from 0 to 2 percent):

For explicit representation of a permeable pavement surface, the assumptions listed in Table 4.21 shall be applied. It is recommended that permeable pavement be modeled as an impervious area with runoff routed to a gravel-filled infiltration trench (of the same area). The pavement area and aggregate storage reservoir depth may be iteratively sized until the Minimum Requirements for Flow Control are met (see Chapter 2) or excess runoff may be routed to a downstream flow control BMP. General sizing procedures for infiltration facilities are presented in Section 4.3.4.

Implicit Method (for slopes from 0 to 5 percent):

The permeable pavement surface may be modeled as lawn and/or impervious surface (Table 4.22). For subgrade slopes less than 2 percent, the surface may be modeled as lawn over the underlying soil type. For slopes from 2 to 5 percent, the surface may be modeled as half lawn over the underlying soil type and half impervious area.

Table 4.21. Continuous Modeling Assumptions for Permeable Pavement Surface (Explicit Method).

Variable	Assumption
Precipitation Series	Seattle 158-year, 5-minute series
Computational Time Step	5-minutes
Inflows to Facility	Model pavement area as impervious basin routed to a gravel-filled trench with infiltration to underlying soil. No additional areas should drain to the pavement.
Precipitation Applied to Facility	No (applied to basin before routing to trench)
Evaporation Applied to Facility	Yes. While evaporation is applied to the impervious basin before routing to the trench, additional evaporation occurs when water is stored in the storage reservoir
Storage Reservoir Depth	Depth of the storage reservoir.
Storage Reservoir Porosity	Assume maximum 20 percent unless test is provided showing higher porosity for aggregate compacted and in place.
Native Soil Design Infiltration Rate	Measured infiltration rate with correction factor applied (Section 4.3.3, Appendix E).
Infiltration Across Wetted Surface Area	No (bottom area only)
Outlet Structure	Overflow elevation set at top of aggregate subbase. May be modeled as weir flow over riser edge or notch. Note that freeboard must be sufficient to allow water surface elevation to rise above the overflow elevation to provide head for discharge.

Table 4.22. Continuous Modeling Assumptions for Permeable Pavement Surface (Implicit Method).

Variable	Assumption
Precipitation Series	Seattle 158-year, 5-minute series
Computational Time Step	5-minutes
Basin	For slopes 0-2%: Model surface area as lawn overlying existing soil type (e.g., till) at existing slope For slopes 2-5%: Model surface area as half lawn overlying existing soil type (e.g., till) at existing slope and half impervious surface at existing slope

Construction Specifications and Criteria

The construction specifications and criteria for permeable pavement surfaces are the same as those presented for permeable pavement facilities (see Section 4.4.7.2).

Operations and Maintenance Requirements

Operations and maintenance requirements are the same as those presented for permeable pavement facilities (see Section 4.4.7.2) and are provided in

Appendix D. A plan shall be submitted demonstrating how the operations and maintenance requirements will be met.

4.4.8 Green Roofs

Green roofs are areas of living vegetation installed on top of buildings to provide flow control via attenuation, soil storage, and losses to interception, evaporation, and transpiration. Green roofs are also known as ecoroofs, vegetated roofs, and roof gardens. An extensive multi-course green roof installation at Seattle City Hall is shown in Figure 4.21.

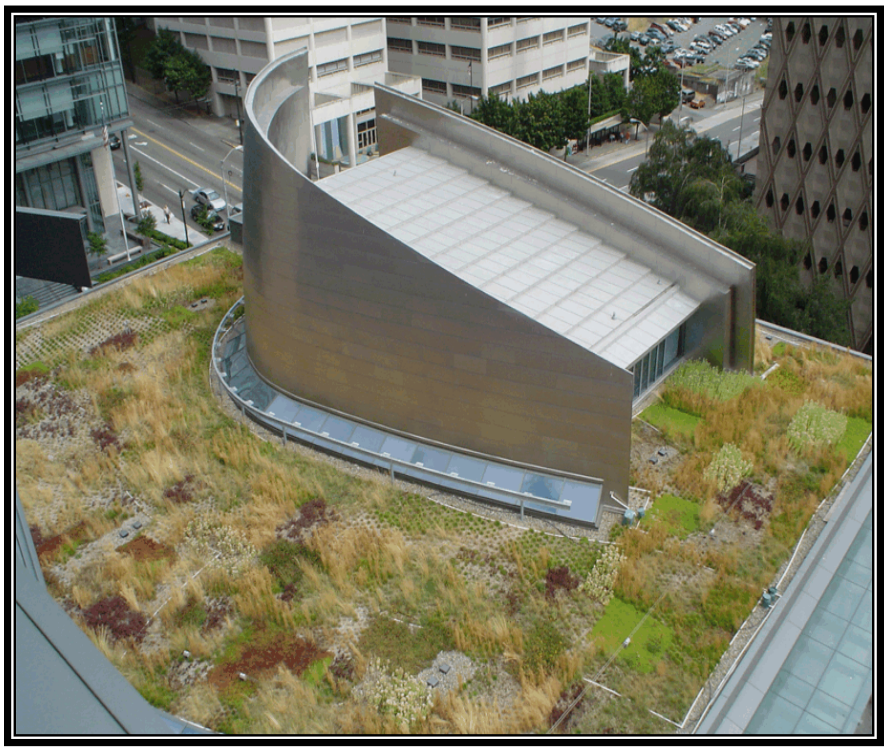


Figure 4.21. Seattle City Hall Green Roof.

A green roof consists of a system in which several materials are layered to achieve the desired vegetative cover and drainage characteristics (see Figure 4.22). Design components vary depending on the green roof type and site constraints, but typically include a waterproofing material, a root barrier, a drainage layer, a separation fabric, a growth medium (soil), and vegetation.

Green roofs are categorized by the depth and the courses used in their construction. Deeper installations, referred to as “intensive” roofs, are comprised of at least 6 inches of growth media and are planted with ground covers, grasses, shrubs and sometimes trees. These systems require regular landscape maintenance. Shallower installations, referred to as “extensive” roofs, are comprised of less than 6 inches of growth media and use a planting palette of drought-tolerant, low maintenance ground covers. Extensive systems are further divided into “single-course” systems which consist of a single media designed to

be freely draining and support plant growth, and "multi-course" systems that include both a growth media layer and a separate, underlying drainage layer. Extensive green roofs have the lowest weight and are typically the most suitable for placement on existing structures.

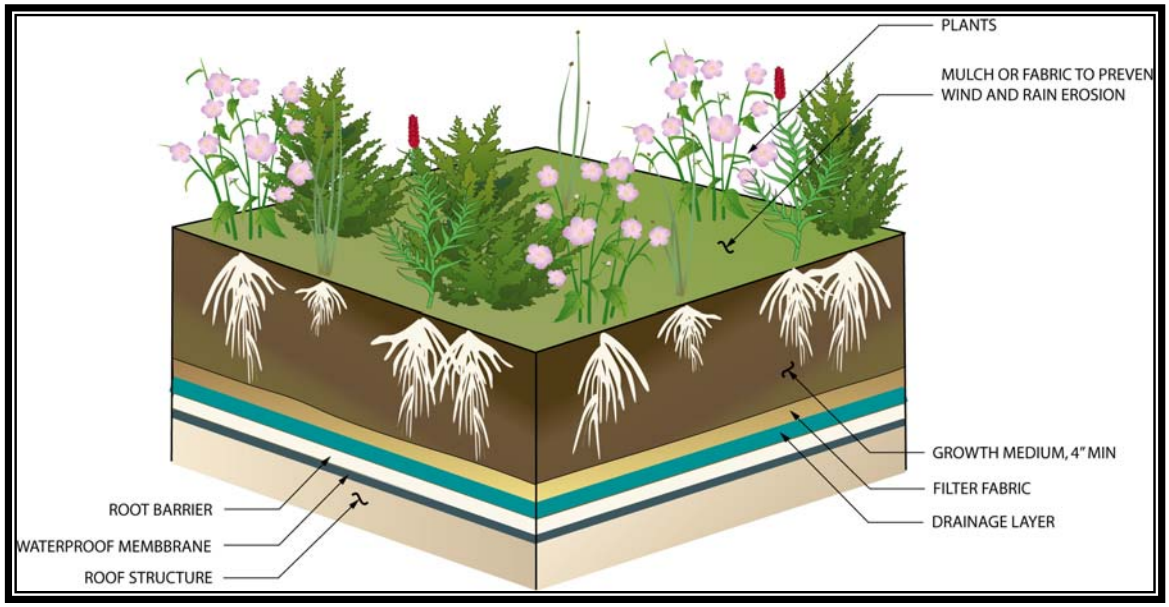


Figure 4.22. Green Roof.

Commercially available "modular" systems consist of prefabricated trays filled with growing media. Modular systems will be evaluated as multi-course systems.

At the time of publication of this manual, the Director accepts the following types of green roofs for flow control compliance:

- Intensive systems
- Extensive multi-course systems (and commercially available modular systems) with at least 4 inches of growth medium
- Extensive single-course systems with at least 4 inches of growth medium for areas less than 1,000 square feet.

Updates will be posted on the SPU GSI website (http://www.seattle.gov/util/greeninfrastructure).

When designed in accordance with the criteria outlined below, green roofs can receive partial credit for the Pre-developed Forest, Peak Flow Control and Pre-developed Pasture Standards (Chapter 2). Green roofs are not an approved water quality treatment BMP.

This section focuses on the guidelines and minimum requirements that must be included in green roof designs to achieve effective stormwater management, and to receive stormwater credit within the City of Seattle. Additional loading, structural, waterproofing, fire resistance, and horticultural considerations must be

taken into account before incorporating green roof systems. While some of these issues are noted, related design methods and requirements are not included. The design team must include qualified professionals to address all design considerations.

4.4.8.1 Applications and Limitations

A primary consideration in the applicability of green roofs is the structural capability of the roof and building structure. Related factors including design load, slipping and shear issues, and wind load are outside the scope of this manual.

Green roofs can be applied to a range of rooftop slopes; however, steeper slopes may result in reduced flow control performance and trigger additional design requirements (e.g., underlying drainage layer and lateral support measures). Nearly flat roofs, those with a pitch of up to 50H:1V (2 percent), are the easiest to install, are the least complex, and generally provide the greatest stormwater storage capacity per inch of growth medium. At the time of publication of this manual, the City of Seattle will only provide stormwater credit for roofs less than 5H:1V (20 percent).

The degree of flow control provided by green roofs varies as a function of growth medium (soil) depth, growth medium composition, drainage layer characteristics, vegetation type, roof slope, and other design choices.

4.4.8.2 Design Criteria

The following sections provide a description and suggested specifications for the common components of green roofs. Submittal for facility review shall include the following elements, described in detail in the subsequent sections:

- Waterproof membrane
- Root barrier
- Drainage layer
- Separation fabric (for multi-course systems)
- Growth medium (soil)
- Vegetation
- Irrigation plan
- Drain system.

Depending upon the system, multiple elements can be combined in one layer. See the SPU GSI website (http://www.seattle.gov/util/greeninfrastructure) for a list of design references.

Waterproof Membrane

Waterproof membranes are made of various materials, including reinforced polyvinyl chloride, synthetic rubber, thermoplastic polyolifins, high-density polyethylene, modified asphalts, and hypalon. Some waterproofing materials come in sheets or rolls and some are available in liquid form. Each material has different strengths and functional characteristics.

The minimum requirements associated with the waterproof membrane design include the following:

- A waterproof membrane shall be included in green roof design.

Root Barrier

To discourage roots from damaging the waterproofing membrane, a physical root barrier may be required. The need for a root barrier depends primarily on the particular waterproof membrane selected. Some waterproofing membranes have root barrier capabilities intrinsic to the material. Modified asphalts usually require a root barrier, while EPDM and reinforced PVC typically do not. The manufacturer must be consulted to determine whether a root barrier is recommended for a particular product.

During installation, treatment to prevent root penetration should not be restricted to parts of the roof that will be covered with vegetation, as the roots will extend beyond the areas in which vegetation shows at the surface. Care should be taken to fully treat the areas at joints, borders, and seams.

The minimum requirements associated with the root barrier design include the following:

- A root barrier shall be included in green roof design
- If waterproofing membrane is to provide root barrier function, supporting manufacturer documentation must be submitted.
- Root barrier shall not contain leachable water quality contaminants (e.g., herbicides, copper, and zinc). To demonstrate, a material safety data sheet must be submitted.

Drainage Layer

For intensive and extensive multi-course green roof systems a drainage layer underlies the growth medium. The drainage layer is a multipurpose layer designed to provide void spaces to hold a portion of the water that passes through the growth medium and to channel the water to the roof drain system. The drainage layer can consist of a layer of aggregate or a manufactured mat or board that provides an open free-draining area. Many manufactured products include “egg carton” shaped depressions that retain a portion of the water for eventual evapotranspiration. Some studies suggest that aggregate drainage layers may provide the better flow control. See the GSI web site (http://www.seattle.gov/util/greeninfrastructure) for updated recommendations.

The minimum requirements associated with the drainage layer design include the following:

- Intensive and extensive multi-course green roofs shall include a drainage layer
- For aggregate drainage layers, the drainage media shall meet the following requirements:
  - Minimum total pore volume of 25 percent by volume (per ASTM E2399)
  - Minimum saturated hydraulic conductivity of 425 inches per hour (per ASTM E2396-05)
  - Maximum total organic matter of 1 percent by mass (per loss-on-ignition test).

For optimal flow control, an aggregate drainage layer with a saturated hydraulic conductivity of less than 4,500 inches per hour is recommended.

Separation Fabric

A separation fabric must be installed between the growth medium (soil) and the drainage layer to prevent fine soil and substrate components from being washed out of the growth medium into the drainage layer. The fabric must be pervious to allow water to percolate into the drainage layer. If a manufactured drainage layer is used, the separation fabric is typically included.

Minimum requirements associated with the separation fabric design include the following:

- For intensive and extensive multi-course green roofs, separation fabric shall be installed between the growth medium and the drainage layer and between the growth medium and all surrounding areas, roof edges, penetrations, and structures
- Separation fabric shall be a non-woven geotextile
- Fabric shall have average opening size sufficient to retain media
- Fabric shall have permissivity sufficient to pass anticipated peak rainfall intensity.

Growth Medium (Soil)

Green roofs use a light-weight growth medium with adequate fertility and drainage capacity to support plants and allow infiltration and storage of water. Growth medium composition (fines content and water holding capacity) is key to flow control performance.

The growth medium typically has a high ratio of mineral to organic material content and can be a mixture of various components including gravel, sand,

compost, soil, or lightweight aggregate material. Because of their excessive weight, particularly when wet, native soils are not typically used for green roofs.

Minimum requirements associated with the growth medium design include the following:

- The growth medium must be a minimum of 4 inches deep, and have the following characteristics:
  - Minimum total pore volume shall be 45 percent by volume for multi-course systems and 30 percent by volume for single-course systems (per ASTM E2399)
  - Water capacity shall be no less than 25 percent for single-course systems, 35 percent for extensive (shallow) multi-course systems, and 45 percent for intensive (deep) multi-course systems (per ASTM E2399)
  - Saturated hydraulic conductivity (permeability) shall be between 14 and 1,200 inches per hour for single-course systems and 2.8 and 28 inches per hour for multi-course systems (per ASTM E2396-05)
  - Minimum air content at maximum water capacity shall be 5 percent by volume (per ASTM E2396-05), or 10 percent by volume (per FLL method)
  - Maximum total maximum organic matter shall be 4 percent by mass for single-course systems, 6 percent by mass for extensive (shallow) multi-course systems, and 8 percent by mass for intensive (deep) multi-course systems (per loss-on-ignition test).

  See the SPU GSI website (http://www.seattle.gov/util/greeninfrastructure) for a growth medium specification that is likely to meet requirements. In addition, approved media testing labs and approved media products are provided.
- Growth medium depth and characteristics must support growth for selected plant species and shall be approved by a licensed landscape architect
- Green roofs must not be subject to any use that will significantly compact the growth medium
- Unless designed for foot traffic, green roof areas that are accessible to the public shall be protected (e.g., signs, railing, and fencing)
- Mulch, mat, or other measures to control erosion of growth media shall be maintained until 90 percent vegetation coverage is achieved.

For increased flow control, it is recommended that the growth medium water holding capacity be designed on the high end of the specified range and the saturated hydraulic conductivity be designed on the low end of the specified range.

Vegetation

Vegetation used on extensive green roofs should be drought tolerant, self-sustaining, low maintenance, and perennial or self-sowing. Appropriate plants should also be able to withstand heat, cold, periodic inundation and high winds. Vegetation with these attributes typically includes succulents, grasses, herbs, and wildflowers that are adapted to harsh conditions. See the Green Factor plant list available via link from the SPU GSI web site (http://www.seattle.gov/util/greeninfrastructure).

Plants can be installed as pre-grown mats, individual plugs, cuttings, or spread as seeds.

- *Pre-grown mats*: Pre-grown mats are sod-like layers that achieve immediate full plant coverage. They provide immediate erosion control, do not need mulch, provide the most rapid establishment for sedums, and minimize weed intrusion. They also need little ongoing watering and weeding after the initial establishment period.
- *Plugs or potted plants*: Plugs or potted plants may provide more design flexibility than mats. However, they take longer to achieve full coverage, are more prone to erosion, need more watering during establishment, require mulching, and require more weeding. Birds sometimes pull out plugs, in which case netting may be needed until they are fully rooted.
- *Cuttings*: While cuttings may have lower initial costs, they establish more slowly than mats and plugs and have a higher mortality rate.
- *Seeds*: Seeds can be either hand broadcast or applied by hydroseeding. Seed plantings require more weeding, erosion control, and watering than mats and plugs.

In the long term, the generation of warm and cold air currents resulting from heating and air-conditioning vents on the rooftop can cause frost and drought damage to plants. Exhaust gases such as sulfur dioxide or grease from chimneys and exhausts can result in direct damage to vegetation, depending on the species. Therefore, areas that are affected by warm air, variable air currents, and exhaust gasses need to be checked carefully to determine whether they are suitable areas for planting and to identify the type of vegetation that is best suited to the particular conditions.

An additional consideration is the effect of providing a green roof habitat. Habitat may be enhanced by using diverse planting and including some larger plants. Some projects sites may not want to encourage wildlife (e.g., birds near air fields).

Minimum requirements associated with vegetation design include the following:

- Plans specify that vegetation coverage of selected plants shall achieve 90 percent coverage within 2 years or additional plantings shall be provided until this coverage requirement is met.
- Plant spacing and plant size shall be designed to achieve specified coverage by licensed landscape architect
- Vegetation must be suitable for harsh (e.g., hot, cold, dry and windy) rooftop conditions
- Plants must not require fertilizer, pesticides or herbicides after 2-year establishment period
- Landscape Management Plan shall be developed and implemented. See Appendix D.

Some project examples are available via link from the SPU GSI web site (http://www.seattle.gov/util/greeninfrastructure).

Irrigation Plan

Provisions must be made for supplemental irrigation during the first two growing seasons after installation to ensure plant survival. Subsurface irrigation methods are preferred. If surface irrigation is the only method available, drip irrigation should be used to deliver water to the base of the plant. At a minimum, a water tap should be available on the roof for manual watering.

A permanent irrigation system using potable water may be used, but an alternative means of irrigation, such as air conditioning condensate or another readily available nonpotable source should be considered to maximize efficient use of resources. Any nonpotable sources must be analyzed to ensure that they do not contain chemicals that might harm or kill the vegetation. Any permanent irrigation system that relies on potable water should be designed to apply no more than 0.2 inches of water every 14 days from June through September, after the 2-year establishment period.

It is recommended that permanent irrigation systems have automatic controls, including a rain shutoff sensor.

Minimum requirements associated with the irrigation design include the following:

- Provisions must be made for irrigation during the first two growing seasons following installation
- Sufficient irrigation shall be provided to achieve and maintain 90 percent plant coverage after 2 years following installation.
- Irrigation design and operation plan shall be included in the Landscape Management Plan. See Appendix D.

Drain System

Drainage facilities must be capable of collecting subsurface and surface drainage and conveying it safely to an approved discharge point. To facilitate subsurface drainage, interceptor drains are often installed at a 15 to 25 foot spacing to prevent excessive moisture build up in the media and convey water to the roof drain.

The roof outlets at green roof sites must be protected from the invasion of plant growth and the entry of loose gravel, and they must be constructed and located so that they are permanently accessible.

Minimum requirements associated with the drain system include the following:

- Green roofs shall include a drain system capable of safely collecting and conveying water to an approved discharge point.

Structural Roof Support and Building Protection

Structural considerations for green roofs include roof slope, design loads (including loads due to ponding), slipping and shear considerations, wind load, and fire resistance. These factors are outside the scope of this manual. All green roof designs should be reviewed and approved by a structural engineer before installation.

4.4.8.3 BMP Sizing

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), flow control credits towards meeting the Pre-developed Pasture and Peak Control Standards may be achieved by applying green roofs. Credits for green roofs are provided in Table 4.23, organized by flow control standard and growth medium depth. These credits can be applied to reduce the effective impervious surface area used in drainage calculations. A 100 percent credit would indicate that the flow control goal is achieved and no further control is required. Because the credits for green roofs are less than 100 percent the standard is not achieved and additional flow control measures will be required. As an example, for a site subject to the Peak Control Standard, a multi-course green roof with 4 inches of growth medium would receive a 71 percent credit. Therefore, 71 percent of the impervious area covered by the green roof can be excluded from drainage calculations. The effective impervious area (area used to size downstream flow control facility) would be calculated as 29 percent of the impervious area covered by the green roof.

The flow control credits outlined above are applicable only if the green roof meets the minimum design requirements outlined in this section and the minimum medium depth listed in Table 4.23.

Table 4.23. Flow Control Credits for Green Roofs.

Courses / Medium Depth	Credit (%)	
	Pre-developed Pasture Standard	Peak Control Standard
Single-Course / 4 inches	46%	71%
Multi-Course / 4 inches	46%	71%
Multi-Course / 8 inches	54%	79%

%- percent
Impervious Area Mitigated = Green Roof Area x Credit (%)/100.

Alternatively, green roofs can be sized using a continuous model as described below.

Facility Modeling

When using continuous modeling to quantify the flow control benefits of green roofs, the assumptions listed in Table 4.24 shall be applied. It is recommended that green roofs be modeled as layers of aggregate with surface flows, interflow, and exfiltrating flow routed to an outlet.

Table 4.24. Continuous Modeling Assumptions for Green Roofs.

Variable	Assumption
Precipitation Series	Seattle 158-year, 5-minute series.
Computational Time Step	5-minutes
Inflows to Facility	None
Precipitation and Evaporation Applied to Facility	Yes
Depth of Material (inches)	Growth medium/soil depth (minimum of 4 inches). Depth of underlying aggregate drainage layer, if any, is neglected.
Vegetative Cover	Ground cover or shrubs. Shrubs are appropriate only when growth medium is at least 6 inches.
Length of Rooftop (ft)	The length of the surface flow path to the roof drain.
Slope of Rooftop (ft/ft)	Flat slope should be set to a minimum slope of 0.001 V:1 H (1,000H:1V).
Discharge from Facility	Surface flow, interflow and exfiltrated flow (groundwater) from green roof module routed to point of compliance.

The medium depth can be modified to achieve various degrees of flow control. Because the Pre-developed Forest, Pre-developed Pasture and Peak Control Standards cannot typically be achieved using a vegetative roof, additional downstream flow control measures may be required.

4.4.8.4 Construction Specifications and Criteria

All green roofs shall be constructed according to the following criteria:

- Growth medium shall be protected from over compaction during construction
- Growth medium and separation fabric shall be isolated from sedimentation during construction
- Root barrier and waterproof membrane shall be checked prior to placement of overlying materials. Any gouges, tears, or stretching shall be repaired in accordance with manufacturer's recommendations.
- Penetrations through waterproof membrane and root barrier shall be checked. Waterproof membrane and root barrier to be affixed to penetration in accordance with manufacturer's recommendations.
- Stockpiling of soil or rock material shall not be permitted on roof structure
- Temporary or permanent point loads from bracing, supports, equipment, and material storage shall not be permitted on drainage matting, root barrier, or waterproof membrane unless shown in an approved plan set.

Flood testing and/or electric field vector mapping to verify the integrity of the membrane before and after green roof materials are installed is highly recommended. For inspection and verification procedures used for facility approval by City staff, see SPU GSI web site (http://www.seattle.gov/util/greeninfrastructure).

4.4.8.5 Operations and Maintenance Requirements

Operations and maintenance requirements for green roof facilities are provided in Appendix D. For non-single family residential projects, a landscape management plan shall be submitted demonstrating how the maintenance requirements will be met. A plan shall be submitted demonstrating how the operations and maintenance requirements will be met.

4.4.9 Full Dispersion

Flow control credit given for "full dispersion" as presented in Ecology's 2005 SWMMWW may also be allowed for projects that can demonstrate they meet all the requirements for full dispersion. Given the large extent of vegetative cover required for full dispersion, these credits will mostly likely only apply to Seattle Parks projects.

4.5 Traditional Infiltration Facilities

Section 4.3 introduced the descriptions, applications and limitations, performance objectives, and general requirements for infiltration facilities. Section 4.3 should be reviewed prior to beginning design of specific infiltration facilities.

In addition to flow control requirements, infiltration facilities can also be designed to meet basic, phosphorus and enhanced water quality treatment requirements (Chapter 2) when the underlying soil meets the treatment soil requirements outlined in Section 5.8.4, and it is shown that at least 91 percent of the total runoff volume for the simulation period is infiltrated. Applicable drawdown requirements must also be met (see Section 5.4.1.2).

The following section provides specific design, maintenance, and construction requirements for infiltration basins and trenches.

4.5.1 Infiltration Basins

Infiltration basins are earthen impoundments used for the collection, temporary storage, and infiltration of stormwater runoff. This section covers siting, sizing, design, and construction criteria specific for infiltration basins (Figure 4.23).

4.5.1.1 Site Considerations

The main site considerations and associated requirements for infiltration basins are outlined in Section 4.3.

In general, infiltration basins typically require a large amount of area. In addition to the area required for the basin, maintenance access must be provided, which can affect the footprint of the infiltration basin and in part determine whether a basin is feasible for a particular site. In a highly developed area like Seattle, large open basins are somewhat uncommon.

4.5.1.2 BMP Sizing

The size of the infiltration basin can be determined by routing the influent runoff file generated by the continuous runoff model through it. In general, an infiltration basin would have two discharge modes. The primary mode of discharge from an infiltration basin is infiltration into the ground. However, when the infiltration capacity of the facility is reached, additional runoff to the basin will cause the facility to overflow. Overflows from an infiltration basin must comply with the Minimum Requirements for Flow Control in Chapter 2, as well as the outflow requirements outlined for detention ponds.

In order to determine compliance with the flow control and/or water quality treatment requirements, an approved continuous hydrologic model (see Chapter 6, Section 6.2.3) must be used. For infiltration facilities sizing procedures see Section 4.3.5.

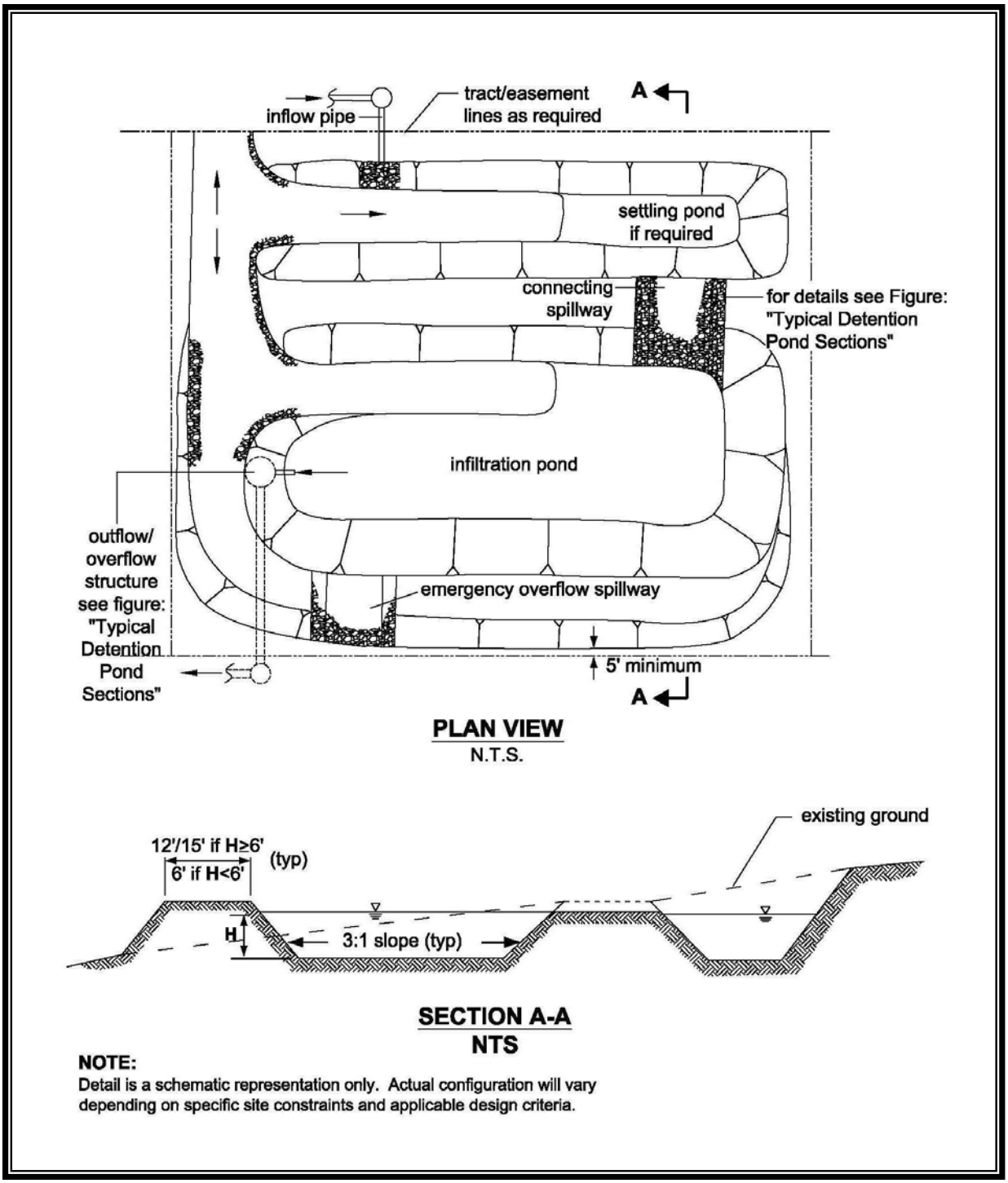


Figure 4.23. Typical Infiltration Pond/Basin.

4.5.1.3 Design Criteria

- Infiltration basins must include a forebay (presettling basin) to remove suspended sediment that might clog the infiltration system and reduce performance. Presettling basins are not required if the basin is preceded by a basic treatment BMP or a presettling BMP (see Chapter 5).
- A minimum of 1 foot of freeboard is required when establishing the design ponded water depth. Freeboard is measured from the rim of the infiltration facility to the maximum ponding level or from the rim down to the overflow point if overflow or a spillway is included.
- The slope of the basin bottom shall not exceed 3 percent in any direction
- Spillways/overflow structures – A non-erodable outlet or spillway with a firmly established elevation must be constructed to discharge overflow. Ponding depth, drawdown time, and storage volume are calculated from that reference point. Approved discharge points shall, at a minimum, maintain natural drainage patterns to the maximum extent feasible, and not cause a significant adverse impact to the downstream conveyance system, receiving surface water, or down-gradient properties. Refer to Section 4.6.3.2 for overflow structure design details.
- Access must be provided for vehicles to easily maintain the forebay (presettling basin) area and not disturb vegetation, or re-suspend sediment any more than is absolutely necessary.
- Lining material – Basins can be open or lined with a 6-to-12-inch layer of filter material such as coarse sand, or a suitable filter fabric to help prevent the buildup of impervious deposits on the soil surface. A non-woven geotextile shall be selected that will function sufficiently without plugging (see geotextile specifications in Appendix F). The filter layer can be replaced or cleaned when/if it becomes clogged.
- For infiltration treatment basins, side-wall seepage is not a concern if seepage occurs through the same stratum as the bottom of the facility. However, for engineered soils or for soils with very low permeability, the potential to bypass the treatment soil through the side-walls may be significant. In those cases, the side-walls must be lined, either with an impervious liner or with at least 18 inches of treatment soil, to prevent seepage of untreated flows through the side walls.
- Treatment infiltration basins must have sufficient vegetation established on the basin floor and side slopes to prevent erosion and sloughing and to provide additional pollutant removal. Erosion protection of inflow points to the basin must also be provided (e.g., riprap, flow spreaders, energy dissipaters). Select suitable vegetative materials for the basin floor and side slopes to be stabilized. Refer to Chapter 5 for required vegetation.

4.5.1.4 Construction Specifications and Criteria

- Initial basin excavation should be conducted to within 1 foot of the final elevation of the basin floor. Excavate infiltration trenches and basins to final grade only after all disturbed areas in the up gradient project drainage area have been permanently stabilized. The final phase of excavation must remove all accumulation of silt in the infiltration facility before putting it in service.
- Infiltration facilities should generally not be used as temporary sediment traps during construction. If an infiltration basin is to be used as a sediment trap, it must not be excavated to final grade until after the up gradient drainage area has been stabilized. Any accumulation of silt in the basin must be removed before putting it in service.
- The development plan sheets must list the proper construction sequence so that the infiltration basin is protected during construction
- Before the development site is graded, the area of the infiltration practices must be roped off and flagged to prevent soil compaction by heavy equipment
- Light earth-moving equipment (backhoes or wheel and ladder type trenchers) must be used to excavate infiltration areas. Heavy equipment can cause soil compaction and reduce infiltration capacity. Compaction of the infiltration area and surrounding soils during construction must be avoided.
- After construction is completed, prevent sediment from entering the infiltration basin by first conveying the runoff water through an appropriate pretreatment system such as a presettling basin, wet pond, or sand filter
- Smearing of soil at the interface of the basin or trench floor and sides must be avoided
- The floor of an infiltration basin must be raked or deep tilled after final grading to restore infiltration rates
- Vegetation – the embankment, emergency spillways, spoil and borrow areas, and other disturbed areas must be stabilized and planted, preferably with grass. Without healthy vegetation the surface soil pores will quickly plug.

4.5.1.5 Operations and Maintenance Requirements

Detailed operations and maintenance requirements for each BMP are provided in Appendix D. Adequate access for operation and maintenance must be included in the design of infiltration basins. A plan shall be submitted demonstrating how the operations and maintenance requirements will be met.

4.5.1.6 Verification of Performance

Verification testing, along with a maintenance program, is strongly recommended during the first 1 to 2 years of operation. Operating and maintaining ground water monitoring wells (specified in Section 4.3.4) is also strongly encouraged.

4.5.2 Infiltration Trenches

Infiltration trenches are backfilled with a coarse stone aggregate, allowing for temporary storage of stormwater runoff in the voids of the aggregate material. Stored runoff gradually infiltrates into the surrounding soil. The surface of the trench can be covered with grating, pavement and/or consist of stone, gabion, sand, or a grassed covered area with a surface inlet. Perforated rigid pipe can also be used to distribute the stormwater in a trench filled with aggregate.

See Figure 4.24 for schematic of an infiltration trench.

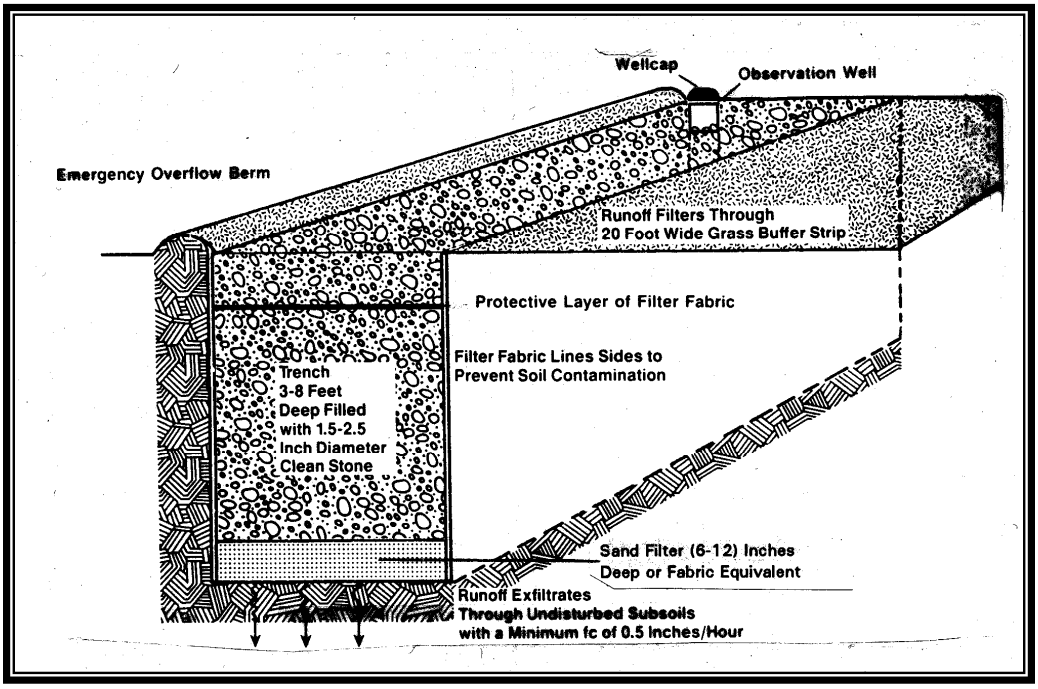


Figure 4.24. Typical Infiltration Trench.

4.5.2.1 Site Considerations

Site considerations for the applicability of infiltration trenches include:

- *Infiltration permitted:* Infiltration trenches must meet the requirements for infiltration facilities presented in Section 4.3. Requirements include horizontal setbacks and a minimum vertical separation from the bottom of the facility to the underlying water table, bedrock, or other impermeable layer.

- *Native Soil Infiltration Rate:* The native soils underlying and surrounding infiltration trenches are a principal factor for determining infiltration capacity and facility sizing. Lower infiltration rates will necessitate a larger trench to achieve flow control goals.
- *Setbacks*: Section 4.3 provides setback for infiltration facilities.
- *Underground Injection Control (UIC)*: Infiltration trenches may be considered UIC systems and subject to Department of Ecology requirements (see Section 4.3.6).

4.5.2.2 Design Criteria

Infiltration trenches must be designed according to the following criteria:

- The minimum width and depth of an infiltration trench shall be 2 feet.
- Trenches shall be filled with uniformly graded washed gravel with nominal size from 1-1/2 – 3/4-inch diameter
- Non-woven geotextile fabric, according to the specifications presented in Appendix F, shall be placed around the walls, bottom and top of the trench aggregate. A 6-inch minimum layer of sand may be used as a filter media at the bottom of the trench instead of geotextile.
- For trenches designed to receive sheet flow, the site must be graded so that runoff is directed in sheet flow across a minimum 10-foot grass buffer strip to remove larger sediment particles
- Trenches designed to receive concentrated flows must include a catch basin with trap and sump upstream of the trench. Concentrated flows must be distributed into the aggregate using a perforated or slotted subsurface pipe. The pipe shall be 4 inches minimum diameter.
- To prevent damage to overlying pavement, trenches located beneath pavement shall be constructed such that the trench pipe is connected to a small yard drain or catch basin with a grate cover so that if the trench infiltration capacity is exceeded, the overflow would occur out of the catch basin at an elevation at least 1 foot below that of any overlying pavement, and in a location that provides a safe path for the overflow.
- The bottom of the trench must be level
- Trenches shall run parallel to site contour lines
- Sides of adjacent trenches shall be a minimum of 5 feet apart
- For projects with more than 5,000 square feet of impervious surface, trenches must be equipped with an observation well to measure the drawdown time following a storm and to monitor sedimentation to determine maintenance needs. Observation wells shall consist of a 4-inch minimum diameter perforated or slotted pipe with a secure locking

well cap that extends to the bottom of the trench, located at a point approximately halfway along the trench length.

- Trenches must have an overflow designed to convey excess flow to an approved discharge point per Section 4.2.5.

4.5.2.3 BMP Sizing

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), pre-sized infiltration trenches may be used to achieve Pre-developed Pasture and Peak Control Standards. Sizing factors for infiltration trenches receiving runoff from an impervious surface are provided in Table 4.25. Factors are organized by flow control standard, trench depth, and native soil design infiltration rate. A 1.5-foot or 3-foot storage reservoir depth may be selected. The storage reservoir is the subsurface aggregate layer below the overflow invert elevation that stores water for infiltration into the underlying native soils. The design rate for the native soils must be rounded down to the nearest infiltration rate in the pre-sized table (i.e., 0.25, 0.5, or 1.0 inch per hour). To use these sizing factors to meet flow control standards, the facility must meet the general requirements for infiltration trenches outlined in this section plus the following specific requirements:

- Trench area shall be sized using the applicable sizing factor
- Storage reservoir depth shall be set at the designated height (1.5 or 3 feet). For intermediate ponding depths (between 1.5 and 3.0 feet), the sizing factor may be linearly interpolated.
- Storage reservoir shall be composed of Mineral Aggregate Type 26 or approved equal
- Bottom area shall be flat (0 percent slope)
- Side slopes shall be vertical
- Invert of overflow shall be set at top of the storage reservoir to allow full depth of storage.

Table 4.25. Sizing Factors for Infiltration Trench Receiving Runoff from Impervious Surface.

Trench Depth	Native Soil Design Infiltration Rate	Sizing Factor for Infiltration Trench Area	
		Pre-developed Pasture Standard	Peak Control Standard
1.5 feet	0.25 inch/hour	27.3%	55.1%
	0.5 inch/hour	16.8%	30.0%
	1.0 inch/hour	10.7%	17.4%
3.0 feet	0.25 inch/hour	17.0%	31.9%
	0.5 inch/hour	12.0%	20.1%
	1.0 inch/hour	7.8%	13.1%

%- percent
Infiltration Trench Area = Contributing Impervious Area x Factor (%)/100.
Impervious Area Mitigated = Trench Area ÷ Factor (%)/100.

The infiltration trench facility area is calculated as a function of the impervious area routed to it. As an example, to meet the Pre-developed Pasture Standard, the trench area would be equal to 27.3 percent of the impervious area routed to it when the native infiltration rate is between 0.25 and 0.5 inches per hour and the trench depth is 1.5 feet.

To use these pre-sized facilities to meet water quality treatment standards in addition to flow control standards, the underlying soil must meet treatment soil requirements specified in Section 5.8.4. To use infiltration trenches for water quality treatment only, see the pre-sized tables in Section 5.8.5.4.

Alternatively, infiltration trench facilities can be sized using a continuous model as described in the subsequent section.

Facility Modeling

When using continuous modeling to size infiltration trenches, the assumptions listed in Table 4.26 shall be applied. It is recommended that infiltration trenches be modeled as a gravel-filled trench with infiltration to underlying soil and an overflow. The tributary areas, trench area, and depth should be iteratively sized until the Minimum Requirements for Flow Control are met (see Chapter 2). General sizing procedures for infiltration facilities are presented in Section 4.3.4.

Table 4.26. Continuous Modeling Assumptions for Infiltration Trench Facilities

Variable	Assumption
Precipitation Series	Seattle 158-year, 5-minute series.
Computational Time Step	5-minutes.
Inflows to Facility	Surface flow and interflow from drainage area routed to facility.
Precipitation and Evaporation Applied to Facility	Yes. If model does not apply precipitation and evaporation to facility, include the facility area in the basin area.
Aggregate Reservoir Depth	Average depth of aggregate below overflow invert.
Aggregate Reservoir Porosity	Assume maximum 20 percent unless test showing higher porosity is provided.
Native Soil Design Infiltration Rate	Measured infiltration rate with correction factor applied (Section 4.3.3, Appendix E).
Infiltration Across Wetted Surface Area	No (bottom area only).
Outlet Structure	Overflow elevation set at average maximum subsurface ponding depth. May be modeled as weir flow over riser edge or notch. Note that freeboard must be sufficient to allow water surface elevation to rise above the overflow elevation to provide head for discharge.

4.5.2.4 Construction Specifications and Criteria

- Infiltration trenches shall not be used as sediment control facilities and all drainage shall be directed away from trench after initial rough grading.

Flow can be directed away from the facility with temporary diversion swales. Trenches shall not be constructed until all contributing drainage areas are stabilized according to erosion and sediment control BMPs and to the satisfaction of the engineer. Erosion and sediment control practices must be inspected and maintained on a regular basis. If deposition of fines occurs in the trench bottom, material shall be removed and the surface scarified to the satisfaction of the engineer.

- The development plan sheets must list the proper construction sequence so that the infiltration trench is protected during construction
- Before the development site is graded, the area of the infiltration must be roped off and flagged to prevent soil compaction by heavy equipment.
- Light earth-moving equipment (backhoes or wheel and ladder type trenchers) must be used to excavate infiltration areas. Heavy equipment can cause soil compaction and reduce infiltration capacity. Compaction of the infiltration area and surrounding soils during construction must be avoided.
- Excavated materials must be placed away from the trench sides to enhance trench wall stability. Care should also be taken to keep this material away from slopes, neighboring property, sidewalks and streets.
- Unstable Excavation Sites – Vertically excavated walls may be difficult to maintain in areas where the soil moisture is high or where soft or cohesionless soils predominate. Trapezoidal, rather than rectangular, cross-sections may be needed.
- Stone Aggregate Placement and Compaction – The stone aggregate must be placed in lifts and compacted using plate compactors. As a rule of thumb, a maximum loose lift thickness of 12 inches is recommended. The compaction process ensures geotextile conformity to the excavation sides, thereby reducing potential piping and geotextile clogging, and settlement problems.
- Potential Contamination – Prevent natural or fill soils from intermixing with the stone aggregate. All contaminated stone aggregate must be removed and replaced with uncontaminated stone aggregate.
- Following the stone aggregate placement, the geotextile must be folded over the stone aggregate to form a 12-inch minimum longitudinal overlap. When geotextile overlaps are required between rolls, the upstream roll must overlap a minimum of 2 feet over the downstream roll in order to provide a shingled effect.
- Voids behind Geotextile – Voids between the geotextile and excavation sides must be avoided. Removing boulders or other obstacles from the trench walls is one source of such voids. Natural soils should be placed in these voids at the most convenient time during construction to ensure

geotextile conformity to the excavation sides. Soil piping, geotextile clogging, and possible surface subsidence will be avoided by this remedial process.

- Smearing of soil at the interface of the trench floor and sides should be avoided
- The sides and bottom of an infiltration trench must be raked or scarified after the trench is excavated to restore infiltration rates.

4.5.2.5 Operations and Maintenance Requirements

General operations and maintenance requirements for infiltration basins apply to infiltration trenches. Detailed operations and maintenance requirements for infiltration facilities are provided in Appendix D. A plan shall be submitted demonstrating how the operations and maintenance requirements will be met.

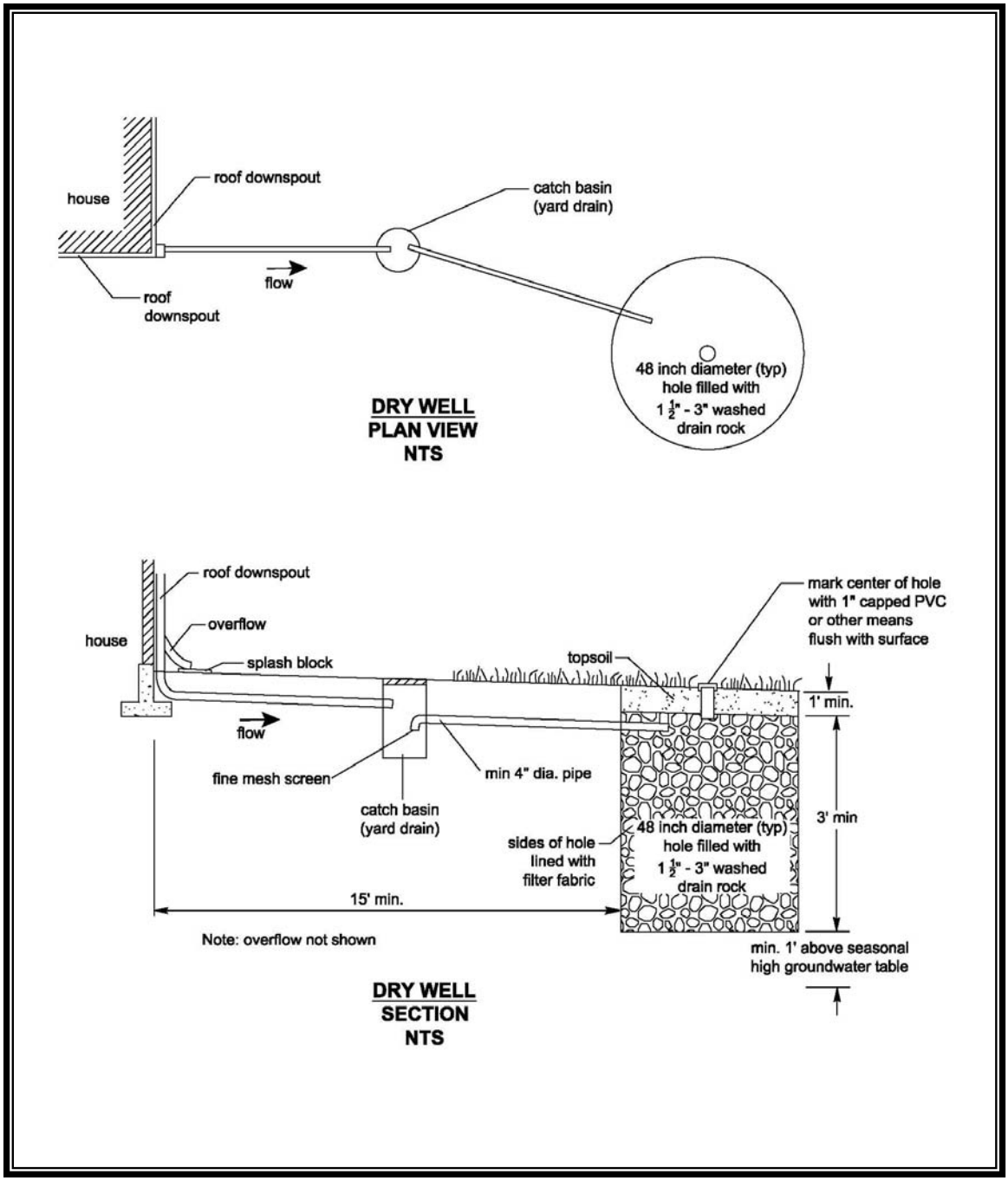
4.5.3 Dry Wells

Dry wells are similar to infiltration trenches but are typically deeper require less site area. Water is delivered to the drywell by pipe. Figure 4.25 shows a typical drywell system.

4.5.3.1 Site Considerations

Site considerations for the applicability of drywells include:

- *Infiltration permitted:* Drywells must meet the requirements for infiltration facilities presented in Section 4.3. Requirements include horizontal setbacks and a minimum vertical separation from the bottom of the facility to the underlying water table, bedrock, or other impermeable layer.
- *Native Soil Infiltration Rate:* The native soils underlying and surrounding drywells are a principal factor for determining infiltration capacity and well sizing. Lower infiltration rates will necessitate a larger facility to achieve flow control goals.
- *Setbacks*: Section 4.3 provides setback for infiltration facilities.
- *Underground Injection Control (UIC)*: Drywells may be considered UIC systems and subject to Department of Ecology requirements (see Section 4.3.6).



Source: King County

Figure 4.25. Typical Infiltration Drywell.

4.5.3.2 Design Criteria

Minimum requirements associated with the drywell design include the following:

- The minimum depth of a drywell shall be 3 feet.
- Drywells shall be filled with uniformly graded washed gravel with nominal size from 1-1/2-inch to 3 inches diameter.
- Non-woven geotextile fabric shall be placed around the walls, bottom and top of the drywell aggregate. A 6-inch minimum layer of sand may be used as a filter media at the bottom of the well instead of geotextile.
- Flows shall be delivered to the drywell aggregate using a perforated or slotted subsurface pipe. The pipe shall be 4 inches minimum diameter.
- Inflows must be routed through a catch basin with trap and sump upstream of the trench.
- Spacing between drywells must be a minimum of 4 feet.
- For projects with more than 5,000 square feet of impervious surface, dry wells must be equipped with an observation well to measure the drawdown time following a storm and to monitor sedimentation to determine maintenance needs Observation wells shall consist of a 4-inch minimum diameter perforated or slotted pipe with a secure locking well cap that extends to the bottom of the well.
- Dry wells must have an overflow designed to convey excess flow to an approved discharge point per Section 4.2.5.

4.5.3.3 BMP Sizing

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), pre-sized drywells may be used to achieve Pre-developed Pasture and Peak Control Standards. Sizing factors for drywells receiving runoff from an impervious surface are provided in Table 4.27. Factors are organized by flow control standard, drywell depth, and native soil design infiltration rate. A 4-foot or 6-foot storage reservoir depth may be selected. The storage reservoir is the subsurface aggregate layer below the overflow invert elevation that stores water for infiltration into the underlying native soils. The design rate for the native soils must be rounded down to the nearest infiltration rate in the pre-sized table (i.e., 0.25, 0.5, or 1.0 inch per hour). To use these sizing factors to meet flow control standards, the facility must meet the general requirements for drywells outlined in this section plus the following specific requirements:

- Drywell area shall be sized using the applicable sizing factor
- Storage reservoir depth shall be set at the designated height (4 or 6 feet). For intermediate ponding depths (between 4 and 6 feet), the sizing factor may be linearly interpolated.

- Storage reservoir shall be composed of Mineral Aggregate Type 26 or approved equal
- Bottom area shall be flat (0 percent slope)
- Side slopes shall be vertical
- Invert of overflow shall be set at top of the storage reservoir to allow full depth of storage.

Table 4.27. Sizing Factors for Drywells Receiving Runoff from Impervious Surface.

Drywell Depth	Native Soil Design Infiltration Rate	Sizing Factor for Drywell Area	
		Pre-developed Pasture Standard	Peak Control Standard
4.0 feet	0.25 inch/hour	13.6%	24.9%
	0.5 inch/hour	10.1%	17.6%
	1.0 inch/hour	6.7%	11.1%
6.0 feet	0.25 inch/hour	9.7%	17.5%
	0.5 inch/hour	7.9%	13.2%
	1.0 inch/hour	5.7%	8.9%

%- percent
Drywell Area = Contributing Impervious Area x Factor (%)/100.
Impervious Area Mitigated = Drywell Area ÷ Factor (%)/100.

The drywell facility area is calculated as a function of the impervious area routed to it. As an example, to meet the Pre-developed Pasture Standard, the well area would be equal to 7.9 percent of the impervious area routed to it when the native infiltration rate is between 0.5 and 1.0 inches per hour and the well depth is 6.0 feet.

To use these pre-sized facilities to meet water quality treatment standards in addition to flow control standards, the underlying soil must meet treatment soil requirements (Section 5.8.4). To use drywells for water quality treatment only, see the pre-sized tables in Section 5.8.5.4.

Alternatively, drywell facilities can be sized using a continuous model as described in the subsequent section.

Facility Modeling

Continuous modeling may be used to size drywells using the procedures presented for infiltration trenches, and the infiltration facility sizing procedures presented in Section 4.3.4.

4.5.3.4 Construction Specifications and Criteria

Construction specifications and criteria are the same as presented for infiltration trenches (Section 4.5.2.4).

4.5.3.5 Operations and Maintenance Requirements

General operations and maintenance requirements for infiltration basins apply to drywells. Detailed operations and maintenance requirements for infiltration facilities are provided in Appendix D.

4.6 Detention Facilities

Detention facilities provide for the temporary storage of increased surface water runoff resulting from development. Stormwater is then released through a control structure (described at the end of this section) at an attenuated rate to meet flow control performance standards.

When designed in accordance with Seattle's Flow Control Minimum Requirements (see Chapter 2), detention facilities reduce flow durations and/or peak flows to more closely match pre-development hydrology, thereby reducing the impacts of the proposed development on downstream receiving waters.

4.6.1 Applications and Limitations

Because detention facilities only detain stormwater runoff and do not provide infiltration, post-development runoff volumes from projects utilizing detention facilities will often exceed predevelopment volumes. This can potentially impact downstream receiving waters by reducing ground water recharge and stream base flow, and contributing to high runoff volumes during storm events. It is preferable to modify the site design to maximize onsite flow control (and treatment) to reduce or eliminate the need for detention. If flow control is required, green stormwater infrastructure is required to the maximum extent feasible, and infiltration facilities are preferable to detention.

Of particular note for detention facilities is that the minimum allowable orifice diameter is 0.5 inches for belowground detention systems and 0.25 inches for aboveground detention systems. In some instances the minimum bottom orifice diameter will be too large to meet standard release rates, even with minimal head. Designers should iteratively increase detention area and decrease live storage depth until the performance criteria are met. However, live storage depth need not be reduced to less than 3 feet in an attempt to meet the flow control standards.

As a rule of thumb, flow control standards cannot be achieved in Seattle using a 0.5 inch diameter bottom orifice with a 3-foot live storage depth in the following scenarios:

– Pre-developed Forest Standard cannot be achieved when the contributing impervious area is less than approximately 35,000 square feet.

– Pre-developed Pasture Standard cannot be achieved when the contributing impervious area is less than approximately 26,000 square feet.

– Peak Control Standard cannot be achieved when the contributing impervious area is less than approximately 2,600 square feet.

In these cases, the following design/modeling approach is recommended. A detention vessel should be sized with an orifice of minimum diameter and a 3-foot live storage depth such that there are no overflows during the 158-year period of record.

Due to the reduced flow control benefits with smaller contributing areas, the minimum area from which runoff is routed to an underground detention facility shall be 2,000 square feet.

Designers and developers are encouraged to consider creative opportunities for providing detention, when it is required. Athletic fields, roofs, parking lots that are not continually in use, and other large surface areas may provide opportunities for stormwater storage in addition to other uses. Additional information is presented in Section 4.6.7.

In addition rainwater harvesting is encouraged when permitted (see Section 4.6.6).

In addition, designers may wish to combine stormwater detention with water quality treatment. Combined detention and water quality wetpool facilities have the appearance of a detention facility but contain a permanent pool of water as well. Chapter 5 presents design procedures, requirements, and recommendations for the following combined detention/water quality treatment facilities:

- Detention/wetpond (basic and large)
- Detention/wetvault
- Detention/stormwater wetland.

4.6.2 Operations and Maintenance Requirements

Operations and maintenance requirements for detention facilities are provided in Appendix D. A plan shall be submitted demonstrating how the operations and maintenance requirements will be met.

4.6.3 Detention Pond

Detention ponds are surface water basins that temporarily store runoff to decrease downstream discharge peak flows and peak flow durations. Detention ponds are typically designed to drain completely between storm events, unless they are designed as a combination water quality treatment and flow control BMP (see Chapter 7).

The design criteria in this section are for detention ponds. However, many of the criteria also apply to infiltration ponds (Section 4.5), as well as water quality wetponds and combined detention/wetponds (Chapter 5).

4.6.3.1 Site Considerations

Detention ponds require a large amount of area. In addition to the area required for the pond, maintenance access must be provided, which can affect the footprint of the infiltration basin and in part determine whether they are feasible for a particular site. In a highly developed area like Seattle, large open ponds are somewhat uncommon.

Setbacks

Setback requirements are intended to protect neighboring properties from flooding and protect receiving waters and other sensitive areas from water quality impacts. The location of the pond relative to site constraints (e.g., buildings, property lines, etc.) shall meet the following criteria:

- A setback of 5 feet from the toe of the exterior slope of the tract or property line
- The tract or property line on a pond cut slope shall be set back 5 feet from the emergency overflow water surface
- The pond water surface at the outlet invert elevation shall be set back 100 feet from existing septic system drainfields**.** This setback may be reduced with written approval of the Seattle-King County Department of Public Health and subsequent acceptance by both SPU and DPD Directors.
- Geotechnical analysis is required for facilities within 20 feet from any structure or property line or within 50 feet up-slope of a building when the slope is greater than 5 percent.
- All facilities must be a minimum of 50 feet from the top of any slope steeper than 5 percent. A geotechnical analysis and report must be prepared addressing the potential impact of the facility on the slope.

The requirements for slopes, embankments, and fencing are intended to accomplish the following objectives:

- To prevent persons from inadvertently slipping into the pond, either by providing gentle interior side slopes (3 H:1 V or gentler) or by fencing or other barrier
- To allow easy egress from the pond (gentle side slopes, safety benches, etc.) when access is not restricted by a fence or other barrier
- To ensure interior and exterior slopes or embankments are stable and will not create a hazardous or damaging situation.

Constraints may also be imposed from natural features such as requirements found in City of Seattle Critical Areas Ordinance SMC 25.09. These must also be reviewed for specific application to the proposed development.

4.6.3.2 BMP Sizing and Design Criteria

Standard details for detention ponds are shown in Figure 4.26 and Figure 4.27 Control structure details are provided in Section 4.6.8.

Design Method

The volume and outflow design for detention ponds must be in accordance with the Minimum Requirements outlined in Chapter 2 and must be designed using continuous runoff modeling as described in Chapter 5.

Ponds must be designed as flow-through systems. Flows must enter through a conveyance system separate from the control structure and outflow conveyance system. Maximizing the distance between the inlet and outlet is encouraged to promote sedimentation. Detention pond bottoms must be level and be located a minimum of 0.5 foot (preferably 1 foot) below the inlet and outlet to provide sediment storage.

Detention Ponds in Infiltrative Soils

Detention ponds may be sited on soils that are sufficiently permeable for a properly functioning infiltration system (see Sections 4.3 and 4.5). These detention ponds have a surface discharge and may also utilize infiltration as a second pond outflow. Detention ponds sized with infiltration as a second outflow must meet all the requirements of Section 4.5.1 for infiltration ponds, including a soils report, testing, groundwater protection, presettling, and construction techniques.

Side Slopes

Interior side slopes must not be steeper than 3 H:1 V unless a fence is provided (see “Fencing”).

Pond walls may be vertical retaining walls, provided they meet the following criteria:

- They are constructed of reinforced concrete and the design is stamped by a licensed civil engineer with structural expertise.
- A fence is provided around the pond.

Overflow

A primary overflow (usually a riser pipe within the control structure (see Section 4.6.8) must be provided to bypass the peak flows with a 1 percent annual probability (100-year recurrence interval) over or around the restrictor system. The design must provide controlled discharge directly into the downstream conveyance system or another approved discharge point.

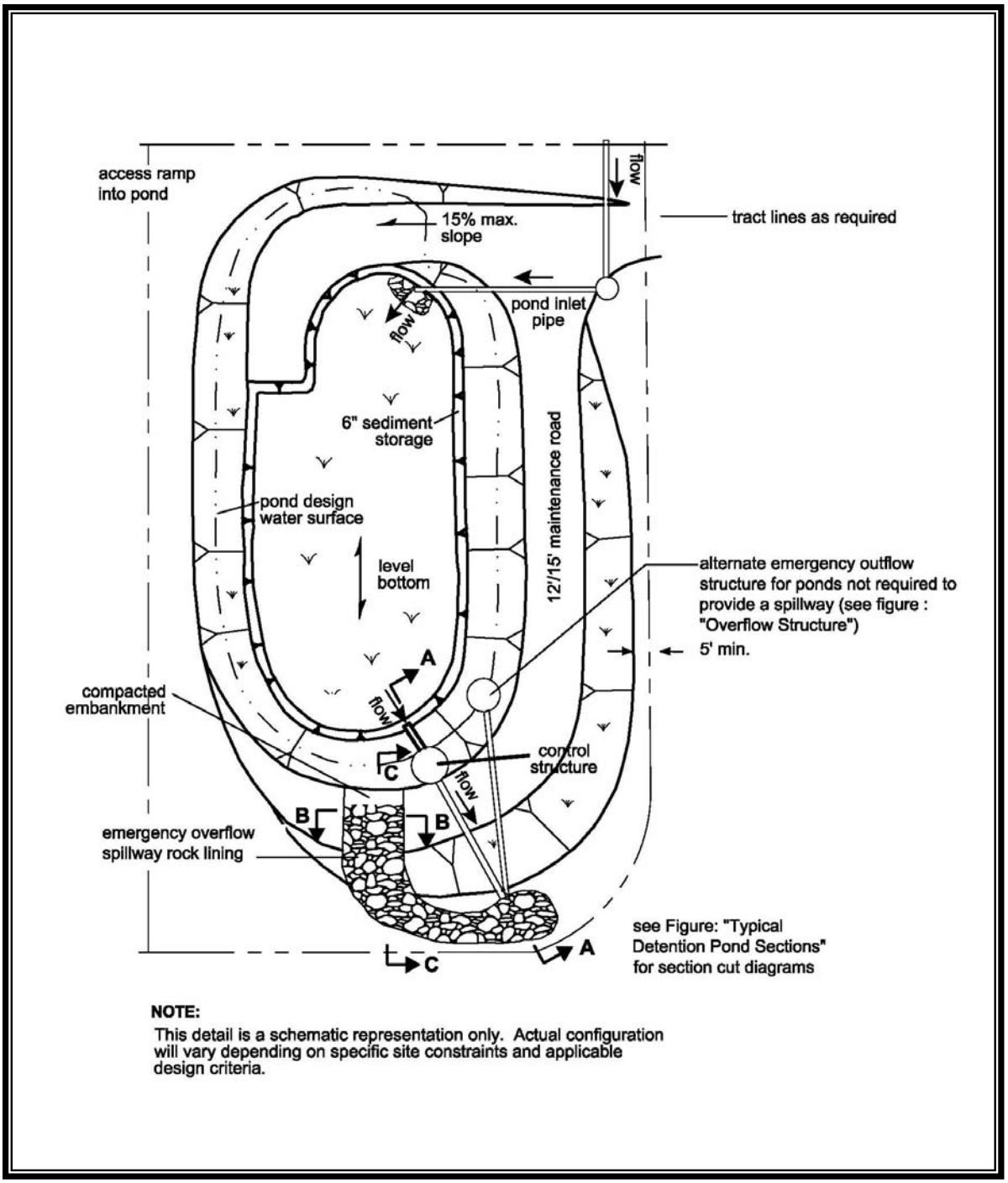


Figure 4.26. Typical Detention Pond.

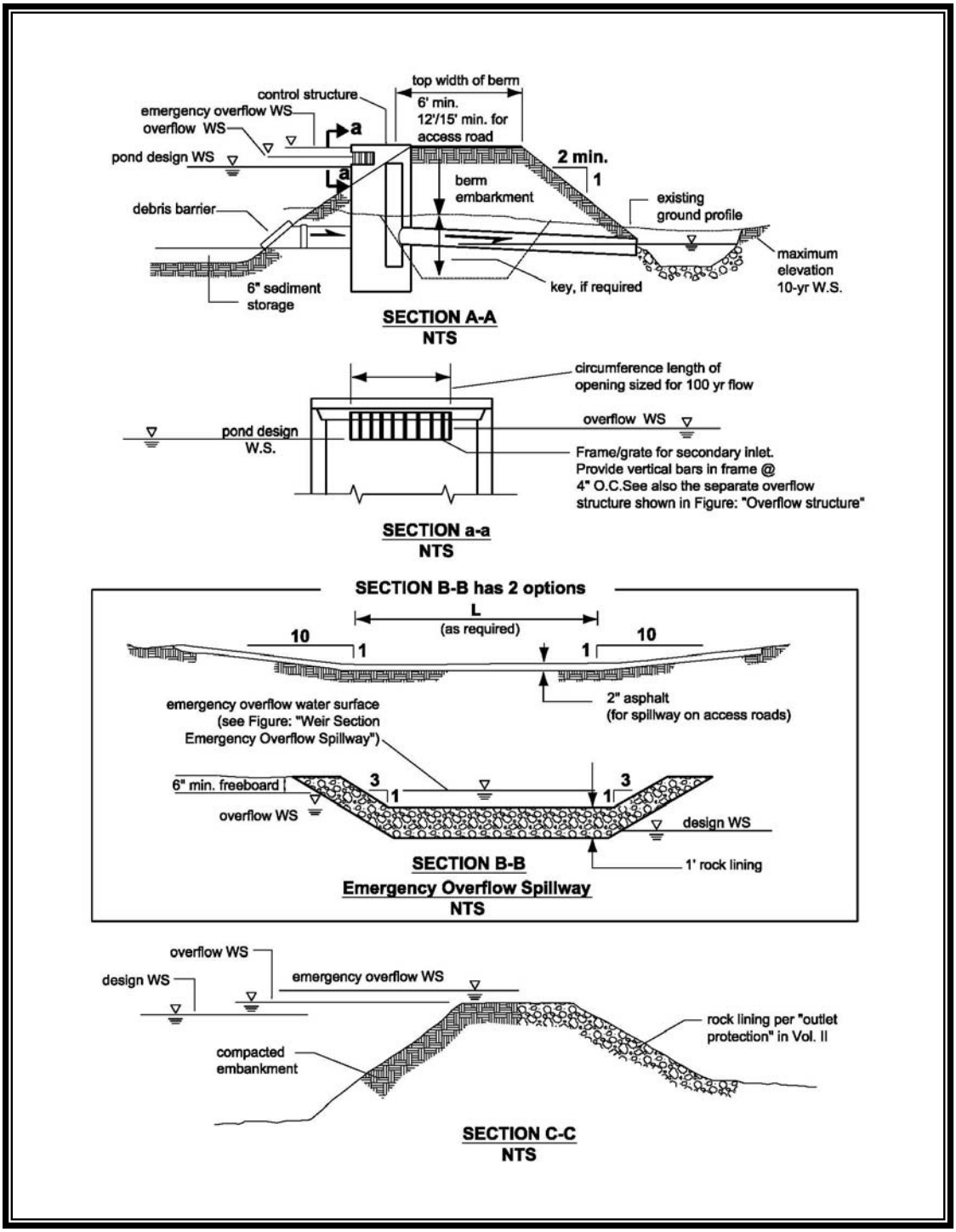


Figure 4.27. Typical Detention Pond Sections.

A second inlet to the control structure must be provided in ponds as additional protection against overtopping should the inlet pipe to the control structure become plugged. A grated opening (i.e., a "jailhouse window") in the control structure manhole functions as a weir when used as a second inlet. The "birdcage" overflow structure as shown in Figure 4.28 may also be used as a secondary inlet.

Emergency Overflow Spillway

In addition to the overflow provisions outlined above, ponds must have an emergency overflow spillway. For impoundments of 10-acre-feet or greater, the emergency overflow spillway must meet the state's dam safety requirements (see below). For impoundments under 10-acre-feet, ponds must have an emergency overflow spillway that is sized to pass the 100-year developed peak flow in the event of total control structure failure (e.g., blockage of the control structure outlet pipe) or extreme inflows. Emergency overflow spillways are intended to control the location of pond overtopping and direct overflows back into the downstream conveyance system or other approved discharge point.

Emergency overflow spillways must be provided for ponds with constructed berms over 2 feet in height, or for ponds located on grades in excess of 5 percent. As an option for ponds with berms less than 2 feet in height and located at grades less than 5 percent, emergency overflow may be provided by an emergency overflow structure, such as a Type 200 Manhole fitted with a birdcage as shown in Figure 4.28. The emergency overflow structure must be designed to pass the 100-year developed peak flow, with a minimum 6 inches of freeboard, directly to the downstream conveyance system or another approved discharge point. Where an emergency overflow spillway would discharge to a steep slope, consideration must be given to providing an emergency overflow structure in addition to the spillway.

The emergency overflow spillway must be armored with riprap in conformance with the "Outlet Protection" BMP in Volume 1, the Construction Stormwater Control Technical Requirements Manual. The spillway must be armored full width, beginning at a point midway across the berm embankment and extending downstream to where emergency overflows re-enter the conveyance system (see Figure 4.27).

Emergency overflow spillway designs must be analyzed as broad-crested trapezoidal weirs as described in Methods of Analysis at the end of this section. Either one of the weir sections shown in Figure 4.27 may be used.

Emergency Overflow Spillway Capacity:

For impoundments under 10-acre-feet, the emergency overflow spillway weir section must be designed to pass the 100-year runoff event for developed conditions assuming a broad-crested weir. The **broad-crested weir equation** for the spillway section in Figure 4.29, for example, would be:

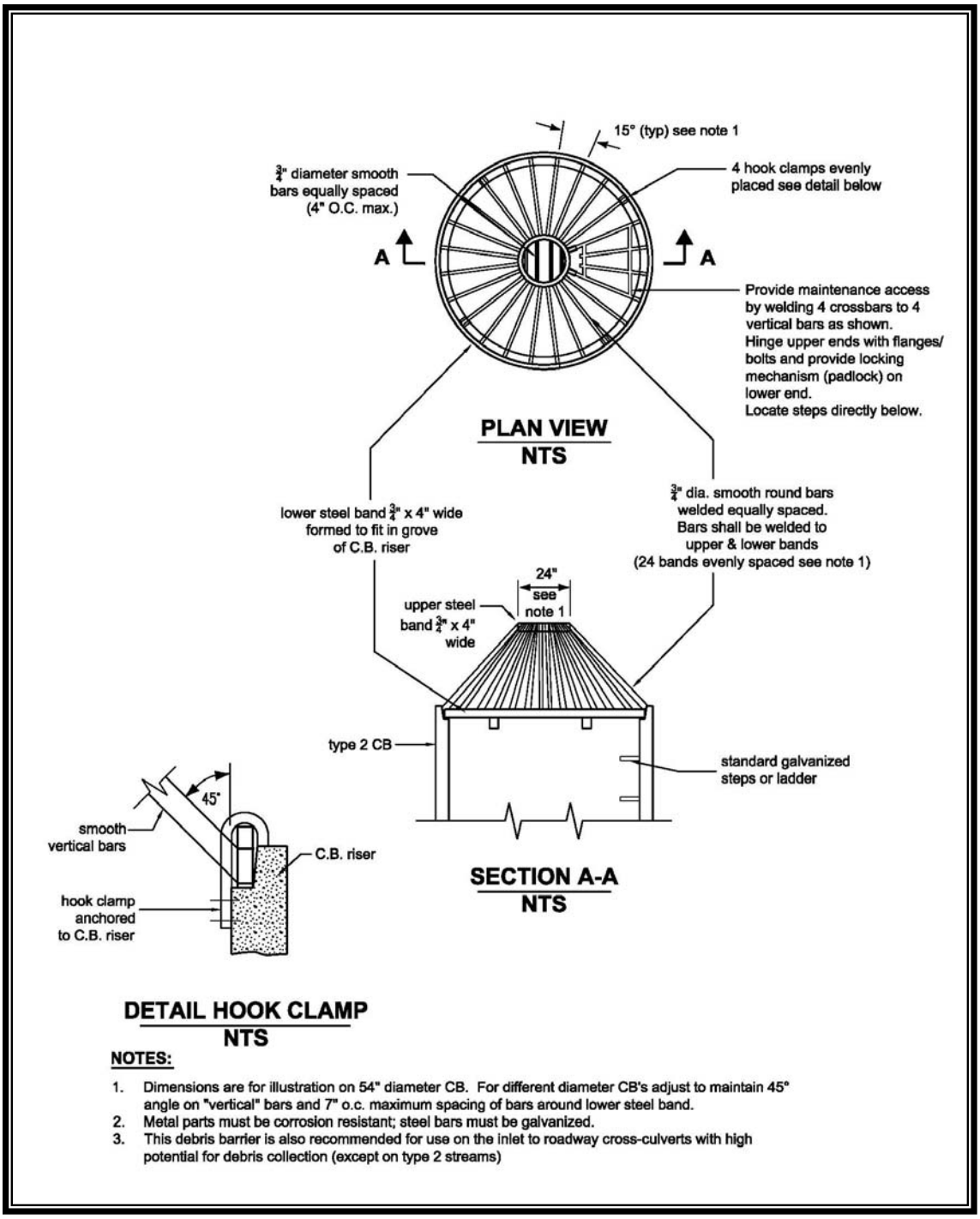


Figure 4.28. Typical Overflow Structure.

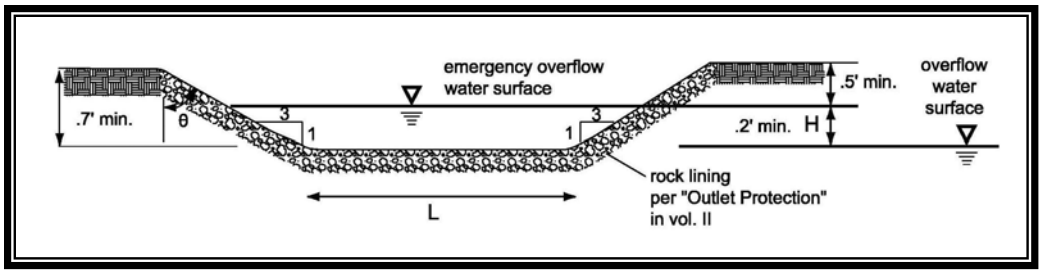


Figure 4.29. Typical Weir Section for Emergency Overflow Spillway.

$$Q_{I00} = C\,(2g)^{1/2}\left[\frac{2}{3}LH^{3/2} + \frac{8}{15}\,(\mathrm{Tan}\,\theta)\,H^{5/2}\right] \qquad (1)$$

Where Q_{I00}	=	peak flow for the 100-year runoff event (cfs)
C	=	discharge coefficient (0.6)
g	=	gravity (32.2 ft/sec^2)
L	=	length of weir (ft)
H	=	height of water over weir (ft)
θ	=	angle of side slopes

Q_{100} is either the peak 10-minute flow computed from the 100-year, 24-hour storm and a Type 1A distribution, or the 100-year, 1-hour flow, indicated by an approved continuous runoff model, multiplied by a factor of 1.6.

Assuming C = 0.6 and Tan θ = 3 (for 3:1 slopes), the equation becomes:

$$Q_{I00} = 3.21[LH^{3/2} + 2.4\,H^{5/2}] \qquad (2)$$

To find width *L* for the weir section, the equation is rearranged to use the computed Q_{I00} and trial values of H (0.2 feet minimum):

$$L = [Q_{I00}/(3.21\,H^{3/2})] - 2.4\,H \quad \text{or} \quad 6 \text{ feet minimum} \qquad (3)$$

Access

The following requirements for access must be used:

- Maintenance access road(s) must be provided to the control structure and other drainage structures associated with the pond (e.g., inlet or bypass structures). Manhole and catch basin lids must be in or at the edge of the access road and at least 3 feet from a property line.
- An access ramp is needed for removal of sediment with a trackhoe and truck. The ramp must extend to the pond bottom if the pond bottom area is greater than 1,500 square feet (measured without the ramp) and it may end at an elevation 4 feet above the pond bottom, if the pond bottom area is less than 1,500 square feet (measured without the ramp).
- On large, deep ponds, truck access to the pond bottom via an access ramp is necessary so loading can be done in the pond bottom. On small deep ponds, the truck can remain on the ramp for loading. On small

shallow ponds, a ramp to the bottom may not be required if the trackhoe can load a truck parked at the pond edge or on the internal berm of a detention pond. If the internal berm is to be used for maintenance access, it must be designed to support a loaded truck, considering the berm is normally submerged and saturated.

- Access ramps must meet the requirements for design and construction of access roads specified below.
- If a fence is required, access shall be limited by a double-posted gate.

Design of Access Roads

The design recommendations for access road are given below:

- Maximum grade shall be 15 percent.
- Outside turning radius shall be a minimum of 40 feet.
- Fence gates shall be located only on straight sections of road.
- Access roads shall be 15 feet in width on curves and 12 feet on straight sections.
- A paved apron must be provided where access roads connect to paved public roadways.
- Access roads may be constructed with an asphalt or gravel surface, or modular grid pavement.

Fencing

It is recommended that fences be 6 feet in height. For example designs, see City of Seattle Standard Plan Number 450a, Type 1 or Type 3 chain link fence.

Embankments

If an embankment is proposed to impound water, early conversations with SPU are encouraged and Director approval is required. Impoundment of a water volume exceeding 10-acre-feet is considered a dam regulated by the Department of Ecology.

Dam Safety

Stormwater detention facilities that can impound 10-acre-feet (435,600 cubic feet; 3.26 million gallons) or more with the water level at the embankment crest are subject to the state's dam safety requirements, even if water storage is intermittent and infrequent (WAC 173-175-020(1)). The principal safety concern is for the downstream population at risk if the dam should breach and allow an uncontrolled release of the pond contents.

The Dam Safety Office is located in the Ecology headquarters building in Lacey. Electronic versions of the guidance documents in PDF format are available on the Department of Ecology Web site at (http://www.ecy.wa.gov/programs/wr/dams/dss.html).

Signage

Detention ponds, infiltration ponds, wetponds, and combined ponds must have a sign placed for maximum visibility from adjacent streets, sidewalks, and paths. The sign must identify the facility as a stormwater detention pond and provide owner contact information.

Planting Requirements

Landscaping is encouraged for most stormwater tract areas (see below for areas not to be landscaped). However, if provided, landscaping must adhere to the criteria that follow so as not to hinder maintenance operations. Landscaped stormwater tracts may, in some instances, provide a recreational space. In other instances, "naturalistic" stormwater facilities may be placed in open space tracts.

The following requirements apply if landscaping is proposed for detention facilities.

- No trees or shrubs may be planted within 10 feet of inlet or outlet pipes or manmade drainage structures such as spillways or flow spreaders. Species with roots that seek water, such as willow, birch or poplar, should be avoided within 50 feet of pipes or manmade structures.
- Planting should be restricted on berms that impound water either permanently or temporarily during storms. This restriction does not apply to cut slopes that form pond banks, only to berms.
    - Trees or shrubs may not be planted on portions of water-impounding berms taller than 4 feet high. Only grasses may be planted on berms taller than 4 feet.
    - Grasses allow unobstructed visibility of berm slopes for detecting potential dam safety problems such as animal burrows, slumping, or fractures in the berm
    - Trees planted on portions of water-impounding berms less than 4 feet high must be small, not higher than 20 feet mature height, and have a fibrous root system. Table 4.28 gives some examples of trees with these characteristics developed for the central Puget Sound.
    - These trees reduce the likelihood of blow-down trees, or the possibility of channeling or piping of water through the root system, which may contribute to dam failure on berms that retain water.
- All landscape material, including grass, must be planted in good topsoil. Native underlying soils may be made suitable for planting if amended with 4 inches of well-aged compost tilled into the subgrade. Compost used must meet the specifications for compost quality as described in Section 4.4.1.
- Soil in which trees or shrubs are planted may need additional enrichment or additional compost top-dressing.

Table 4.28. Small Trees and Shrubs with Fibrous Roots.

Small Trees / High Shrubs	Low Shrubs
*Red twig dogwood (*Cornus stolonifera*)	*Snowberry (*Symporicarpus albus*)
*Serviceberry (*Amelanchier alnifolia*)	*Salmonberry (*Rubus spectabilis*)
*Filbert (*Corylus cornuta*, others)	Rosa rugosa (avoid spreading varieties)
Highbush cranberry (*Vaccinium opulus*)	Rock rose (*Cistus* spp.)
Blueberry (*Vaccinium spp.*)	Ceanothus spp. (choose hardier varieties)
Fruit trees on dwarf rootstock	New Zealand flax (*Phormium penax*)
Rhododendron (native and ornamental varieties)	Ornamental grasses (e.g., *Miscanthis, Pennisetum*)

* Native species.

- For a naturalistic effect as well as ease of maintenance, trees or shrubs must be planted in clumps to form *"landscape islands"* rather than evenly spaced
- The landscaped islands must be a minimum of 6 feet apart, and if set back from fences or other barriers, the setback distance must also be a minimum of 6 feet. Where tree foliage extends low to the ground, the 6 foot setback should be counted from the outer drip line of the trees (estimated at maturity). This setback allows a 6-foot-wide mower to pass around and between clumps.
- Evergreen conifers are preferred, but deciduous trees with relatively little leaf-fall (such as Oregon ash, Serviceberry, and our native Hawthorn) can be utilized in areas draining to the pond
- Trees must be set back so that branches do not extend over the pond (to prevent leaf-drop into the water)
- Drought tolerant species are recommended.

4.6.3.3 Construction Specifications and Criteria

Detention ponds may be used for sediment control during site construction, but sediment must be removed upon completion.

Exposed earth on the pond bottom and interior side slopes must be vegetated or seeded with an appropriate seed mixture.

4.6.3.4 Operation and Maintenance Requirements

Facility components should be designed to facilitate maintenance operations. See Appendix D for specific operations and maintenance requirements. A plan shall be submitted demonstrating how the operations and maintenance requirements will be met.

4.6.4 Detention Pipe

Detention pipes, also referred to as detention tanks, are underground storage facilities.

4.6.4.1 Site Considerations

Primary site constraints for a detention pipe system include conflicts with existing underground utilities and site setback requirements. While there are no specific setback requirements for detention pipe, DPD must approve the detention pipe location and may specify pipe material (e.g., may require a concrete pipe) or require geotechnical analysis.

Note that belowground detention facilities are discouraged for single-family residential sites and are not required to meet flow control standards.

4.6.4.2 Design Criteria

Typical design guidelines for detention pipe are shown in City of Seattle Standard Plans 270 through 272 and provided in the City of Seattle Side Sewer Directors' Rule. Flow control structure details are shown in Section 4.6.10.

Pipe Materials and Specifications

Galvanized metals leach zinc into the environment, especially in standing water situations. This can result in zinc concentrations that can be toxic to aquatic life. Therefore, galvanized materials shall not be used in stormwater facilities and conveyance systems.

The material, diameter, and specification of pipe selected must be indicated on the Side Sewer Permit application, required before installing the drainage system. If the detention pipe is located under a building, a load analysis must be evaluated by a licensed civil engineer to determine the pipe specifications. The pipe must not be located under the foundation or have pressure exerted on it by the foundation. Corrugated Metal Pipe should not be used for detention pipes constructed in a public right-of-way, an Environmental Critical Area or Geologic Hazard Area.

Buoyancy

In moderately pervious soils where seasonal groundwater may induce flotation, buoyancy tendencies must be balanced either by ballasting with backfill or concrete backfill, providing concrete anchors, increasing the total weight.

Structural Stability

Detention pipes must meet structural requirements for overburden support and traffic loading if appropriate. Metal detention pipe end plates must be designed for structural stability at maximum hydrostatic loading conditions. Flat end plates generally require thicker gage material than the pipe and/or require reinforcing ribs. Detention pipes must be placed on a stable, well consolidated foundation

and have a suitable bedding. Detention pipes must not be placed in fill slopes, unless analyzed in a geotechnical report for stability and constructability.

Detention Pipe End Plates and Connections

The upstream end of the detention pipe must have a maintenance hole or watertight end plate or plug of standard manufacture (not constructed in the field) and must be made from the same material as the detention pipe. The inlet pipes may connect to the flow control structure or the detention pipe. Connections to the detention pipe are most easily made through the end plate. For connections through the side of a corrugated metal detention pipe, consult Standard Plan Number 279.

Access

The following access requirements apply to detention pipes:

- Detention pipes more than 50 feet long must provide a cleanout. Detention pipes more than 100 feet long must have access risers at each end to allow for maintenance and repair. Detention pipes over 200 feet long must have an access riser at the upstream end and a cleanout at least every 100 feet.
- Access openings shall be positioned a maximum of 50 feet from any location within the detention pipe.
- All detention pipe access openings shall have round, solid locking lids (usually 1/2 to 5/8-inch diameter Allen-head cap screws).
- Thirty-six-inch minimum diameter CMP riser-type manholes of the same gage as the detention pipe material may be used for access along the length of the detention pipe and at the upstream terminus of the detention pipe in a backup system. The top slab is separated (1-inch minimum gap) from the top of the riser to allow for deflections from vehicle loadings without damaging the riser pipe.
- All detention pipe access openings must be readily accessible by maintenance vehicles (including flow control structures located under buildings).
- Detention pipes must comply with the OSHA and WISHA confined space requirements, which include, but are not limited to, the preparation of ventilation plans and clearly marking entrances to confined space areas.

4.6.4.3 BMP Sizing

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), pre-sized detention pipes may used to achieve Pre-developed Pasture and Peak Control Standards. Sizing factors for detention pipe receiving runoff from an impervious surface are provided in Table 4.29. Factors are organized by flow control standard, pipe diameter and contributing area. To use these sizing factors to meet flow control

standards, the facility must meet the general requirements for detention pipe outlined in this section plus the following specific requirements:

- Sizing equations are applicable for contributing impervious surface areas between 2,000 and 10,000 square feet.
- Pipe length shall be sized using the applicable sizing equation.
- The low flow orifice diameter shall be 0.5 inches.
- Detention pipe shall be the designated diameter (24 or 36 inches). For intermediate diameters (between 24 and 36 inches), the pipe length may be linearly interpolated.
- The entire volume of the pipe shall be available for storage.

Table 4.29. Sizing Equations for Detention Pipe Receiving Runoff from Impervious Surface.

Detention Pipe Diameter	Contributing Area	Sizing Equation for Pipe Length	
		Pre-developed Pasture Standard	Peak Control Standard
24 inches	2,000 – 5,000 sf	0.0000263A^1.94	0.00000508A^2.10
	5,000 – 10,000 sf	0.120A – 235	0.123A - 314
36 inches	2,000 – 5,000 sf	0.000000219A^2.40	0.00000277A^2.05
	5,000 – 10,000 sf	0.0515A – 110	0.0546A- 167

A–contributing impervious area; ft- feet; sf-square feet.
For A < 5,000sf: Pipe Length (ft) = [Factor x Area (sf)] +Integer.
Impervious Area Mitigated (sf) = [Pipe Length (ft)- Integer]÷ Factor.
For A 5,000 to 10,000 sf: Pipe Length (ft) = Factor x [A (sf)^Integer].
Impervious Area Mitigated (sf) = [Pipe Length (ft) ÷ Factor]^(1 ÷ Integer).

The pipe length is calculated as a function of the impervious area routed to it. As an example, for the Pre-developed Pasture Standard, the pipe length for a 24-inch diameter pipe receiving runoff from between 5,000 to 10,000 square feet of impervious surface would be calculated as: 0.120 x impervious area (square feet) – 235 feet. All area values must be in square feet and length values must be in feet.

Alternatively, detention pipe for small sites can be sized using a continuous model as described in the subsequent section.

Facility Modeling

When using the continuous runoff model vault module for pipe sizing, the assumptions listed in Table 4.30 shall be applied. It is recommended that pipes be modeled as horizontal cylinders with an outlet structure that includes a low flow orifice. The tributary areas, pipe diameter, pipe length and orifice configuration should be iteratively sized until the Minimum Requirements for Flow Control are met (see Chapter 2).

Table 4.30. Continuous Modeling Assumptions for Detention Pipe.

Variable	Assumption
Precipitation Series	Seattle 158-year, 5-minute series
Computational Time Step	5-minutes
Inflows to Facility	Surface flow and interflow from drainage area should be connected to facility.
Precipitation and Evaporation Applied to Facility	No
Infiltration	No
Total Depth	The total depth is the pipe diameter.
Outlet Structure	Low flow orifice, riser height and diameter.
Low Flow Orifice	Invert of low flow orifice should be set at a minimum of 6 inches above the bottom of the pipe.

In some instances, the minimum diameter for the low flow orifice (0.25-inch for an aboveground facility and 0.5-inch for a belowground facility) will be too large to meet standard release rates, even with minimal head. See Section 4.6.1 for recommended modeling approach.

4.6.4.4 Construction Specifications and Criteria

See the construction-related issues outlined above as part of the design criteria.

- *Cover, Bedding, and Slope:* The bedding required for detention pipes is the same as for other pipes in the drainage system. Use washed, crushed aggregate Type 22 for corrugated metal pipes. Use 3/8–inch washed gravel (City–Mineral Aggregate, Type 9) for other pipes. Place at least 4 inches of gravel under the pipe. The gravel must fill the trench to a point half–way up the sides of the pipe (to the "spring line"). Provide at least 2 feet of cover over a detention pipe. For single family and duplex residences, 18 inches of cover is allowable. Before a side sewer permit is signed–off as completed, a City inspector must approve the installed system, including the detention pipe and the flow control structure, after it is bedded but before it is covered with soil. The standard slope for detention pipes is 0.5 percent. The inlet pipe to the detention pipe and the outlet pipe from the flow control structure must have at least a 2 percent slope, the same as required for other service drain pipes.

4.6.4.5 Operations and Maintenance Requirements

Facility components should be designed to facilitate maintenance operations. See Appendix D for specific operations and maintenance requirements. A plan shall be submitted demonstrating how the operations and maintenance requirements will be met.

4.6.5 Detention Vault

Detention vaults are box-shaped, underground storage facilities typically constructed with reinforced concrete (Figure 4.30).

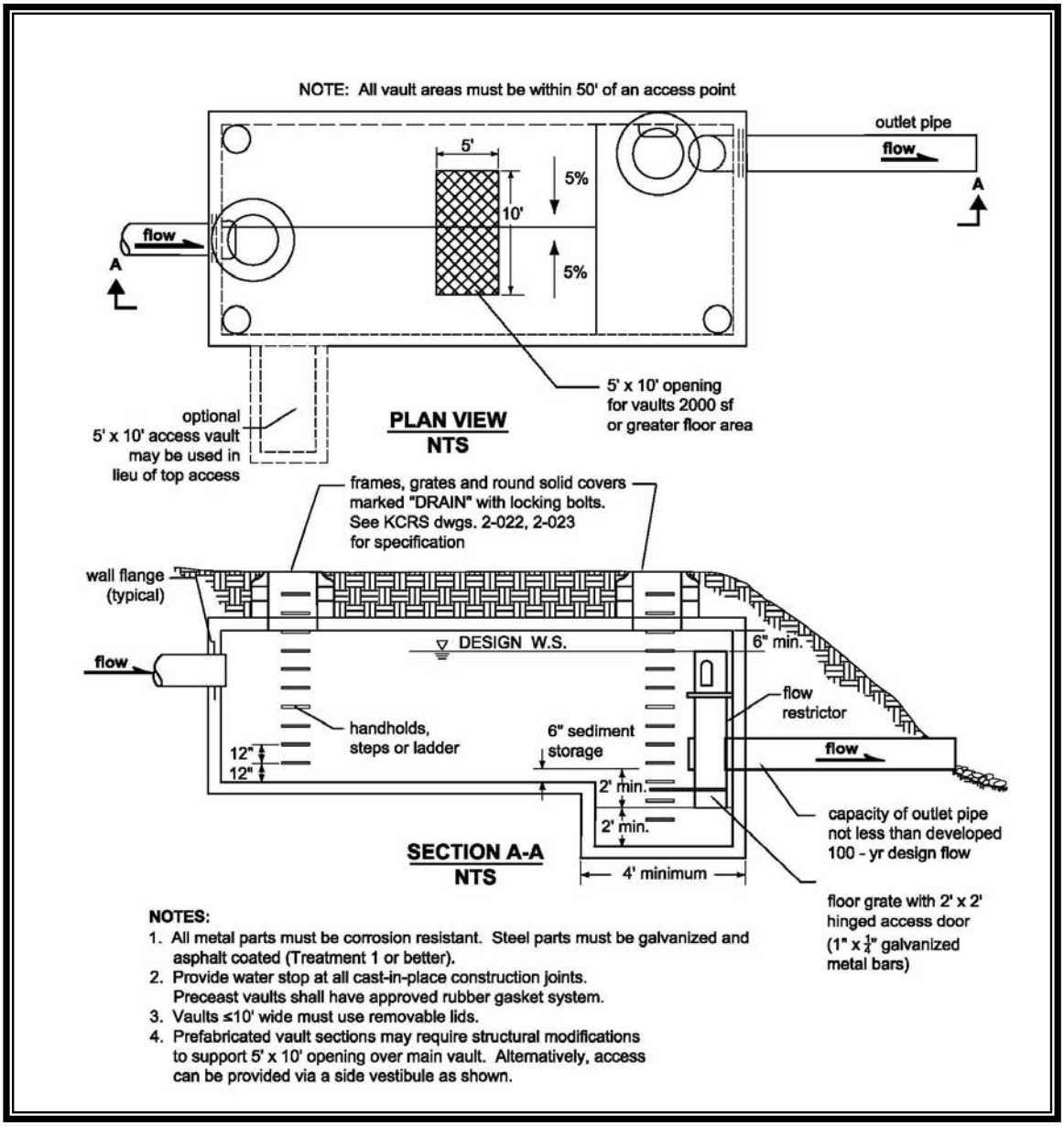


Figure 4.30. Typical Detention Vault.

4.6.5.1 Site Considerations

Primary site constraints for a detention vaults include conflicts with existing underground utilities and site setback requirements. While there are no specific setback requirements for detention pipe, DPD must approve the detention pipe location and may specify pipe material (e.g., may require a concrete pipe) or require geotechnical analysis.

Note that belowground detention facilities are discouraged for single-family residential sites and are not required to meet flow control standards.

4.6.5.2 Design Criteria

Materials

Minimum 3,000 psi structural reinforced concrete may be used for detention vaults. All construction joints must be provided with water stops.

Buoyancy

In moderately pervious soils where seasonal groundwater may induce flotation, buoyancy tendencies must be balanced either by ballasting with backfill or concrete backfill, providing concrete anchors, increasing the total weight.

Structural Stability

Detention pipes must meet structural requirements for overburden support and traffic loading if appropriate. Structural designs for vaults must be stamped by a licensed civil engineer with structural expertise. Vaults must be placed on a stable, well-consolidated foundation and have a suitable bedding. Vaults must not be placed in fill slopes, unless analyzed in a geotechnical report for stability and constructability.

Access

The following access requirements apply to detention vaults:

- Access may be provided by use of removable panels, hatches, or ring and cover.
- All detention vault access openings must be readily accessible by maintenance vehicles (including flow control structures located under buildings).
- Access shall be provided over *both* the inlet pipe and outlet structure. Access openings shall be positioned a maximum of 50 feet from any location within the detention vault. Additional access points may be needed on large vaults. If more than one “v” is provided in the vault floor for sediment removal, access to each “v” must be provided.
- Internal structural walls of large vaults shall be provided with openings sufficient for maintenance access between cells. The openings shall be sized and situated to allow access to the maintenance “v” in the vault floor.
- The recommended minimum internal height is 7 feet from the highest point of the vault floor (not sump), and the recommended minimum width is 4 feet. However, concrete vaults may be a minimum 3 feet in height and width if used as detention pipes with access manholes at each end, and if the width is no larger than the height. Also the minimum internal

height requirement may not be needed for any areas covered by removable panels.

- Vaults must comply with the OSHA and WISHA confined space requirements, which include, but are not limited to, the preparation of ventilation plans and marking entrances to confined space areas.

4.6.5.3 BMP Sizing

A standard detention vault detail is shown in Figure 4.30. Control structure details are outlined in Section 4.6.8.

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), pre-sized detention vaults may used to achieve Pre-developed Pasture and Peak Control Standards. Sizing factors for rectangular detention vaults receiving runoff from an impervious surface are provided in Table 4.31. Factors are organized by flow control standard, vault overflow depth and contributing area. To use these sizing factors to meet flow control standards, the facility must meet the general requirements for vaults outlined in this section plus the following specific requirements:

- Sizing equations are applicable for contributing impervious surface areas between 2,000 and 10,000 square feet
- Vault area shall be sized using the applicable sizing equation
- The low flow orifice diameter shall be 0.5 inches
- Invert of overflow shall be set at the designated detention depth (3 or 4 feet) above invert of low flow orifice. For intermediate depths (between 3 and 4 feet), the vault area may be linearly interpolated.
- The vault shall have vertical walls to the designated overflow height.

Table 4.31. Sizing Equations for Vaults Receiving Runoff from Impervious Surface.

Detention Depth	Contributing Area	Sizing Equation for Vault Area	
		Pre-developed Pasture Standard	Peak Control Standard
3 feet	2,000 – 5,000 sf	0.0000428A^1.87	0.0000616A^1.77
	5,000 – 10,000 sf	0.121A – 255	0.0700A - 130
4 feet	2,000 – 5,000 sf	NA	0.00000441A^2.04
	5,000 – 10,000 sf	NA	0.0508A – 98.0

A–contributing impervious area; NA-not applicable; sf-square feet.
For A < 5,000sf: Vault Area (sf) = [Factor x A (sf)] + Integer.
Impervious Area Mitigated (sf) = [Vault Area (sf)- Integer] ÷ Factor.
For A 5,000 to 10,000 sf: Vault Area (sf) = Factor x [A (sf)^Integer].
Impervious Area Mitigated (sf) = [Vault Area (sf) ÷ Factor]^(1 ÷ Integer).

The vault area is calculated as a function of the impervious area routed to it. As an example, for the Peak Control Standard, the area for a vault with an overflow

invert set at 4.0 feet above the low flow orifice and receiving runoff from less than 5,000 square feet of impervious surface would be calculated as: 0.00000441 x impervious area (square feet) ^2.04. All area units must be square feet.

A vault with 4 feet of head above the low flow orifice is not applicable for site subject to the Pre-developed Pasture Standard because the designer is required to reduce the head to at least 3 feet in an attempt to meet this standard (see Section 4.6.1.

Alternatively, vaults can be sized using a continuous model as described in the subsequent section.

Facility Modeling

When using the continuous runoff model for vault sizing, the assumptions listed in Table 4.32 shall be applied. It is recommended that vaults be modeled as a flat-bottomed detention vault or tank with an outlet structure that includes a low flow orifice. The tributary areas, detention bottom area, overflow depth and orifice configuration should be iteratively sized until the Minimum Requirements for Flow Control are met (see Chapter 2).

Table 4.32. Continuous Modeling Assumptions for Vault Detention.

Variable	Assumption
Precipitation Series	Seattle 158-year, 5-minute series
Computational Time Step	5-minutes
Inflows to Facility	Surface flow and interflow from drainage area should be connected to facility.
Precipitation and Evaporation Applied to Facility	No
Infiltration	No
Total Depth	The total depth is the vault height (including freeboard) above the vault bottom.
Outlet Structure	Low flow orifice, riser height and diameter.
Overflow	The top of the overflow orifice should be set a minimum of 6 inches below the top of the vault.
Low Flow Orifice	Invert of low flow orifice should be set at a minimum of 6 inches above the bottom of the vault.

In some instances, the minimum diameter for the low flow orifice (0.25-inch for an aboveground facility and 0.5-inch for a belowground facility) will be too large to meet standard release rates, even with minimal head. See Section 4.6.1 for recommended modeling approach.

4.6.5.4 Construction Specifications and Criteria

See the construction-related issues outlined above as part of the design criteria.

4.6.5.5 Operations and Maintenance Requirements

Facility components should be designed to facilitate maintenance operations. See Appendix D for specific operations and maintenance requirements. . A plan shall be submitted demonstrating how the operations and maintenance requirements will be met.

4.6.6 Detention Cistern

Detention cisterns are tanks used for the capture and detention of stormwater runoff. Runoff from roof downspouts can be routed to cisterns for detention and slow release to an approved discharge point (see Figure 4.31). Like other detention facilities, cisterns can be used to achieve reductions in peak flows and flow durations. The detention performance of cisterns is a function of contributing area, cistern height, and orifice size. When designed in accordance with the criteria outlined below, cistern detention systems may meet Pre-developed Pasture and Peak Control Standards (Chapter 2), depending upon the orifice diameter used. Detention cisterns are typically not capable of achieving the Pre-developed Forest Standard for small sites. A schematic for a typical detention cistern in shown in Figure 4.32.

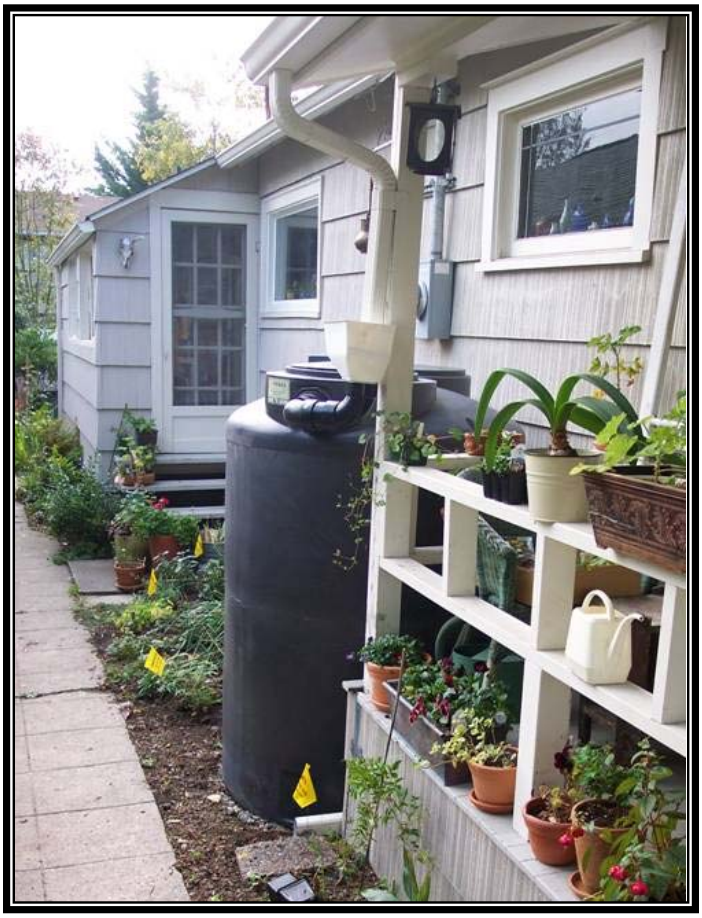


Figure 4.31. Cistern Collecting Roof Runoff in Fremont.

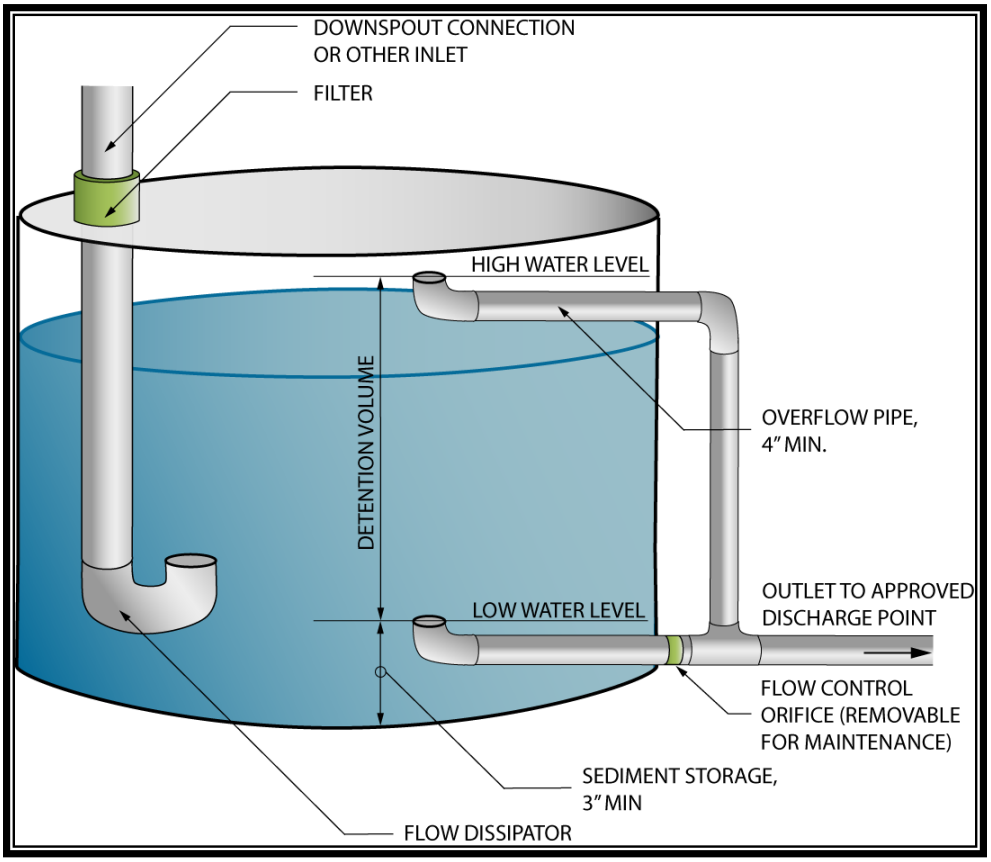


Figure 4.32. Detention Cistern.

Cisterns are more likely to be aboveground than other detention facilities. The major differences between above- and below-ground facilities include the following:

- A smaller orifice diameter is acceptable for aboveground detention (0.25 inches versus 0.5 inches)
- Additional requirements apply to aboveground installations (e.g., earthquake strapping).

In addition to detention, cisterns can be used to harvest rainwater for outdoor non-potable uses (e.g., irrigation). If indoor use of harvested water is desired, the system is not considered a "detention cistern" as defined in this manual and must be designed per the Rainwater Harvesting section (Section 4.4.6).

Examples of detention cisterns combined with rainwater harvesting for irrigation are shown in Figures 4.34 and 4.35. For most projects, the detention volume required to meet flow control goals must be separate from any additional volume stored for reuse. In this case, the lower portion of a cistern (below the low flow orifice) may be used for rainwater harvesting (see Figure 4.33).

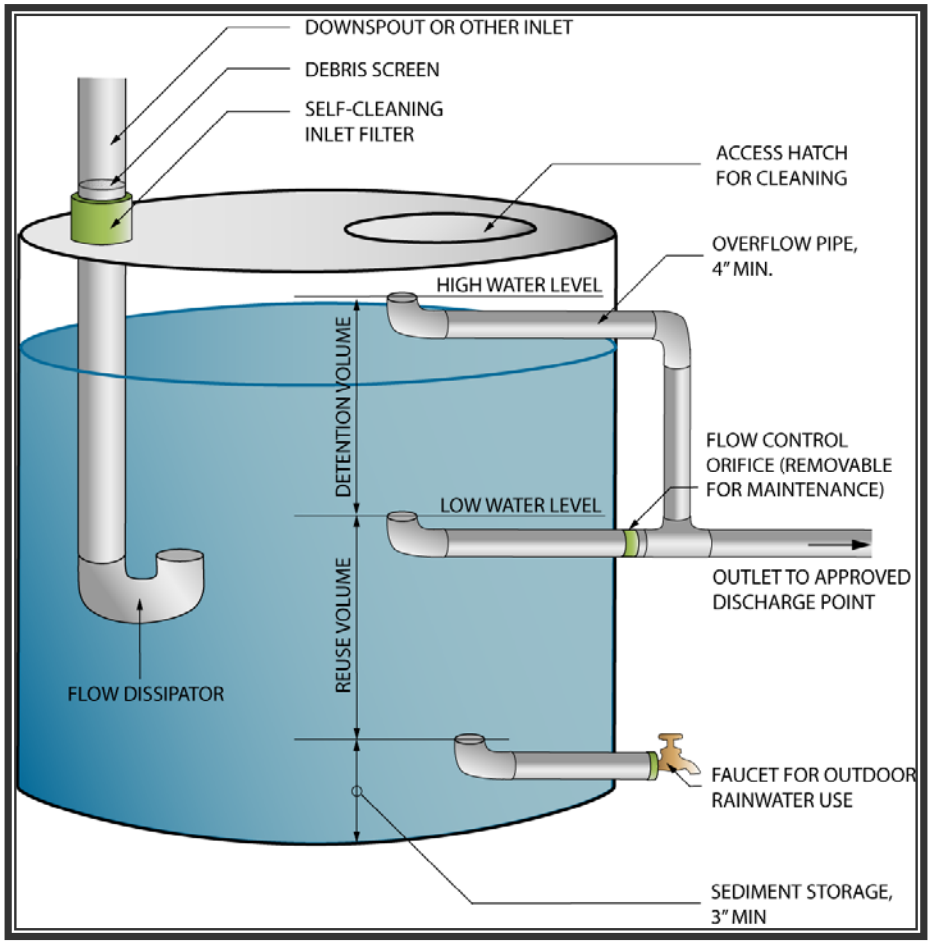


Figure 4.33. Detention Cistern with Rainwater Use.

For cisterns on single family residential parcels, the entire cistern volume may be used for harvesting (see Figure 4.34). This configuration requires seasonal operation of a valve to detain water through the winter months.

While there are benefits of harvesting for outdoor, non-potable uses, additional flow control credit (beyond credit for detention) is not applicable for this type of system nor it is considered green stormwater infrastructure unless it is designed per the rainwater harvesting requirements of Section 4.4.6. An exception to this is for single family residential projects, where detention cisterns with harvesting capacity are considered green stormwater infrastructure. In this case, the cistern must be aboveground and have an orifice diameter of 0.25 inches.

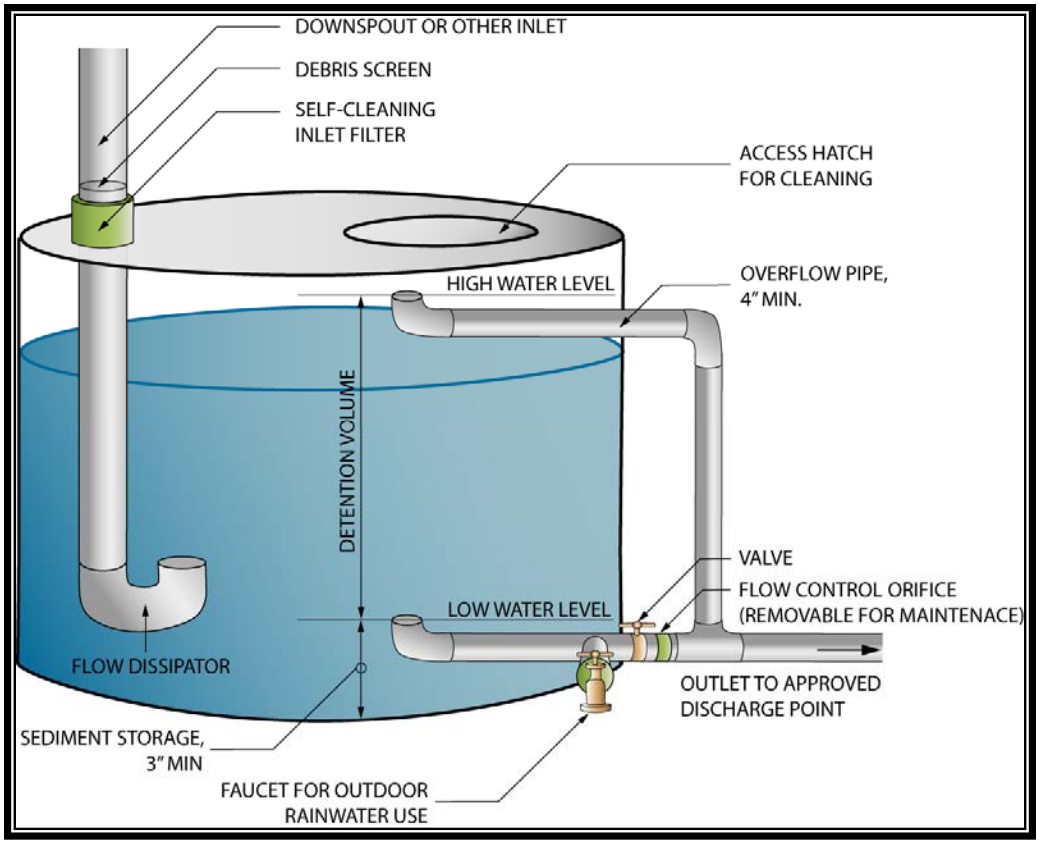


Figure 4.34. Detention Cistern with Rainwater Use for Single Family Residential Only.

4.6.6.1 Applications and Limitations

Detention cisterns can be used to detain rooftop runoff in any type of new or retrofit development project. Cisterns may be used individually or connected to each other in series for greater detention and storage capacity. Detained stormwater and system overflows may be conveyed to an approved discharge point or to another stormwater BMP such as a bioretention area.

While underground detention facilities are discouraged for single-family residential sites, the use of aboveground detention cisterns with harvesting capacity is considered GSI and is encouraged for single-family residential sites.

4.6.6.2 Design Criteria

The following provides recommendations and requirements for the common components of cistern detention systems. Submittal for facility review shall include the following elements, described further in the subsequent sections:

- Collection system
- Screen/debris excluder

- Cistern
- Low flow orifice
- Overflow.

Catchment Area / Collection System

Collection systems include gutters and downspouts, as well as piping and any other conveyance needed to route runoff from the roof to the cistern.

Screens/Debris Excluder

A filter screen or other debris barrier is required to prevent insects, leaves, and other larger debris from entering the system. A self-cleaning inlet filter is recommended.

Cistern

Cisterns are commonly constructed of fiberglass, polyethylene, concrete, metal, or wood. Tanks can be installed at or below grade and individually or in series. Cisterns shall have tight fitting covers to exclude contaminants and animals, and aboveground tanks shall not allow penetration of sunlight to limit algae growth.

Minimum requirements associated with cistern design include the following:

- All cisterns must be installed in accordance with manufacturer's installation instructions.
- Cisterns must be designed to prevent mosquitoes and other life forms from entering the cistern system. This can be done with appropriate screening at all openings to the cistern.
- Opaque containers must be used for aboveground cisterns to minimize algae growth.
- Minimum cistern size shall be that of a rain barrel (typically 55 gallons).

Low Flow Orifice

Minimum requirements associated with low flow orifice design include the following:

- As for other detention systems, the minimum diameter shall be 0.25 inches for orifices located above ground, and 0.5 inches for orifices located below ground. (Note: belowground facilities are not permitted for single-family residential sites unless approved by the Director.)
- The low flow orifice invert must be at least 3 inches above the bottom of the cistern to prevent entraining sediment.

Overflow

Cisterns must have an overflow to convey water exceeding the detention capacity of the system to an approved discharge point or another BMP (e.g.,

bioretention area, vegetated cell, or infiltration trench). Conveyance may be provided by gravity flow or by pumps, but gravity flow is preferred. Minimum requirements associated with overflow design include the following:

- Overflows shall be designed to convey excess flow to approved discharge point per Section 4.2.5.

Rainwater Use Equipment (if any)

If the lower portion of the cistern is being used to harvest stormwater, an outlet spigot can be installed near the bottom of the tank. Minimum requirements associated with rainwater use include the following:

- Rainwater use must be for outdoor non-potable uses only
- For non-single family residential projects, the spigot must be located below the low flow orifice.

Additional requirements for rainwater harvesting are presented in Section 4.4.6.

4.6.6.3 BMP Sizing

Pre-Sized Approach

Under the Pre-Sized Approach (see Section 4.2.2), pre-sized detention cisterns may used to achieve Pre-developed Pasture and Peak Control Standards. Sizing factors for aboveground cisterns receiving runoff from an impervious surface are provided in Table 4.33. Factors are organized by flow control standard, cistern overflow depth and contributing area. To use these sizing factors to meet flow control standards, the facility must meet the general requirements for cisterns outlined in this section plus the following specific requirements:

- The cistern area shall be sized using the applicable sizing equation.
- The low flow orifice diameter shall be 0.25 inches.
- Invert of overflow shall be set at the designated detention depth (3 or 4 feet) above invert of low flow orifice. For intermediate ponding depths (between 3 and 4 feet), the cistern area may be linearly interpolated.
- The cistern shall have vertical walls to the designated overflow height.
- The cistern bottom area is calculated as a function of the impervious area routed to it. As an example, to meet the Peak Control Standard, the area of a cistern with an overflow invert set at 3 feet above the low flow orifice and receiving runoff from less than 6,140 square feet would be calculated as: 0.00599 x contributing impervious area (square feet) ^1.35. All areas values must be in units of square feet. For the same cistern receiving runoff from between 6,140 and 8,000 square feet, the cistern area would be 760 square feet.

Table 4.33. Sizing Equations for Aboveground Cisterns

Detention Depth	Contributing Area	Sizing Equation for Cistern Area	
		Pre-developed Pasture Standard	Peak Control Standard
3 feet	< 6,140 sf	0.00599A^1.35	0.000531A^1.59
	6,140 – 8,000 sf	760 sf	
	8,000 – 10,000 sf	0.131A – 289	
4 feet	NA	NA	0.000417A^1.58

A–contributing impervious area; NA-not applicable; sf-square feet.
Cistern Area (sf) = [Factor x A (sf)] + Integer or Factor x [A (sf)^Integer].
Impervious Area Mitigated (sf) = [Cistern Area (sf) – Integer] ÷ Factor or [Cistern (sf) Area/Factor]^(1/Integer)

A cistern with 4 feet of head above the low flow orifice is not applicable for site subject to the Pre-developed Pasture Standard because the designer is required to reduce the head to at least 3 feet in an attempt to meet this standard (see Section 4.6.1.

Alternatively, cisterns can be sized using a continuous model as described in the next section.

Facility Modeling

Continuous modeling may be used to size cisterns using the procedures presented for vaults in Section 4.6.5.2. The assumptions provided in Table 4.33 shall be applied.

4.6.6.4 Operations and Maintenance Requirements

Operations and maintenance requirements for cistern detention systems are provided in Appendix D. For non-single family residential projects, a landscape management plan shall be submitted demonstrating how the maintenance requirements will be met. A plan shall be submitted demonstrating how the operations and maintenance requirements will be met.

4.6.7 Other Detention Options

This section presents other design options for detaining flows to meet flow control facility requirements.

4.6.7.1 Use of Parking Lots for Additional Detention

Private parking lots may be used to provide additional detention volume for runoff events greater than the 50 percent annual probability (2-year recurrence interval) runoff event provided all of the following are met:

- Depth of storage in parking lots must be 3 inches or less for parking lots serving retail and office buildings, and 6 inches or less for parking serving commercial truck traffic only, for runoff events up to and including the

storm event with a 1 percent annual probability (100-year recurrence interval flow)

- The gradient of the parking lot area subject to ponding is 1 percent or greater
- The emergency overflow path is identified and noted on the engineering plan. The overflow must not create a significant adverse impact to downhill properties or drainage system.
- Fire lanes used for emergency equipment are free of ponding water for all runoff events up to and including the storm event with a 1 percent annual probability (100-year recurrence interval flow).

4.6.7.2 Use of Roofs for Detention

Detention ponding on roofs of structures may be used to meet flow control requirements provided all of the following are met:

- The roof support structure is analyzed by a structural engineer to address the weight of ponded water.
- The roof area subject to ponding is sufficiently waterproofed to achieve a minimum service life of 30 years.
- The minimum pitch of the roof area subject to ponding is 1/4 inch per foot.
- An overflow system is designed to safely convey the peak flow with a 1 percent annual probability (100-year recurrence interval flow).
- A mechanism is included in the design to allow the ponding area to be drained for maintenance purposes or in the event the restrictor device is plugged.

4.6.8 Control Structures for Flow Control Facilities

Control structures are catch basins or manholes with a restrictor device for controlling outflow from a facility to meet the desired performance. Riser type restrictor devices (“tees” or “FROP-Ts”) also provide some incidental oil/water separation to temporarily detain oil or other floatable pollutants in runoff due to accidental spill or illegal dumping.

The restrictor device usually consists of two or more orifices and/or a weir section sized to meet performance requirements. Standard control structure details are shown in Figures 4.35 through 4.38.

For design requirements related conveyance and drainage see City of Seattle Side Sewer Code and Directors’ Rule.

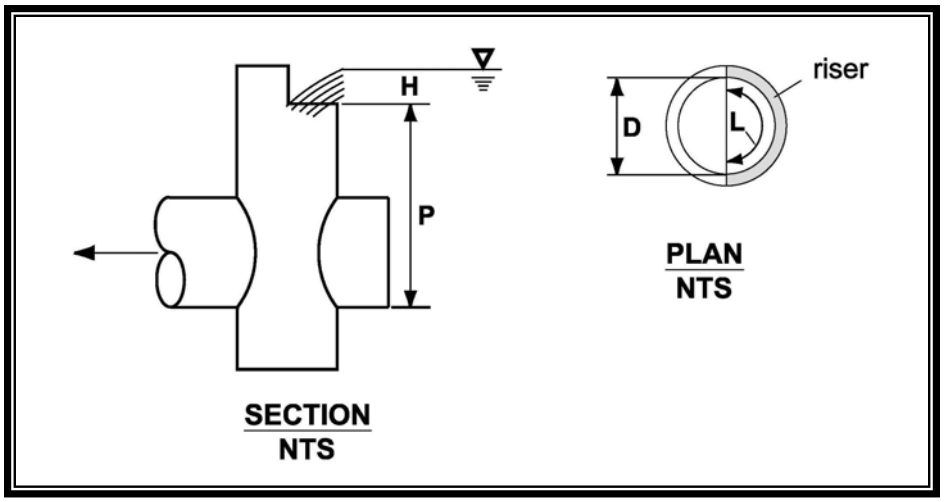


Figure 4.35. Rectangular, Sharp-Crested Weir.

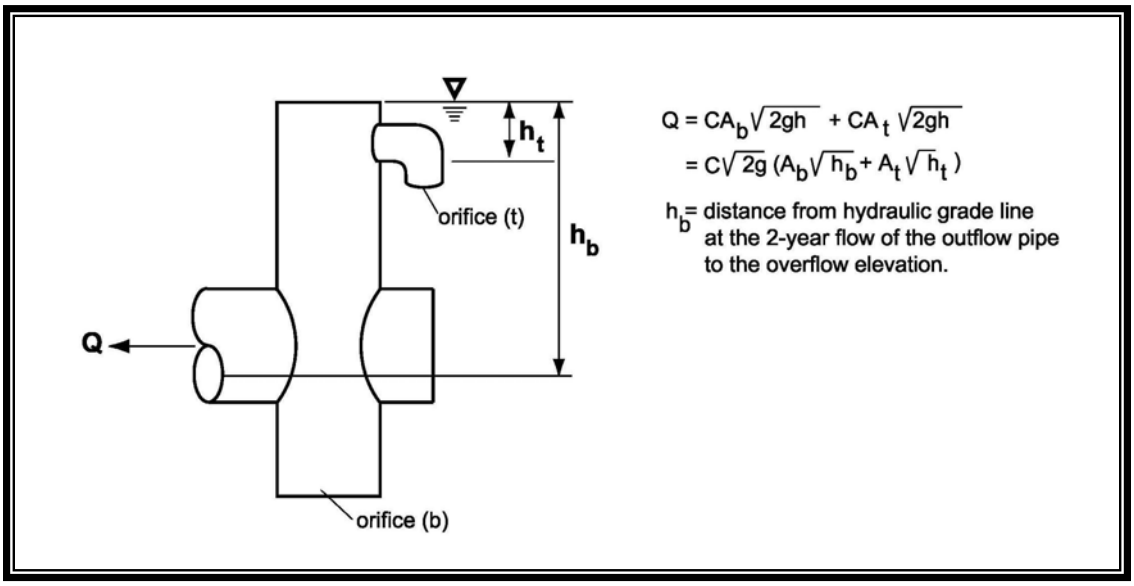


Figure 4.36. Simple Orifice.

4.6.8.1 General Requirements

Flow control structures must comply with the specifications outlined in the City's Standard Plans numbers 270 and 272A. Additional general requirements are presented below.

Permit submittal must include flow control structure rim elevation, the storage pipe invert elevation, the outlet pipe invert elevation, and, the elevation of the top of the storage pipe, and the elevation of the top of the overflow pipe.

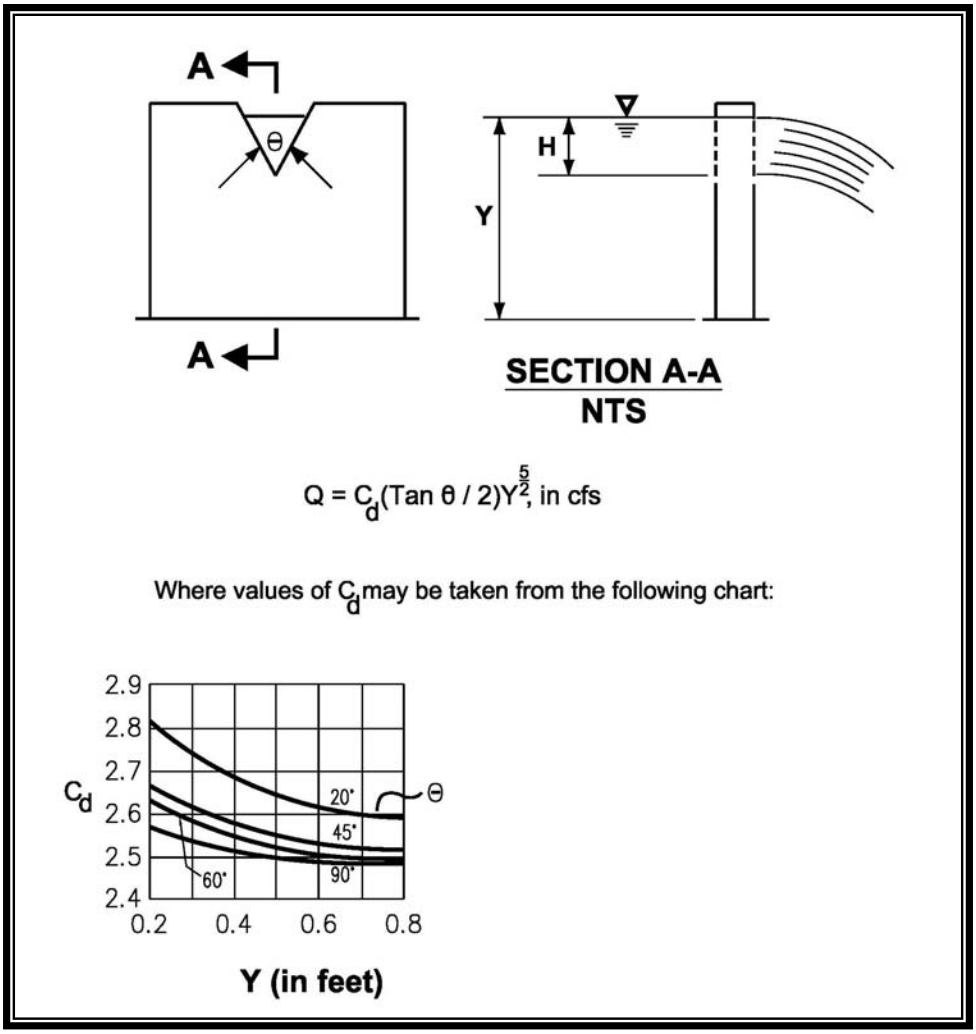


Figure 4.37. V-Notch, Sharp-Crested Weir.

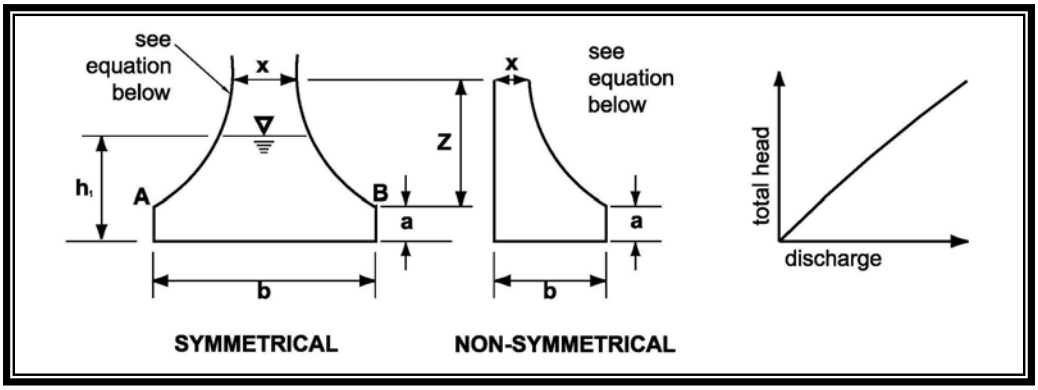


Figure 4.38. Sutro Weir.

Access

The following access requirements apply to control structures:

- Access must be provided to the flow control structure from the ground surface with a three bolt locking maintenance hole ring and cover (see City of Seattle Standard Plan 230). Rim elevations must match proposed finish grade. A rectangular cover, or a cover that allows water to enter through the top of the flow control structure, may not be used. The ring and cover must be set so the flow control device or the ladder is visible at the edge of the access opening.
- The inside diameter of the flow control structure must be at least 4 feet to allow maintenance and repair access, and to accommodate stormwater overflow.
- Manholes and catch basins must meet the OSHA and WISHA confined space requirements, which include, but are not limited to, clearly marking entrances to confined space areas. This may be accomplished by hanging a removable sign in the access riser, just under the access lid.
- The flow control device must be PVC, not Corrugated Metal Pipe (CMP). The mounting straps and the outlet adapter must be installed in a manner that will make the flow control device easily removable for maintenance, repair, or replacement. The flow control device must be designed and located under the maintenance hole ring and cover for inspection from the surface. The outlet pipe adapter may be a plastic, bell-end pipe or a plastic coupling with rubber gaskets. The outside of the pipe or coupling must be sanded, epoxy coated, and sand impacted to bond with the flow control structure. This preparation must be done by the manufacturer or supplier, not in the field.

Information Plate

A brass or stainless steel plate must be permanently attached inside each control structure with the following information engraved on the plate:

- Name and file number of project
- Name and company of 1) developer, 2) engineer, and 3) contractor
- Date constructed
- Date of manual used for design
- Flow performance criteria
- Release mechanism size, type, and invert elevation
- List of stage, discharge, and volume at 1-foot increments
- Elevation of overflow
- Required frequency of maintenance.

4.6.8.2 Design Criteria

Multiple Orifice Restrictor

In most cases, control structures need only two orifices: one at the bottom and one near the top of the riser, although additional orifices may best utilize detention storage volume. Several orifices may be located at the same elevation if necessary to meet performance requirements.

Design requirements for multiple orifice flow restrictors are presented below.

- The minimum allowable orifice diameter is 0.5 inches for below ground outlets and 0.25 inches for aboveground outlets. In some instances, a 0.5-inch bottom orifice will be too large to meet target release rates, even with minimal head. In these cases, the live storage depth need not be reduced to less than 3 feet in an attempt to meet the performance standards. Also, under such circumstances, flow-throttling devices may be a feasible option. These devices will throttle flows while maintaining a plug-resistant opening.
- Orifices may be constructed on a tee section as shown in City of Seattle Standard Plan 270 or on a baffle.
- In some cases, performance requirements may require the top orifice/elbow to be located too high on the riser to be physically constructed (e.g., a 13-inch-diameter orifice positioned 0.5 feet from the top of the riser). In these cases, a notch weir in the riser pipe may be used to meet performance requirements (see Figure 4.35).
- Consideration must be given to the backwater effect of water surface elevations in the downstream conveyance system. High tailwater elevations may affect performance of the restrictor system and reduce live storage volumes. Backwater calculations may be required.

Riser and Weir Restrictor

Design requirements for multiple orifice flow restrictors are presented below.

- Properly designed weirs may be used as flow restrictors. However, they must be designed to provide for primary overflow of the developed 100-year peak flow discharging to the detention facility.
- The combined orifice and riser (or weir) overflow may be used to meet performance requirements; however, the design must still provide for primary overflow of the developed 100-year peak flow assuming all orifices are plugged. Figure 4.39 can be used to calculate the head in feet above a riser of given diameter and flow.

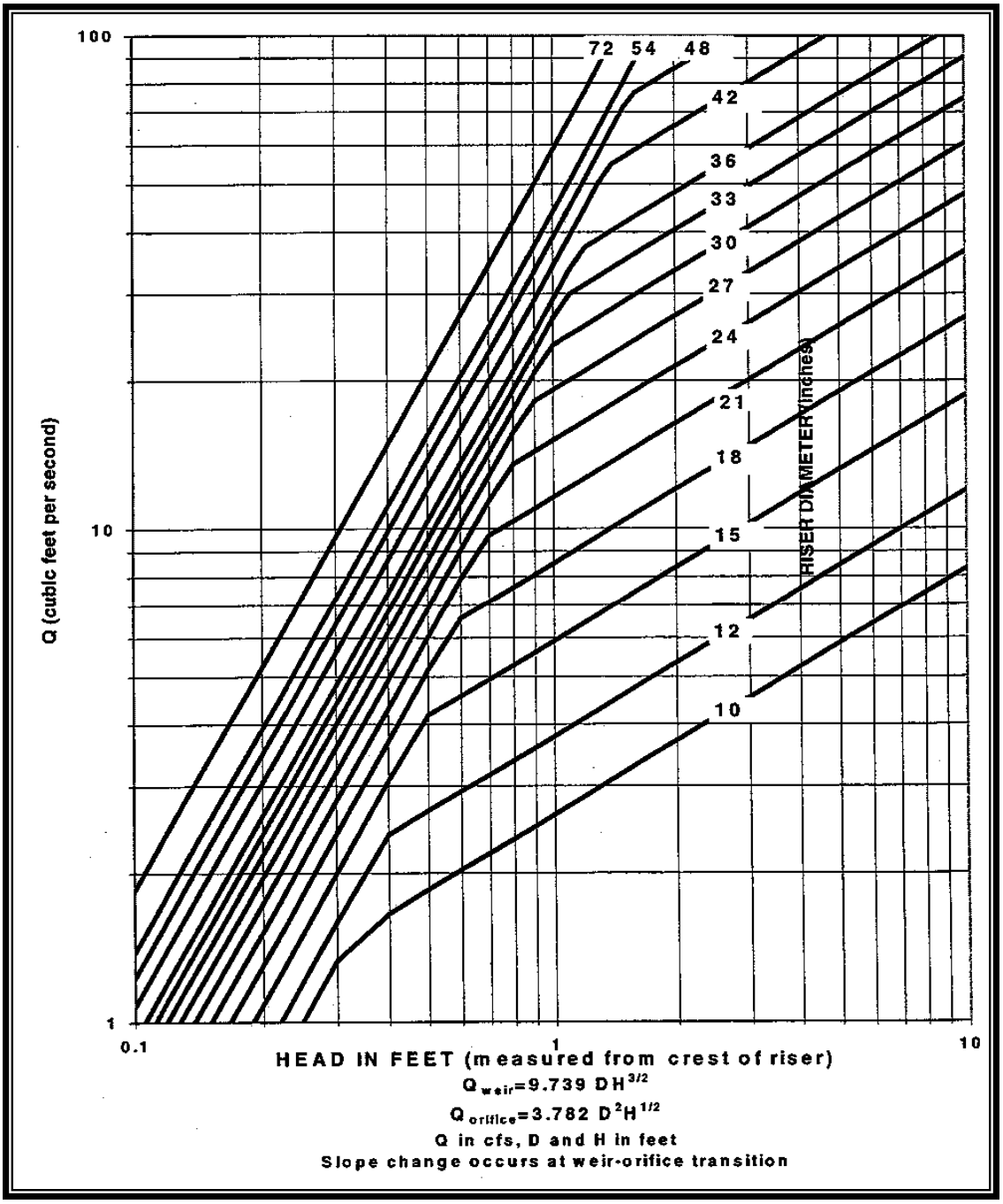


Figure 4.39. Riser Inflow Curves.

4.6.8.3 Flow Control Device Sizing

Orifices

Flow-through orifice plates in the standard tee section or turn-down elbow may be approximated by the general equation:

$$Q = C\,A\sqrt{2gh}$$

where Q = flow (cfs)
C = coefficient of discharge (0.62 for plate orifice)
A = area of orifice (ft^2)
h = hydraulic head (ft)
g = gravity (32.2 ft/sec^2)

Figure 4.36 illustrates this simplified application of the orifice equation.

The diameter of the orifice is calculated from the flow. The orifice equation is often useful when expressed as the orifice diameter in inches.

$$d = \sqrt{\frac{36.88Q}{\sqrt{h}}}$$

where *d* = orifice diameter (inches)
Q = flow (cfs)
h = hydraulic head (ft).

The combined orifice and riser (or weir) overflow may be used to meet performance requirements; however, the design must still provide for primary overflow of the developed 100-year peak flow assuming all orifices are plugged. Figure 4.39 can be used to calculate the head in feet above a riser of given diameter and flow.

Rectangular Sharp-Crested Weir

The rectangular sharp-crested weir design shown in Figure 4.35 may be analyzed using standard weir equations for the fully contracted condition.

$$Q=C\,(L - 0.2H)H^{3/2}$$

where Q = flow (cfs)
C = 3.27 + 0.40 H/P (ft)
H, P are as shown in Figure 4.35
L = length (ft) of the portion of the riser circumference as necessary not to exceed 50 percent of the circumference
D = inside riser diameter (ft)

Note that this equation accounts for side contractions by subtracting 0.1H from L for each side of the notch weir.

V-Notch Sharp-Crested Weir

V-notch weirs as shown in Figure 4.37 may be analyzed using standard equations for the fully contracted condition.

Proportional or Sutro Weir

Sutro weirs are designed so that the discharge is proportional to the total head. This design may be useful in some cases to meet performance requirements.

The sutro weir consists of a rectangular section joined to a curved portion that provides proportionality for all heads above the line A-B (see Figure 4.38). The weir may be symmetrical or non-symmetrical.

For this type of weir, the curved portion is defined by the following equation (calculated in radians):

$$\frac{x}{b} = 1 - \frac{2}{\pi} Tan^{-1} \sqrt{\frac{Z}{a}}$$

where a, b, x, and Z are as shown in Figure 4.38. The head-discharge relationship is:

$$Q = C_d \, b \sqrt{2ga} \left(h_1 - \frac{a}{3}\right)$$

Values of C_d for both symmetrical and non-symmetrical sutro weirs are summarized in Table 4.34. (Note: When b> 1.50 or a > 0.30, use C_d=0.6.)

Table 4.34. Values of C_d for Sutro Weirs.

Cd Values, Symmetrical					
	b (ft)				
a (ft)	0.50	0.75	1.0	1.25	1.50
0.02	0.608	0.613	0.617	0.6185	0.619
0.05	0.606	0.611	0.615	0.617	0.6175
0.10	0.603	0.608	0.612	0.6135	0.614
0.15	0.601	0.6055	0.610	0.6115	0.612
0.20	0.599	0.604	0.608	0.6095	0.610
0.25	0.598	0.6025	0.6065	0.608	0.6085
0.30	0.597	0.602	0.606	0.6075	0.608
Cd Values, Non-Symmetrical					
	b (ft)				
a (ft)	0.50	0.75	1.0	1.25	1.50
0.02	0.614	0.619	0.623	0.6245	0.625
0.05	0.612	0.617	0.621	0.623	0.6235
0.10	0.609	0.614	0.618	0.6195	0.620
0.15	0.607	0.6115	0.616	0.6175	0.618
0.20	0.605	0.610	0.614	0.6155	0.616
0.25	0.604	0.6085	0.6125	0.614	0.6145
0.30	0.603	0.608	0.612	0.6135	0.614

Riser Overflow

The nomograph in Figure 4.39 can be used to determine the head (in feet) above a riser of given diameter and for a given flow (usually the 100-year peak flow for developed conditions).

4.7 References

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Chapter 5 - Water Quality Treatment Design

5.1 Introduction

The focus of this chapter is on permanent water quality treatment best management practices (BMPs). Water quality BMPs are designed to remove sediment and other pollutants from stormwater runoff. The need to provide a treatment facility for a project depends on the applicable Minimum Requirements for Treatment as outlined in the Stormwater Code and Chapter 2 of this manual.

Requirements for controlling pollutants from the source (source control) and for BMPs applicable to construction sites are discussed in Volume 1, the Source Control Technical Requirements Manual and Volume 2, the Construction Stormwater Control Technical Requirements Manual and, respectively.

5.1.1 Organization

This chapter contains 12 sections:

- Section 5.1 serves as an introduction and summarizes available options for treatment of stormwater.
- Section 5.2 outlines a step-by-step process for selecting treatment facilities for new development and redevelopment projects.
- Section 5.3 presents treatment facility “menus” that are used in applying the step-by-step process presented in Section 5.2. These menus cover different treatment needs that are associated with different sites.
- Section 5.4 discusses general requirements for treatment facilities.
- Sections 5.5 through 5.11 provide detailed information regarding specific types of treatment facilities identified in the menus.
    - Pretreatment Facilitie
    - Biofiltration Swales
    - Filter Strips
    - Infiltration Facilities
    - Sand Filtration and Media Filtration
    - Wet Pool Facilities
    - Oil Control Facilities
    - Emerging Technologies.
- Section 5.12 discusses special considerations for emerging technologies for stormwater treatment.

Under each water quality treatment category (i.e., pretreatment, filter strips, etc), specific water quality BMPs are described in detail. Descriptions are organized as follows:

- ***Site Considerations*** identify the limitations associated with siting each water quality BMP. The applications of a BMP may be constrained by factors such as approximate footprint, elevation requirements, soil characteristics, and other site-specific requirements. The intention is to allow the designer to quickly evaluate whether a proposed BMP might be suitable for a particular site or project.
- ***BMP Sizing and Design Criteria*** provide a summary of basic BMP sizing guidelines and requirements, the procedures for designing each BMP, as well as any unique design requirements and considerations for each BMP. This typically includes information on the design treatment flow or volume for each BMP. However, specific guidelines on modeling procedures are primarily contained in Chapter 6, Hydrologic Analysis. Likewise, the overall water quality treatment requirements are outlined in Chapter 2, Minimum Requirements.
- ***Construction Specifications and Criteria*** describe critical considerations during construction of the BMP, such as erosion control, landscape stabilization, and timing of facility installation.

5.1.2 Relevant Information from Other Chapters

Chapter 2 describes the Minimum Requirements contained in the Stormwater Code, including water quality treatment standards and where they apply.

Chapter 3 summarizes site planning, site assessment and drainage control review procedures.

Chapter 4 details selection procedures and design requirements for flow control BMPs.

Chapter 6 provides detailed guidance for hydrologic analysis.

The appendices to this volume contain more detailed information on selected topics described in the various sections. Appendices are located at the end of the volume.

5.1.3 Summary of Minimum Requirements for Treatment

The Minimum Requirements for Treatment are presented in the Stormwater Code and Chapter 2 of this manual. This section provides a brief summary and designers must refer to the detailed requirements in Chapter 2 to confirm all applicable requirements are met.

Parcel-based and roadway projects **not** discharging to a public combined sewer that have at least 5,000 square feet of new plus replaced pollution generating impervious surface (PGIS) or at least 3/4 acres of pollution generating pervious surface (PGPS) are subject to WQ#1—Basic Treatment and must evaluate all other treatment minimum requirements (WQ#2, WQ#3 and WQ#4) to determine if they are applicable.

- **Treatment** Minimum Requirement #1 (WQ#1) – Basic Treatment. Install and maintain a basic water quality treatment facility. The requirements for determining the applicable water quality treatment volume and/or rate are presented in Section 2.4.7, with additional modeling guidance presented in Chapter 6.
- **Treatment** Minimum Requirement #2 (WQ#2) – Oil Control. Install and maintain an oil control treatment facility for high-use sites.
- **Treatment** Minimum Requirement #3 (WQ#3) – Phosphorus Treatment. Install and maintain a phosphorus treatment facility for projects discharging into nutrient-critical receiving waters.
- **Treatment** Minimum Requirement #4 (WQ#4) – Enhanced Treatment. Install and maintain an enhanced treatment facility.

5.1.4 Water Quality Runoff Treatment Facility Categories

Runoff treatment facilities are designed to remove pollutants contained in stormwater runoff. The pollutants of concern include sand, silt, and other suspended solids; metals such as copper, lead, and zinc; nutrients (e.g., nitrogen and phosphorus); certain bacteria and viruses; and organic contaminants such as petroleum hydrocarbons and pesticides. Methods of pollutant removal include sedimentation/settling, filtration, plant uptake, ion exchange, adsorption, and bacterial decomposition. Floatable pollutants such as oil, debris, and scum can be removed with separator structures.

Categories of runoff treatment facilities and common terms used in runoff treatment are presented below.

- **Pretreatment.** Presettling basins are often used to remove sediment from runoff prior to discharge into other treatment or flow control facilities. The presettling is intended to remove larger solids, but not typically designed to meet water quality treatment goals or sizing guidelines for pretreatment facilities. For green stormwater infrastructure facilities, if the catchment area exceeds 2,000 square feet and flow is concentrated, the BMP shall be preceded by a presettling technique (e.g., variation on filter strip, presettling catch basin, or vault). For all other filtration and infiltration facilities, appropriate pretreatment devices include a presettling basin, wet pond/vault, biofiltration swale, or constructed wetland. Basic treatment facilities, listed in Step 6 – Figure 5.1, can also be used to provide pretreatment.

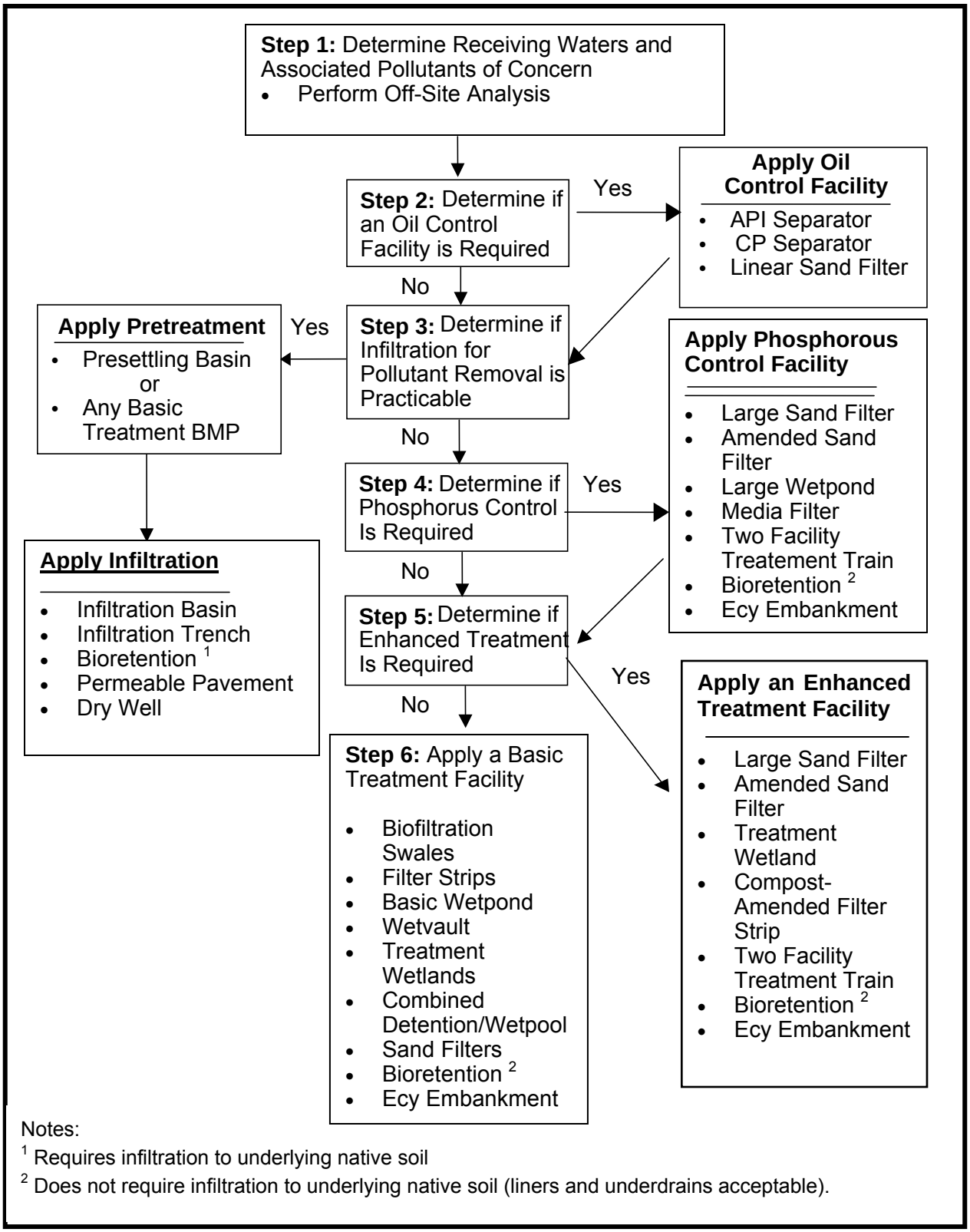


Figure 5.1. Treatment Facility Selection Flow Chart.

- **Biofiltration.** Biofiltration uses vegetation in conjunction with slow and shallow-depth flow for runoff treatment. As runoff passes through the vegetation, pollutants are removed through the combined effects of filtration, infiltration, and settling. These effects are aided by the reduction in the velocity of runoff as it passes through the swale. Biofiltration facilities include swales that are designed to convey and treat concentrated runoff at shallow depths and slow velocities, and filter strips that consist of broad areas of vegetation for treating sheet flow runoff. Biofiltration facilities may result in infiltration to underlying native soils. Projects must be evaluated on a case-by-case basis to determine if the infiltration facilitiy siting restrictions and setbacks requirements presented in Section 4.3 apply.

- **Filtration.** Filtration systems are similar to infiltration systems except filters use an engineered filter media rather than native soil to treat stormwater runoff. In addition, water that passes through the filter is typically collected in an underdrain system and discharged to the downstream conveyance system rather than infiltrating into the ground. Various media such as sand, perlite, zeolite, carbon, peat, and compost media have been used to treat stormwater runoff. Most filters are effective in removing particulates and particulate-bound pollutants. However, some media such as zeolite and leaf compost have been shown to be effective in removing dissolved metals. Filter systems are commonly configured as basins, trenches, vaults, or proprietary cartridge filtration systems. Filtration facilities may result in infiltration to underlying native soils. Projects must be evaluated on a case-by-case basis to determine if the infiltration facilitiy siting restrictions and setbacks requirements presented in Section 4.3 apply.

- **Infiltration.** These facilities infiltrate water to underlying native soils and use filtration, adsorption, and biological decomposition properties of soils to remove pollutants. Infiltration can provide multiple benefits including pollutant removal, peak flow control, ground water recharge, and flood control. Infiltration facilities are subject to the siting restrictions and setbacks requirements presented in Section 4.3. Potential adverse impact on ground water quality can also limit the use of infitlration. To adequately address the protection of ground water when evaluating infiltration it is important to understand the difference between soils that are suitable for runoff treatment and soils only suitable for flow control. Section 5.8.4 outlines the specific requirements for infiltration soil quality. In general, sufficient organic content and sorption capacity to remove pollutants must be present for soils to provide runoff treatment. Examples are silty and sandy loams. Coarser soils, such as gravelly sands, can provide flow control but are not suitable for providing runoff treatment. The use of coarser soils to provide flow control for runoff from pollutant generating surfaces must be preceded by treatment to protect ground water quality. Thus, there will be instances when soils are suitable for treatment but not flow control, and vice versa. In addition, note that

infiltration is regulated by the Department of Ecology and the Underground Injection Control (UIC) Program (WAC 173-218). Additional information on UIC and how it applies to infiltration and stormwater management is included in Section 4.3.4.

- **Wetpools**. Wet pool systems – including ponds, constructed wetlands, and wetvaults – treat runoff primarily by providing quiescent conditions using a permanent pool of standing water, which enhances the natural tendency of particles and particulate-bound pollutants to settle. In addition, depending on the design features, wet ponds and constructed wetlands may provide pollutant removal via biological uptake and vegetative filtration. Wetpools may be single-purpose facilities, providing only runoff treatment, or they may be combined with a detention pond or vault to also provide flow control. If combined, the wetpool portion of the facility can often be incorporated below the detention facility with little further loss of development area.

- **Oil/Water Separation.** Oil/water separators remove oil floating on the top of the water. There are two general types of separators – the American Petroleum Institute (API) separators and coalescing plate (CP) separators. Both use gravity to remove floating and dispersed oil. API separators, or baffle separators, are generally composed of three chambers separated by baffles. The efficiency of these separators is dependent on hydraulic residence time and on droplet size. CP separators use a series of parallel plates, which improve separation efficiency by providing more surface area, thus reducing the space needed for the separator. Oil/water separators must be located off-line from the primary conveyance/detention system, bypassing flows greater than the water quality design flow. Other devices/facilities that may be used for removal of oil include linear sand filters. Oil control devices/facilities must always be placed upstream of other treatment facilities and as close to the source of oil generation as possible.

- **Emerging Technologies.** Emerging technologies are new technologies that have not been evaluated using approved protocols, but for which preliminary data indicate that they may provide a desirable level of stormwater pollutant removal. They have not been evaluated in sufficient detail to be acceptable as stand alone BMPs for general usage in new development or redevelopment situations requiring Basic Treatment. Section 5.3 provides additional information on the use and application of emerging technologies in the City of Seattle. Note that the recommendations for use of these emerging technologies will change as the City and the Department of Ecology collect more data on their performance.

- **On-line vs. Off-line Systems.** Most treatment facilities can be designed as on-line systems with flows above the water quality design flow or volume simply passing through the facility with lesser or no pollutant removal efficiency. However, it is sometimes desirable to restrict flows to

treatment facilities and bypass the remaining higher flows around them. These are called off-line systems. Design guidelines for flow splitters for use in off-line facilities are provided in Section 5.4.5.

BMPs that provide both water quality treatment and flow control are presented in Table 4.3. These include some GSI infiltration facilities (e.g., bioretention without underdrains and permeable pavement), some traditional infiltration facilities (e.g., infiltration trenches), and combination detention/wetpool facilities (e.g., wetvaults).

5.2 BMP Selection Process for Water Quality Treatment Facilities

After it is determined which, if any, Minimum Requiremetns for Treatment apply to a project (see Chapter 2 and Section 5.1.3), BMPs may be selected. This chapter describes a step-by-step process for selecting the type of treatment facilities that will apply to individual projects. Physical features of sites that are applicable to treatment facility selection are also discussed. Refer to Section 5.3 for additional detail on the four treatment menus – oil control treatment, phosphorus treatment, enhanced treatment, and basic treatment.

5.2.1 Step-by-Step Selection Process for Treatment Facilities

Please refer to Figure 5.1. Use the step-by-step process outlined below to determine the type of treatment facilities applicable to the project.

Step 1: Determine the Receiving Waters and Associated Pollutants of Concern

An off-site analysis is required in order to obtain a more complete determination of the potential impacts of stormwater discharge from the project. The project proponent must determine the natural receiving water for the stormwater drainage from the project site (ground water, wetland, lake, stream, or salt water). This is necessary to determine the applicable treatment menu from which to select treatment facilities. If the discharge is to the municipal storm drainage system, the receiving water for the drainage system must be determined.

An analysis of the proposed land use(s) of the project shall also be used to determine the stormwater pollutants of concern. Urbanization of a watershed is often characterized by commercial, industrial, and transportation land uses such as roadways, airports, smelters, factories, and landfills. These land uses are typically associated with the production of metals, organic compounds, and other toxic wastes that can be entrained in precipitation and runoff (through air pollution or deposition on the ground surface) and ultimately enter aquatic areas. Residential land uses can also produce water quality pollutants resulting from landscaping activities (pesticides and fertilizers), pet waste, and various home and automobile maintenance activities.

Proceed to Step 2.

Step 2: Determine if an Oil Control Facility/Device is Required

The use of oil control devices and facilities is dependent upon the specific land use proposed for development.

The Oil Control Menu (Section 5.3.2) applies to projects that have "high-use sites" or have NPDES permits that require application of oil control. According to the Stomrwater Code (SMC 22.801.090), high-use sites are those that typically generate high concentrations of oil due to high traffic turnover or the frequent transfer of oil. High-use sites include:

- An area of a commercial or industrial site subject to an expected average daily traffic (ADT) count equal to or greater than 100 vehicles per 1,000 square feet of gross building area
- An area of a commercial or industrial site subject to petroleum storage and transfer in excess of 1,500 gallons per year, not including routinely delivered heating oil
- An area of a commercial or industrial site subject to parking, storage, or maintenance of 25 or more vehicles that are over 10 tons gross weight (trucks, buses, trains, heavy equipment, etc.)
- A road intersection with a measured average daily traffic (ADT) count of 25,000 vehicles or more on the main roadway and 15,000 vehicles or more on any intersecting roadway, excluding projects proposing primarily pedestrian or bicycle use improvements.

Note: The project proponent must develop an ADT estimate, and contact SDOT for approval of the ADT estimate, early in the design process. The City may also require oil control facilities from this menu to be used on sites that generate high concentrations of oil.

If oil control is required for the site, please refer to the General Requirements in Section 5.4. These requirements may affect the design and placement of facilities on the site (e.g., flow splitting). Please refer to the Oil Control Menu for a listing of oil control facility options. Then see Section 5.11 of this volume for information on the proper selection of options and design details.

If an Oil Control Facility is required, select and apply an Oil Control Facility. Please refer to the Oil Control Menu in Section 5.3.2. After selecting an Oil Control Facility, proceed to Step 3.

If an Oil Control Facility is not required, proceed directly to Step 3.

Step 3: Determine if Infiltration for Pollutant Removal is Practicable

Check the infiltration treatment design criteria in Section 5.8 of this volume. Infiltration into underlying native soils can be effective at treating stormwater runoff, but soil properties (or the propoerties of imported filter media) must be

appropriate to achieve effective treatment while not adversely impacting ground water resources. The location and depth to bedrock, the water table, or impermeable layers (such as glacial till), and the proximity to wells, foundations, septic tank drainfields, and unstable slopes can preclude the use of infiltration. See the infiltration facility siting restrictions and setbacks requirements presented in Section 4.3. If drainage area exceeds 2,000 square feet or is not thoroughly stabilized, infiltration facilities must be preceded by a presettling facility (e.g., filter strip, presettling basin or vault) to reduce the occurrence of plugging. Dispersed flow should not be concentrated for presettling purposes. Any of the basic treatment facilities, and detention ponds designed to meet flow control requirements, can also be used for pre-settling. If an oil/water separator is necessary for oil control, it can also function as the presettling basin as long as the influent suspended solids concentrations are not high. However, frequent inspections are necessary to determine when accumulated solids exceed the 6-inch depth at which clean-out is required (see Appendix D).

If infiltration is planned, please refer to the General Requirements in Section 5.4. They can affect the design and placement of facilities on your site. For non-residential developments, if your infiltration site is within 1/4 mile of a fish-bearing stream, a tributary to a fish-bearing stream, or a lake, please refer to Step 5 below to determine if part or all of the site is subject to the Enhanced Treatment Menu (Section 5.3.4). If the Enhanced Treatment Menu applies, read the comment under “Infiltration with appropriate pretreatment” to identify special pretreatment needs. If your infiltration site is within 1/4 mile of a phosphorus-sensitive receiving water, please refer to the Phosphorus Treatment Menu (Section 5.3.3) for special pretreatment needs.

If infiltration is practicable, select and apply pretreatment and an infiltration facility.

If infiltration is not practicable, proceed to Step 4.

Step 4: Determine if Phosphorus Control is Required

The requirement to provide phosphorus control is determined by the City of Seattle, the Department of Ecology, or the USEPA. At the time this manual was developed, there were no established phosphorus-specific control requirements for project-scale treatment BMPs in the City. **However, the use of leaf compost in media filtration systems is not allowed when the drainage water is discharged to the following receiving waters: Green Lake, Bitter Lake, Haller Lake, and Lake Washington.** In the future, the City may develop a management plan and implementing ordinances or regulations for control of phosphorus from new development and redevelopment for the receiving water(s) of the stormwater drainage. The project proponent must comply with all applicable legal requirements.

If phosphorus control is required, select and apply a phosphorus treatment facility. Refer to the Phosphorus Treatment Menu in Section 5.3.3. Select an

option from the menu after reviewing the applicability and limitations, site suitability, and design criteria of each for compatibility with the site.

If you have selected a phosphorus treatment facility, refer to the General Requirements in Section 5.4. They may affect the design and placement of the facility on the site.

Note: Project sites subject to the Phosphorus Treatment requirement could also be subject to the Enhanced Treatment requirement (see Step 5). In that event, apply a facility or a treatment train that is listed in both the Enhanced Treatment Menu and the Phosphorus Treatment Menu.

If phosphorus treatment is not required for the site, proceed to Step 5.

Step 5: Determine if Enhanced Treatment is Required

Enhanced treatment is required for the following project sites that discharge to fish-bearing streams, lakes, or to waters or conveyance systems tributary to fish-bearing streams or lakes:

- Industrial project sites
- Commercial project sites
- Multi-family project sites
- Fully controlled and partially controlled limited access highways with Annual Average Daily Traffic (AADT) counts of 15,000 or more
- All other roads with an AADT of 7,500 or greater.

However, sites listed above that discharge directly (or, indirectly through a municipal storm sewer system) to Basic Treatment Receiving Waters (see below) are not subject to Enhanced Treatment requirements. Likewise, any portions of the above-listed project sites that are identified as subject to Basic Treatment requirements only (see Step 6) are not subject to Enhanced Treatment requirements. For developments with a mix of land use types, the Enhanced Treatment requirement shall apply when the runoff from the areas subject to the Enhanced Treatment requirement comprises 50 percent or more of the total runoff from the project area.

Basic Treatment Receiving Waters include:

- All marine waters
- Lake Union
- Lake Washington
- Ship Canal and bays between Lake Washington and Puget Sound
- Duwamish River.

If the project must apply Enhanced Treatment, select and apply an appropriate Enhanced Treatment facility. Please refer to the Enhanced Treatment Menu in Section 5.3.4. Select an option from the menu after reviewing the applicability and limitations, site suitability, and design criteria of each for compatibility with the site.

Note: Projects subject to the Enhanced Treatment requirement could also be subject to a phosphorus treatment requirement if located in an area designated for phosphorus control. In that event, apply a facility or a treatment train that is listed in both the Enhanced Treatment Menu and the Phosphorus Treatment Menu. If you have selected an Enhanced Treatment facility, refer to the General Requirements in Section 5.4. They may affect the design and placement of the facility on the site.

If Enhanced Treatment does not apply to the site, please proceed to Step 6.

Step 6: Select a Basic Treatment Facility

The Basic Treatment Menu is generally applied to:

- Project sites that discharge stormwater to the ground (i.e., via infiltration) **UNLESS**:
  - The soil suitability criteria for infiltration treatment are met (see Section 5.8.4), or
  - The project uses infiltration strictly for flow control – not treatment – and the discharge is within 1/4 mile of a phosphorus sensitive lake (use the Phosphorus Treatment Menu), or within 1/4 mile of a fish-bearing stream, or a lake (use the Enhanced Treatment Menu).
- Residential projects not otherwise needing phosphorus control in Step 4 as designated by USEPA, the Department of Ecology, or the City of Seattle
- Project sites discharging directly to Basic Treatment Receiving Waters (listed under Step 5)
- Project sites that drain to streams that are not fish-bearing, or to waters not tributary to fish-bearing streams
- Landscaped areas of industrial, commercial, and multi-family project sites, and parking lots of industrial and commercial project sites, dedicated solely to parking of employees' private vehicles that do not involve any other pollution-generating activities (e.g., industrial activities; customer parking; storage of erodible or leachable material, wastes, or chemicals; vehicle maintenance).

For developments with a mix of land use types, the Basic Treatment requirement shall apply when the runoff from the areas subject to the Basic Treatment requirement comprises 50 percent or more of the total runoff within a threshold discharge area.

Refer to the Basic Treatment Menu in Section 5.3.5. Select an option from the menu after reviewing the applicability and limitations, site suitability, and design criteria of each for compatibility with the site. After selecting a Basic Treatment Facility, refer to the General Requirements in Section 5.4. They may affect the design and placement of the facility on the site.

You have completed the treatment facility selection process.

5.2.2 Other Treatment Facility Selection Factors

Factors that influence facility selection and the associated requirements are briefly summarized below.

5.2.2.1 Soil Type

Soil conditions can affect the performance of some treatment systems, particularly infiltration systems, which rely on soil hydraulic and physical/chemical characteristics to treat runoff (see Section 5.8.4). Similarly, wet ponds constructed in permeable soil require a liner to help maintain the permanent pool necessary to provide treatment. Maintaining a permanent pool in the first cell is necessary to avoid resuspension of settled solids. Biofiltration swales in coarse soils may require soil amendments to support vegetation growth and/or to reduce the infiltration rate.

5.2.2.2 High Sediment Input

Runoff containing elevated concentrations of TSS, particularly sites with a large amount of fine-grained particles like silt and sand, can clog infiltration and filtration facilities. High TSS loads can also hinder the function of oil/water separators, especially CPS systems, if sediment clogs the coalescing plates. Pretreatment to remove suspended solids is required to extend the maintenance cycle for these systems.

5.2.2.3 Other Physical Factors

Infiltration facilities must be sited appropriately to ensure performance. Issues that must be considered include the presence of Landslide–Prone Critical Areas, the proximimty to Steep Slope Critical Areas, site setbacks (e.g., property lines, structures, wells, septic tanks, and contaminated sites), and vertical separation from the water table, bedrock or other impermeable layer. Site restrictions and setbacks for infiltration facilities are provided in Section 4.3.4.2. These requirements apply to facilities characterized as “infiltration facilities” and may also apply to other BMPs that infiltrate water to underlying native soils, such as biofiltration and filtration BMPs.

In addition, land slopes may restrict the use of several BMPs. For example, biofiltration swales are usually situated on sites with slopes of less than 5 percent, although greater slopes can be considered.

5.3 Treatment Facility Menus

This section identifies choices that comprise the treatment facility menus referred to in Section 5.2. The menus in this section are discussed in the order of the decision process shown in Figure 5.1 (selection flow chart, Section 5.2) and are as follows:

- Oil Control Menu, Section 5.3.2
- Phosphorus Treatment Menu, Section 5.3.3
- Enhanced Treatment Menu, Section 5.3.4
- Basic Treatment Menu, Section 5.3.5.

5.3.1 Guide to Applying Menus

Read the step-by-step selection process for treatment facilities in Section 5.2.1.

Determine which menus apply to the project. This will require knowledge of (1) the receiving water(s) that the project site ultimately discharges to, (2) whether the City of Seattle, the Department of Ecology or the USEPA, has identified the receiving water as subject to phosphorus control requirements, and (3) whether the site qualifies as subject to oil control.

Determine if your project requires oil control.

If the project requires oil control, or if you elect to provide enhanced oil pollution control, choose one of the options presented in the Oil Control Menu, Section 5.3.2. Detailed designs for oil control facilities are given in subsequent sections.

Note: One of the other three treatment menus will also need to be applied along with oil control.

Find the Treatment Menu that applies to the project – Basic, Enhanced, or Phosphorus.

Each menu presents treatment options. A project site may be subject to both the Enhanced Treatment requirement and the Phosphorus Treatment requirement. In that event, select a facility or a treatment train that is listed in both treatment menus. Note: if flow control requirements apply, it will usually be more economical to use the combined detention/wetpool facilities. Detailed facility designs for all the possible options are given in subsequent sections in this volume.

Read Section 5.4 concerning general facility requirements.

They apply to all facilities and may affect the design and placement of facilities on the site.

5.3.2 Oil Control Menu

Note: Where this menu is applicable, it is in addition to facilities required by one of the other Treatment Menus.

Where Applied: The Oil Control Menu applies to projects that have high-use sites, or are subject to NPDES permits that require oil control. Specific applicability criteria are described in Section 5.2.1, Step 2.

Application on the Project Site: Oil control facilities are to be placed upstream of other facilities, as close to the source of oil generation as practical. For high-use sites located within a larger commercial center, only the impervious surface associated with the high-use portion of the site is subject to oil treatment requirements. If common parking for multiple businesses is provided, treatment shall be applied to the number of parking stalls required for the high-use business only. However, if the treatment collection area also receives runoff from other areas, the treatment facility must be sized to treat all stormwater passing through it.

High-use roadway intersections shall treat lanes where vehicles accumulate during the signal cycle, including left and right turn lanes and through lanes, from the beginning of the left turn pocket. If no left turn pocket exists, the treatable area shall begin at a distance equal to three car lengths from the stop line. If runoff from the intersection drains to more than two collection areas that do not combine within the intersection, treatment may be limited to any two of the collection areas.

Performance Goal: The facility choices in the Oil Control Menu are intended to achieve the goals of no ongoing or recurring visible sheen, and to have a 24-hour average Total Petroleum Hydrocarbon (TPH) concentration no greater than 10 mg/l, and a maximum of 15 mg/l for a discrete sample (grab sample).

Note: Use the method for NWTPH-Dx in Ecology Publication No. ECY 97-602, Analytical Methods for Petroleum Hydrocarbons. If the concentration of gasoline is of interest, the method for NWTPH-Gx should be used to analyze grab samples.

Options: Any one of the following options may be chosen to satisfy the phosphorus treatment requirement. The API and coalescing plate oil/water separators in particular require frequent maintenance.

- **API-Type Oil/Water Separator** (see Section 5.11.4)
- **Coalescing Plate Oil/Water Separator** (see Section 5.11.5)
- **Linear Sand Filter** (see Section 5.9.7).

Note: The linear sand filter is used in the Basic, Enhanced, and Phosphorus Treatment menus also. If used to satisfy one of those treatment requirements, the same facility shall not also be used to satisfy the oil control requirement

unless enhanced maintenance (quarterly cleaning) is assured. This is to prevent clogging of the filter by oil so that it will function for suspended solids and phosphorus removal as well.

5.3.3 Phosphorus Treatment Menu

Where Applied: The Phosphorus Treatment Menu applies to projects within watersheds that have been determined by the City of Seattle, the Department of Ecology, or the USEPA to be sensitive to phosphorus and are being managed to control phosphorus inputs from stormwater. At the time this manual was published, no water bodies in the City of Seattle were subject to phosphorus control requirements.

This menu applies to stormwater conveyed to water bodies by surface flow as well as to stormwater infiltrated within one-quarter mile of a phosphorus-sensitive waterbody in soils that do not meet the soil suitability criteria in Section 5.8.4.

Performance Goal: The Phosphorus Menu facility choices are intended to achieve a goal of 50 percent total phosphorus removal for a range of influent concentrations of 0.1 to 0.5 mg/l total phosphorus. In addition, the choices are intended to achieve the Basic Treatment performance goal. The performance goal applies to the water quality design storm volume or flow rate, whichever is applicable, and on an annual average basis. The incremental portion of runoff in excess of the water quality design flow rate or volume can be routed around the facility (off-line treatment facilities), or can be passed through the facility (on-line treatment facilities) provided a net pollutant reduction is maintained. Off-line facilities are recommended to prevent resuspension and washout of accumulated phosphorus during large storm events.

Options: Any one of the following options may be chosen to satisfy the phosphorus treatment requirement:

- Infiltration with appropriate pretreatment – see Section 5.5.
    - Infiltration treatment

      If infiltration is through soils meeting the minimum criteria for infiltration treatment (see Section 5.8.4), a presettling basin or a basic treatment facility can serve for pretreatment.
    - Infiltration preceded by Phosphorus Treatment

      If the infiltration soils do not meet the soil suitability criteria (see Section 5.8.4) and the infiltration site is within 1/4 mile of a phosphorus-sensitive receiving water, or a tributary to that water, treatment must be provided by one of the other treatment facility options listed below.
- **Large Sand Filter** – see Section 5.9
- **Amended Sand Filter** – see the Ecology 2005 *Stormwater Management Manual for Western Washington*.

Note: Processed steel fiber and crushed calcitic limestone are the only sand filter amendments for which data are available that documents increased dissolved metals removal. Use of amended sand filters for phosphorus treatment requires prior Director approval for all applications.

- **Large Wetpond** – see Section 5.10
- **Media Filter targeted for phosphorus removal.** Some proprietary Media Filter treatment systems have been approved by Ecology for phosphorus treatment. Those facilities approved by Ecology may be used in the City of Seattle. See Section 5.12 for guidance on emerging technologies.
- **Ecology Embankment** – see Section 5.7.5, or the 2008 WSDOT *Highway Runoff Manual*, Chapter 5.
- **Two-Facility Treatment Trains** – see Table 5.1.

Table 5.1. Treatment Trains for Phosphorus Removal.

First Basic Treatment Facility	Second Treatment Facility
Biofiltration Swale (5.6.3)	Basic Sand Filter or Sand Filter Vault (5.9)
Filter Strip (5.7)	Linear Sand Filter (5.9.7 [no presettling needed])
Linear Sand Filter (5.9.7)	Filter Strip (5.7)
Basic Wetpond (5.10.3)	Basic Sand Filter or Sand Filter Vault (5.9)
Wetvault (5.10.4)	Basic Sand Filter or Sand Filter Vault (5.9)
Stormwater Treatment Wetland (5.10.5)	Basic Sand Filter or Sand Filter Vault (5.9)
Basic Combined Detention and Wetpool (5.10.6)	Basic Sand Filter or Sand Filter Vault (5.9)

5.3.4 Enhanced Treatment Menu

Where Applied: Enhanced treatment is required for particular project sites that discharge to fish-bearing streams, lakes, or to waters or conveyance systems tributary to fish-bearing streams or lakes. A description of the project sites that are subject to this requirement is provided in Section 5.2.1, Step 5.

Performance Goal: The Enhanced Menu facility choices are intended to provide a higher rate of removal of dissolved metals than Basic Treatment facilities, but also achieve the Basic Treatment performance goal. The performance goal assumes that the facility is treating stormwater with dissolved copper typically ranging from 3 to 20 µg/l, and dissolved zinc ranging from 20 to 300 µg/l.

The performance goal applies to the water quality design storm volume or flow rate, whichever is applicable, and on an annual average basis. The incremental portion of runoff in excess of the water quality design flow rate or volume can be routed around the facility (off-line treatment facilities), or can be passed through the facility (on-line treatment facilities) provided a net pollutant reduction is

maintained. Where possible, off-line facilities are recommended to prevent resuspension and washout of accumulated sediment and other particulate-bound pollutants during large storm events.

Options: Any one of the following options may be chosen to satisfy the enhanced treatment requirement:

- **Infiltration with appropriate pretreatment** – see Section 5.8:
  - Infiltration treatment

    If infiltration is through soils meeting the minimum criteria for infiltration treatment (see Section 5.8.4), a presettling basin or a basic treatment facility can serve for pretreatment.
  - Infiltration preceded by Enhanced Treatment

    If the soils do not meet the soil suitability criteria (see Section 5.8.4) and the infiltration site is within 1/4 mile of a fish-bearing stream, a tributary to a fish-bearing stream, or a lake, treatment must be provided by one of the other treatment facility options listed below.
- **Large Sand Filter** – see Section 5.9.
- **Amended Sand Filter** – see the Ecology 2005 *Stormwater Management Manual for Western Washington*.

  Note: Processed steel fiber and crushed calcitic limestone are the only sand filter amendments for which data is available that documents increased dissolved metals removal. Use of amended sand filters for enhanced treatment requires prior Director approval.
- **Stormwater Treatment Wetland** – see Section 5.10.5
- **Compost-amended Filter Strip** – see Section 5.7.4
- **Ecology Embankment** – see Section 5.7.5, or the 2008 WSDOT *Highway Runoff Manual* (http://www.wsdot.wa.gov/Environment/WaterQuality/Runoff/HighwayRunoffManual.htm)
- **Two Facility Treatment Trains** – see Table 5.2.
- **Bioretention** – See Section 5.8.6.4, and Chapter 4.

  Note: Bioretention systems that are constructed using the soil mix specified in Section 5.8.4 will qualify as Enhanced Treatment, provided that these systems are designed to infiltrate 91 percent of the total runoff volume (as determined using an approved continuous runoff model).
- **Other Ecology approved options** – As other BMPs are approved by Ecology for meeting enhanced treatment requirements, these may be added to the City of Seattle's approved list of acceptable enhanced treatment BMPs. Additional BMPs will be accepted by the Director on a case-by-case basis, after approval by Ecology.

Table 5.2. Treatment Trains for Dissolved Metals Removal.

First Basic Treatment Facility	Second Treatment Facility
Biofiltration Swale (5.6)	Basic Sand Filter or Sand Filter Vault or Media Filter[1] (5.9)
Filter Strip (5.7)	Linear Sand Filter with no presettling cell needed (5.9.7)
Linear Sand Filter (5.9.7)	Filter Strip (5.7)
Basic Wetpond (5.10.3)	Basic Sand Filter or Sand Filter Vault or Media Filter[1] (5.9)
Wetvault (5.10.4)	Basic Sand Filter or Sand Filter Vault or Media Filter[1] (5.9)
Basic Combined Detention/Wetpool (5.10.6)	Basic Sand Filter or Sand Filter Vault or Media Filter[1] (5.9)
Basic Sand Filter or Sand Filter Vault with a presettling cell if the filter isn't preceded by a detention facility (5.9)	Media Filter[1]

[1] The media must be of a nature that has the capability to remove dissolved metals effectively as approved by Ecology and accepted by the Director.

5.3.5 Basic Treatment Menu

Where Applied: The Basic Treatment Menu is generally applied to projects not subject to phosphorus or enhanced treatment requirements. See Section 5.2 for specific applicability.

Performance Goal: The Basic Treatment Menu facility choices are intended to achieve 80 percent removal of total suspended solids for influent concentrations greater than 100 mg/l, but less than 200 mg/l. For influent concentrations greater than 200 mg/l, a higher treatment goal may be appropriate. For influent concentrations less than 100 mg/l, the facilities are intended to achieve an effluent goal of 20 mg/l total suspended solids.

The performance goal applies to the water quality design storm volume or flow rate, whichever is applicable. The goal also applies on an average annual basis to the entire annual discharge volume (treated plus bypassed). The incremental portion of runoff in excess of the water quality design flow rate or volume can be routed around the facility (off-line treatment facilities), or can be passed through the facility (on-line treatment facilities) provided a net TSS reduction is maintained.

Options: Any one of the following options may be chosen to satisfy the basic treatment requirement:

- **Biofiltration Swales** – see Section 5.6
- **Filter Strips** – see Section 5.7
- **Infiltration** – see Section 5.8
- **Bioretention** – see Section 5.8.5.4, and Chapter 4.

- **Ecology Embankment** – see Section 5.7.5, or the 2008 WSDOT *Highway Runoff Manual*
- **Sand Filters** – see Section 5.9
- **Basic Wetpond** – see Section 5.10
- **Wetvault** – see Section 5.10 (see note)
- **Stormwater Treatment Wetland** – see Section 5.10
- **Combined Detention and Wetpool Facilities** – see Section 5.10
- Some proprietary media filtration systems are also approved for basic treatment. Those facilities approved by Ecology may be used in the City of Seattle. See Section 5.12 for guidance on emerging technologies.
- **Other Ecology approved options** – As other BMPs are approved by Ecology for meeting basic treatment requirements, these may be added to the City of Seattle's approved list of acceptable basic treatment BMPs. Additional BMPs will be accepted by the Director on a case-by-case basis, after approval by Ecology.

Note: A wetvault may be used for commercial, industrial, or road projects if there are space limitations. The City of Seattle does not permit the use of wetvaults for single-family residential projects.

5.4 General Requirements for Stormwater Treatment Facilities

Note: All Figures in Section 5.4 are courtesy of King County.

This section addresses general requirements applicable to all treatment facilities. Requirements discussed in this section include design flow rates and volumes, facility sequencing for treatment trains, liners, flow splitters, and flow dispersion devices. Specific design criteria and requirements for individual treatment facilities are provided in subsequent sections.

5.4.1 Design Volume and Flow

Water quality treatment facilities shall be installed and maintained to treat flows from the pollution generating pervious and impervious surfaces on the site being developed. When stormwater flows from other areas, including non-pollution generating surfaces (e.g., roofs) and offsite areas, cannot be separated or bypassed, treatment BMPs shall be designed for the entire area draining to the treatment facility.

The water quality design volumes and rates are identified in Chapter 2, Section 2.4.7, and outlined again below.

5.4.1.1 Water Quality Design Storm Volume

Stormwater treatment facilities are designed based on the stormwater runoff volume from the contributing area or a peak flow rate. According to SMC 22.805.090.B.1, water quality treatment facilities must be designed to treat the daily runoff volume at or below which 91 percent of the total runoff volume for the simulation period occurs, as determined using an approved continuous model. This volume is calculated as follows (see also Chapter 6):

1. Rank the daily runoff volumes from highest to lowest
2. Sum all the daily volumes and multiply by 0.09
3. Sequentially sum daily runoff volumes, starting with the highest value, until the total equals 9 percent of the total runoff volume as calculated in Step 2. The last daily value added to the sum is defined as the water quality design volume.

5.4.1.2 Water Quality Design Flow Rate

According to SMC 22.805.090.B.1, different design flow rates have been established depending on whether the proposed treatment facility will be located upstream or downstream of a detention facility.

Facilities Located Upstream of Detention Facilities or when Detention Facilities are not required: The design flow rate is the flow rate at or below which 91 percent of the total runoff volume for the simulation period is treated, as determined using an approved continuous runoff model. Chapter 6 provides additional details on how the continuous models perform this calculation.

Facilities Located Downstream of Detention Facilities: The design flow rate is the release rate from the detention facility that has a 50 percent annual probability of occurring in any given year (2-year recurrence interval), as determined using an approved continuous runoff model. Treatment facilities that are located downstream of detention facilities shall only be designed as on-line facilities. High flow bypasses are not permitted.

Treatment facilities located upstream of a detention system can be designed as online or off-line facilities.

- On-line facilities: Runoff flow rates in excess of the water quality design flow rate can be routed through the facility provided a net pollutant reduction is maintained, and the applicable annual average performance goal is likely to be met.
- Off-line facilities: For treatment facilities not preceded by an equalization or storage basin, flows exceeding the water quality design flow rate may be bypassed around the treatment facility. However, during bypass events, the facility shall continue to receive and treat the water quality

design flow rate. Only the higher incremental portion of flow rates are bypassed around a treatment facility. Flow splitter design guidelines are provided in Section 5.4.5.

Treatment facilities preceded by an equalization or storage basin may identify a lower water quality design flow rate provided that at least 91 percent of the total runoff volume predicted by an approved continuous runoff model is treated.

Infiltration Facilities Providing Water Quality Treatment: Infiltration facilities designed for water quality treatment must infiltrate 91 percent of the total runoff volume as determined using an approved continuous runoff model. The procedure is the same as for designing infiltration for flow control (see Chapter 4, Section 4.5), except that the target is to infiltrate 91 percent of the total runoff volume. In addition, to prevent the onset of anaerobic conditions, an infiltration facility designed for water quality treatment must be designed to drain the water quality design treatment volume (the 91st percentile, 24-hour volume) within 48 hours. This can be calculated by using a horizontal projection of the infiltration basin mid-depth dimensions and the estimated long-term infiltration rate.

5.4.1.3 Runoff Requiring Treatment

Runoff from pollution-generating impervious or pervious surfaces must be treated. Pollution-generating impervious surfaces (PGIS) are those impervious surfaces considered to be a significant source of pollutants in stormwater runoff (SMC 22.801.170).

Such surfaces include those which are subject to: vehicular use; industrial activities; or storage of erodible or leachable materials, wastes, or chemicals, and which receive direct rainfall or the run-on or blow-in of rainfall. Erodible or leachable materials, wastes, or chemicals are those substances which, when exposed to rainfall, measurably alter the physical or chemical characteristics of the rainfall runoff. Examples include erodible soils that are stockpiled, uncovered process wastes, manure, fertilizers, oily substances, ashes, kiln dust, and garbage dumpster leakage. Metal roofs are also considered to be PGIS unless they are coated with an inert, non-leachable material.

A surface, whether paved or not, shall be considered subject to vehicular use if it is regularly used by motor vehicles. The following are considered regularly-used surfaces: roads, unvegetated road shoulders, bike lanes within the traveled lane of a roadway, driveways, parking lots, unfenced fire lanes, vehicular equipment storage yards, and airport runways.

The following are not considered regularly-used surfaces: paved bicycle pathways separated from and not subject to drainage from roads for motor vehicles, fenced fire lanes, infrequently used maintenance access roads, and sidewalks.

Pollution-generating pervious surfaces (PGPS) are any non-impervious surface subject to the use of pesticides and fertilizers or loss of soil. Typical PGPS include lawns, landscaped areas, golf courses, parks, cemeteries, and sports fields (SMC 22.801.170).

The Stomrwater Code and the glossary in Appendix A provides definitions of these terms.

Summary of Areas Needing Treatment

- All runoff from pollution-generating impervious surfaces is to be treated through the water quality facilities specified in Sections 5.2 and 5.3.
- If runoff from pollution-generating pervious surfaces drains to a treatment facility, the pervious area runoff must be estimated and added to the runoff from impervious areas to size treatment facilities.
- If runoff from non-pollution generating surfaces (pervious or impervious) reaches a runoff treatment BMP, flows from those areas must be included in the sizing calculations for the facility. Once runoff from non-pollution generating areas is mixed with runoff from pollution-generating areas, it cannot be separated before treatment.
- Runoff from backyards can drain into native vegetation in areas designated as open space or buffers. In these cases, the area in native vegetation may be used to provide the requisite water quality treatment, provided it meets the requirements for full dispersion (see Section 4.4.9).
- Runoff from impervious surfaces that are not pollution-generating need not be treated and may bypass runoff treatment, if it is not mingled with runoff from pollution-generating surfaces.
- Roof runoff is still subject to flow control per the Minimum Requirements for Flow Control in Chapter 2. Note that metal roofs are considered pollution generating unless they are coated with an inert non-leachabale material.
- Drainage from areas in native vegetation should not be mixed with untreated runoff from streets and driveways, if possible. It is best to infiltrate or disperse this relatively clean runoff to maximize recharge to shallow ground water, wetlands, and streams.

5.4.2 Sequence of Facilities

The Phosphorus Removal and Enhanced Treatment Menus, described in Sections 5.3.3 and 5.3.4 respectively, include treatment options in which more than one type of treatment facility is used. In those options, the sequence of facilities is prescribed. This is because the specific pollutant removal role of the second or third facility in a treatment often assumes that significant solids settling has already occurred. For example, phosphorus removal using a two-facility

treatment train relies on the second facility (sand filter) to remove a finer fraction of solids than those removed by the first facility.

There is also the question of whether treatment facilities should be placed upstream or downstream of detention facilities that are needed for flow control purposes. In general, all treatment facilities may be installed upstream of detention facilities, although presettling basins are needed for sand filters and infiltration systems. However, not all treatment facilities can function effectively if located downstream of detention facilities. For example, filter strips and continuous inflow biofiltration swales cannot be installed downstream of detention facilities because runoff enters these facilities as sheet or unconcentrated flow. Once runoff is collected and conveyed in a channel or piped system, it cannot be effectively re-dispersed to enable these systems to function properly.

Other types of treatment facilities present special problems that must be considered before placement downstream of detention facilities. For example, prolonged flows discharged by a detention facility that is designed to meet the flow duration standard may interfere with proper functioning of basic biofiltration swales and sand filters. Grasses typically specified in the basic biofiltration swale design will not survive. A wet biofiltration swale design would be a better choice.

Similarly, for sand filters located downstream of detention facilities, the prolonged flows may cause extended saturation periods within the filter. Saturated sand can lose all oxygen and become anoxic. If that occurs, some amount of phosphorus captured within the filter may become soluble and released. To prevent long periods of sand saturation, adjustments may be necessary after the sand filter is in operation to bypass some areas of the filter. This bypassing will allow them to drain completely. It may also be possible to employ a different type of facility that is less sensitive to prolonged flows.

Oil control facilities for runoff treatment must be located upstream of treatment and detention facilities and as close to the source of oil-generating activity as possible.

In any case, all facilities receiving runoff must provide an outlet and overflow to an approved discharge point. Approved discharge points shall, at a minimum, maintain natural drainage patterns to the maximum extent practicable, and not cause a significant adverse impact to the downstream conveyance system, receiving surface water, or down-gradient properties.

Table 5.3 summarizes placement considerations of treatment facilities in relation to detention.

Table 5.3. Treatment Facility Placement in Relation to Detention.

Water Quality Facility	Preceding Detention	Following Detention
Basic biofiltration swale (Section 5.6.3)	OK	OK. Prolonged flows may reduce grass survival. Consider wet biofiltration swale.
Wet biofiltration swale (Section 5.6.4)	OK	OK
Filter strip (Section 5.7)	OK	No—must be installed before flows concentrate.
Basic or large wetpond (Section 5.10)	OK	OK—less water level fluctuation in ponds downstream of detention may improve aesthetic qualities and performance.
Basic or large combined detention and wetpond (Section 5.10.6)	Not applicable	Not applicable
Wetvault (Section 5.10)	OK	OK
Basic or large sand filter or sand filter vault (Section 5.9)	OK, but presettling and control of floatables needed	OK—sand filters downstream of detention facilities may require field adjustments if prolonged flows cause sand saturation and interfere with phosphorus removal.
Stormwater treatment wetland/pond (Section 5.10.5)	OK	OK—less water level fluctuation and better plant diversity are possible if the stormwater wetland is located downstream of the detention facility.

5.4.3 Setbacks, Slopes, and Embankments

This section provides information on required setbacks, slopes, embankments, and liners applicable to most water quality treatment BMPs.

5.4.3.1 Setbacks

Setback requirements are intended to protect adjacent properties from flooding and to protect receiving waters and other sensitive areas from water quality impacts. Setbacks are typically specified relative to site restrictions such as property lines, easements, structures, etc. that impose constraints on development. Constraints may also be imposed from natural features such as land slopes, landslide prone areas, sensitive habitat areas, and related requirements found in City of Seattle Critical Areas Ordinance SMC 25.09. These should also be reviewed for specific application to the proposed development.

In general, it is recommended that water quality treatment facilities be a minimum of 5 feet from any structure, property line, and any vegetative buffer. (Note: restrictions and setbacks for infiltration facilities are more restrictive, as outlined in Section 4.3.3.2.) The pond water surface at the outlet invert elevation shall be setback 100 feet from existing septic system drainfields. This setback may be

reduced with written approval of the Seattle-King County Department of Public Health. Any variations from these requirements are outlined under the detailed criteria for each individual BMP.

5.4.3.2 Side Slopes

Interior side slopes for water quality treatment facilities shall not exceed 3H:1V, unless fencing is provided. Moderately undulating slopes can provide a more natural setting for the facility. In general, gentle side slopes improve the aesthetic attributes of the facility and enhance safety.

Exterior side slopes shall not be steeper than 2H:1V unless recommended by a geotechnical engineer.

5.4.3.3 Fencing

If the water quality facility will hold open standing water deeper than 2 feet (e.g., wetponds, sand filters, or stormwater wetlands), fencing is required along any interior slopes steeper than 3H:1V. If only sections of the slope are steeper than 3H:1V and those sections are shorter than 20 feet, barrier shrubs, such as barberry, may be used rather than fencing. Planting climbing vines at the base of a fence can enhance its aesthetic qualities.

If required, fencing shall be placed at or above the overflow water surface. Side slope and attendant fencing requirements are not applicable to slopes above the overflow water surface.

Interior side slopes may be retaining walls, provided that the design is prepared per the City of Seattle Building Code. A fence shall be provided along the top of the wall.

If facilities are privately owned and maintained, the fencing requirements of this manual are recommended rather than required. However, the site must still comply with any fencing requirements in SMC 15.22.00.

5.4.3.4 Embankments

See embankment and dam safety requirements presented in Section 4.6.3.

5.4.4 Facility Liners

Liners discussed in this section are intended to reduce the likelihood that pollutants in stormwater will reach ground water when runoff treatment facilities are constructed. In addition to groundwater protection considerations, some facility types require permanent standing water for proper functioning. An example is the first cell of a wetpond.

There are two types of facility liners: treatment liners and low permeability liners.

Treatment liners amend the soil with materials that treat stormwater before it reaches more freely draining soils. They have slow rates of infiltration, generally less than 2.4 inches per hour (1.7 x 10 -3 cm/s), but not as slow as low permeability liners. Treatment liners may use in-place native soils or imported soils, provided that the design criteria outlined below are met.

Low permeability liners reduce infiltration to a very slow rate, generally less than 0.02 inches per hour (1.4 x 10 -5 cm/s). These types of liners are often used for industrial or commercial sites with a potential for high pollutant loading in the stormwater runoff. Low permeability liners may be fashioned from compacted till, clay, geomembrane, or concrete.

Liners may also be required in areas where infiltration is not permitted (e.g., sloped areas).

Table 5.4 shows the type of liner required for use with various runoff treatment facilities. Other liner configurations may be used with prior approval from the Director.

Table 5.4. Lining Types Required for Runoff Treatment Facilities.

WQ Facility	Area to be Lined	Type of Liner Required
Presettling basin	Bottom and sides	Low permeability liner or Treatment liner (If the basin will intercept the seasonal high ground water table, a treatment liner may be recommended.)
Wetpond	First cell: bottom and sides to WQ design water surface	Low permeability liner or Treatment liner
	Second cell: bottom and sides to WQ design water surface	Treatment liner
Combined detention/WQ facility	First cell: bottom and sides to WQ design water surface	Low permeability liner or Treatment liner
	Second cell: bottom and sides to WQ design water surface	Treatment liner
Stormwater wetland	Bottom and sides, both cells	Low permeability liner
Sand filter basin	If over a critical aquifer recharge area. See Section 5.9.2.2.	Low permeability or treatment liner
Sand filter vault	Not applicable	No liner needed
Linear sand filter	Not applicable if in vault Bottom and sides of presettling cell if not in vault	No liner needed Low permeability or treatment liner
Media filter (in vault)	Not applicable	No liner needed
Wet vault	Not applicable	No liner needed

Liners shall be evenly placed over the bottom and/or sides of the treatment area of the facility as indicated in Table 5.4. Areas above the treatment volume that are required to pass flows greater than the water quality treatment flow (or volume) need not be lined. However, the lining must be extended to the top of the interior side slope and be anchored if it cannot be permanently secured by other means.

5.4.4.1 Design Criteria for Treatment Liners

This section presents the design criteria for treatment liners.

- A 2-foot thick layer of soil with a minimum organic content of 5 percent AND a minimum cation exchange capacity (CEC) of 5 milliequivalents/ 100 grams can be used as a treatment layer beneath a water quality or detention facility.
- To demonstrate that in-place soils meet the above criteria, one sample per 1,000 square feet of facility area shall be tested. Each sample shall be a composite of subsamples collected throughout the depth of the treatment layer (usually 2 to 6 feet below the expected facility invert).
- Typically, side wall seepage is not a concern if the seepage flows through the same stratum as the bottom of the treatment BMP. However, if the treatment soil is an engineered soil or has very low permeability, the potential to bypass the treatment soil through the side walls may be significant. In those cases, the treatment BMP side walls should be lined with at least 18 inches of treatment soil, as described above, to prevent untreated seepage. This lesser soil thickness is based on unsaturated flow as a result of alternating wet-dry periods.
- Organic content shall be measured on a dry weight basis using ASTM D2974.
- Cation exchange capacity (CEC) shall be tested using EPA laboratory method 9081.
- Certification by a soils testing laboratory that imported soil meets the organic content and CEC criteria above shall be provided to the City.
- Animal manures used in treatment soil layers must be sterilized because of potential for bacterial contamination of the groundwater.
- If a treatment liner will be below the seasonal high water level, the pollutant removal performance of the liner and facility must be evaluated by a geotechnical or groundwater specialist and found to be as protective as if the liner and facility were above the level of the groundwater.

5.4.4.2 Design Criteria for Low Permeability Liners

This section presents the design criteria for each of the following four low permeability liner options: compacted till liners, clay liners, geomembrane liners, and concrete liners. For low permeability liners, the following criteria apply:

- Where the seasonal high groundwater elevation is likely to contact a low permeability liner, liner buoyancy may be a concern. In these instances, use of a low permeability liner shall be evaluated and recommended by a geotechnical engineer.
- Where grass must be planted over a low permeability liner per the facility design, a minimum of 6 inches of good topsoil or compost-amended native soil (2 inches compost tilled into 6 inches of native till soil) must be placed over the liner in the area to be planted. Twelve inches of cover is preferred.

Compacted Till Liners

- Liner thickness shall be 18 inches after compaction.
- Soil shall be compacted to 95 percent minimum dry density, modified proctor method (ASTM D-1557).
- A different depth and density sufficient to retard the infiltration rate to 2.4×10^{-5} inches per minute (1×10^{-6} cm/s) may also be used instead of Criteria 1 and 2 above.
- Soil shall be placed in 6-inch lifts.
- Soils must meet the gradation outlined in Table 5.5.

Table 5.5. Compacted Till Liners.

Sieve Size	Percent Passing
6-inch	100
4-inch	90
#4	70 – 100
#200	20

Clay Liners

- Liner thickness shall be 12 inches.
- Clay shall be compacted to 95 percent minimum dry density, modified proctor method (ASTM D-1557).
- A different depth and density sufficient to retard the infiltration rate to 2.4 x 10-5 inches per minute (1 x 10-6 cm/s) may also be used instead of the above criteria, if approved by the Director.

- Plasticity index shall not be less than 15 percent (ASTM D-423, D-424).
- Liquid limit of clay shall not be less than 30 percent (ASTM D-2216).
- Clay particles passing shall not be less than 30 percent (ASTM D-422).
- The slope of clay liners must be restricted to 3H: 1V for all areas requiring soil cover; otherwise, the soil layer must be stabilized by another method so that soil slippage into the facility does not occur. Any alternative soil stabilization method must take maintenance access into consideration.
- Where clay liners form the sides of ponds, the interior side slope shall not be steeper than 3H: 1V, irrespective of fencing. This restriction is to ensure that anyone falling into the pond may safely climb out.

Geomembrane Liners

- Geomembrane liners shall be ultraviolet (UV) light resistant and have a minimum thickness of 30 mils. A thickness of 40 mils shall be used in areas of maintenance access or where heavy machinery must be operated over the membrane.
- The geomembrane fabric shall be protected from puncture, tearing, and abrasion by installing geotextile fabric on the top and bottom of the geomembrane determined to have a high survivability per Appendix F, and the WSDOT Standard Specifications as Amended, specifically Section 9-33 Construction Geotextile (2006 or the latest version as amended). Equivalent methods for protecting the geomembrane liner may be permitted, subject to approval by Director. Equivalency will be judged on the basis of ability to protect the geomembrane from puncture, tearing, and abrasion.
- Geomembranes shall be bedded according to the manufacturer's recommendations.
- Liners must be covered with 12 inches of top dressing forming the bottom and sides of the water quality facility, except for linear sand filters. Top dressing shall consist of 6 inches of crushed rock covered with 6 inches of native soil. The rock layer is to mark the location of the liner for future maintenance operations. As an alternative to crushed rock, 12 inches of native soil may be used if orange plastic “safety fencing” or another highly-visible, continuous marker is embedded 6 inches above the membrane.
- If possible, liners should be of a contrasting color so that maintenance workers are aware of any areas where a liner may have become exposed when maintaining the facility.
- Geomembrane liners shall not be used on slopes steeper than 5H:1V to prevent the top dressing material from slipping. Textured liners may be used on slopes up to 3H:1V upon recommendation by a geotechnical

engineer that the top dressing will be stable for all site conditions, including maintenance.

Concrete Liners

- Concrete liners may also be used for sedimentation chambers and for sedimentation and filtration basins less than 1,000 square feet in area. Concrete shall be 5-inch-thick Class 3000 or better and shall be reinforced by steel wire mesh. The steel wire mesh shall be 6 gage wire or larger and 6 inch by 6 inch mesh or smaller. An "Ordinary Surface Finish" is required. When the underlying soil is clay or has an unconfined compressive strength of 0.25 ton per square foot or less, the concrete shall have a minimum 6 inch compacted aggregate base consisting of coarse sand and river stone, crushed stone or equivalent with diameter of 0.75 to 1 inch. Where visible, the concrete shall be inspected annually and all cracks shall be sealed.
- Portland cement liners are allowed irrespective of facility size, and shotcrete may be used on slopes. However, specifications must be developed by a professional engineer who certifies the liner against cracking or losing water retention ability under expected conditions of operation, including facility maintenance operations. Weight of maintenance equipment can be up to 80,000 pounds when fully loaded.
- Asphalt concrete may not be used for liners due to its permeability to many organic pollutants.
- If grass is to be grown over a concrete liner, slopes must be no steeper than 5H:1V to prevent the top dressing material from slipping. Textured liners may be used on slopes up to 3H:1V upon recommendation by a geotechnical engineer that the top dressing will be stable for all site conditions, including maintenance.

5.4.5 Hydraulic Structures

The following sections provide design information on flow splitters and flow spreaders. Both types of facilities are commonly used as part of water quality facility designs. Flow splitters are used to direct specific rates and volumes of runoff to (or away from) a water quality facility, while flow spreaders are typically used to spread flows across the inflow portion of certain water quality facilities to help maximize and maintain the treatment facility effectiveness.

5.4.5.1 Flow Splitter Designs

Many water quality facilities can be designed as flow-through or on-line systems with flows above the water quality design flow simply passing through the facility at a lower pollutant removal efficiency. However, it is sometimes desirable to restrict flows to water quality treatment facilities and bypass the remaining higher flows around them (i.e., off-line facilities). This can be accomplished by splitting

flows in excess of the water quality design flow upstream of the facility and diverting higher flows to a bypass pipe or channel. The bypass typically enters a detention pond or the downstream receiving drainage system, depending on applicable flow control requirements. In most cases, it is a designer's choice whether water quality facilities are designed as on-line or off-line; an exception is oil/water separators and sand filters, which must be designed off-line.

A crucial factor in designing flow splitters is to ensure that low flows are delivered to the treatment facility up to the water quality design flow rate. Above this rate, additional flows are diverted to the bypass system with minimal increase in head at the flow splitter structure to avoid surcharging the water quality facility under high flow conditions.

Flow splitters are typically manholes or vaults with concrete baffles. In place of baffles, the splitter mechanism may be a half tee section with a solid top and an orifice in the bottom of the tee section. A full tee option may also be used as described below in the "General Design Criteria." Two possible design options for manhole-based flow splitters are shown in Figure 5.2 and Figure 5.3. Figure 5.4 shows another option for an isolation/diversion structure. Other equivalent designs that achieve the result of splitting low flows and diverting higher flows around the facility may also be acceptable.

5.4.5.2 General Design Criteria

- A flow splitter must be designed to deliver the water quality design flow rate specified in this volume to the water quality treatment facility. For the basic size sand filter, which is sized based on volume, use the water quality design flow rate to design the splitter. For the large sand filter, use the flow rate that corresponds with treating 95 percent of the runoff volume predicted by an approved continuous runoff model.
- The top of the weir must be located at the water surface for the design flow. Remaining flows enter the bypass line. Flows modeled using a continuous simulation model must be at a 15-minute time step or less.
- The maximum head must be minimized for flow in excess of the water quality design flow. Specifically, flow to the water quality facility at the 100-year water surface must not increase the design water quality flow by more than 10 percent.
- As an alternative to using a solid top plate in Figure 5.3, a full tee section may be used with the top of the tee at the 100-year water surface. This alternative would route emergency overflows (if the overflow pipe were plugged) through the water quality facility rather than generate back up from the manhole.
- Special applications, such as roads, may require the use of a modified flow splitter. The baffle wall may be fitted with a notch and adjustable weir plate to proportion runoff volumes other than high flows.

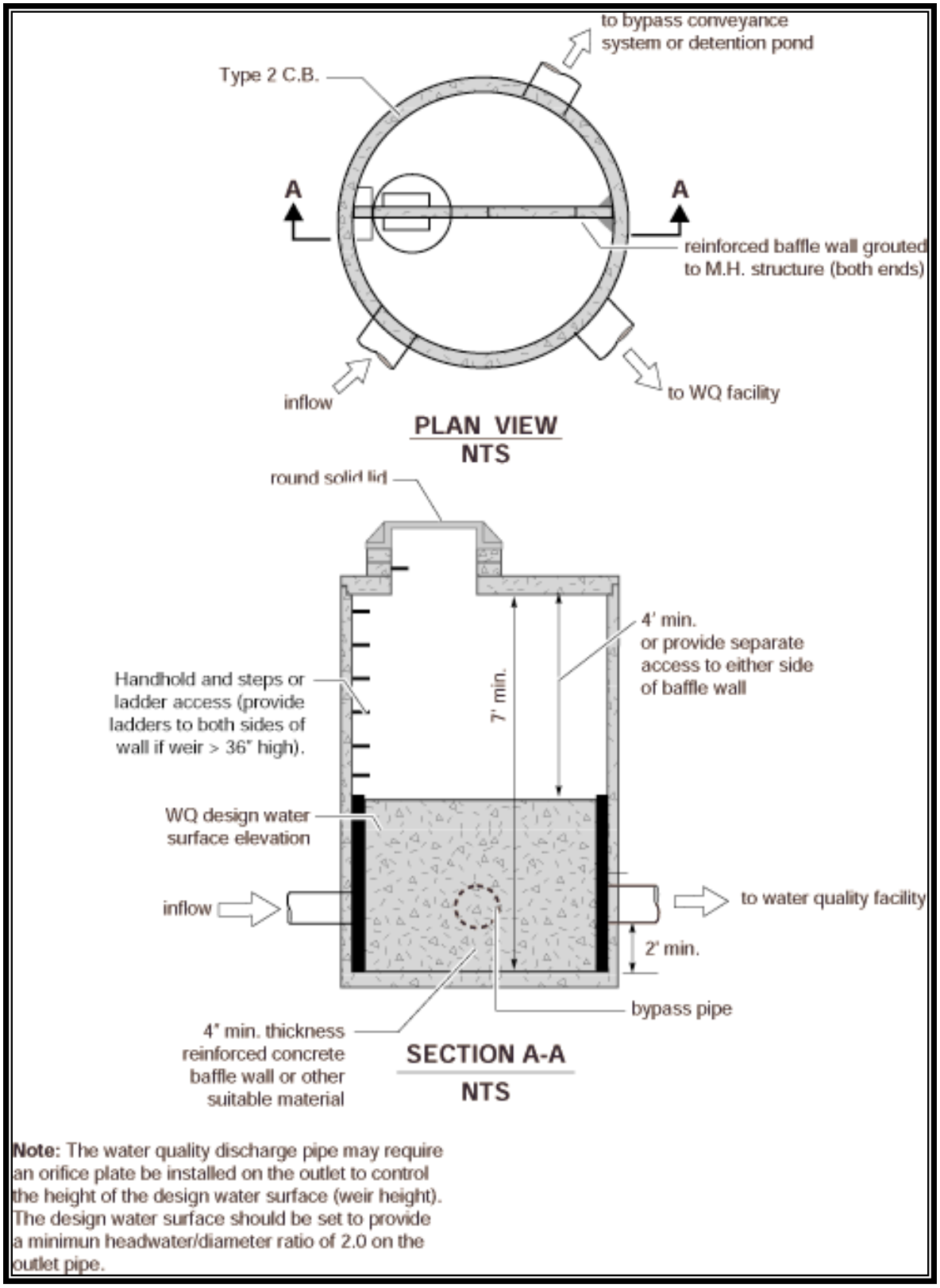


Figure 5.2. Flow Splitter, Option A.

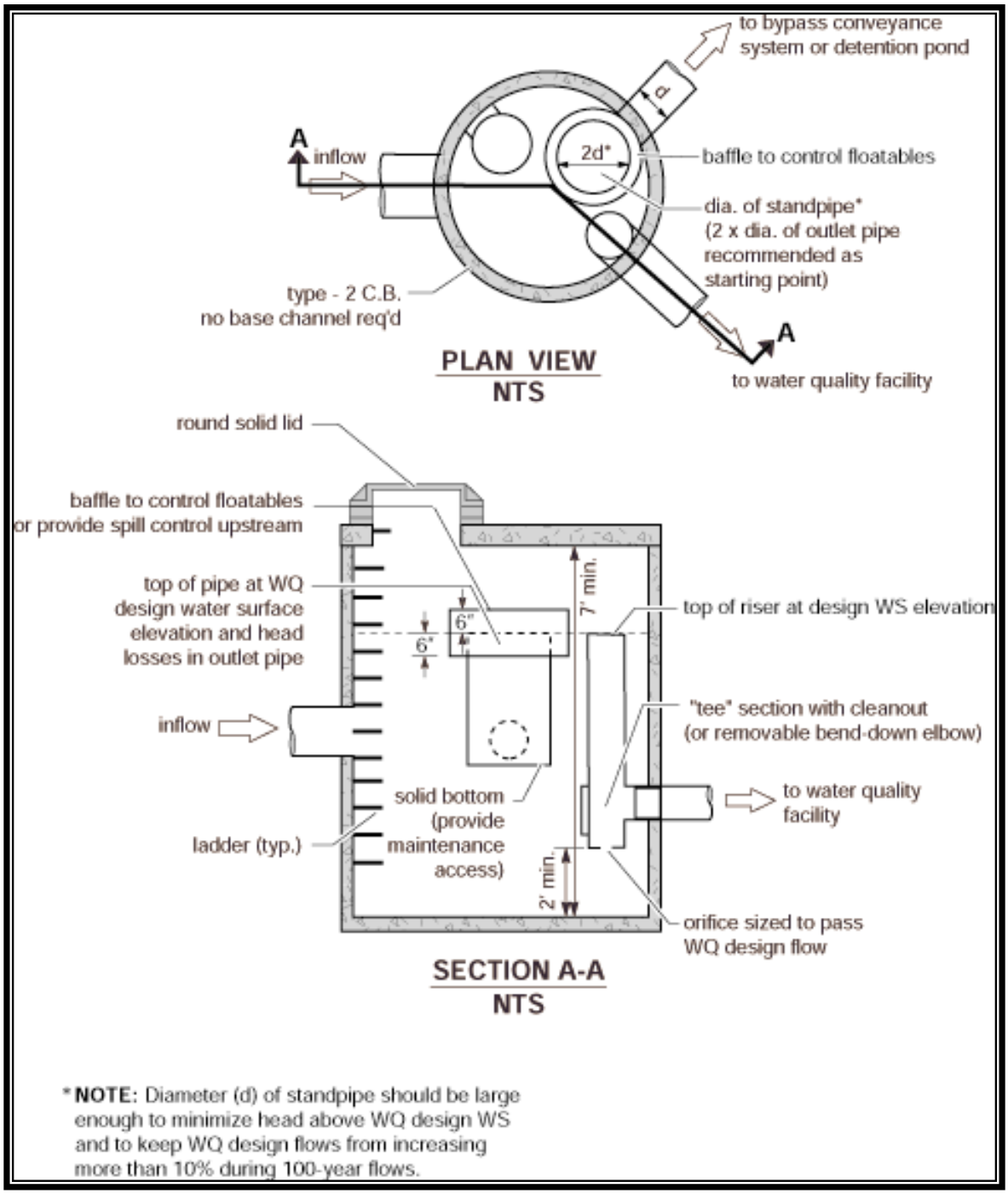
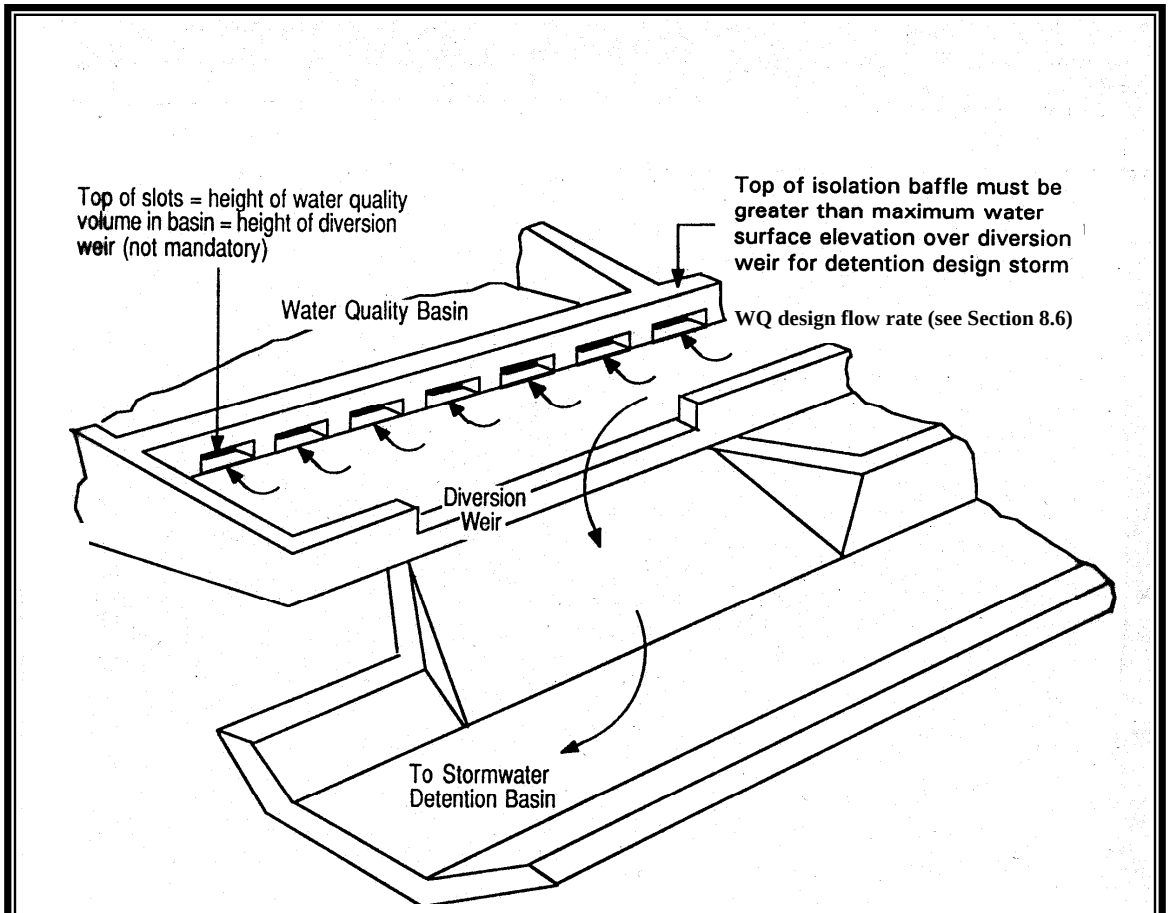


Figure 5.3. Flow Splitter, Option B.



Source: City of Austin

Figure 5.4. Example Isolation/Diversion Structure.

- For ponding facilities, backwater effects must be included in designing the height of the standpipe in the manhole.
- Ladder or step and handhold access must be provided. If the baffle wall is higher than 36 inches, two ladders, one to either side of the wall, must be used. The minimum clearance between the top of the baffle wall and the bottom of the manhole cover must be 4 feet; otherwise, dual access points should be provided.

Materials

- The splitter baffle may be installed in a Type 2 manhole or vault.
- The baffle wall must be made of reinforced concrete or another suitable material resistant to corrosion, and have a minimum 4-inch thickness.
- All metal parts must be corrosion resistant. Examples of preferred materials include aluminum, stainless steel, and plastic. Zinc and galvanized materials are prohibited because of aquatic toxicity. Painted metal parts should not be used because of poor longevity.

5.4.5.3 Flow Spreading Options

Flow spreaders uniformly spread flows across the inflow portion of water quality facilities (e.g., sand filter, biofiltration swale, or filter strip). There are five flow spreader options presented in this section:

- Option A – Anchored plate
- Option B – Concrete sump box
- Option C – Notched curb spreader
- Option D – Through-curb ports
- Option E – Interrupted curb.

Options A through C can be used for spreading flows that are concentrated. Any one of these options can be used when spreading is required by the facility design criteria. Options A through C can also be used for unconcentrated flows, and in some cases must be used, such as to correct for moderate grade changes along a filter strip.

Options D and E are only for flows that are already unconcentrated and enter a filter strip or continuous inflow biofiltration swale. Other flow spreader options are possible with prior approval by the Director.

General Design Criteria

- Where flow enters the flow spreader through a pipe, it is recommended that the pipe be submerged to the extent practical to dissipate energy as much as possible.

- For higher inflows (greater than 5 cfs for the 100-year storm), a Type 1 catch basin should be positioned in the spreader and the inflow pipe should enter the catch basin with flows exiting through the top grate. The top of the grate should be lower than the level spreader plate, or if a notched spreader is used, lower than the bottom of the v-notches.

Option A – Anchored Plate (Figure 5.5)

- An anchored plate flow spreader must be preceded by a sump having a minimum depth of 8 inches and minimum width of 24 inches. If not otherwise stabilized, the sump area must be lined to reduce erosion and to provide energy dissipation.

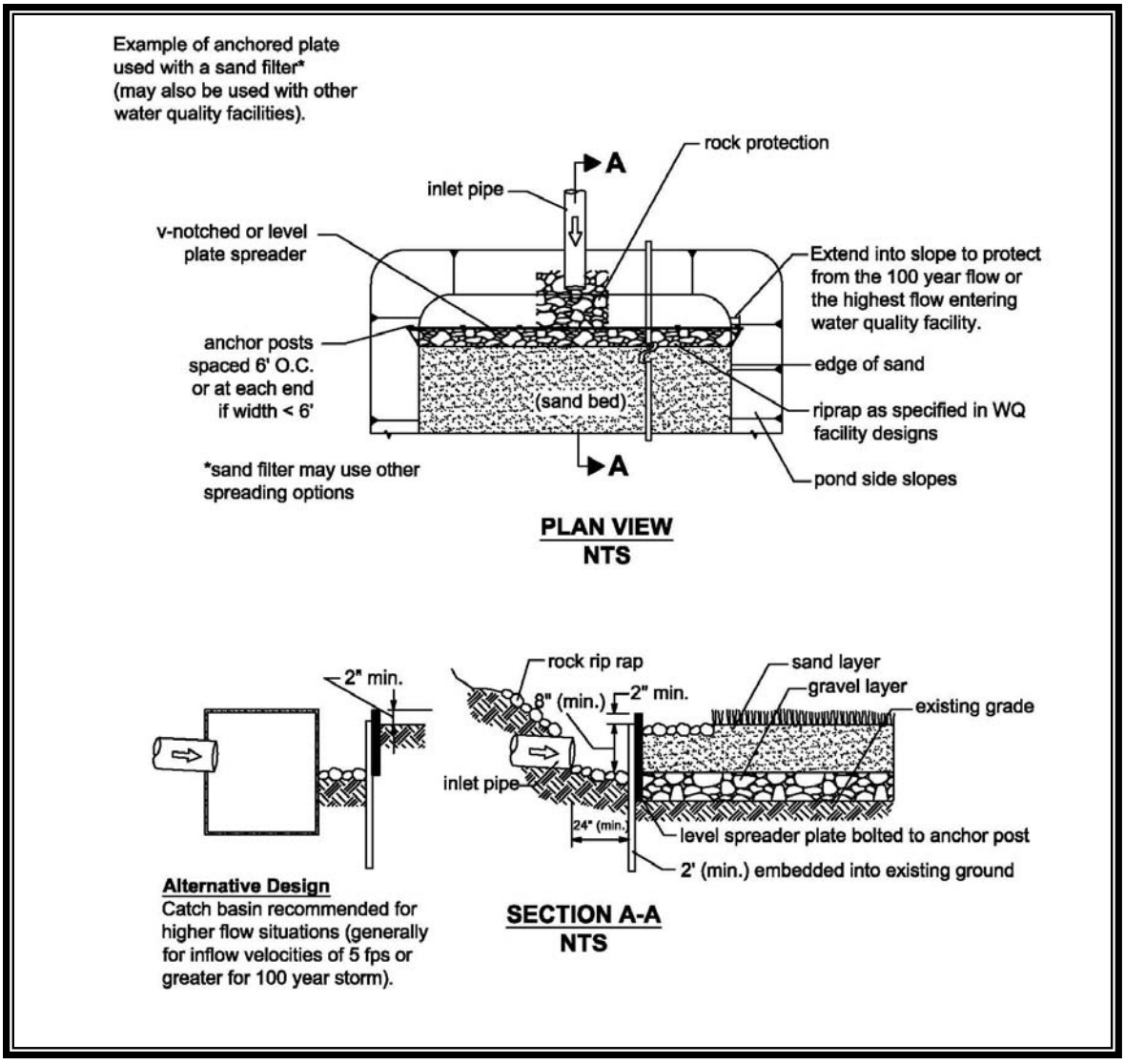


Figure 5.5. Flow Spreader Option A: Anchored Plate.

- The top surface of the flow spreader plate must be level, projecting a minimum of 2 inches above the ground surface of the water quality facility, or v-notched with notches 6 to 10 inches on center and 1 to 6 inches deep (use shallower notches with closer spacing). Alternative designs may also be considered.
- A flow spreader plate must extend horizontally beyond the bottom width of the facility to prevent water from eroding the side slope. The horizontal extent should be such that the bank is protected for all flows up to the 100-year flow, or the maximum flow that will enter the water quality facility.
- Flow spreader plates must be securely fixed in place
- Flow spreader plates may be made of either wood, metal, fiberglass reinforced plastic, or other durable material. If wood, pressure treated 4- by 10-inch lumber or landscape timbers are acceptable.
- Anchor posts must be 4-inch square concrete, tubular stainless steel, or other material resistant to decay.

Option B – Concrete Sump Box (Figure 5.6)

- The wall of the downstream side of a rectangular concrete sump box must extend a minimum of 2 inches above the treatment bed. This serves as a weir to spread the flows uniformly across the bed.
- The downstream wall of a sump box must have “wing walls” at both ends. Side walls and returns must be slightly higher than the weir so that erosion of the side slope is minimized.
- Concrete for a sump box can be either cast-in-place or precast, but the bottom of the sump must be reinforced with wire mesh for cast-in-place sumps.
- Sump boxes must be placed over bases that consists of 4 inches of crushed rock, 5/8-inch minus to help assure the sump remains level.

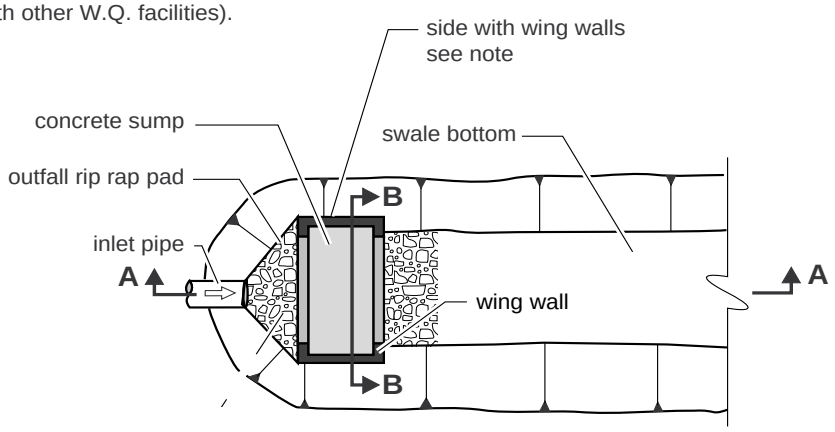
Option C – Notched Curb Spreader (Figure 5.7)

Notched curb spreader sections must be made of extruded concrete laid side-by-side and level. Typically, five “teeth” per 4-foot section provide good spacing. The space between adjacent teeth forms a v-notch.

Option D – Through-Curb Ports (Figure 5.8)

Unconcentrated flows from paved areas entering filter strips or continuous inflow biofiltration swales can use curb ports or interrupted curbs (Option E) to allow flows to enter the strip or swale. Curb ports use fabricated openings that allow concrete curbing to be poured or extruded while still providing an opening through the curb to admit water to the water quality facility.

Example of a concrete sump flow spreader used with a biofiltration swale (may be used with other W.Q. facilities).



PLAN VIEW
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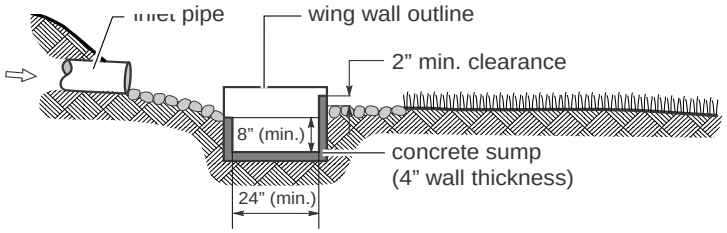
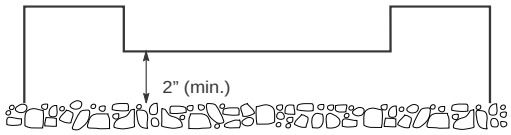


Note: Extend sides into slope. Height of side wall and wing walls must be sufficient to handle the 100 year flow or the highest flow entering the facility.

SECTION A-A
NTS



SECTION B-B
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Figure 5.6. Flow Spreader Option B: Concrete Sump Box.

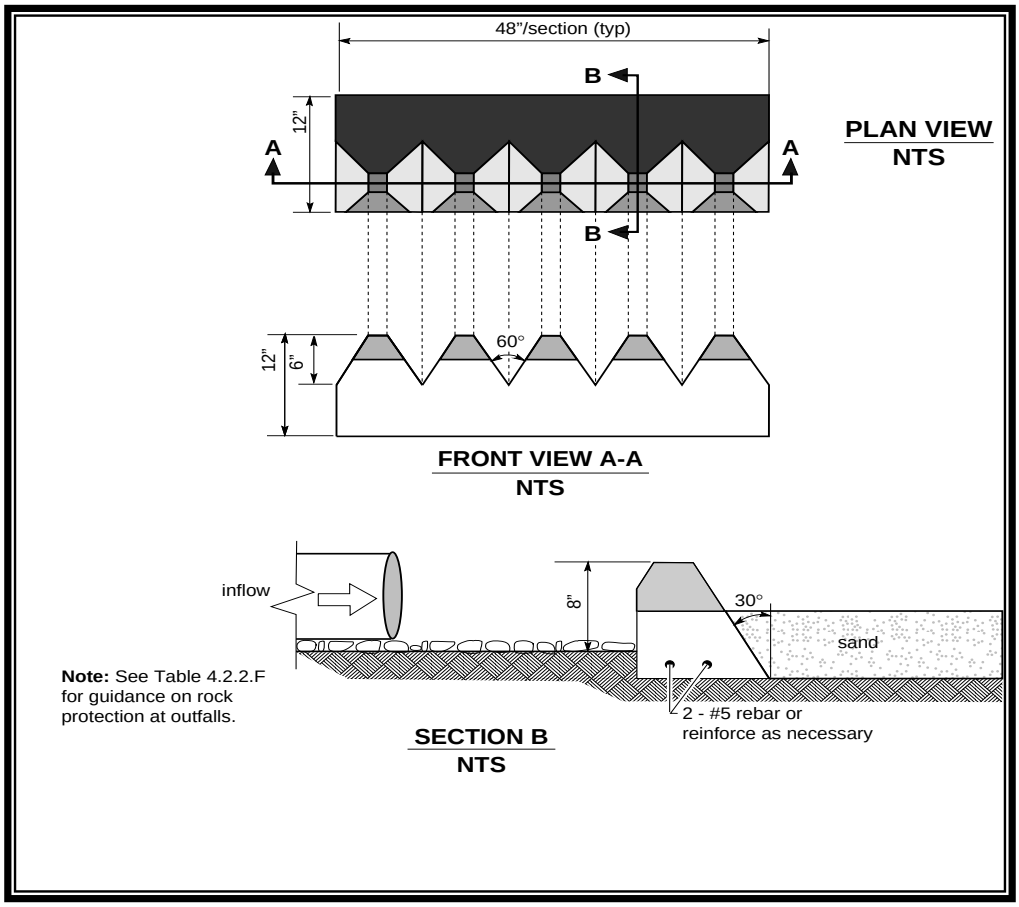


Figure 5.7. Flow Spreader Option C: Notched Curb Spreader.

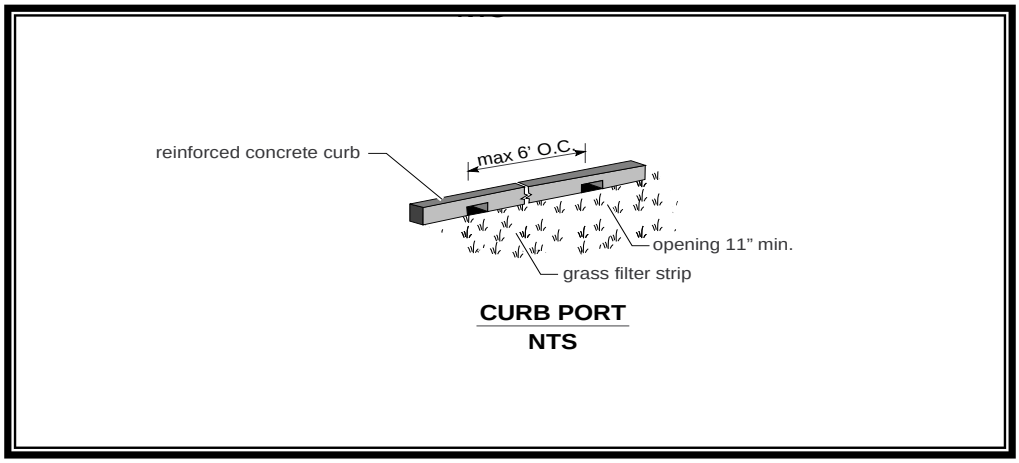


Figure 5.8. Flow Spreader Option D: Through-Curb Port.

Openings in the curb must be at regular intervals but at least every 6 feet (minimum). The width of each curb port opening must be a minimum of 11 inches. Approximately 15 percent or more of the curb section length should be in open ports, and no port should discharge more than about 10 percent of the flow.

Option E – Interrupted Curb (No Figure)

Interrupted curbs are sections of curb placed to have gaps spaced at regular intervals along the total width (or length, depending on facility) of the treatment area. At a minimum, gaps must be every 6 feet to allow distribution of flows into the treatment facility before they become too concentrated. The opening must be a minimum of 11 inches. As a general rule, no opening should discharge more than 10 percent of the overall flow entering the facility.

5.5 Pretreatment Facilities

Pretreatment is essential to effective long-term BMP performance. Where the primary treatment or flow control mechanism of a treatment BMP is biofiltration, infiltration, or settling; excessive sediment can reduce the effectiveness over time by reducing stormwater contact with vegetation or clogging sands and other filtration media. Pretreatment protects BMPs from excessive siltation and debris by reducing the total suspended solids (TSS) concentrations in influent stormwater flows.

Pretreatment must be provided in the following applications:

- For sand and media filtration and infiltration BMPs to protect them from excessive siltation and debris
- Where the basic treatment facility or the receiving water may be adversely affected by non-targeted pollutants (e.g., oil), or may by overwhelmed by a heavy load of targeted pollutants (e.g., suspended solids).

Where water quality treatment is required, runoff treated by a pretreatment facility may not be discharged directly to a receiving water; it must be further treated by a basic or enhanced water quality treatment BMP.

5.5.1 Applications and Limitations

Pretreatment facilities generally fall into one of two categories: 1) presettling basins, and 2) hydrodynamic separators.

The Department of Ecology is responsible for evaluating and approving new water quality treatment BMPs, including new pretreatment BMP designs. After these designs are formally approved for use as pretreatment facilities by Ecology, the City of Seattle will determine whether to allow these facilities to be used in the City. Those facilities approved by Ecology and the Director for use as pretreatment facilities are identified below. Refer to the Department of

Ecology's web site for design requirements and related information for newly approved BMPs (http://www.ecy.wa.gov/).

Presettling basins are a typical pretreatment BMP used to remove suspended solids. A presettling basin is a vault or pond that is located upstream of a flow control or water quality treatment facility and is intended to collect sediment that could otherwise clog or impair the function of the primary BMP. For a constructed wetland or other pond or basin facility, the presettling basin will typically consist of a presettling cell.

Several proprietary media filtration systems and hydrodynamic separators have been approved by Ecology for various levels of treatment. Those facilities approved by Ecology and the Director for use as pretreatment facilities are listed below.

All of the basic water quality treatment facilities may also be used for pretreatment to reduce suspended solids. A detention pond sized to meet the flow control standard may also be used to provide pretreatment for suspended solids removal. Catch basin inserts may not be used for oil or TSS control.

5.5.2 Operations and Maintenance Requirements

Pretreatment facility operations and maintenance requirements are provided in Appendix D.

5.5.3 Presettling Basins

Presettling basins shall be designed with a wetpool (i.e., permanent storage). The treatment volume shall be at least 30 percent of the total volume of runoff from the water quality design storm event (see Chapter 6 for guidance on hydrologic modeling to determine water quality design flows and volumes).

Presettling Basins shall conform to the following:

- The length-to-width ratio shall be at least 3:1. Berms or baffles may be used to lengthen the flowpath.
- The minimum water storage depth shall be 4 feet; the maximum storage depth shall be 6 feet.
- One foot minimum sediment storage depth shall be provided
- One foot minimum freeboard (above the design water surface elevation) shall be provided.

Inlets and outlets shall be designed to minimize velocity and reduce turbulence. Inlet and outlet structures must be located at opposite ends of the presettling basin to prevent short-circuiting and maximize particle settling.

If the runoff in the presettling basin will be in direct contact with the soil, it must be lined, in accordance with the provisions in Section 5.4.4.

Refer to the Department of Ecology's web site for design requirements and related information for newly approved BMPs (http://www.ecy.wa.gov/programs/WQ/stormwater/newtech/technologies.html).

5.5.4 Hydrodynamic Separators

Hydrodynamic separators are flow-through structures with a settling or separation unit to remove sediments and particle-bound pollutants. The facility name refers to the application of the energy of the flowing water to facilitate sediment separation and removal. Depending on the type of unit, particle settling may occur by means of swirl action or indirect filtration.

The following hydrodynamic separator systems have been approved by the Director for pretreatment applications only. Refer to the Department of Ecology's web site for design requirements and related information for newly approved BMPs (http://www.ecy.wa.gov/programs/WQ/stormwater/newtech/technologies.html).

- CONTECH Stormwater Solutions Inc. Vortechs® System.

5.6 Biofiltration Swales

A biofiltration swale (Figures 5.9 and 5.10) is an open, gently sloped, vegetated channel designed to treat stormwater. Pollutant removal occurs by filtration as stormwater moves through the grass blades, which enhances sedimentation, as well as trapping and adhesion of pollutants to the grass and thatch. Biofiltration swales are designed so that stormwater will flow evenly across the entire width of a densely vegetated channel. A swale can be designed for both treatment and conveyance of stormwater flow. This combined use can reduce development costs by eliminating the need for separate conveyance and treatment systems. Biofiltration swales are typically configured as flow-through systems, with little or no detention or storage. All biofiltration facilities meet basic water quality treatment requirements. When installed upstream of a sand filter (basin or vault) in a treatment train, biofiltration swales can also help meet phosphorus and enhanced treatment requirements. Similarly, when installed upstream of a media filter capable of removing dissolved metals in a treatment train, biofiltration swales can help meet enhanced treatment requirements (see treatment menus in Section 5.3).

The three biofiltration BMPs described in this section are:

1. Basic biofiltration swale: standard swale, with all runoff entering at the head of the swale.
2. Wet biofiltration swale: similar to basic swale, but due to site conditions and/or influent conditions, this swale is designed to accommodate saturated soil conditions
3. Continuous inflow biofiltration swale: same as basic swale, but runoff enters at multiple locations along the length of the swale.

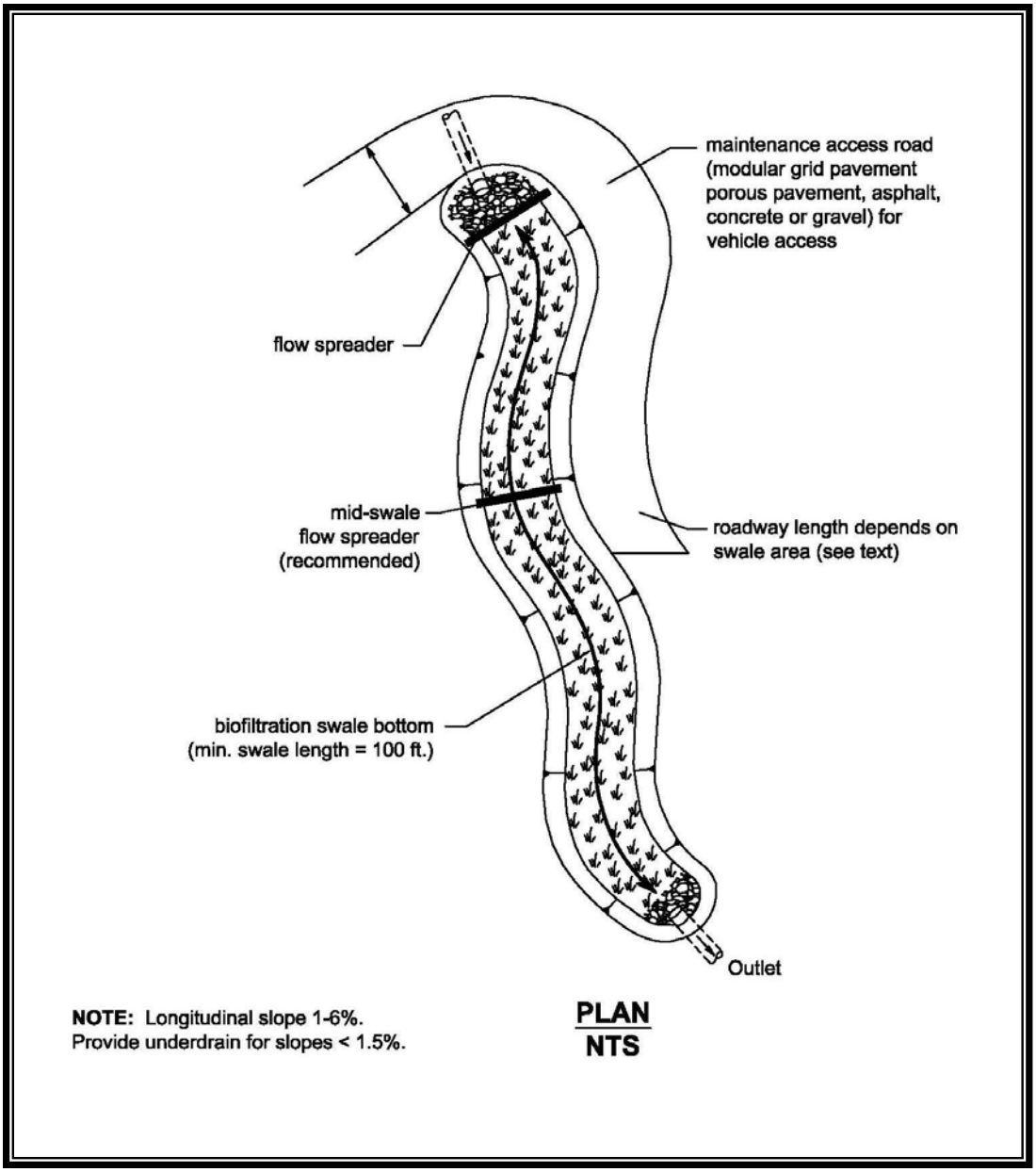


Figure 5.9. Typical Biofiltration Swale Layout.

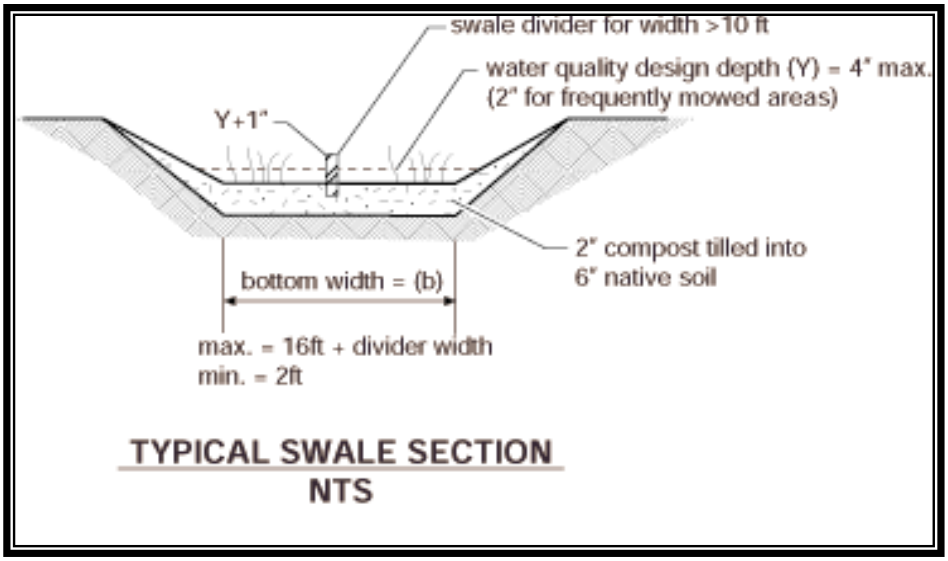


Figure 5.10. Typical Swale Section.

All biofiltration facilities must provide an outlet and overflow to an approved discharge point. Approved discharge points shall, at a minimum, maintain natural drainage patterns to the maximum extent practicable, and not cause a significant adverse impact to the downstream conveyance system, receiving surface water, or down-gradient properties.

5.6.1 Applications and Limitations

Biofiltration swales are best applied on relatively small sites (generally less than 5 acres of contributing area). They work well along roadways, driveways, and parking lots, and can be easily incorporated into a project's landscape design. Swales are generally not recommended in situations where the channel would be deep, because shading can inhibit the grass growth, resulting in poor pollutant removal performance. See Section 5.4 for general requirements that may be applicable to bioretention swales. Specific considerations for biofiltration swale applications are listed below:

- A biofiltration swale should not be located in a shaded area. For healthy grass growth, a swale should receive a minimum of 6 hours of sunlight daily during the summer months throughout the length of the swale.
- To maintain healthy grass growth, a swale must dry out between storms. It should not receive continuous base flows (such as seepage from a hill slope throughout the winter) or be located in a high groundwater area, because saturated soil conditions will kill grass. If these conditions are likely to occur, adjust the design appropriately or use a wet biofiltration swale (see Section 5.6.4).
- Stormwater runoff containing high concentrations of oil and grease impairs the treatment capability of a swale. Oil control options described in Section 5.11 should be applied in these situations.

- Most biofiltration swales are designed to be on-line facilities. However, an off-line design may be preferred in some cases to avoid scour and damage to vegetation during high flows. An additional benefit of designing swales to be off-line is that the stability check (see BMP Sizing and Design Criteria in the Basic Biofiltration Swale section below), which may make the swale larger, is not necessary.

5.6.2 Operations and Maintenance Requirements

Biofiltration BMP operations and maintenance requirements are provided in Appendix D.

5.6.3 Basic Biofiltration Swales

5.6.3.1 Site Considerations

Biofiltration swales require a minimum footprint of approximately 100 feet by 20 feet. The actual footprint will depend on the bottom width, side slopes, and length, which are all dependent on the design flows. The following design criteria and requirements influence whether swales are feasible for a particular site:

- 100-foot minimum length
- 2-foot minimum bottom width
- 10-foot-wide maintenance access road unless access is provided along the base of the biofiltration swale channel
- Maximum side slope 3H:1V (4H:1V preferred)
- 0.5 foot of freeboard.

5.6.3.2 BMP Sizing and Design Criteria

Biofiltration swales must be designed to meet the criteria listed in Table 5.6, with the following exceptions:

- Slopes may be less than 1.5 percent if an underdrain is installed (see Design Specifications section below). Alternately, a wet biofiltration swale can be considered for slopes less than 1.5 percent.
- For swale widths greater than 10 feet and up to 16 feet, the cross-section can be divided with a berm (concrete, plastic, compacted earthfill) using a flow spreader at the inlet (Figure 5.11).
- For swale slopes greater than 2.5 percent, the slope can be reduced by installing check dams (with a recommended12 to 15 inch vertical drop).

Table 5.6. Basic Biofiltration Swale Design and Sizing Criteria.

Design Parameter	Basic Biofiltration Swale
Longitudinal Slope	1.5% to 2.5%
Maximum velocity	1 ft/sec @ WQ design flow rate 3 ft/sec @ 100-yr discharge
Maximum water depth	2 inches if mowed frequently, 4 inches if mowed infrequently
Manning's roughness coefficient	0.2 – 0.3 (0.24 if mowed infrequently)
Bed width (bottom)	2 – 10 ft (up to 16 ft with divider berm)
Freeboard height	0.5 ft
Minimum hydraulic residence time at Water Quality Design Flow Rate	9 min
Minimum length	100 ft
Maximum sideslope	3H:1V 4H:1V preferred

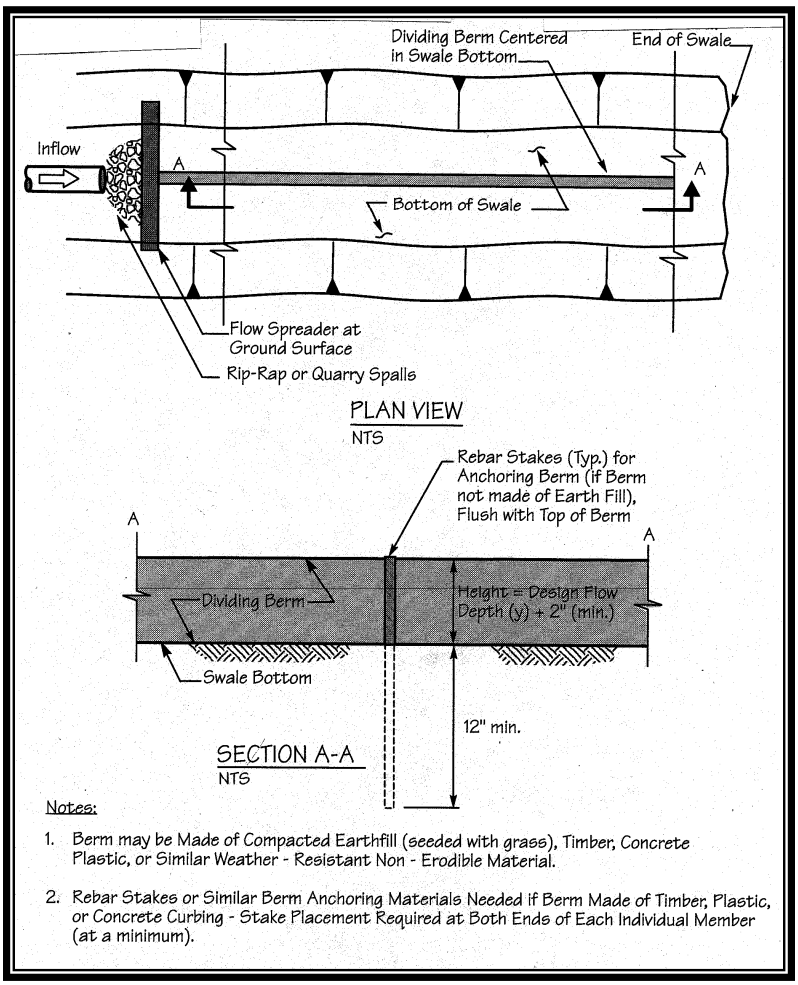


Figure 5.11. Typical Swale Dividing Berm.

The following section provides biofiltration swale design procedures in full detail, along with examples.

Preliminary Steps (P)

P-1. Determine the Water Quality design flow rate (Q) in 15-minute time steps using an approved continuous hydrologic model. Refer to Chapter 5 for approved models and requirements for performing hydrologic modeling.

- For off-line swales, the high flow bypass must be designed so that all flows up to and including the water quality design flow rate are directed to the swale. The water quality design flow rate is calculated by multiplying the design flow determined by an approved continuous runoff model by the off-line ratio specified by the Department of Ecology (use a value of 3.0 for Seattle). This modified design flow rate is an estimate of the design flow rate determined by using SBUH procedures. The intent is to maintain recent biofiltration sizing recommendations (9 minutes detention at the peak design flow rate estimated by SBUH for a 6-month, 24-hour storm with a Type 1A rainfall distribution) until more definitive information is collected concerning biofiltration swale performance.
- For online swales, the water quality design flow rate is determined by multiplying the design flow determined by an approved continuous runoff model by the online ratio (use 1.65 for Seattle).

P-2. Establish the longitudinal slope of the proposed biofiltration swale.

P-3. Select a vegetation cover suitable for the site. Refer to Tables 5.8, 5.9, and 5.10 to select vegetation for western Washington.

Design Calculations for Basic Biofiltration Swales

There are a number of ways to apply the design procedure introduced by Chow (Chow 1959). The variations depend on the order in which steps are performed, what constants are established at the beginning of the process and which ones are calculated, and what values are assigned to the variables selected.

The procedure recommended here is an adaptation appropriate for biofiltration applications of the type being installed in the Puget Sound region. This procedure reverses Chow's order, designing first for treatment capacity and then for stability. The treatment capacity analysis emphasizes the promotion of biofiltration, rather than transporting flow with the greatest possible hydraulic efficiency. Therefore, it is based on criteria that promote sedimentation, filtration, and other pollutant removal mechanisms. Because these criteria include a lower maximum velocity than permitted for stability, the swale dimensions usually do not have to be modified after a stability check.

Design Steps (D):

D-1. Select the type of vegetation, and design depth of flow (based on frequency of mowing and type of vegetation) (Table 5.6)

D-2. Select a value of Manning's n roughness coefficient (Table 5.6)

D-3. Select swale shape (typically trapezoidal)

D-4. Use Manning's equation for open channel flow to solve for appropriate swale dimensions as follows:

$$Q = \frac{1.49AR^{0.67}s^{0.5}}{n}$$

Where:

Q = modified water quality design flow rate from Step 1 (cfs)
n = Manning's roughness coefficient (dimensionless – see Table 5.6)
s = longitudinal slope as a ratio of vertical rise/horizontal run (dimensionless)
A = cross sectional area of the channel at the design flow (ft^2)
R = hydraulic radius (ft).

Because the depth of flow in most biofiltration swales is shallow relative to the bottom width, channel side slopes can be ignored in the calculation of bottom width. Use the following equation (a simplified form of Manning's formula) to estimate the swale bottom width:

$$b \approx \frac{Qn_{wq}}{1.49y^{1.67}s^{0.5}} - Zy$$

Where:

b = bottom width of swale (ft)
Q = modified water quality design flow from step P-1 (cfs)
n_{wq} = Manning's roughness coefficient for shallow flow conditions (unitless – see Table 5.6)
y = design flow depth (ft)
s = longitudinal slope (along direction of flow) (ft/ft).

Note: For a trapezoid, select a side slope Z of at least 3. Compute b and then top width T, where T = b + 2yZ.

If b for a swale is greater than 10 feet, either investigate how Q can be reduced, divide the flow by installing a low berm, or arbitrarily set b = 10 feet and continue with the analysis. For other swale shapes, refer to Figure 5.12.

D-5. Compute A:

$$A_{trapezoid} = by + Zy^2$$

D-6. Compute the flow velocity at the design flow rate:

$$V = Q/A$$

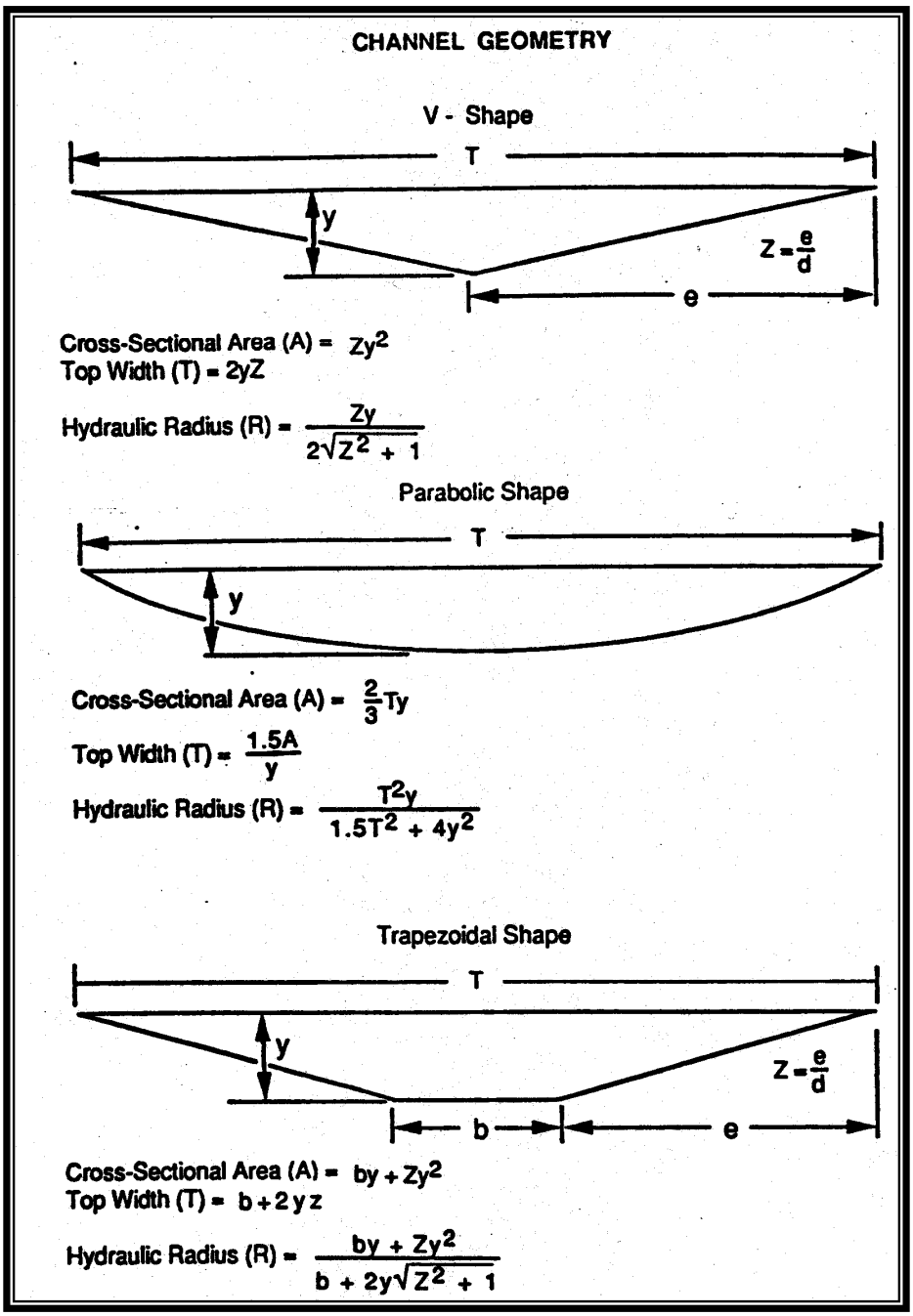


Figure 5.12. Geometric Formulas for Common Swale Shapes.

If V >1.0 ft/sec, repeat steps D-1 to D-6 until the condition is met. A velocity greater than 1.0 ft/sec has been found to flatten grasses, thus reducing filtration. A velocity lower than this maximum value will allow a 9-minute hydraulic residence time criterion in a shorter swale. If the value of V suggests that a longer swale will be needed than space permits, investigate how Q can be reduced (e.g., use of low impact development BMPs), or increase y and/or T (up to the allowable maximum values) and repeat the analysis.

D-7. Compute the swale length (L, ft)

L = Vt (60 sec/min)

Where: t = hydraulic residence time (min)

Use t = 9 minutes for this calculation (use t = 18 minutes for a continuous inflow biofiltration swale). If a swale length is greater than the space permits, follow the advice in step D-6.

If the analysis results in a length less than 100 feet, increase the swale length to 100 feet, the minimum allowed. In this case, it may be possible to reduce the swale width and still meet all criteria. This can be checked by computing V in the 100 ft swale for t = 9 minutes, recalculating A (if V < 1.0 ft/sec) and recalculating T.

D-8. If there is still not sufficient space for the swale, the project proponent should consider the following potential solutions (listed in order of preference):

1. Divide the site drainage to flow to multiple swales
2. Use infiltration to provide lower discharge rates to the swale (only if the soil treatment criteria in Section 5.8.4 are met)
3. Increase the vegetation height and design depth of flow (note: the design must ensure that vegetation remains standing during design flow)
4. Reduce the developed surface area to gain space for biofiltration
5. Increase the longitudinal slope
6. Increase the side slopes
7. Nest the swale within or around another BMP.

Check for Stability (Minimizing Erosion)

The stability check must be performed for the combination of highest expected flow and least vegetation coverage and height. A check is not required for biofiltration swales that are located off-line from the primary conveyance/detention system. Maintain the same units as in the biofiltration capacity analysis.

SC-1. Perform the stability check for the 100-year return frequency flow using a 15-minute time step in an approved continuous runoff model.

SC-2. Estimate the vegetation coverage ("good" or "fair") and height on the first occasion that the swale will receive flow, or whenever the coverage and height will be least. Avoid flow introduction during the vegetation establishment period by timing planting or bypassing.

SC-3. Estimate the degree of retardance from Table 5.7. When uncertain, be conservative by selecting a relatively low degree.

Table 5.7. Guide for Selecting Degree of Retardance (a).

Coverage	Average Grass Height (inches)	Degree of Retardance
Good	<2	E. Very Low
	2-6	D. Low
	6-10	C. Moderate
	11-24	B. High
	>30	A. Very High
Fair	<2	E. Very Low
	2-6	D. Low
	6-10	D. Low
	11-24	C. Moderate
	>30	B. High

See Chow (1959). In addition, Chow recommended selection of retardance C for a grass-legume mixture 6-8 inches high and D for a mixture 4-5 inches high. No retardance recommendations have appeared for emergent wetland species. Therefore, judgment must be used. Since these species generally grow less densely than grasses, using a "fair" coverage would be a reasonable approach.

The maximum permissible velocity for erosion prevention (V_{max}) is 3 feet per second.

Stability Check Steps (SC)

SC-4. Select a trial Manning's n roughness coefficient for the high flow condition. The minimum value for poor vegetation cover and low height (possibly, knocked from the vertical by high flow) is 0.033. A good initial choice under these conditions is 0.04.

SC-5. Refer to Figure 5.13 to obtain a first approximation for VR based on Manning's n roughness coefficient and the degree of retardance.

SC-6. Compute hydraulic radius, R, from VR in Figure 5.13 and a V_{max} of 3 feet/second (from SC-3).

$$R = \frac{VR_{appx}}{V_{max}}$$

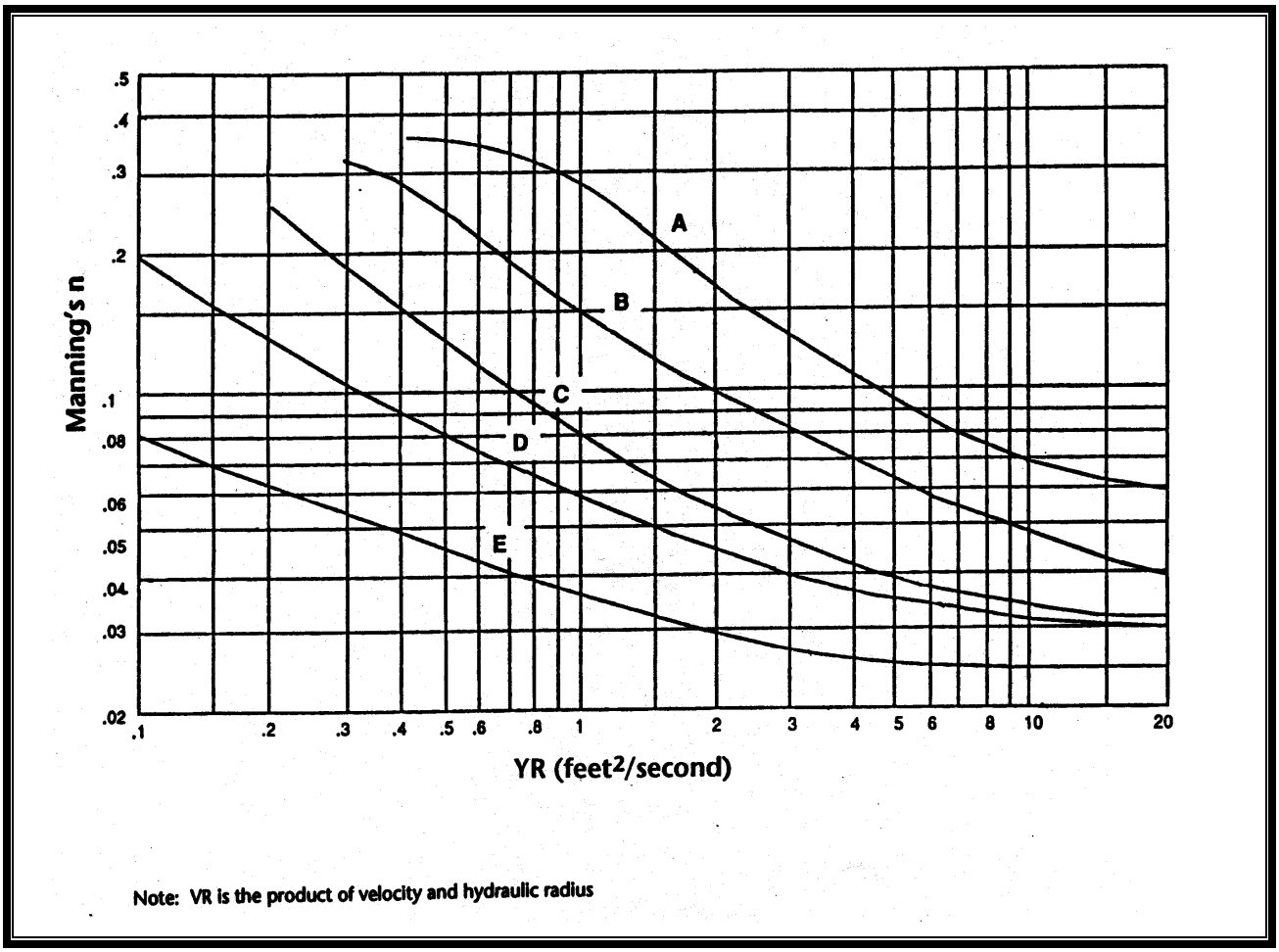


Figure 5.13. The Relationship of Manning's n with VR for Various Degrees of Flow Retardance (A-E).

SC-7. Use Manning's equation to solve for the actual VR.

SC-8. Compare the actual VR from step SC-7 and first approximation from step SC-5. If they do not agree within 5 percent, repeat steps SC-4 to SC-8 until acceptable agreement is reached. If n<0.033 is needed to get agreement, set n = 0.033, repeat step SC-7, and then proceed to step SC-9.

SC-9. Compute the actual V for the final design conditions:

Check to be sure $V < V_{max}$ of 3 feet/second.

SC-10. Compute the required swale cross-sectional area, A, for stability.

SC-11. Compare the A, computed in step SC-10 of the stability analysis, with the A from the biofiltration capacity analysis (step D-5).

If less area is required for stability than is provided for capacity, the capacity design is acceptable. If not, use A from step SC-10 of the stability analysis and recalculate channel dimensions.

SC-12. Calculate the depth of flow at the stability check design flow rate condition for the final dimensions and use A from step SC-10.

SC-13. Compare the depth from step SC-12 to the depth used in the biofiltration capacity design (Step D-1). Use the larger of the two and add 0.5 feet of freeboard to obtain the total depth (y_t) of the swale. Calculate the top width for the full depth using the appropriate equation.

SC-14. Recalculate the hydraulic radius: (use b from Step D-4 calculated previously for biofiltration capacity, or Step SC-11, as appropriate, and y_t = total depth from Step SC-13).

SC-15. Make a final check for capacity based on the stability check design storm (this check will ensure that capacity is adequate if the largest expected event coincides with the greatest retardance). Apply Manning's Equation (Step D-4), a Manning's n roughness coefficient selected in Step D-2, and the calculated channel dimensions including freeboard, to compute the flow capacity of the channel under these conditions. Use R from Step SC-14, above, and $A = b(y_t) + Z(y_t)^2$ using b from Step D-4, D-15, or SC-11 as appropriate.

If the flow capacity is less than the stability check design storm flow rate, increase the channel cross-sectional area as needed for this conveyance. Specify the new channel dimensions.

Completion Step (CO)

CO. Review all of the criteria and guidelines for swale planning, design, installation, and operation above and specify all of the appropriate features for the application.

Level Spreaders

Install level spreaders (min. 1-inch gravel) at the head and every 50 feet in swales of ≥4 feet width. Include sediment cleanouts (weir, settling basin, or equivalent) at the head of the biofiltration swale as needed. Level spreaders are also required at the toe of vertical drops (check dams). Design guidelines and example design figures for level spreaders are provided in Section 5.4.5.

Underdrains

If the slope is less than 1.5 percent install an underdrain using a slotted pipe, or equivalent. Amend the soil if necessary to allow effective percolation of water to the underdrain. Install the low-flow drain 6 inches deep in the soil. Slopes greater than 2.5 percent need check dams (riprap) at vertical drops of 12 to 15 inches. Underdrains shall be 6-inch minimum Schedule 40 PVC perforated pipe with 6 inches of clean drain rock (5/8-inch minus) above the pipe (see Figures 5.14 and 5.15). The gravel and pipe must be enclosed by geotextile (i.e., filter) fabric. Refer to Appendix F for required geotextile properties. The underdrain must infiltrate into the subsurface or drain freely to an acceptable discharge point.

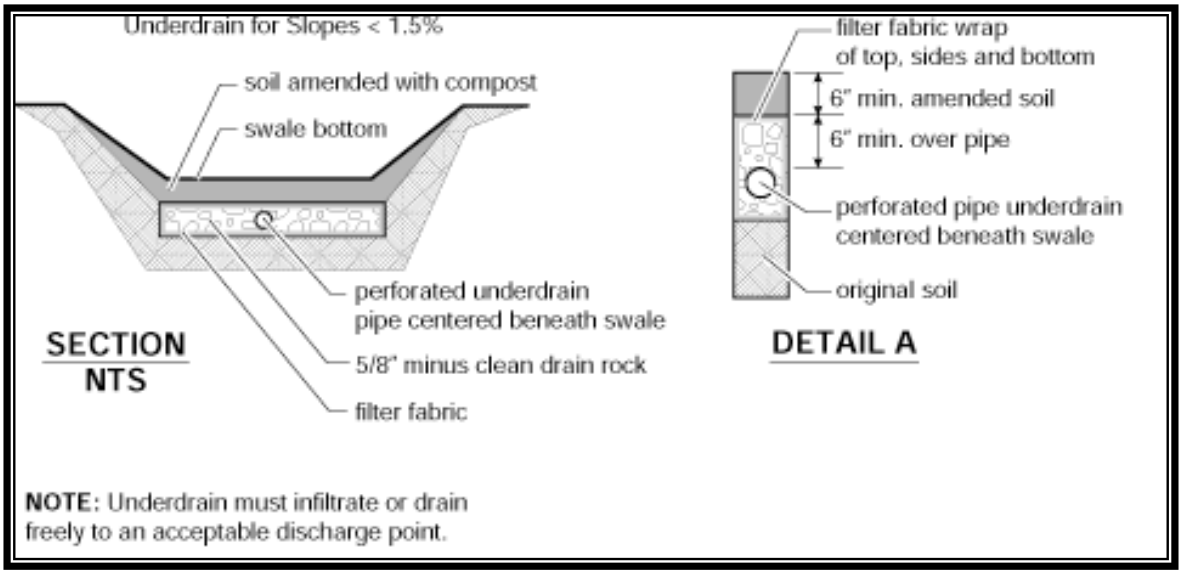


Figure 5.14. Typical Biofiltration Swale Underdrain Detail.

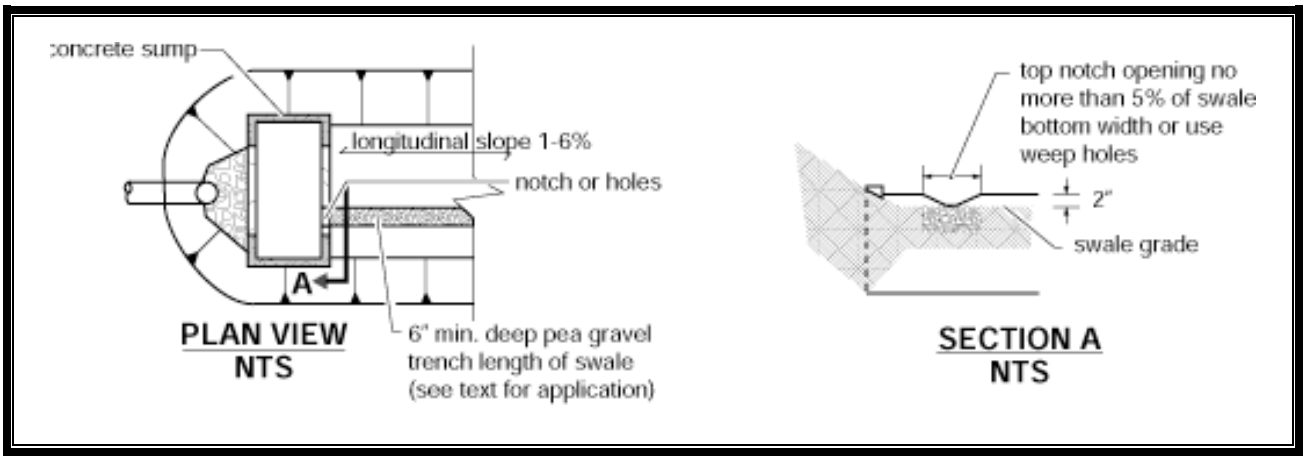


Figure 5.15. Typical Biofiltration Swale Low-Flow Drain Detail.

Low Flow Drains

If a swale will receive base flows because of seeps and springs on site, then either a low-flow drain shall be provided or a wet biofiltration swale shall be used. Low-flow drains are narrow surface drains filled with pea gravel that run lengthwise through the swale to bleed off base flows; they should not be confused with underdrains. In general, base flows less than 0.01 cfs per acre can be handled with a low-flow drain. If flows are likely to be in excess of this level, a wet biofiltration swale should be used.

If a low-flow drain is used, it shall extend the entire length of the swale. The drain shall be a minimum of 6 inches deep, and its width shall be no greater than 5 percent of the calculated swale bottom width. Adjust the bottom width accordingly to maintain the necessary design bottom width for treatment. If an anchored plate or concrete sump is used for flow spreading at the swale inlet, the

plate or sump wall shall have a v-notch (maximum top width = 5 percent of swale width) or holes to allow preferential exit of low flows into the drain. See Figure 5.15 for low-flow drain specifications and details. Additional design guidelines for level spreaders are provided in Section 5.4.5.

Access

Access along half of the length of a biofiltration swale must be maintained according to the following criteria:

- Access road must be 10 feet wide, minimum.
- Curves must have a minimum width of 15 feet and a minimum outside radius of 40 feet.

Alternately, maintenance vehicle access can be provided along the base of the biofiltration swale channel using wheel strips made of modular grid pavement built into the swale bottom (see Figure 5.9). The subgrade for the strips must be engineered to support a vehicle weight of 16,000 pounds and installed according to the manufacturer's recommendations on firm native soil or structural fill, not on the amended topsoil. Each strip shall be 18 inches wide and shall not be counted as treatment area; therefore, the swale bottom width must be increased accordingly.

The wheel strip must be filled or covered with native soil (no amendments required) and overseeded with grass. If a low-flow drain is also needed, a portion of the wheel strip may be filled with pea gravel as appropriate to form the drain. Wheel strips can not be used in biofiltration swales with underdrains due to concerns about compaction of drain rock (see also Figures 5.14 and 5.15).

Soil Amendment

The condition of the soil is critical to support healthy grass growth. Native topsoil that has been stockpiled onsite or in-situ soil may be used provided that it meets the soil quality criteria described in Section 4.4.1.2. Soil amendments are required if native soil is not suitable. The following summarizes the requirements for soil amendments:

- Option 1) – Amend existing site topsoil or subsoil. The default pre-approved rates are:
  - In turf areas, place 1.75 inches of compost and till in to an 8 inch depth
  - Subsoil shall be scarified (loosened) 4 inches below amended layer, to produce 12-inch depth of un-compacted soil.
- Option 2) – Import topsoil mix of sufficient organic content and depth to meet the requirements. Imported soils should not contain excessive clay or silt fines (more than 5 percent passing the No. 200 sieve) because that could restrict stormwater infiltration. The default pre-approved rates for imported topsoils include a mix by volume of 20 percent compost with 80 percent mineral soil is pre-approved to achieve the requirement of a

minimum 4 percent (target 5 percent) organic matter by loss-on-ignition test. Subsoil shall be scarified (loosened) 4 inches below amended layer, to produce 12-inch depth of un-compacted soil.

- The organic content for "pre-approved" amendment rates can be met only using compost that meets the definition of "composted materials" in WAC 173-350 section 220. This code is available at the Dept. of Ecology's website (http://www.ecy.wa.gov/programs/swfa/compost/). The compost must also have an organic matter content of 40 percent to 65 percent, and a carbon to nitrogen ratio below 25:1. The carbon to nitrogen ratio may be as high as 35:1 for plantings composed entirely of plants native to the Puget Sound Lowlands region.

Planting Requirements

Grass shall be established throughout the entire treatment area of the swale subject to the following provisions:

- Seeding is best performed in spring (mid-March to June) or fall (late September to October). For summer seeding, sprinkler systems or other measures for watering grass seed must be provided.
- Seed may be applied via hydroseeding or broadcast application.
- Irrigation is required during the first summer following installation if seeding occurs in spring or summer. Swales seeded in the fall may not need irrigation. Site planning must address the need for sprinklers or other means of irrigation.

Swales are subject to both dry and wet conditions, as well as accumulation of sediment and debris. A mixture of dry-area and wet-area grass species that can continue to grow through silt deposits is most effective. Acceptable grass seed mixes for the Seattle area are provided in the City of Seattle Standard Specifications (9-14). As an alternative to these mixes, a horticultural or erosion control specialist may develop a seed specification tailored to the site. Table 5.8 lists grasses or other plants that are particularly tolerant of wet conditions. Some of these seed types, however, may not be commercially available.

Sod may be used where needed to initiate adequate growth. If sod is used, the sod must be grown from a seed mix suitable for a biofiltration swale and clay content must be less than 10 percent.

A newly constructed swale shall be protected from stormwater flows until grass has been established. This may be done by diverting flows or by covering the swale bottom with clear plastic until the grass is well rooted. If these actions are not feasible, an erosion control blanket per Seattle standard 9-14.5(2) shall be placed over the freshly applied seed mix. Sod may be used as a temporary cover during the wet season, but sodded areas must be reseeded with a suitable grass mix as soon as the weather is conducive to seed germination. Sod must be removed before reseeding.

Table 5.8. Finely-textured Plants Tolerant of Frequent Saturated Soil Conditions or Standing Water.

Grasses		Wetland Plants	
Water Foxtail	*Alopecurus geniculatus*	Sawbeak Sedge	*Carex stipata*
Shortawn Foxtail	*Alopecurus aequalis*	Spike Rush	*Eleocharis palustris*
Bentgrass	*Agrosits* spp.	Slender Rush	*Juncus tenuis*
Spike Bentgrass	*A. exarata*	Grass-leaf rush	*Juncus marginatus*
Redtop	*A. alba* or *gigantea*		
Colonial Bentgrass	*A. tenuis or capillaris*		
Mannagrass	*Glyceria* spp.		
Western	*G. occidentalis*		
Northern	*G. borealis*		
Slender-Spiked	*G. leptostachya*		
Rough-Stalked Bluegrass	*Poa trivialis*		
Velvet Grass	*Holcus mollis*		

The side slopes of the swale above the maximum water surface elevation can be planted with either a typical lawn seed mix or landscape plants. For swales designed to convey high flows, the soil binding capacity of the vegetation should be considered. Acceptable grasses and groundcovers are presented in Table 5.9. Plant material other than that listed in Table 5.9 may be used if the swale is privately maintained and the selected plants will not spread into the treatment area. Ivy may not be used because of its tendency to spread. Native plants (e.g., kinnikinnick) are preferred.

The following features should be incorporated into the biofiltration swale designs where site conditions allow:

Recommended Design Features

The following features should be incorporated into biofiltration swale designs where site conditions allow:

Swale Layout and Grading

- If the longitudinal slope is less than 1.5 percent (requiring the use of underdrains along the swale length), the subgrade should contain 10 percent or more of sand to promote infiltration of standing water. If sand is added to promote drainage, the soil or sand substrate must still be amended with compost.
- Underdrains are also recommended for swales greater than 1.5 percent longitudinal slope on till soils, especially if it is likely that the swale will intercept groundwater.

Table 5.9. Groundcovers and Grasses Suitable for the Upper Side Slopes of a Biofiltration Swale in Western Washington

Groundcovers	
kinnikinnick*	*Arctostaphylos uva-ursi*
Epimedium	*Epimedium grandiflorum*
creeping forget-me-not	*Omphalodes verna*
--	*Euonymus lanceolata*
yellow-root	*Xanthorhiza simplissima*
--	*Genista*
white lawn clover	*Trifolium repens*
white sweet clover*	*Melilotus alba*
-------	*Rubus calycinoides*
strawberry*	*Fragaria chiloensis*
broadleaf lupine*	*Lupinus latifolius*
Grasses (drought-tolerant, minimum mowing)	
dwarf tall fescues	*Festuca* spp. (e.g., Many Mustang, Silverado)
hard fescue	*Festuca ovina duriuscula* (e.g., Reliant, Aurora)
tufted fescue	*Festuca amethystine*
buffalo grass	*Buchloe dactyloides*
red fescue*	*Festuca rubra*
tall fescue grass*	*Festuca arundinacea*
blue oatgrass	*Helictotrichon sempervirens*

* Native species
Notes: 1) Many other ornamental grasses which require only annual mowing are suitable. 2) Ivy is not permitted because of its tendency to spread.

- Biofiltration swales should be aligned to avoid sharp bends where erosion of the swale side slope can occur. However, gradual meandering bends in the swale are desirable for aesthetic purposes and to promote slower flow.

Location and Landscaping

- During seeding, slow-release fertilizers may be applied to speed the growth of grass. If the swale is located in a sensitive lake watershed, low phosphorus fertilizers (such as formulations in the proportion 3: 1: 3 N-P-K or less) or a slow-release phosphorus formulation such as rock phosphate or bone meal should be used. A typical fertilizer application rate should be 2 pounds per 1,000 square feet. If animal manures are used in the fertilizer, they must be sterilized to avoid leaching fecal coliform bacteria into receiving waters.

- Consultation with a landscape or erosion control specialist is recommended for project-specific recommendations on grass seed, fertilizer, and mulching applications to ensure healthy grass growth. The grass mix should be capable of surviving and remaining healthy under both dry and wet conditions with limited maintenance.
- A grassy swale should be incorporated into the project site landscape design. Shrubs may be planted along the edges of a swale (above the WQ treatment level) provided that exposure of the swale bottom to sunlight and maintenance accessibility are not compromised. Note: For swales used to convey high flows, the plant material selected must bind the soil adequately to prevent erosion.
- Swales should not be located in areas where trees will drop leaves or needles that can smother the grass or clog part of the swale flowpath. Likewise, landscaping plans should take into consideration the problems that falling leaves and needles can cause for swale performances and maintenance. Landscape planter beds should be designed and located so that soil does not erode from the beds and enter and nearby biofiltration swale.

5.6.3.3 Construction Specifications and Criteria

Biofiltration swales must not be put into operation until areas of exposed soil in the contributing drainage catchment have been sufficiently stabilized. Deposition of eroded soils can impede the growth of grass in the swale and reduce swale treatment effectiveness. Thus, effective erosion and sediment control measures must remain in place until the swale vegetation is established (see also Volume 2, the Construction Stormwater Control Technical Requirements Manual). Avoid compaction during construction. Grade swales to attain uniform longitudinal and lateral slopes.

5.6.3.4 Operations and Maintenance Requirements

Biofiltration swale operations and maintenance requirements are provided in Appendix D.

5.6.4 Wet Biofiltration Swale

The wet biofiltration swale is designed with vegetation specifically adapted to saturated soil conditions, therefore it is appropriate for swales where the longitudinal slope is very low, water tables are high, or continuous low base flow is likely to result in saturated soil conditions. Where saturation exceeds about 2 weeks, typical grasses will die. Consequently, vegetation specifically adapted to saturated soil conditions is needed. Different vegetation requires changes to several of the design parameters used for basic biofiltration swales.

5.6.4.1 Site Considerations

The site considerations for a wet biofiltration swale are generally the same as those for a basic biofiltration swale. Wet biofiltration swales are applied where a basic biofiltration swale is desired but not allowed or advisable because one or more of the following conditions exist:

- The swale is on till soils and is downstream of a detention pond providing flow control.
- Saturated soil conditions are likely because of seeps or base flows on the site.
- Longitudinal slopes are slight (generally less than 2 percent).

5.6.4.2 BMP Sizing and Design Criteria

The swale geometry for a wet biofiltration swale is the same as specified for basic biofiltration swales, with the following modifications:

- ***Criterion 1:*** The bottom width may be increased to 25 feet maximum, but a length-to-width ratio of 5:1 must be provided. No longitudinal dividing berm is needed. *Note: The minimum swale length is still 100 feet.*
- ***Criterion 2:*** If longitudinal slopes are greater than 2 percent, the wet swale must be stepped so that the slope within the stepped sections averages 2 percent. Steps may be made of retaining walls, log check dams, or short riprap sections. **No underdrain or low-flow drain is required**.

These and other sizing and design criteria for wet biofiltration swales are summarized in Table 5.10.

Table 5.10. Wet Biofiltration Swale Design and Sizing Criteria.

Design Parameter	BMP 6.20-Wet Biofiltration Swale
Longitudinal Slope	< 2%
Maximum velocity	1 ft/sec @ WQ design flow rate; 3 ft/sec @ 100-yr discharge
Maximum water depth	4 inches
Manning's roughness coefficient	0.24 – 0.3
Bed width (bottom)	2 – 25 ft
Freeboard height	0.5 ft
Minimum hydraulic residence time at Water Quality Design Flow Rate	9 min
Minimum length	100 ft
Maximum sideslope	3H:1V 4H:1V preferred

Use the same design approach as for basic biofiltration swales except to add the following:

Adjust for extended wet season flow. If the swale will be downstream of a detention pond providing flow control, multiply the treatment area (bottom width times length) of the swale by 2, and readjust the swale length, if desired. Maintain a 5:1 length to width ratio.

Intent: An increase in the treatment area of swales following detention ponds is required because of the differences in vegetation established in a constant flow environment. Flows following detention are much more prolonged. These prolonged flows result in more stream-like conditions than are typical for other wet biofiltration swale situations. Since vegetation growing in streams is often less dense, this increase in treatment area is needed to ensure that equivalent pollutant removal is achieved in extended flow situations.

High-Flow Bypass: Wet biofiltration swales must be designed as off-line facilities to protect wetland vegetation from damage. Unlike grass, wetland vegetation will not quickly regain an upright attitude after being laid down by high flows. New growth, usually from the base of the plant, often taking several weeks, is required to regain its upright form. The bypass may be an open channel parallel to the wet biofiltration swale.

Water Depth and Base Flow: Same as for basic biofiltration swales except the design water depth shall be 4 inches for all wetland vegetation selections, and no underdrains or low-flow drains are required.

Flow Velocity, Energy Dissipation, and Flow Spreading: Same as for basic biofiltration swales.

Access

Access requirements are the same as for basic biofiltration swales except access is only required to the inflow and the outflow of the swale; access along the length of the swale is not required. Also, wheel strips may not be used for access in the swale.

Intent: An access road is not required along the length of a wet swale because of infrequent access needs. Frequent mowing or harvesting is not desirable. In addition, wetland plants are fairly resilient to sediment-induced changes in water depth, so the need for access should be infrequent.

Soil Amendment

Same as for basic biofiltration swales.

Planting Requirements

Same as for basic biofiltration swales except for the following modifications:

- A list of acceptable plants and recommended spacing is shown in Table 5.11. In general, it is best to plant several species to increase the likelihood that at least some of the selected species will find growing conditions favorable.

Table 5.11. Recommended Plants for Wet Biofiltration Swale.

Common Name	Scientific Name	Spacing (on center)
Shortawn foxtail	*Alopecurus aequalis*	seed
Water foxtail	*Alopecurus geniculatus*	seed
Spike rush	*Eleocharis spp.*	4 inches
Slough sedge*	*Carex obnupta*	6 inches or seed
Sawbeak sedge	*Carex stipata*	6 inches
Sedge	*Carex* spp.	6 inches
Western mannagrass	*Glyceria occidentalis*	seed
Velvetgrass	*Holcus mollis*	seed
Slender rush	*Juncus tenuis*	6 inches
Watercress*	*Rorippa nasturtium-aquaticum*	12 inches
Water parsley*	*Oenanthe sarmentosa*	6 inches
Hardstem bulrush	*Scirpus acutus*	6 inches
Small-fruited bulrush	*Scirpus microcarpus*	12 inches

* Good choices for swales with significant periods of flow, such as those downstream of a detention facility.

Note: Cattail (Typha latifolia) is not appropriate for most wet swales because of its very dense and clumping growth habit which prevents water from filtering through the clump.

- A wetland seed mix may be applied by hydroseeding, but if coverage is poor, planting of rootstock or nursery stock is required. Poor coverage is considered to be more than 30 percent bare area through the upper 2/3 of the swale after 4 weeks.

5.6.4.3 Construction Specifications and Criteria

Same as for basic biofiltration swales, see Section 5.6.3.3.

5.6.4.4 Operations and Maintenance Requirements

Wet biofiltration swale operations and maintenance requirements are provided in Appendix D.

5.6.5 Continuous Inflow Biofiltration Swale

A continuous inflow biofiltration swale is used in situations where stormwater enters the swale continuously along the length of the channel rather than at a single location at the head of the swale. The basic swale design is modified by increasing the swale length to achieve an equivalent average residence time.

5.6.5.1 Site Considerations

A continuous inflow biofiltration swale is to be used when inflows are not concentrated or when flow enters at frequent points along the swale. Unconcentrated inflow occurs along roadways that that have no curbs, where

runoff sheet flows across the shoulder to the swale. Multiple inflow points via curb cuts can occur along roadways and parking lots. In general, no inlet port should carry more than about 10 percent of the flow.

A continuous inflow swale is not appropriate for a situation in which significant lateral flows enter a swale at some point downstream from the head of the swale. In this situation, the swale length must be recalculated from the point of entry to provide adequate treatment for the increased flow.

5.6.5.2 BMP Sizing and Design Criteria

The design method for continuous inflow swales is the same as for the basic biofiltration swale except for the following modifications:

- The design flow must include runoff from the pervious side slopes draining to the swale along the entire length of the swale.
- The water quality design flow can be variable to reflect the increase in flows along the swale length. If only a single design flow is used, the total flow at the outlet should be used to size the swale.
- Continuous inflow biofiltration swales are designed as on-line facilities. To determine the design flow for facility sizing purposes, adjust the output design flow from the continuous runoff model using the on-line adjustment factor of 1.65 (see basic biofiltration swale preliminary step P-1).
- Double the hydraulic residence time so that it is a minimum of 18 minutes.

Design criteria for continuous inflow biofiltration swales are provided in Table 5.12.

Table 5.12. Continuous Inflow Biofiltration Swale Design and Sizing Criteria.

Design Parameter	Continuous Inflow Biofiltration Swale
Longitudinal Slope	1.5%- 2.5%
Maximum velocity	1 ft/sec @ WQ design flow rate; 3 ft/sec @ 100-yr discharge
Maximum water depth	2 inches if mowed frequently; 4 inches if mowed infrequently
Manning's roughness coefficient	0.2 – 0.3 (0.24 if mowed infrequently)
Bed width (bottom)	2 – 10 ft (up to 16 ft with divider berm)
Freeboard height	0.5 ft
Minimum hydraulic residence time at Water Quality Design Flow Rate	18 min
Minimum length	Sufficient to achieve hydraulic residence time
Maximum sideslope	3H:1V 4H:1V preferred

For continuous inflow biofiltration swales, interior side slopes above the water quality design treatment elevation shall be planted in grass. A typical lawn seed mix or the biofiltration seed mixes are acceptable. Landscape plants or groundcovers other than grass may not be used anywhere between the runoff inflow elevation and the bottom of the swale.

Intent: The use of grass on interior side slopes reduces the chance of soil erosion and transfer of pollutants from landscape areas to the biofiltration treatment area.

5.6.5.3 Construction Specifications and Criteria

Requirements for construction of continuous inflow biofiltration swales are the same as for basic biofiltration swales.

5.6.5.4 Operations and Maintenance Requirements

Continuous inflow biofiltration swale operations and maintenance requirements are provided in Appendix D.

5.7 Filter Strips

A filter strip is a grassy slope that receives unconcentrated runoff from an adjacent impervious area such as a parking lot, driveway, or roadway. It is graded to maintain sheet flow over its entire width and removes pollutants primarily by filtration as stormwater moves through the grass blades, which enhances sedimentation, as well as by trapping and adhesion of pollutants to the grass and thatch (Figure 5.16). Pollutants can also be adsorbed by the underlying soil when infiltration occurs, but the extent of infiltration depends on the type of soil, the density of grass, and the slope of the filter strip. The following three types of filter strip BMPs are described in this section:

- Basic Filter Strip
- Compost Amended Vegetated Filter Strip
- Narrow Area Filter Strip
- Ecology Embankments

The *basic filter strip* is flat with no side slopes (Figure 5.16). Contaminated stormwater is distributed as sheet flow across the inlet width of the filter strip.

The *compost-amended filter strip* serves as an enhanced treatment option. The compost-amended filter strip is similar to the basic filter strip, but the filter area is compost-amended to a minimum of 10 percent organic content (in accordance with the compost amendment requirements in Chapter 4, Section 4.4.1), and planted with hydroseeded grass maintained at 95 percent density and a 4-inch length by mowing and periodic re-seeding (possible landscaping with herbaceous shrubs).

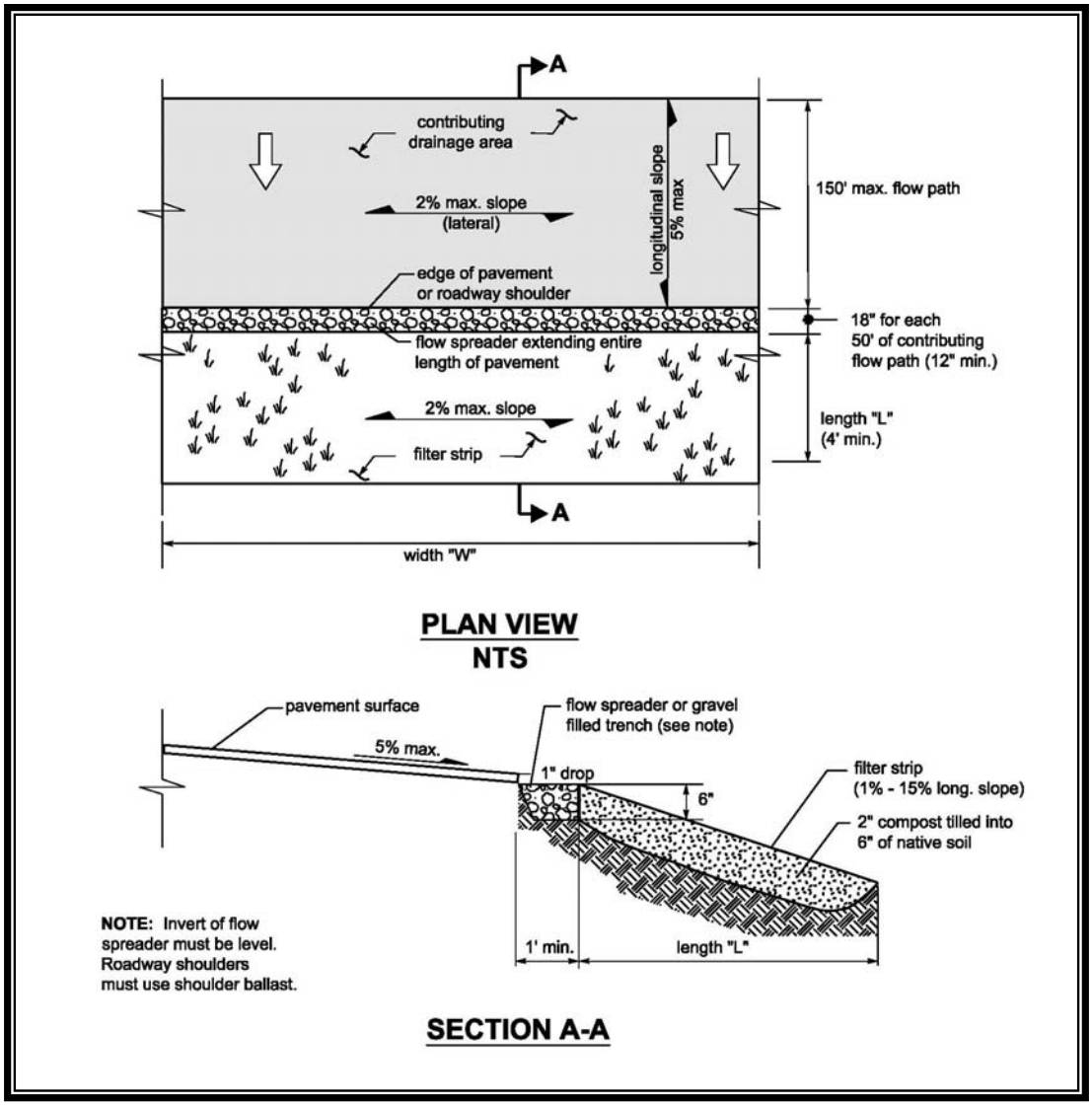


Figure 5.16. Typical Filter Strip.

The *narrow area filter strip* is designed for impervious areas with flowpaths of 30 feet or less that can drain along their widest dimension.

5.7.1 Applications and Limitations

Filter strips are designed based on the expectation that water will flow fairly evenly across the entire width and length of the filter strip area. Therefore, paved sites without underground stormwater collection systems, gutters, or other runoff control features are good candidates for filter strips.

Filter strips are generally suitable for sites that meet the following conditions:

- Stormwater runoff from the area requiring treatment can be uniformly distributed along the entire top of the filter strip. In areas where

stormwater runoff cannot be spread evenly along the top of the filter strip, a different BMP should be used.

- The flow path draining to the filter strip should not exceed 150 feet. Runoff traveling greater distances tends to concentrate before entering the filter strip.
- The lateral slope of the drainage area contributing flows to the filter strip (parallel to the edge of the pavement) should be less than 2 percent.
- The longitudinal slope of the contributing drainage area (parallel to the direction of flow entering the filter strip) should be less than 5 percent.

The *basic filter strip* is typically used on-line and adjacent and parallel to a paved area such as parking lots, driveways, and roadways. The *CAVFS* is a variation of the basic vegetated filter strip that improves the stormwater management capabilities of the roadside embankment. The soil amendments improve infiltration characteristics, increase surface roughness, and improve plant sustainability.

Narrow area filter strips could be used at roadways with limited right-of-way, or for narrow parking strips. If space is available to use the basic filter strip design, that design must be used in preference to the narrow filter strip.

All filter strips meet basic water quality treatment requirements. A filter strip that is designed as a compost-amended filter strip in accordance with Chapter 4, Section 4.4.1 can meet enhanced treatment requirements.

In general, filter strips require more land area than biofiltration swales because the flow depth is much shallower than in a swale. Although space requirements may be greater, the filter strip is a viable stormwater treatment option in locations where grassy slopes already exist, or where a slope can easily be incorporated into the landscape design for the site. See Section 5.4 for general requirements that may be applicable to filter strips. Other limitations that should be considered include:

1. Filter strips are susceptible to short-circuiting via flow channelization because they rely on a large smoothly graded area. If rills, gullies, or channels develop in the filter strip area, stormwater will travel too quickly through the strip, reducing contact time and pollutant removal performance. A filter strip with uneven grading perpendicular to the sheet flow path will develop flow channels over time. These problems can be overcome with careful site planning, good soil compaction, skillful grading, and periodic maintenance.
2. Filter strip areas cannot be used for material storage or any activities that will cause disturbance of the ground surface in a manner that could create or promote preferential flow paths (i.e., rills or gullies) in the filter strip.

3. Filter strips should not be located in shaded areas because sunlight is required to ensure healthy grass growth.

5.7.2 Operations and Maintenance Requirements

Operations and maintenance requirements for filter strip BMPs are provided in Appendix D.

5.7.3 Basic Filter Strip and Compost Amended Vegetated Filter Strip

5.7.3.1 Site Considerations

Filter strips are designed as online facilities. They are designed to receive continuous sheet flow from contributing surfaces and should not be located downstream of detention facilities or other concentrated flows. The following design criteria and requirements influence whether a filter strip is feasible for a particular site:

- A maximum longitudinal slope along the contributing roadway surface of 2 percent. A stepped series of flow spreaders installed at the head of the strip could compensate for slightly steeper slopes.
- A maximum cross-slope along the contributing surface of 5 percent. Contributing drainage areas with slopes steeper than 5 percent should either use a different BMP or must provide energy dissipation and flow spreading mechanisms upslope of the upper edge of the filter strip.
- A flow spreader adjacent to the roadway is required
- Uncurbed contributing areas are preferred. If the roadway being treated by the filter strip is curbed, a 12-inch minimum curb cut is required at frequent intervals. In general, no inlet port should carry more than about 10 percent of the flow.
- Curb cuts should be a minimum of 1 inch above the filter strip inlet.

5.7.3.2 BMP Sizing and Design Criteria

The general BMP sizing requirements for basic and compost amended filter strips are included in Table 5.13. In addition, the WSDOT *Highway Runoff Manual* (WSDOT 2008) includes Ecology-approved design guidelines for compost amended vegetated filter strips. These guidelines may also be used for designs in the City of Seattle. The City's general design procedures for filter strips are as follows.

Table 5.13. Basic and Compost Amended Vegetated Filter Strip Design and Sizing Criteria.

Design Parameter	Basic Filter Strip	Compost Amended Vegetated Filter Strip
Longitudinal Slope	1% - 15%	1% - 15%
Maximum velocity	0.5 ft/sec	0.5 ft/sec
Maximum water depth	1-inch max.	1-inch max.
Manning's roughness coefficient	0.35	0.45 if compost-amended, and mowed to maintain grass height ≤4"
Minimum hydraulic residence time at Water Quality Design Flow Rate	9 minutes	9 minutes
Minimum length	100 ft	100 ft
Maximum sideslope	Inlet edge ≥ 1" lower than contributing paved area	Inlet edge ≥ 1" lower than contributing paved area
Max. tributary drainage flowpath	150 ft	150 ft
Max. longitudinal slope of contributing area	5% (steeper than 5% need upslope flow spreading and energy dissipation)	5% (steeper than 5% need upslope flow spreading and energy dissipation)
Max. lateral slope of contributing area	2% (at the edge of the strip inlet)	2% (at the edge of the strip inlet)

Calculate the design flow depth using Manning's equation as follows:

$$KQ = (1.49A\ R^{0.67}\ s^{0.5})/n$$

Substituting for AR:

$$KQ = (1.49Ty^{1.67}\ s^{0.5})/n$$

Where:

$Ty = A_{rectangle}\ ft^2$
$y \approx R_{rectangle}$, design depth of flow, ft (1 inch maximum)
Q = peak Water Quality design flow rate based on continuous modeling, ft^3/sec
K = 1.65 (as in swale design, the adjustment factor is required by the Department of Ecology to account for the differential between continuous runoff model design flow rates and SBUH design flow rates)
n = Manning's roughness coefficient
s = Longitudinal slope of filter strip parallel to direction of flow
T = Width of filter strip perpendicular to the direction of flow, ft
A = Filter strip inlet cross-sectional flow area (rectangular), ft^2
R = hydraulic radius, ft.

Rearranging for y (y must not exceed 1 inch):

$$y = [KQn/1.49Ts^{0.5}]^{0.6}$$

Calculate the design flow velocity V, ft./sec., through the filter strip:

$$V = KQ/Ty$$

V must not exceed 0.5 ft./sec

Calculate required length, in feet, of the filter strip at the minimum hydraulic residence time, t, of 9 minutes:

$$L = tV = 540V$$

5.7.3.3 Construction Specifications and Criteria

Filter strips must not be put into operation until areas of exposed soil in the contributing drainage catchment have been sufficiently stabilized. Deposition of eroded soils can impede the growth of grass in the filter strip and reduce treatment effectiveness. Erosion and sediment control measures must remain in place until the filter strip vegetation is established (see Volume 2, the Construction Stormwater Control Technical Requirements Manual for erosion and sediment control BMPs). It is also important to avoid compaction of the filter strip areas during construction.

5.7.3.4 Operations and Maintenance Requirements

Operations and maintenance requirements for basic and compost amended vegetated filter strip BMPs are provided in Appendix D.

5.7.4 Narrow Area Filter Strip

5.7.4.1 Site Considerations

Narrow area filter strips are used for impervious areas that have a short flow path (30 feet or less) that can drain along their widest dimension to a grassy area. Basic filter strips are preferred over narrow area filter strips and should be used whenever space is available. However, the narrow area filter strip may be used along roadways with limited right-of-way or for narrow parking strips.

5.7.4.2 BMP Sizing and Design Criteria

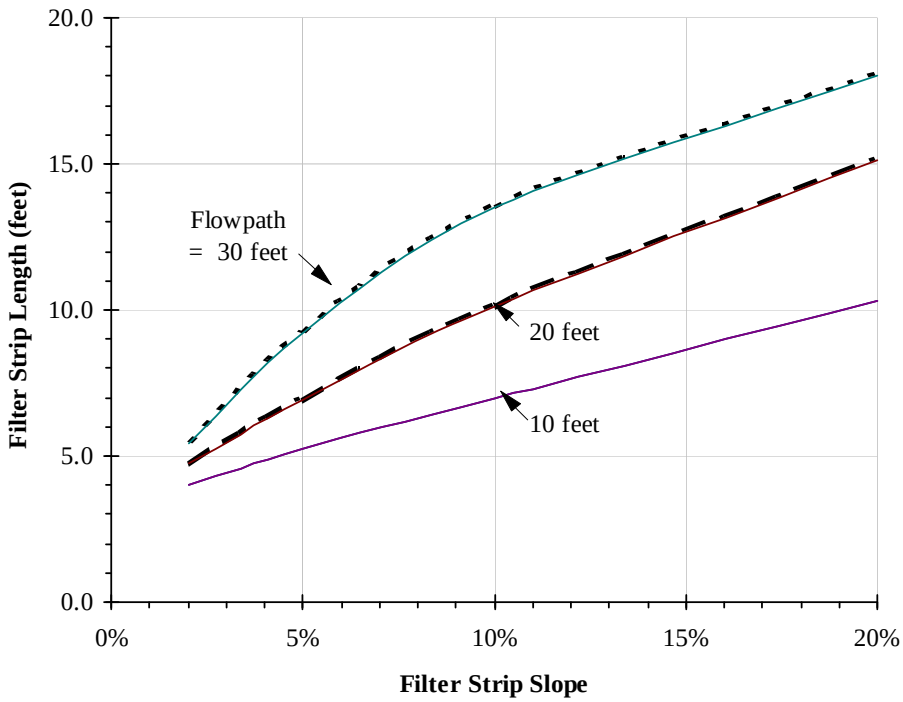
Design and construction criteria for narrow area filter strips are generally the same as for basic filter strips. The sizing of a narrow area filter strip is based on the length of flowpath draining to the filter strip and the longitudinal slope of the filter strip itself (parallel to the flowpath). Sizing and design modifications (relative to basic filter strips) are outlined below.

Step 1: Determine the length of the flow path from the upstream to the downstream edge of the contributing impervious area that will drain via sheet flow to the filter strip. Normally this is the same as the width of the paved area,

but if the site is sloped, the flow path may be longer than the width of the impervious area.

Step 2: Calculate the longitudinal slope of the filter strip (along the direction of unconcentrated flow), averaged over the total width of the filter strip. The minimum slope used to size the filter strip is 2 percent. If the slope is less than 2 percent, use 2 percent for sizing purposes. The maximum allowable filter strip slope is 20 percent. If the slope exceeds 20 percent, the filter strip must be stepped down the slope so that the treatment areas between drop sections do not have a longitudinal slope greater than 20 percent. Drop sections must be provided with erosion protection at the base and flow spreaders to re-spread flows (see Section 5.4.5 for design guidelines for level spreaders). Vertical drops along the slope must not exceed 12 inches in height. If this is not possible, a different treatment facility must be selected.

Step 3: Select the appropriate filter strip length for the flowpath length and filter strip longitudinal slope (Steps 1 and 2 above) from the graph in Figure 5.17. The filter strip must be designed to provide the specified filter strip length along the entire stretch of pavement draining into it.



Note: minimum allowable filter strip length is 4 feet

Figure 5.17. Filter Strip Lengths for Narrow Right-of-Way.

To use the graph: Find the length of the flowpath on one of the curves (interpolate between curves as necessary). Move along the curve to the point where the design longitudinal slope of the filter strip (x-axis) is directly below. Read the filter strip length on the y-axis which corresponds to the intersection point.

5.7.4.3 Construction Specifications and Criteria

Requirements for construction of narrow area filter strips are the same as for basic filter strips.

5.7.4.4 Operations and Maintenance Requirements

Operations and maintenance requirements for narrow area filter strip BMPs are provided in Appendix D.

5.7.5 Ecology Embankments

The WSDOT ecology embankment (now referred to as a media filter drain; see WSDOT *Highway Runoff Manual* BMP RT.07 [WSDOT 2008]) is a linear flow-through stormwater treatment device that can be sited along roadway side-slopes (conventional design) and medians (dual ecology embankment), borrow ditches, or other linear depressions. Cut-slope applications may also be considered. The ecology embankment can be used where available right-of-way is limited, sheet flow from the roadway surface is feasible, lateral gradients are generally less than 25 percent (4H:1V), and longitudinal gradients are less than 5 percent. In 2006, the ecology embankment BMP received a general-use level designation for basic, phosphorus, and enhanced treatment applications.

Ecology embankments have four basic components: a gravel no-vegetation zone, a vegetated filter strip, the ecology mix bed, and an optional gravel-filled underdrain trench. The ecology embankment removes suspended solids, phosphorus, and metals from roadway runoff through physical straining, ion exchange, carbonate precipitation, and biofiltration.

Additional descriptions, applications, and design details are provided in the WSDOT *Highway Runoff Manual* under BMP RT.07 (WSDOT 2008). As of the date of publication for this Directors' Rule, the City of Seattle allows the use of ecology embankments per the Ecology-approved designs outlined in the *Highway Runoff Manual*.

5.8 Infiltration BMPs

Infiltration systems can function as both flow control and stormwater treatment facilities. To minimize duplication, information regarding siting criteria, design methods and details, and guidance of a general nature for infiltration systems is presented in Chapter 4 (Sections 4.3 and 4.4) and not repeated here. Infiltration system modeling guidelines are provided in Chapter 6, as well as Chapter 4.

With regards to runoff treatment, infiltration systems remove pollutants primarily via physical filtration as stormwater passes through the underlying soil, but also via chemical adsorption and precipitation reactions. Biological uptake by plants may also occur. In addition, some pollutants such as nutrients may also be absorbed by microbes present in the soil. As such, the physical and chemical characteristics of the soil (e.g., organic content, soil sorptive capacity, soil aerobic conditions) are critical in establishing pollutant removal performance. Although the design methods are the same whether these systems are used for flow control or treatment, the design criteria, particularly those related to soil characteristics are quite different. Chapter 6 highlights where guidance applies specifically to treatment systems as opposed to flow control systems. Information specific to treatment performance, such as soil suitability is provided in this section.

The following infiltration BMPs are accepted stormwater treatment facilities in Seattle, and are summarized in this section:

1. Infiltration basins. An impoundment that stores stormwater so that it can infiltrate into the underlying soil (see Section 4.5.1).
2. Infiltration trenches. Similar to a basin, except trenches are long and linear, and stormwater is usually introduced below the surface via a perforated pipe rather than at the surface (see Section 4.5.2).
3. Drywells. Similar to a trench, except drywells are typically deeper than they are wide (see Section 4.5.3).
4. Bioretention systems. Bioretention systems incorporate various designs using soil and plant complexes to both treat and detain stormwater through infiltration, evapotranspiration, and impoundment (see Section 4.4.5).
5. Permeable pavement systems. Permeable pavement is a paving system which allows rainfall to percolate into an underlying soil or aggregate storage reservoir, where stormwater is stored and infiltrated to underlying soil (see Section 4.4.7).

Note that the soil infiltration requirements for water quality treatment are substantially different from those for flow control. Infiltration treatment soils must contain sufficient organic matter and/or clays to sorb, decompose, and/or filter stormwater pollutants. Pollutant/soil contact time, soil sorptive capacity, and soil aerobic conditions are all important design considerations. Specific requirements are outlined in Section 5.8.4 below.

5.8.1 Applications and Limitations

Infiltration treatment facilities meet the requirements for basic, phosphorus, and enhanced treatment provided that the facilities meet the water quality treatment infiltration requirements outlined in Chapter 2 and Section 5.4, and the underlying

soil or a specially engineered imported soil mix meets certain specifications (see also 5.8.4). In general, these treatment techniques can capture and remove or reduce the target pollutants to levels that:

- Will not adversely affect public health or beneficial uses of surface and ground water resources
- Will not cause a violation of ground water quality standards.

Infiltration treatment systems are typically installed:

- As off-line systems
- As a treatment for street/highway runoff after pretreatment
- As part of a treatment train
- As retrofits at sites with limited land area, such as residential lots, commercial areas, parking lots, and open space areas
- With appropriate pretreatment for oil and silt control to prevent clogging. Appropriate pretreatment devices are outlined in Section 5.5.

5.8.2 General Considerations

See Section 4.3 of Chapter 4.

5.8.3 Operations and Maintenance Requirements

Infiltration facility operations and maintenance requirements are provided in Appendix D.

5.8.4 Soil Requirements for Infiltration for Water Quality Treatment

The soil requirements for infiltration treatment vary depending on the type of facility. Many infiltration facilities (e.g., infiltration basins and trenches) rely on the properties of the native soils (i.e., existing underneath the facility) to meet treatment requirements. Bioretention systems utilize imported soils meeting specific criteria to meet treatment requirements. The following sections summarize the applicable soil requirements for each facility.

5.8.4.1 Native Soil Requirements for Infiltration Facilities

Infiltration basins, trenches, drywells, and permeable pavement meet the requirements for basic, phosphorus, and enhanced treatment provided that the following soil suitability criteria (SSC) are met:

SSC#1: The short-term soil infiltration rate (field measured, before correction factors applied) must be 4.0 inches per hour or less to a depth of 2.5 times the maximum pond design water depth, or a minimum of 6 feet below the base of the infiltration facility, whichever is greater. A long-term (i.e., design, after correction factors) infiltration rate of up to 2.0 inches/hour may be used, provided that the

facility is not located within a critical aquifer recharge area and the designer documents that the underlying soil has characteristics comparable to those specified below in SSC #2. The site infiltration rate must be determined using the modified PIT method described in Appendix E.

SSC#2: The underlying soil to a depth of at least 18 inches, must meet the following conditions:

- Cation exchange capacity (CEC), as determined by U.S. EPA Method 9081, of the soil must be greater than or equal to 5 milliequivalents per 100 milligrams of dry soil. Lower CEC content may be considered if it is based on a soil loading capacity determination for the target pollutants that is approved by the Director.
- Organic content of the treatment soil (ASTM D 2974): Organic matter can increase the sorptive capacity of the soil for some pollutants. Soil organic content should be at least 1 percent, however the designer should evaluate whether the organic matter content is sufficient for control of the target pollutant(s).

SSC#3: Waste materials shall not be used as infiltration media, nor may an infiltration system be constructed over uncontrolled or non-engineered fill soils.

5.8.4.2 Imported Soil Requirements for Bioretention Systems

Bioretention facilities (see also Chapter 4, Section 4.4.5) meet the requirements for basic, phosphorus, and enhanced treatment, but are not subject to the same soil infiltration treatment requirements for basins and trenches discussed above (i.e., SSC #1 through #3). Instead, bioretention facilities utilize the City of Seattle-specific standards for the imported underlying soil mix. Soil requirements for bioretention systems designed to meet water quality treatment requirements are summarized below:

1. The bioretention soil mix shall meet City of Seattle Bioretention Soil Standard Specification 7-21.
2. The minimum depth of treatment soil must be 18 inches.

Soil depths of 24 inches and greater should be considered to provide improved removal of nutrients as needed, including phosphorus. Metals uptake has been shown to be greater at 22 inches than 10 inches (with no additional improvement in deeper soils); however, most significant metal uptake occurs in the mulch layer that can retain a large portion of the total metals loads (WSU 2005).

5.8.5 Best Management Practices (BMPs) for Infiltration Treatment

The five BMPs discussed previously are recognized as effective treatment techniques using infiltration. Selection of a specific BMP should be coordinated with the treatment facility menus provided in Section 5.3. The following sections summarize (or reference) specific design guidelines for each BMP.

5.8.5.1 Infiltration Basins

See Section 4.5 of Chapter 4.

5.8.5.2 Infiltration Trenches

Infiltration trench requirements for siting, design, modeling, construction, and operations and maintenance are presented in Section 4.5, Chapter 4. Infiltration trenches may be sized to achieve water quality treatment standards using the sizing factors presented in Table 5.14. A pre-sized facility must meet the requirements set forth under the "Pre-Sized Approach" section for infiltration trenches in Section 4.5.

Table 5.14. Sizing Factors for Infiltration Trench Receiving Runoff from Impervious Surface.

Trench Depth	Native Soil Design Infiltration Rate	Sizing Factor for Trench Area
1.5 feet	0.25 inch/hour	12.7%
	0.5 inch/hour	8.3%
	1.0 inch/hour	5.3%
3.0 feet	0.25 inch/hour	8.6%
	0.5 inch/hour	5.9%
	1.0 inch/hour	4.0%

%- percent
Infiltration Trench Area = Contributing Impervious Area x Factor (%)/100.
Impervious Area Mitigated = Trench Area ÷ Factor (%)/100.

5.8.5.3 Drywells

Drywell requirements for siting, design, modeling, construction, and operations and maintenance are presented in Section 4.5, Chapter 4. Drywells may be sized to achieve water quality treatment standards using the sizing factors presented in Table 5.15. A pre-sized facility must meet the requirements set forth under the "Pre-Sized Approach" section for drywells in Section 4.5.

Table 5.15. Sizing Factors for Drywells Receiving Runoff from Impervious Surface.

Drywell Depth	Native Soil Design Infiltration Rate	Sizing Factor for Drywell Area
4.0 feet	0.25 inch/hour	7.5%
	0.5 inch/hour	5.1%
	1.0 inch/hour	3.5%
6.0 feet	0.25 inch/hour	6.1%
	0.5 inch/hour	4.2%
	1.0 inch/hour	2.8%

%- percent
Drywell Area = Contributing impervious Area x Factor (%)/100.
Impervious Area Mitigated = Drywell Area ÷ Factor (%)/100.

5.8.5.4 Bioretention Facilities

Bioretention facility requirements for siting, design, modeling, construction, and operations and maintenance are presented in Section 4.4.5, Chapter 4. Bioretention cells (without underdrains) and bioretention planters may be sized to achieve water quality treatment standards using the sizing factors presented in Tables 5.16 and 5.17, respectively. A pre-sized facility must meet the requirements set forth under the applicable "Pre-Sized Approach" section for bioretention cells and planters in Section 4.4.5. Note that bioretention facilities without infiltration to underlying soils (including bioretention planters) are not considered infiltration facilities. Therefore, they are not subject to the same siting prohibitions and setbacks.

Table 5.16. Sizing Factors for Bioretention Cell (without underdrain).

Ponding Depth	Native Soil Design Infiltration Rate	Sizing Factor for Cell Bottom Area
6-inches	0.25 inch/hour	5.0%
	0.5 inch/hour	2.9%
	1.0 inch/hour	1.6%
12-inches	0.25 inch/hour	3.0%
	0.5 inch/hour	1.7%
	1.0 inch/hour	0.9%

%- percent
Bioretention Cell Bottom Area = Contributing Impervious Area x Factor (%)/100.
Impervious Area Mitigated = Bioretention Cell Bottom Area ÷ Factor (%)/100.

Table 5.17. Sizing Factors for Bioretention Planter.

Ponding Depth	Sizing Factor Bioretention Planter Area
6-inches	2.6%
12-inches	2.0%

%- percent
Bioretention Planter Area = Contributing Impervious Area x Factor (%)/100.
Impervious Area Mitigated = Bioretention Planter Area ÷ Factor (%)/100.

5.8.5.5 Permeable Pavement

See Section 4.4.7, Chapter 4.

5.9 Sand Filtration Treatment Facilities

This section presents criteria for the design and construction of runoff treatment sand filters including basin, vault, and linear filters. The following three BMPs are discussed in this section:

- *Sand filter basins*. Like an infiltration pond, the sand filter basin is an impoundment that temporarily stores stormwater runoff so that it can infiltrate, but instead of infiltrating through the native soil, stormwater passes through a constructed sand bed that is specifically designed to treat stormwater runoff. In addition, sand filter basins are designed with underdrains to collect and route runoff following treatment to the downstream conveyance system.
- *Sand filter vaults*. A sand filter vault is similar to a filter basin except the media and underdrain system are installed below grade in a concrete vault.
- *Linear sand filters*. Linear sand filters are similar to sand filter vaults, except the vault is configured as a long, shallow, linear system. The vault contains two cells or chambers, one for removing coarse sediment and the other containing sand overlying an underdrain. Runoff usually enters the settling chamber as unconcentrated flow from an adjacent impervious area and overflows a central weir into the sand portion of the vault.

Sand filters treat stormwater primarily via physical filtration. As stormwater passes through the sand media, pollutants are trapped in the small spaces between sand grains, or adhere to the sand surface. Over time, soil bacteria may also grow in the sand bed and some biological removal may occur. Sand filter performance objectives depend on whether the facility is a “basic” or “large” sand filter, as summarized below.

<u>Basic sand filters</u> are expected to achieve the performance goals for Basic Treatment. Based upon experience in King County and Austin, Texas basic sand filters achieve the following average pollutant removals:

- 80 percent TSS at influent Event Mean Concentrations (EMCs) of 30 to 300 mg/L (King County, 1998;Chang, 2000)
- Oil and grease to below 10 mg/L daily average and 15 mg/L at any time, with no ongoing or recurring visible sheen in the discharge.

Sand filter media can also be amended to provide Enhanced Treatment. At the time this manual was written, processed steel fiber and crushed calcitic limestone are the only sand filter amendments for which data are available that documents increased dissolved metals removal (the target of enhanced treatment). Use of amended sand filters requires prior Director approval.

<u>Large sand filters</u> hold 1.5 times the amount of stormwater as a basic filter. Because they will treat a larger volume of stormwater (approximately 95 percent of the annual runoff volume), they are expected to remove at least 50 percent of the total phosphorus. Consequently, large sand filters meet the requirements for phosphorus control. As with basic sand filters above, various amendments may also be added to improve the removal of specific pollutants (subject to Director approval).

5.9.1 Applications and Limitations

Sand filters can be used in most residential, commercial, and industrial developments where site topography and drainage provide adequate hydraulic head to operate the filter. An elevation difference of approximately 4 feet between the inlet and the outlet of the filter is usually required.

Sand filters must drain freely. Sand media cannot be saturated for extended periods because under these conditions, oxygen can be depleted, releasing pollutants such as dissolved metals and phosphorus that are more mobile under anoxic conditions. To prevent this release of pollutants that have accumulated in the media, sand filters must be designed to drain the water quality design storm volume within 72 hours.

Similarly, there must be sufficient clearance (at least 2 feet) between the seasonal high groundwater level and the bottom of the sand filter to obtain adequate drainage. High groundwater may damage underground structures or affect the performance of filter underdrain systems. Likewise, to the extent base flow conditions can be identified, it is recommended that base flows be bypassed around sand filter basins and sand filter vaults to keep the sand from remaining saturated for extended periods of time. Base flows must be bypassed for all linear sand filters.

Sand filters should not be used in areas where heavy sediment loads or oils and greases will clog or prematurely overload the sand, unless adequate pretreatment can be provided or maintenance frequencies are increased to prevent clogging.

If a project must comply with the Minimum Requirements for Flow Control, any bypassed flows as well as the discharge from the filter (i.e., underdrains) must meet any applicable flow control requirements or be routed to a flow control facility. If the project does not need to meet flow control requirements, the filter outlet and overflow must discharge to an approved discharge point. Approved discharge points shall, at a minimum, maintain natural drainage patterns to the maximum extent practical, and not cause significant adverse impact to the down stream conveyance system, receiving water body, or down-gradient properties.

See Section 5.4 for additional general requirements that may be applicable to sand filter facilities.

5.9.2 General Requirements

5.9.2.1 Pretreatment

Sand filters are prone to clogging because sediment tends to build up on the surface of the filter media, which causes the hydraulic capacity of the filter and its ability to treat the required stormwater volume to decline. Clogging generally occurs gradually over the service life of the filter, but can occur during a single

large storm if large quantities of fine-grained material are deposited on the surface of the filter. Consequently, pretreatment is required to prevent clogging and extend the service life of the filter media. Pretreatment design requirements are described in Section 5.5, but generally can consist of a presettling basin integrated into the sand filer, or a dedicated pretreatment BMP upstream of the filter.

5.9.2.2 Liners

A liner is required if the site is located within a critical aquifer recharge area. See Section 5.4.4 for additional information on liner design criteria.

5.9.2.3 Online Versus Off-line Facilities

Sand filters may be designed as online or off-line facilities. However, it is recommended that filters be designed off-line to prevent the filter surface from scour during large storm events and to avoid excessive solids loading, which could clog the filter surface.

On-line sand filters must not be placed ***upstream*** of a detention facility. This is to prevent exposure of the sand filter surface to high flow rates that could cause loss of media and previously removed pollutants.

On-line sand filters placed ***downstream*** of a detention facility must be sized to filter 91 percent of the total runoff volume (95 percent for large sand filter).

Off-line sand filters placed ***upstream*** of a detention facility must have a flow splitter designed to send all flows at or below 91 percent of the total runoff volume, as predicted by an approved continuous runoff model, to the sand filter. The sand filter must be sized to filter all the runoff sent to it (no overflows from the treatment facility should occur). Note that some continuous hydrologic models allow any bypasses and the runoff filtered through the sand to be directed to the downstream detention facility.

Off-line sand filters can be located either upstream or downstream of detention facilities. Off-line sand filters placed **downstream** of a detention facility must have a flow splitter designed to send all flows at or below the 2-year flow frequency from the detention pond, as predicted by an approved continuous runoff model, to the treatment facility. The treatment facility must be sized to filter all the runoff sent to it (no overflows from the treatment facility should occur).

On-line sand filters shall be equipped with **overflows** (primary, secondary, and emergency) in accordance with the design criteria for wetponds (see Section 5.10.3). *Note: The primary overflow may be incorporated into the emergency spillway in cases where the spillway discharges into a downstream detention facility, or where overflows can be safely controlled and redirected through an approved discharge point into the downstream conveyance system.*

Approved discharge points shall, at a minimum, maintain natural drainage patterns to the maximum extent practicable, and not cause a significant adverse impact to the downstream conveyance system, receiving surface water, or down-gradient properties.

5.9.3 Construction Specifications and Criteria

Extra care is required during construction maintain the filter design hydraulic capacity. Construction requirements applicable to all sand filter BMPs are described below. Construction requirements specific to each sand filter BMP are provided in the individual BMP sections.

- Runoff from construction activities should not be allowed to enter a sand filtration system unless the sand bed is replaced periodically during construction and after the site is stabilized (i.e., after all project improvements are completed and all exposed ground surfaces are stabilized with vegetation or landscaping).
- An alternative is to install a sand filter basin including near-full excavation for the filter sand and underdrain layers, delaying placement of the sand and underdrains until the site is stabilized. The partially complete sand filter will then function like a small wetpond. Later, the accumulated sediment shall be cleaned, final excavation performed, and the underdrain and sand layers placed. A second alternative is to place only the gravel underdrain during the construction phase. Then clean the gravel and place the sand layer after the site is stabilized.
- Sand media characteristics are the key element of any sand filtration system. Increased content of fine-grained material can severely inhibit filtration capacity. Therefore it is important that material delivered to the site be checked to ensure that it meets the design specifications.
- Overcompaction of the sand media must be avoided to ensure adequate filtration capacity. Sand is best placed with a low pressure bulldozer (4 psig or less). The number of passes over the sand fill should be minimized during placement. Use of large rubber-tired vehicles can minimize ground pressure and compaction.
- Careful placement of the sand is necessary to avoid formation of voids within the sand that could lead to short-circuiting (particularly around penetrations for underdrain cleanouts) and to prevent damage to the underdrain system and associated geomembranes. Voids between the trench wall and geotextile fabric should also be avoided.

5.9.4 Operations and Maintenance Requirements

Operations and maintenance requirements for all sand filter BMPs are provided in Appendix D.

5.9.5 Basic and Large Sand Filter Basins

A sand filter basin contains a storage reservoir to temporarily store runoff so it can infiltrate into the sand media and sand filter bed through which runoff must infiltrate for treatment. The depth of storage over the filter media determines the hydraulic head and determines the rate of flow through the sand media. Figures 5.18 through 5.20b at the end of this section provide examples of various sand filter basin designs and configurations.

5.9.5.1 Site Considerations

See Section 5.9.2.3 previously. In addition, the location of the facility relative to site constraints (e.g., buildings, property lines, etc.) shall be the same as for wet ponds (see Section 5.10.3).

5.9.5.2 BMP Sizing and Design Criteria

See Section 5.9.2 for general sizing and design criteria applicable to all sand filters. Sand filters must be designed to capture and treat 91 percent of the total runoff volume (95 percent for large sand filter) as predicted by an approved continuous runoff model. Only 9 percent of the total runoff volume (5 percent for large sand filter) may bypass or overflow from the sand filter facility. The following design criteria apply to all sand filter BMPs, unless otherwise noted under the subsequent BMP descriptions for Sand Filter Vaults and Linear Sand Filters.

Two methods are provided for sizing sand filters (simplified method and modeling method), both of which are based on Darcy's law, which models flow through a porous media like sand or soil:

$$Q = KiA$$

Where:

Q = water quality design flow (cfs)
K = hydraulic conductivity of the media (fps)
A = surface area perpendicular to the direction of flow (sf)
i = hydraulic gradient (ft/ft) for a constant head and constant media depth

$$i = \frac{h + L}{L}$$

Where:
h = average depth of water above the filter (ft), defined as d/2
d = maximum water storage depth above the filter surface (ft)
L = thickness of sand media (ft)

Although it is not seen directly, Darcy's law underlies both the simple and the modeling design methods. V, or more correctly, 1/V, is the direct input in the sand filter design. The relationship between V and K is revealed by equating Darcy's law and the equation of continuity, Q = VA. (Note: When water is flowing into the ground, V is commonly called the filtration rate. It is ordinarily measured via a soil infiltration test.)

Specifically:

$Q = KiA$ and $Q = VA$ so,

$VA = KiA$ or $V = Ki$

Note that $V \neq K$. The filtration rate is not the same as the hydraulic conductivity, but they do have the same units (distance per time). K can be equated to V by dividing V by the hydraulic gradient i, which is defined above. The hydraulic conductivity K does not change with head nor is it dependent on the thickness of the media, only on the characteristics of the media and the fluid. The hydraulic conductivity of 1 inch per hour (2.315×10^{-5} fps) used in this design is based on bench-scale tests of conditioned rather than clean sand. This design hydraulic conductivity represents the average sand bed condition as silt is captured and held in the filter bed. Unlike the hydraulic conductivity, the filtration rate V changes with head and media thickness, although the media thickness is constant in the sand filter design. Table 5.18 shows values of V for different water depths d (d=2h).

Table 5.18. Sand Filter Design Parameters.

	Sand Filter Design Parameters					
Facility ponding depth d (ft)	1	2	3	4	5	6
Filtration rate V (in/hr) [a]	1.33	1.67	2.00	2.33	2.67	3.00
1/V (min/in)	45	36	30	26	22.5	20

[a] The filtration rate is not used directly, but is provided for information. V equals the hydraulic conductivity, K, times the hydraulic gradient, i. The hydraulic conductivity used is 1 inch/hr. The hydraulic gradient = (h + L)/L, where h = d/2 and L = the sand depth (1.5 ft).

Pre-Sized Method

The Pre-Sized method is taken directly from the *King County Surface Water Design Manual* (King County 1998). It uses standard values to define filter hydraulic characteristics for determining the sand surface area. This method is useful for planning purposes, for a first approximation to begin iterations in the modeling method, or when use of a computer model is not desired or not available. The simplified sizing method very often results in a larger filter than the modeling method. More robust calculation methods, using an approved continuous runoff model, may be used (see following section on modeling method).

King County (1998) developed the simple method to design sand filters that meet the required treatment volume without performing detailed modeling. Steps for the simple sizing procedure are summarized below.

Step 1: Determine maximum depth of water above sand filter. This depth is defined as the depth at which water begins to overflow the reservoir pond, and it depends on site topography and hydraulic constraints. The depth is chosen by the designer.

Step 2: Determine site characteristics. Determine the total number of impervious acres and the total number of grass acres draining to the sand filter. Determine whether the site is on till or outwash soils.

Step 3: Calculate minimum required surface area for the sand filter. Determine the sand filter area by multiplying the values in Table 5.19 by the site acreage from Step 2 using the following equation:

$$A_{sf} = 0.7(T_i A_i + T_{tg} A_{tg} + T_{og} A_{og})$$

Where:

A_{sf} = sand filter area (sf)
0.7 = adjustment factor to account for routing effect on size
$T_{i,tg,og}$ = tributary area per soil/cover type (acres)
$A_{i,tg,og}$ = filter area per soil/cover type (sf/acre) from Table 5.19.

Table 5.19. Sand Filter Area Increments for Various Soil and Cover Types.

Treatment Goal	Maximum Depth above filter (ft)	SOIL AND COVER TYPES [filter area (sf) / tributary area (acre)]		
		A_i Impervious	A_t Till Grass	A_{og} Outwash Grass
BASIC	6	760	160	140
	3	1,140	240	210
	1	1,711	360	314
LARGE	6	1,179	279	250
	3	1,769	419	370
	1	2,654	629	550
Forested areas may be ignored. Vegetated areas other than grass may still be represented as grass for the simple sizing method, or the detailed routing method may be employed using actual cover types.				

Note: The values in Table 5.19 were derived as follows. Flows were estimated using the KCRTS model for one acre of the cover types selected in the table. Darcy's law ($Q = Ki\ A$) was then used to determine sand filter area using this flow Q, the hydraulic gradient i for the various ponding depths given, and a hydraulic conductivity k of 2.3×10^{-5} fps (1 inch/hr). The hydraulic gradient i was calculated as $(h+l)/l$, where h = the average depth of water above the filter, taken to be the ponding depth $d/2$, and l = the thickness of the sand layer, which is 1.5 feet. The hydraulic conductivity represents a partially plugged sand condition found by bench-scale testing using successive trials with turbid water.

For depths between the values given in the table, areas can be interpolated. For depths outside the range presented in the table, the detailed modeling method must be used.

Step 4. Size the underdrain system. The underdrain system is sized to convey the peak filtered flows to the outlet. The design criteria presented below in the section on Underdrains can be used in lieu of analyzing conveyance capacity for feeder pipes. Strip drains, if used, must be analyzed for conveyance per manufacturer's specifications.

The collector pipe (i.e., the pipe collecting flows from the rest of the underdrain system) shall be sized to convey the 2-year, 15-minute peak flow with 1 foot of head above the invert of the upstream end of the collector pipe.

Intent: The underdrain must be able to remove standing water from beneath the sand. If standing water remains, the sand will remain saturated. This could cause oxygen depletion and reduced conditions in the sand, allowing some pollutants to become mobile and be released from the filter to downstream receiving waters.

Simple Method Sizing Example

For a site with 2 acres of impervious area and 2 acres of till grass draining to the sand filter, and 3 feet of head above the filter, the required sand area for a basic size sand filter would be found as follows:

	Site Areas		Table 5.19 Values for Basic Size	
	2 acres	x	1,140 sf/acre	= 2,280 sf
+	2 acres	x	240 sf/acre	= 480 sf
				= 2,760 sf

Because the site is located in Seattle, the “regional scale factor (see Step 1) is 1.0. Multiply 2,760 square feet by the 0.7 adjustment factor (see Step 4).

2,760 sf x 1.0 x 0.7 = 1,930 sf

The required sand bed area is therefore 1,930 square feet.

Note: Find the total facility area by adding 3H:1V side slopes for the 3-foot ponding depth plus extra vertical height to convey the 100-year flow. If the total pond depth is 3.5 feet, the sand filter will require a total land area of (44 ft + 10.5 ft) x (44 ft + 10.5 ft) = 2.970 square feet, plus access and setback requirements.

Modeling Method

When using continuous modeling to size a sand filter, apply the assumptions listed in Table 5.20. It is anticipated that available modeling programs will develop built-in modules to size sand filters.

Table 5.20. Sand Filter Design and Sizing Criteria.

Variable	Assumption
Precipitation Series	Seattle 158-year, 5-minute series
Computational Time Step	15-minutes
Inflows to Facility	Continuous model output for applicable water quality design flow rate and volume (see Section 5.9.2.3)
Ponding Depth	Maximum water depth over the filter media
Precipitation Applied to Facility	Yes
Evaporation Applied to Facility	Yes
Media depth	18 inches or other as designed
Sand Media Hydraulic Conductivity	1 inch per hour
Use Wetted Surface Area	Only if side slopes are 3H:1V or flatter

Sand Filter Geometry

- Any shape sand bed may be used, including circular or free-form designs. Note: The treatment process is governed by vertical flow, so short-circuiting is not a concern as with wetponds.
- Sand depth shall be 18 inches (1.5 feet) minimum
- Depth of storage over the filter media (d) shall be 6 feet maximum
- Side slopes for earthen/grass embankments must not exceed 3H:1V to facilitate mowing.
- Runoff to be treated by the sand filter must be pretreated (e.g., presettling basin or other pretreatment facility in Section 5.5).

Inlets and Outlets

- Inlet bypass and flow spreading structures (e.g., flow spreaders, weirs or multiple orifice openings) must be designed to capture the applicable design flow rate, minimize turbulence, and to spread the flow uniformly across the surface of the sand filter. Design guidelines for flow splitters and level spreaders are provided in Section 5.4.5. Stone riprap or other energy dissipation devices must be installed to prevent gouging of the sand medium and to promote uniform flow. Include emergency spillway or overflow structures (see additional guidelines in Chapter 4, Sections 4.6.3.2 and 4.6.7).
    - If the sand filter is curved or an irregular shape, a flow spreader shall be provided for a minimum of 20 percent of the filter perimeter.

- If the length-to-width ratio of the filter is 2:1 or greater, a flow spreader must be located on the longer side and for a minimum length of 20 percent of the facility perimeter.

Underdrains

Underdrains are required to allow the sand media to dry out between events. Types of underdrains include: a central collector pipe with lateral feeder pipes, or a geotextile drain strip in an 8-inch gravel backfill or drain rock bed, or longitudinal pipes in an 8-inch gravel backfill or drain rock with a collector pipe at the outlet end.). Design criteria are listed below.

- At least 8 inches of gravel backfill must be maintained over all underdrain piping or drain strip, and 6 inches must be maintained on either side to prevent damage by heavy equipment during maintenance. Either drain rock or gravel backfill may be used between pipes or drain strip. *Note: If drain strip is used, it may be easier to install the central collector pipe in an 8-inch trench filled with drain rock, making the cover over the drain strip and the collector pipe the same thickness. In this case the pipe shall be wrapped with geotextile to prevent clogging. Use the geotextile specification provided in Appendix F.*
- If a drain strip is used for lateral drainage, the strip must be placed at the slope specified by the manufacturer but at least at 0.5 percent. All drain strips must extend to the central collector pipe. Drain strips installations must be analyzed for conveyance because manufactured products vary in the amount of flow they are designed to handle.
- Upstream of detention facilities the underdrain piping must be sized to handle the 2-year return frequency flow indicated by an approved continuous runoff model (using a 5-minute time step). Downstream of detention facilities the underdrain piping must be sized for the 2-year return frequency release rate from the detention system. In both instances there must be at least 1 foot of hydraulic head above the invert of the upstream end of the collector pipe.
- Internal diameters of underdrain pipes must be a minimum of 6 inches and two rows of 3/8-inch holes spaced 6 inches apart longitudinally (maximum), with rows 120 degrees apart (laid with holes downward). Maximum perpendicular distance between two feeder pipes must be 10 feet. All piping is to be schedule 80 PVC or greater wall thickness. Drain piping could be installed in basin and trench configurations.
- Main collector underdrain pipe must be at a slope of 1.0 percent minimum
- A geotextile fabric (specifications in Appendix F) must be used between the sand layer and drain rock or gravel and placed so that 2 inches of drain rock/gravel is above the fabric. Drain rock must be 0.75 to 1.5 inch 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. Access for cleaning all underdrain piping must be provided. This may consist of installing cleanout ports, which tee into the underdrain system and surface above the top of the sand bed.

Sand Media

- Use hydraulic conductivity of 1 inch per hour when sizing sand filters.
- Sand bed depth shall be a minimum of 18 inches. The sand in a filter must consist of a medium sand meeting the size gradation (by weight) given in Table 5.21 below. The contractor must obtain a grain size analysis from the supplier to certify that the No. 100 and No. 200 sieve requirements are met. (Note: Standard backfill for sand drains, Wa. Std. Spec. 9-03.13, does not meet this specification and shall not be used for sand filters.)

Table 5.21. Sand Media 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

Source: *King County Surface Water Design Manual*, September 1998

Grass Cover

- No top soil may be added to sand filter beds because fine-grained materials (e.g., silt and clay) reduce the hydraulic capacity of the filter.
- Growing grass will require selecting species that can tolerate the demanding environment of the sand bed. Sand filters experience long periods of saturation during the winter wet season, followed by extended dry periods during the summer. Modeling predicts that sand filters will be dry about 60 percent of the time in a typical year. Consequently, vegetation must be capable of surviving drought as well as wetness.

  The grasses and plants listed in Table 5.22 are good choices for basin sides. They are facultative (i.e., they can tolerate fluctuations in soil water). These species can generally survive approximately 1 month of submersion while dormant in the winter (until about February 15), but they can withstand only about 1 to 2 weeks of submersion after mid-February.

Table 5.22. Recommended Plants for Saturated Areas.

RECOMMENDED PLANTS FOR POND SIDES	
Scientific Name	Common Name
Agrostis alba	Redtop
Agrostis palustris	Creeping bentgrass
Alopecurus pratensis	Meadow foxtail
Calamagrostis nutkaensis	Pacific reed grass
Glyceria borealis	Northern mannagrass
Holcus lanatus	Common velvet grass
Poa palustris	Fowl bluegrass
Poa pratensis	Kentucky bluegrass
Juncus acuminatus	Tapertip rush
Juncus effusus	Soft rush
RECOMMENDED PLANTS FOR POND BOTTOM (SAND SURFACE)	
Agrostis tenuis	Colonial bentgrass (Highland strain good)
Buchloe dactyloides	Buffalo grass
Festuca elatior	Tall fescue
Festuca elatior "Many Mustang", "Silverado"	Dwarf tall fescues
Festuca rubra	Red fescue
Lolium perenne	Perennial ryegrass
Zoysia tenuifolia	Korean grass
Trifolium repens	White lawn clover

Note: Other grasses may be used if recommended by a horticultural or erosion control specialist for the specific site.

The lower portion of Table 5.22 lists grass species that are good choices for the sand filter bottom. They can withstand summer drying and are fairly tolerant of infertile soils. In general, planting a mixture of three or more species is recommended. This ensures better coverage since tolerance of the different species is somewhat different, and the best adapted grasses will spread more rapidly than the others. Legumes, such as clover, fix nitrogen and hence can thrive in low-fertility soils such as sands. This makes them particularly good choices for planting the sand filter bed.

- A sport-field sod grown in sand may be used on the sand surface. No other sod may be used due to the high clay content in most sod soils.
- To prevent overuse that could compact and potentially damage the filter surface, permanent structures (e.g., playground equipment or bleachers) are not permitted. Temporary structures or equipment must be removed for filter maintenance.

- Low phosphorus fertilizers (such as formulations in the proportion 3: 1: 3 N-P-K or less) or a slow-release phosphorus formulation should be used.

Access

- An access road shall be provided to the inlet and outlet of a sand filter for inspection and maintenance purposes. Requirements for access roads are the same as for wet ponds (see Section 5.10.3).
- Include an access ramp with a slope not to exceed 7H:1V, or equivalent, for maintenance purposes at the inlet and the outlet of a surface filter. Consider an access port for inspection and maintenance.

5.9.5.3 Recommended Design Features

The following design features should be incorporated into sand filter designs where site conditions allow:

- A horticultural specialist should be consulted for advice on planting
- Seed should be applied in spring or mid to late fall unless irrigation is provided. If the filter is seeded during the dry summer months, surface irrigation is needed to ensure that the seeds germinate and survive. Seed should be applied at 80 lbs./acre.
- Slow-release fertilizers may be applied to improve germination
- A sand filter can add landscape interest and should be incorporated into the project landscape design. Interior side slopes may be stepped with flat areas to provide informal seating with a game or play area below. Perennial beds can be planted above the overflow water surface elevation. However, large shrubs and trees are not recommended because shading limits evaporation and can inhibit drying of the filter surface. In addition, falling leaves and needles can clog the filter surface, requiring more frequent maintenance. Note: Examples of areas with stepped side slopes can be found at the Ballard Locks in Seattle and at Luther Burbank Park on Mercer Island.

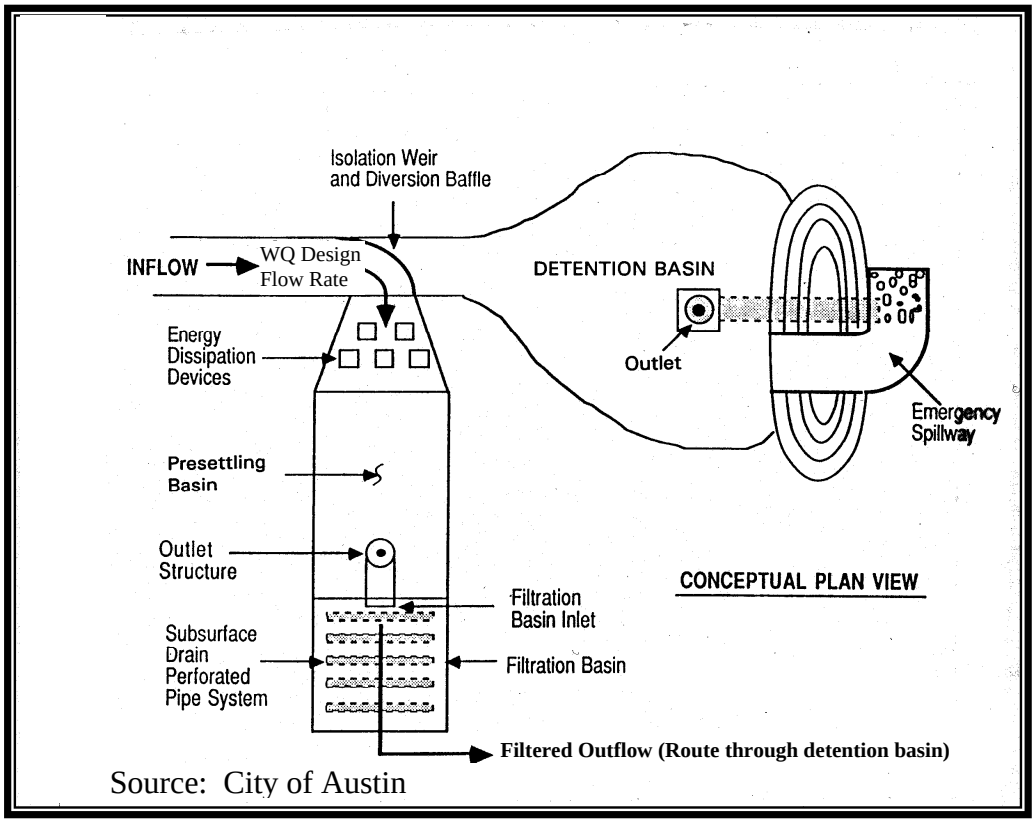


Figure 5.18. Sand Filtration Basin Preceded by Presettling Basin (Variation of a Basic Sand Filter).

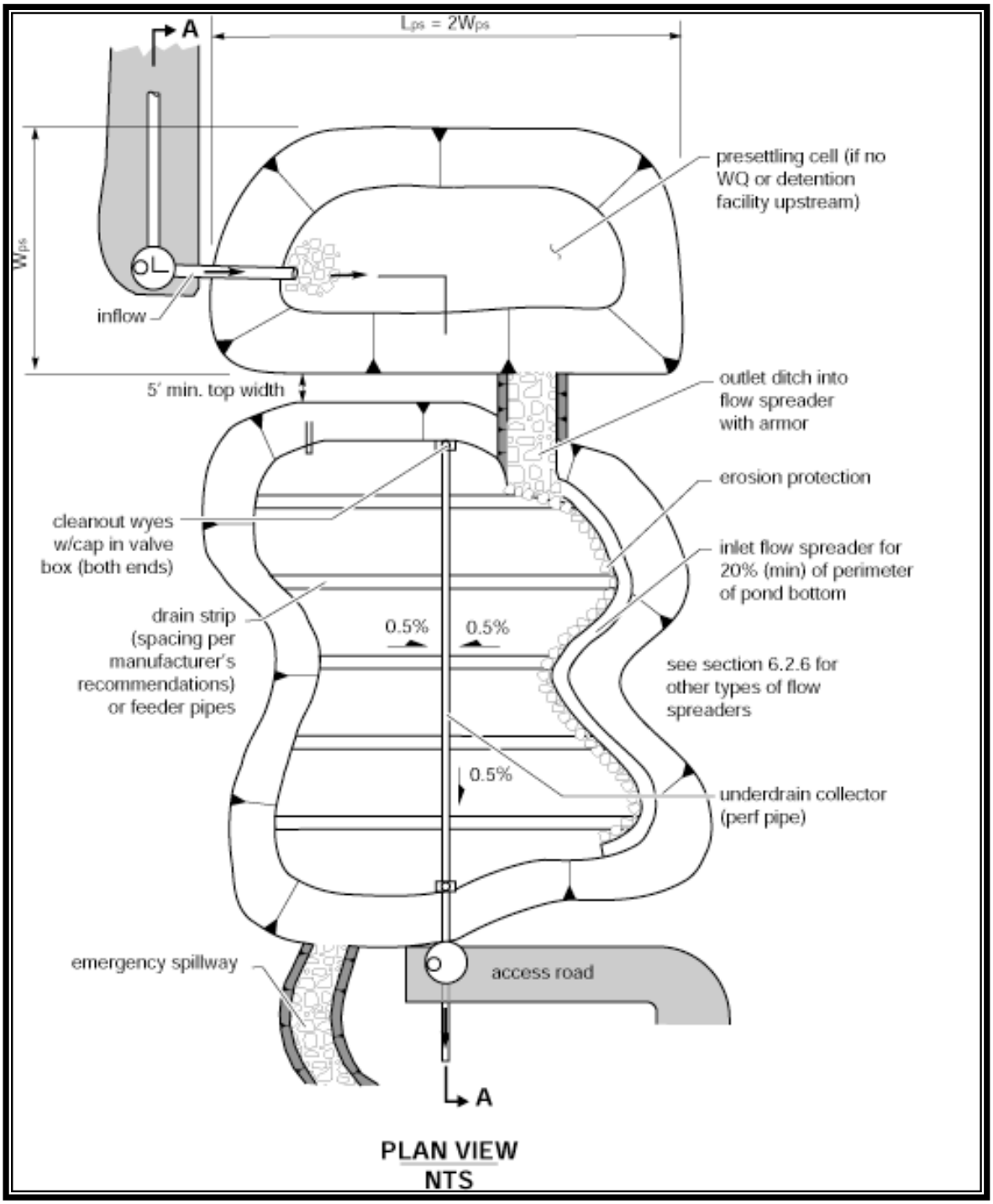


Figure 5.19a. Typical Sand Filter Basin with Pretreatment Cell.

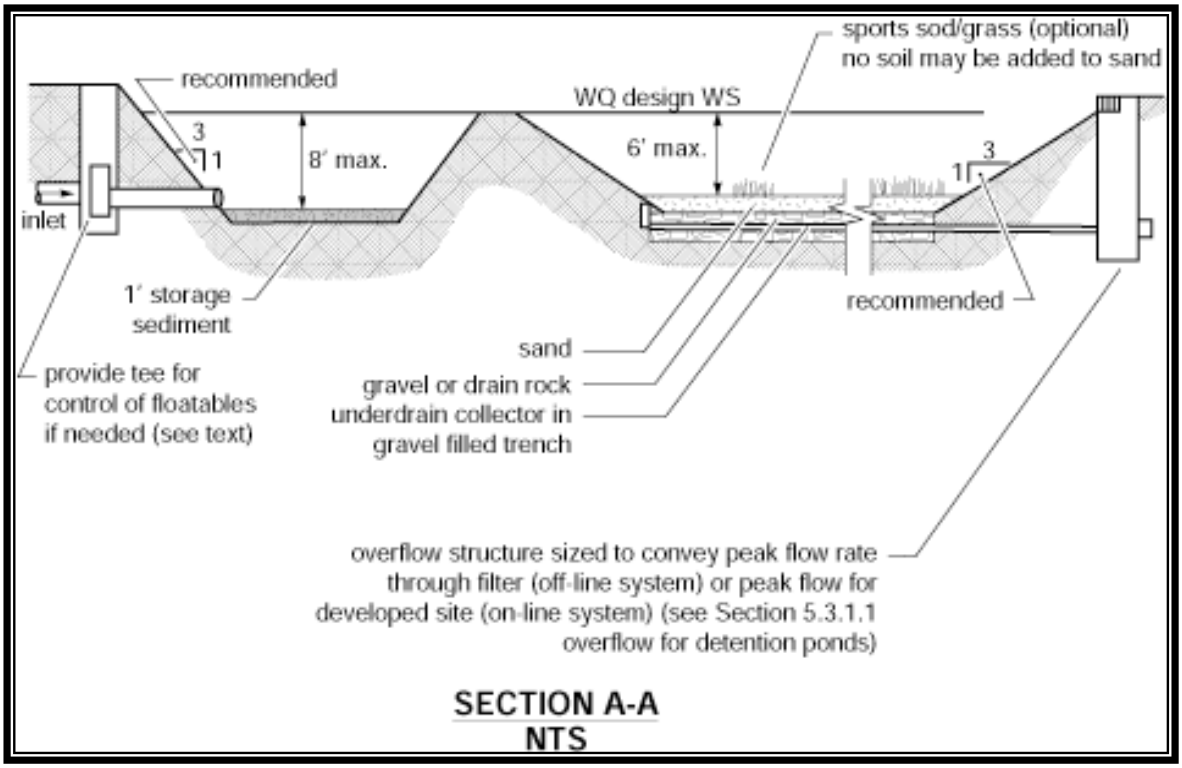


Figure 5.19b. Typical Sand Filter Basin with Pretreatment Cell.

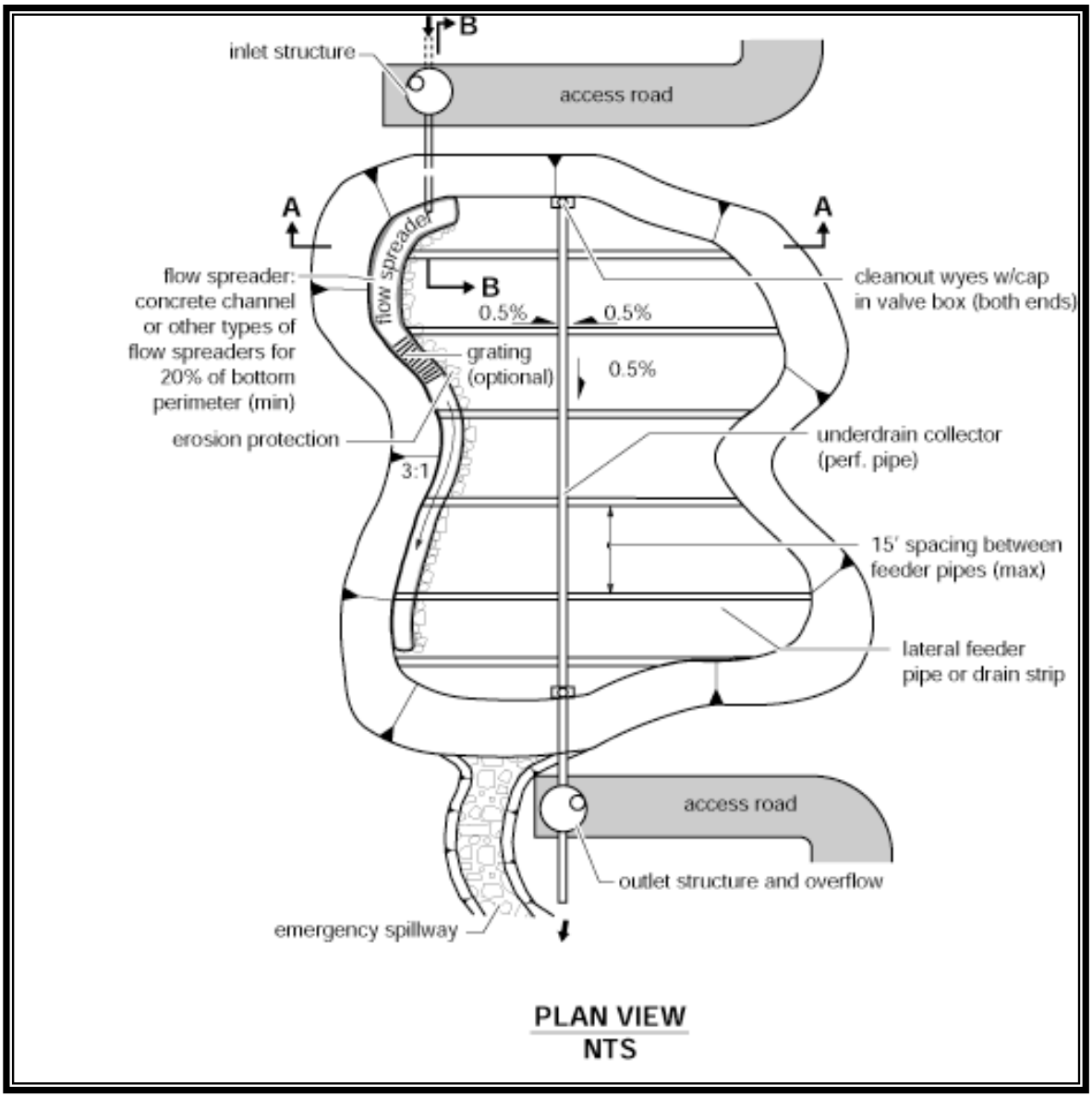


Figure 5.20a. Typical Sand Filter Basin with Level Spreader.

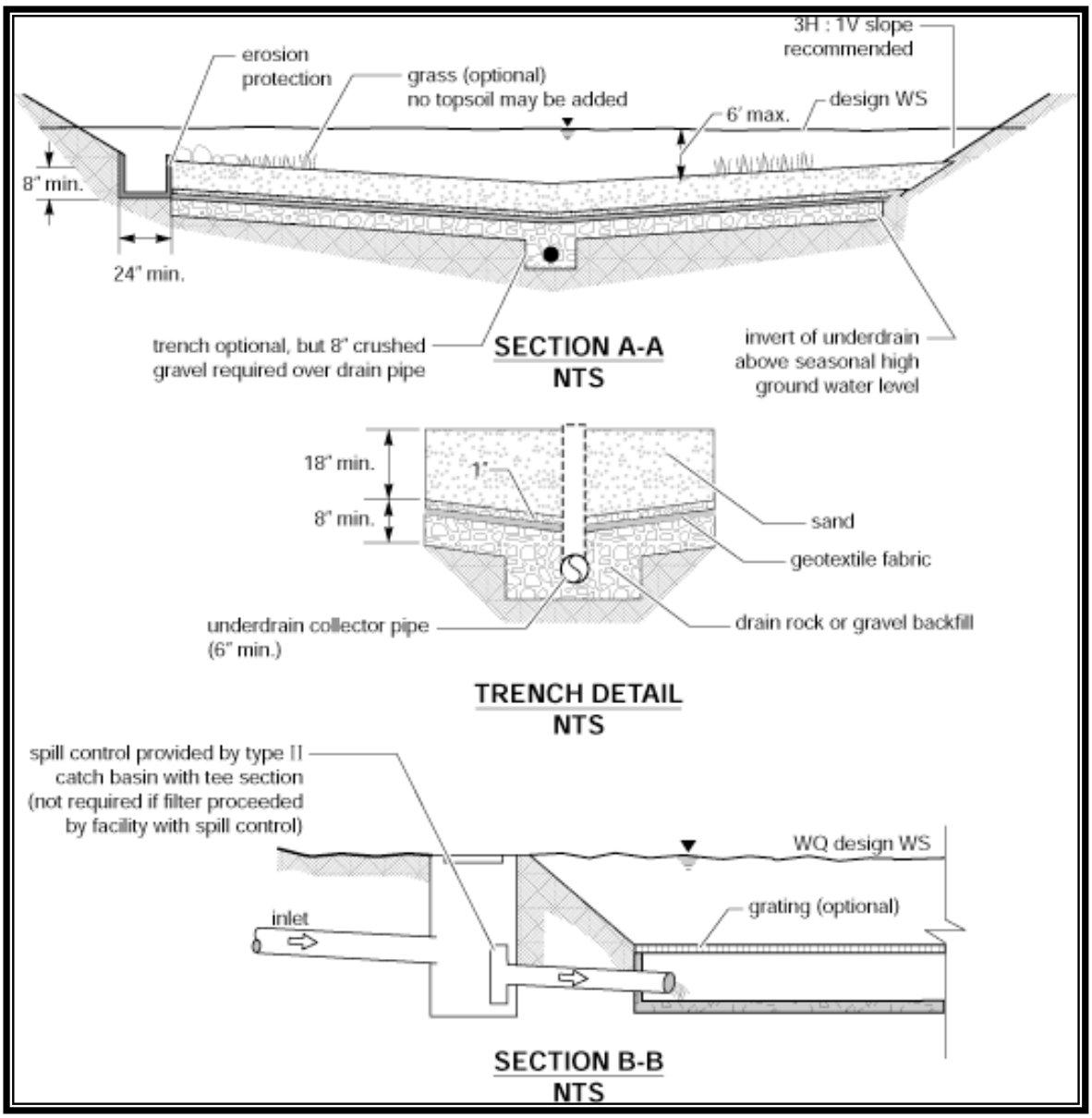


Figure 5.20b. Typical Sand Filter Basin with Level Spreader.

5.9.6 Sand Filter Vault

A sand filter vault is similar to a sand filter basin, except that the entire facility is installed below grade in a concrete vault. It typically consists of a presettling cell (if pretreatment is not already provided) and a sand filtration cell. Like a sand filter basin, a vault can be sized as either a basic or a large facility to meet different water quality objectives. The basic sand filter vault is designed to meet the basic water quality requirements (i.e., 80 percent TSS removal). The large sand filter vault is expected to meet the phosphorus control requirement (i.e., 50 percent total phosphorus removal). See Figures 5.21a and 5.21b at the end of this section for examples of various sand filter vault designs and configurations.

5.9.6.1 Applications and Limitations

See Section 5.9.1 for applications and limitations that apply to all filter systems. The following are additional requirements specific to sand filter vaults:

- Use where space limitations preclude above ground facilities
- Not suitable where high water table and heavy sediment loads are expected.

5.9.6.2 BMP Sizing and Design Criteria

Design criteria for sand filter vaults are the same as for sand filter basins (see Section 5.9.5). The following additional criteria apply specifically to sand filter vaults:

Vault Geometry and Composition

- The filter bed must consist of a sand top layer, and a geotextile fabric second layer with an underdrain system
- The geotextile fabric shall cover the entire sand bed and shall be flexible, highly permeable, three-dimensional matrix, and adequately secured (see also Appendix F). This is useful in trapping trash and litter.
- Provide a sand filter inlet shutoff/bypass valve for maintenance
- To prevent anoxic conditions, a minimum of 24 square feet of ventilation grate must be provided for each 250 square feet of sand bed surface area. For sufficient distribution of airflow 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.
- Sand filter vaults must conform to the materials and structural suitability criteria specified for wet vaults.

Presettling

- If a presettling cell is provided as an integral part of the vault, design the presettling cell for sediment collection and removal. A V-shaped bottom,

removable bottom panels, or equivalent sludge handling system shall be used. One-foot of sediment storage in the presettling cell must be provided. The cell bottom may be longitudinally level or inclined toward the inlet. To facilitate sediment removal, the bottom shall also slope from each side towards the center at a minimum of 5 percent, forming a broad "v".

Note: More than one "v" may be used to minimize cell depth.

- If a presettling cell is provided, the chamber must be sealed to trap oil and trash. This chamber is usually connected to the sand filtration chamber through an invert elbow to protect the filter surface from oil and trash.
- Where pipes enter and leave the presettling cell below the water quality design water surface, they shall be sealed using a non-porous, non-shrinking grout.
- If a retaining baffle is necessary for oil/floatables in the presettling cell, it must extend at least 1 foot above to 1 foot below the design flow water level. 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.

Inlet

- Optimize sand inlet flow distribution with minimal sand bed disturbance. A maximum of 8-inch distance between the top of the spreader and the top of the sand bed is suggested. Flows may enter the sand bed by spilling over the top of the wall into a flow spreader pad or 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. Design guidelines for level spreaders are provided in Section 5.4.5.2.
- If an inlet pipe and manifold system is used, the minimum pipe size must be 8 inches. 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 equivalent method, may be used.

Access

- Provision for access is the same as for wet vaults. However, the arch culvert sections allowed for wet vaults may not be used for sand filter vaults. Free access to the entire sand bed is needed for maintenance. Removable panels must be provided over the entire sand bed. Removable panels shall be at grade, have stainless steel lifting eyes, and weigh no more than 5 tons per panel.
- An access road shall be provided to the inlet and outlet of a sand filter for inspection and maintenance purposes.

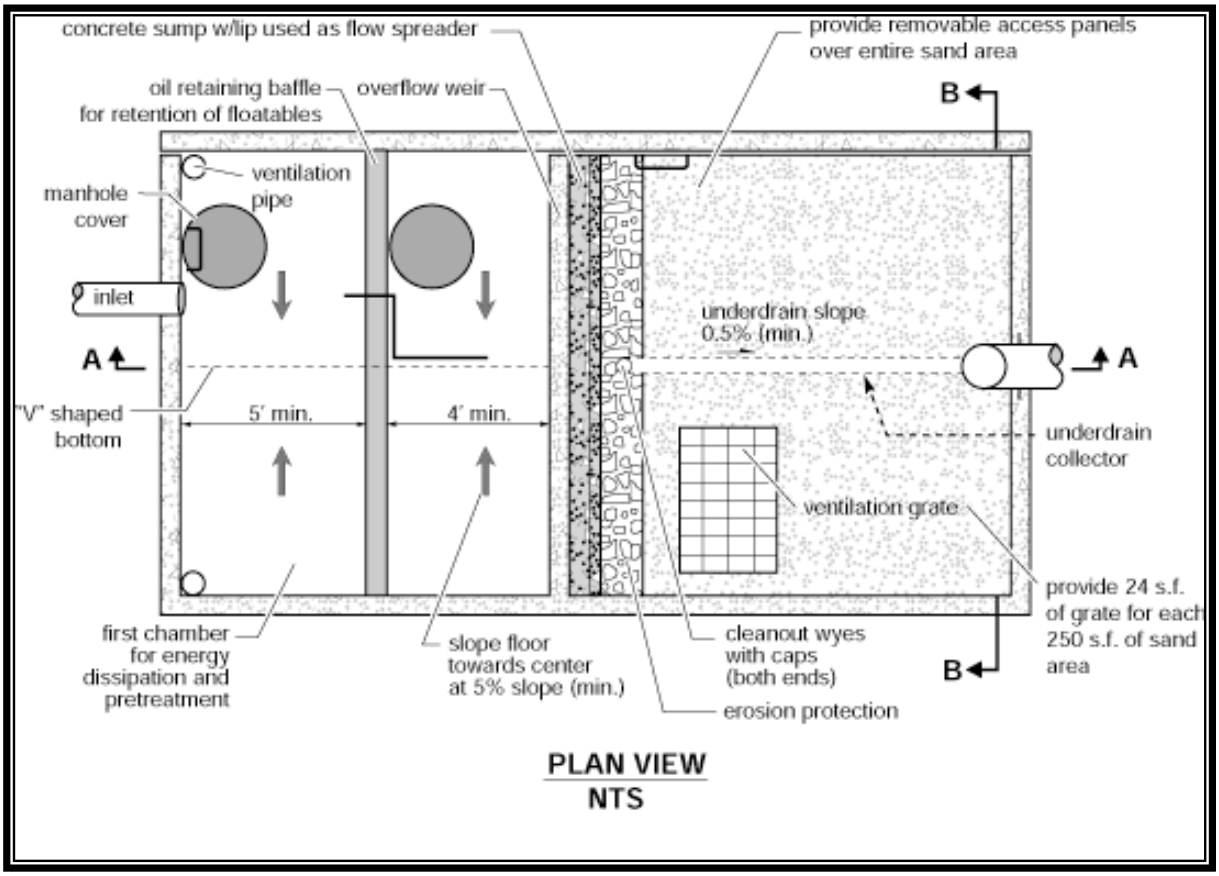


Figure 5.21a. Typical Sand Filter Vault.

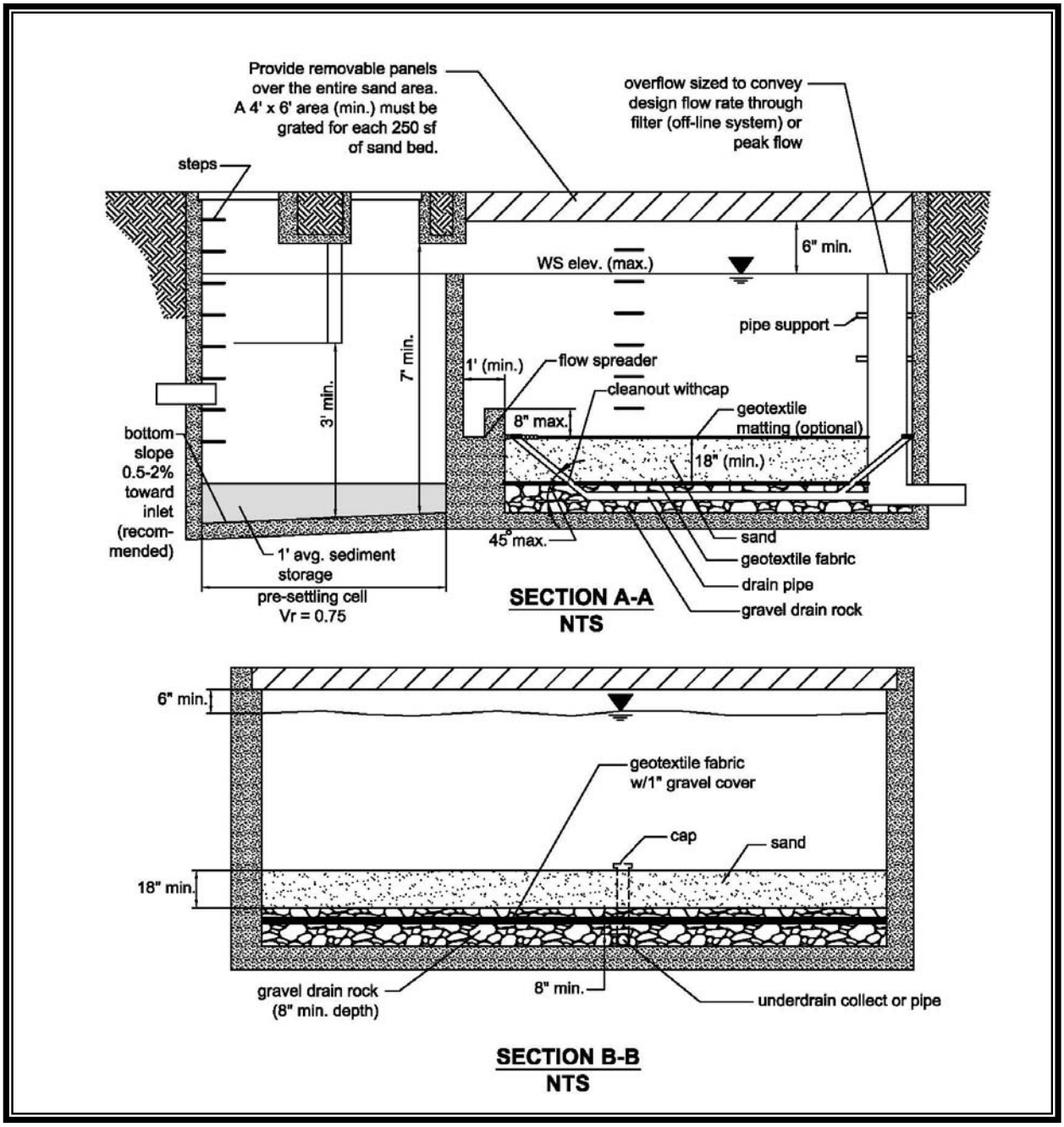


Figure 5.21b. Typical Sand Filter Vault (cont.).

5.9.7 Linear Sand Filter

Linear sand filters (see Figure 5.22 at the end of this section) are similar to sand filter vaults, except the vault is configured as a long, shallow, rectangular vault with a pretreatment cell or chamber for settling coarse particulates on one side and a treatment chamber with sand on the opposite side. Runoff usually enters the pretreatment cell as unconcentrated flow from an adjacent impervious area and overflows a central weir into the media portion of the vault.

5.9.7.1 Application and Limitations

See Section 5.9.1 for applications and limitations that apply to all filter systems. Linear sand filters are best suited for treating runoff from small sites (less than 2 acres of impervious area), particularly long, narrow paved areas. They are used primarily for the following applications:

- To provide basic treatment or as part of a treatment train
- To treat runoff from high use sites that generate higher than typical concentrations of oil and grease.

The goal is to keep linear sand filters fairly shallow and narrow in width. A linear sand filter can be located along the perimeter of a paved impervious surface or can be installed downstream of a filter strip where additional treatment is needed. If used for oil control, the filter should be located upstream from the main water quality treatment facility (e.g., wet pond, biofiltration swale, or combined detention/wet pond).

5.9.7.2 BMP Sizing and Design Criteria

Design criteria for linear sand filters are the same as for sand filter basins (see Section 5.9.5) unless otherwise specified below. The following additional criteria apply specifically to linear sand filters:

Vault Geometry and Composition

- The two cells must be divided by a divider wall that is level and extends a minimum of 12 inches above the sand bed
- Set sediment cell width as follows:

Sand filter width, (w) inches	12-24	24-48	48-72	72+
Sediment cell width, inches	12	18	24	w/3

- The sand filter bed must be a minimum of 12 inches deep and have an 8-inch layer of drain rock with slotted drainpipe beneath the sand layer
- The width of the sand cell must be 1 foot minimum to 15 foot maximum
- Maximum sand bed ponding depth shall be 1 foot

- If separated from traffic areas, a linear sand filter may be covered or open, but if covered, the cover must be removable for the entire length of the filter. Covers must be grated if flow to the filter is from sheet flow.

Inlet

- Stormwater may enter the sediment cell by sheet flow or a piped inlet
- The drainpipe must be 6-inch diameter minimum and be wrapped in geotextile and sloped a minimum of 0.5 percent

Structural Requirements

- A linear sand filter vault shall be concrete (precast/prefabricated or cast-in-place). The concrete must conform to the "Material" requirements for wetvaults (see Chapter 4, Section 4.6.5).
- At the discretion of DPD, the sediment cell may be made of materials other than concrete, provided water can be evenly spread for uniform delivery into the sand filter cell.
- Where linear sand filters are located in traffic areas, they must meet the structural requirements specified for wetvaults (see Section 5.10.4.2 and Chapter 4, Section 4.6.5). The sediment cell shall have a removable grated cover that meets HS-25 traffic loading requirements. The cover over the sand filter cell may be either solid or grated.
- A minimum of 24 square feet of ventilation grate must be provided for each 250 square feet of sand bed surface area. Grates located over the sediment chamber are preferred. Grates may be in one central location or dispersed over the entire sand bed. Vertical grates may also be used, such as at a curb inlet. Grates are important to allow air exchange above the sand. Poor air exchange will hasten anoxic conditions which may result in release of pollutants such as phosphorus and metals and cause objectionable odors.

5.9.7.3 Construction Specifications and Criteria

If put into service before the site is stabilized, placement of the sand layer must be delayed, and the linear sand filter may be used with the gravel layer only. The gravel layer must be replaced and the vault cleaned when the site is stabilized and the sand bed installed.

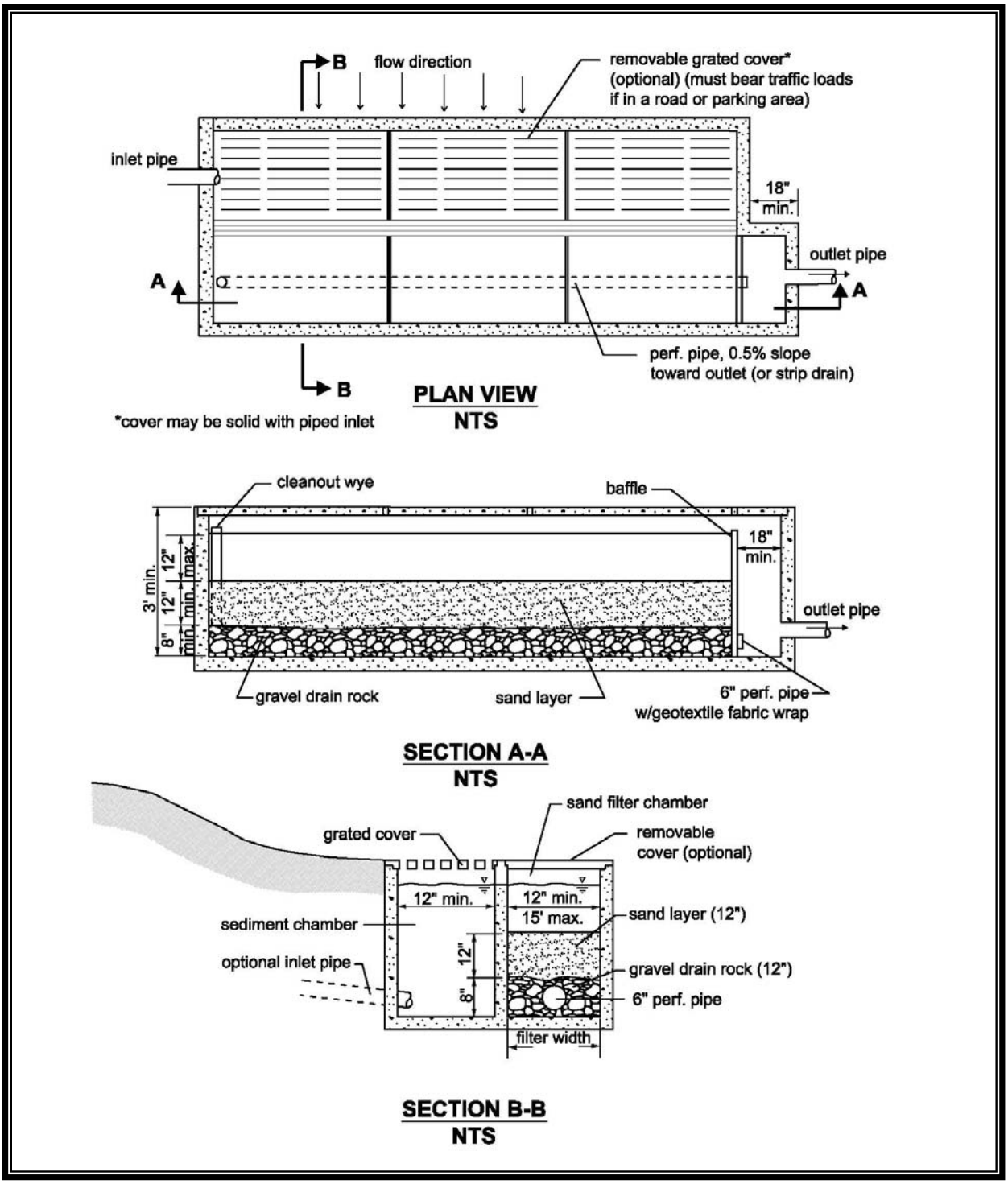


Figure 5.22. Typical Linear Sand Filter.

5.10 Wetpool Facilities

This section presents the methods, criteria, and details for analysis and design of wetponds, wetvaults, and stormwater treatment wetlands. These facilities have as a common element a permanent pool of water – the wetpool. Each of the wetpool facilities can be combined with a detention (flow control) facility to create a combined flow control and treatment facility (see Section 5.10.6). Included are the following specific facility designs:

- *Wetponds* – are stormwater ponds that retain a permanent pool of water (i.e., a wet pool or dead storage) at least during the wet season.
- *Wetvaults* – are similar to a wet pond, except the facility is constructed below grade in a concrete (or similar) vault.
- *Stormwater Treatment Wetlands* – are similar to wet ponds, but also provide a shallow marsh area to allow the establishment of emergent wetland aquatic plants, which improve pollutant removal.
- *Combined Detention and Wetpool Facilities* – can be either ponds or vaults that provide storage for both flow control and a wet pool for treatment. The volume required for flow control or "live storage" is provided above the treatment wet pool.

5.10.1 Applications and Limitations

The applications and limitations associated with each wetpool BMP vary depending on the BMP type. Detailed site suitability information is presented in the Site Considerations section for each BMP.

The four wetpool facility designs described in this Section will achieve the performance objectives cited in Section 5.3 for specific treatment menus. Pollutant removal in wet pool facilities occurs primarily by gravity settling. In open systems like ponds and constructed wetlands, biological uptake of phosphorus by algae and microorganisms can also occur. Pollutant removal efficiency is largely a function of the volume of the permanent pool. The larger the volume, the greater the potential for removal. The following provides additional general information on wetpool facility performance.

- There are two varieties of wetponds that provide different levels of pollutant removal: the basic wetpond (basic treatment only) and the large wetpond (basic and phosphorus treatment). A large wetpond requires a wetpool volume at least 1.5 times larger than for a basic wetpond.
- Wetvaults are believed to be ineffective in removing dissolved pollutants such as soluble phosphorus or metals. There is also concern that oxygen levels will decline, especially in warm summer months, because of limited contact with air and wind. However, the extent to which this potential problem occurs has not been documented.

- If oil control is required for a project, a wetvault may be combined with an API oil/water separator.
- Stormwater wetlands perform well to remove sediment, metals, and pollutants that bind to humic or organic acids. Phosphorus removal in stormwater wetlands is highly variable.

In general, wet pool facilities function best when stormwater entering the pond moves through the pond as a single wave or unit, fully displacing the wet pool volume, a phenomena known as plug flow. By preventing short-circuiting from occurring, this flow pattern maximizes the hydraulic retention time, which enhances particle settling. For proper function and to prevent sediment and associated pollutants from eroding and flushing out of wet pools during storms, it is also important that the permanent pool retain water, particularly during the wet season. Likewise, seepage of contaminated stormwater runoff could result in contamination of shallow or deep groundwater. As such, wet pond cells are required to be lined, as outlined in Section 5.4.4.

5.10.2 Operations and Maintenance Requirements

Operations and maintenance requirements for wet pool BMPs are provided in Appendix D.

5.10.3 Wetponds – Basic and Large

Wetponds are constructed stormwater ponds that retain a permanent pool of water (i.e., a wet pool or dead storage) at least during the wet season. The volume of the wet pool, which slows down the velocity of incoming stormwater, allowing particulates and particulate-bound pollutants to settle, is a key factor in determining wet pond effectiveness. Figures 2.23a and 5.23b illustrate a typical wetpond BMP.

5.10.3.1 Site Considerations

Wetponds require a larger area than a biofiltration swale or a sand filter, but can be integrated to the contours of a site fairly easily and function well for any size project. Wetponds may be single-purpose facilities, providing only runoff treatment, or they may be combined with a detention pond to also provide flow control. If combined, the wetpond storage can often be situated under the detention pond with little loss of development area. See Section 5.10.6 for a description of combined detention and wetpool facilities.

Wetponds are also subject to the dam safety requirements outlined in Section 4.6.3.2 under detention ponds.

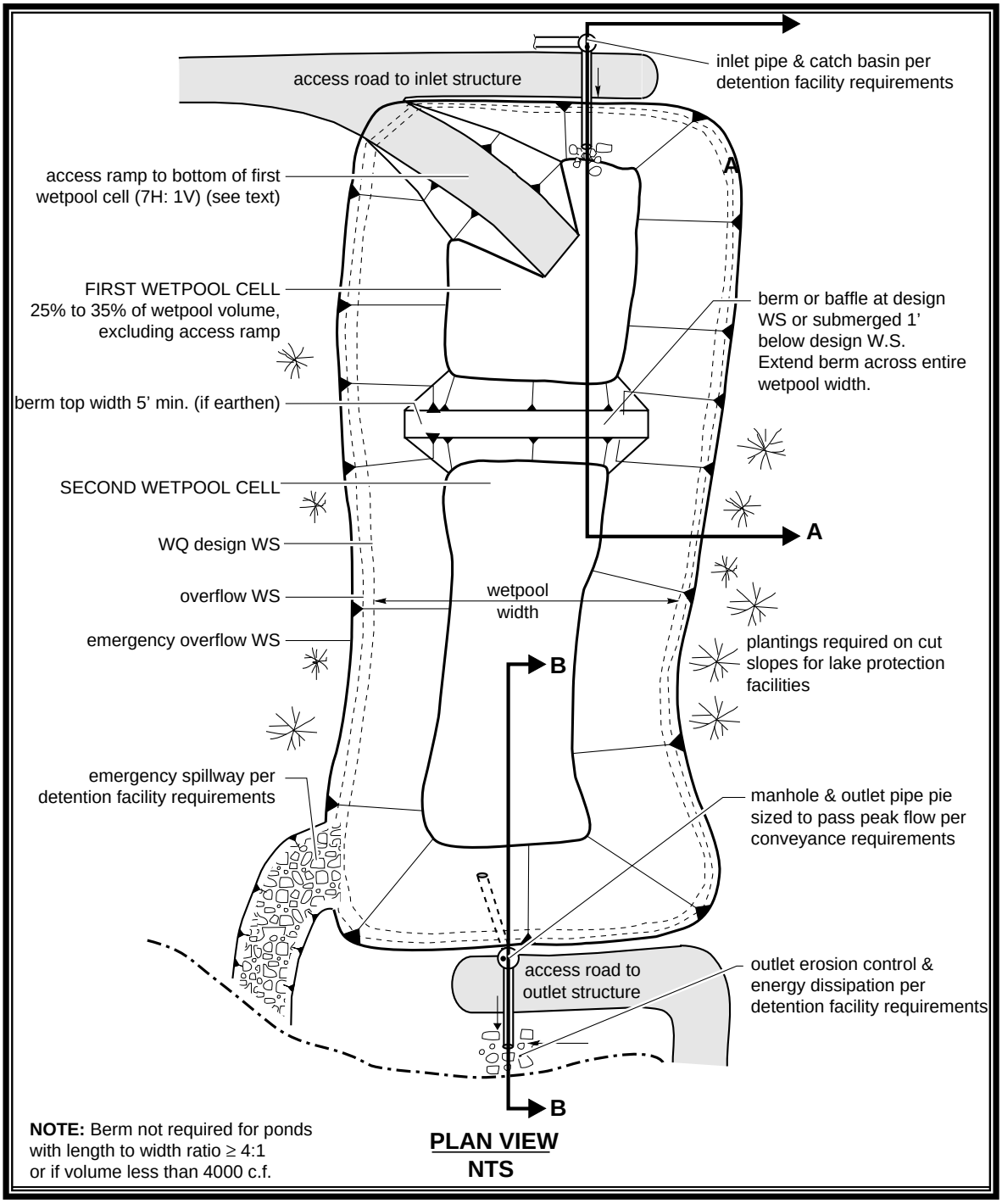


Figure 5.23a. Typical Wetpond.

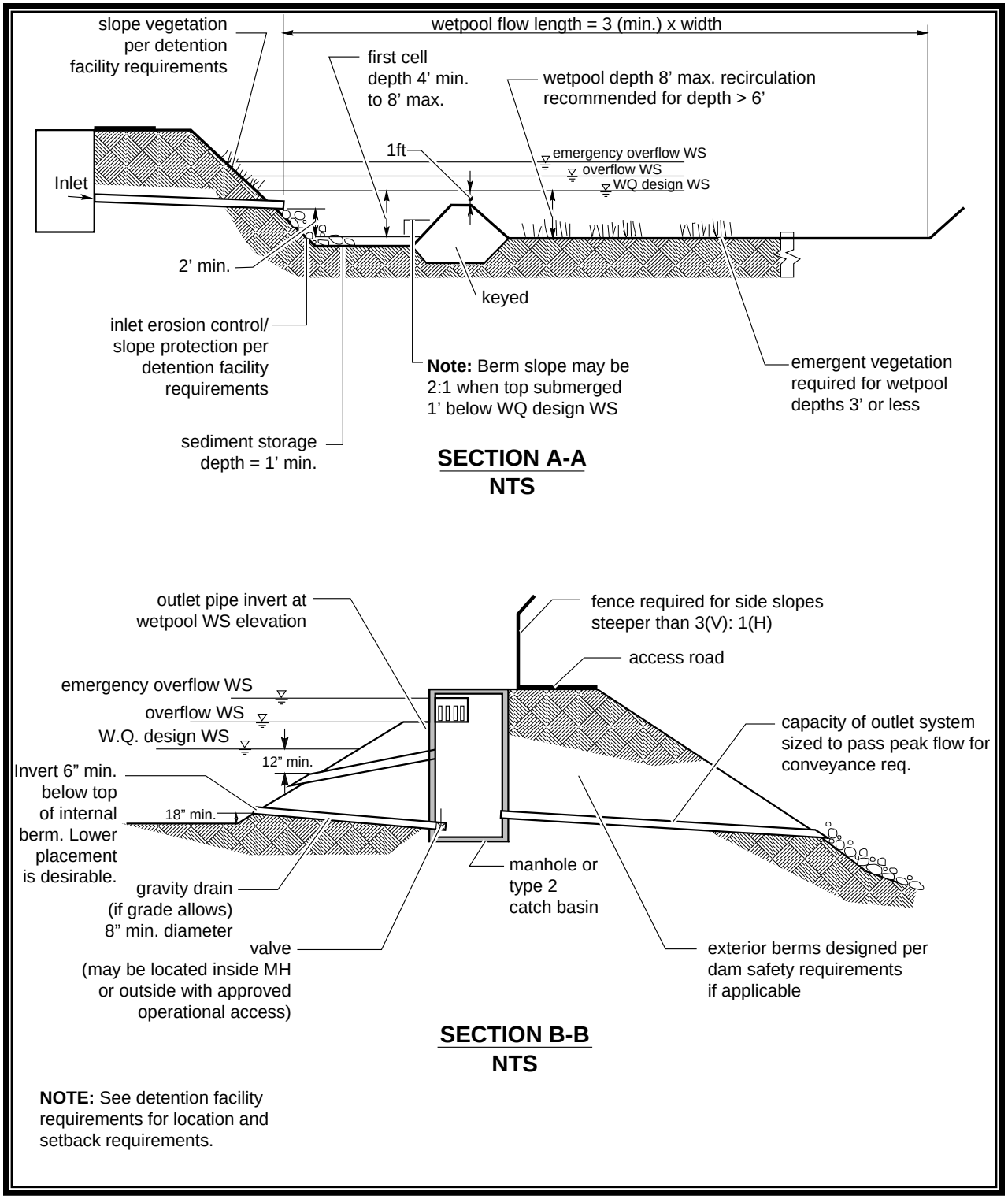


Figure 5.23b. Typical Wetpond.

Setbacks

The location of the pond relative to site constraints (e.g., buildings, property lines, etc.) shall meet the following criteria:

- A setback of 5 feet from the toe of the exterior slope of the tract or property line
- The tract or property line on a pond cut slope shall be setback 5 feet from the emergency overflow water surface
- The pond water surface at the outlet invert elevation shall be setback 100 feet from existing septic system drainfields**.** This setback may be reduced with written approval of the Seattle-King County Department of Public Health.
- Geotechnical analysis is required for facilities within 20 feet from any structure or property line or within 50 feet up-slope of a building when the slope is greater than 5 percent.
- All facilities must be a minimum of 50 feet from the top of any slope steeper than 5 percent. A geotechnical analysis and report must be prepared addressing the potential impact of the facility on the slope.

5.10.3.2 BMP Sizing and Design Criteria

Table 5.23 summarizes design and sizing criteria for basic and large wetponds. More detailed information on specific design elements are summarized below.

Table 5.23. Wetpond Design and Sizing Criteria.

Design Parameter	Basic, and Large Wetpond
Wetpool volume	Total water quality volume, or 1.5 times the total water quality volume[1] for large wetponds
Volume of first cell	25-35% total volume[1]
Minimum flowpath length to width ratio	3:1
Baffle or berm	Required unless volume ≤4,000 cf and flowpath length:width ≥4:1
Minimum sediment storage depth[2]	1’ (6” for one-cell pond)
Minimum depth of first cell	4’ (excluding sediment storage depth)
Maximum depth of both cells[3]	8’ (excluding sediment storage depth)

[1] The berm/baffle volume shall not count as part of the total wetpool volume

[2] A fixed sediment depth monitor shall be installed in the first cell to gauge sediment accumulation unless an alternative gauging method is proposed

[3] For wetpool depths in excess of 6 feet, it is recommended that some form of recirculation be provided in the summer, such as a fountain, aerator, or a small amount of base flow, to prevent stagnation and low dissolved oxygen conditions.

Design Procedures

Procedures for determining a wetpond's dimensions and volume are outlined below.

Step 1 – Identify required wetpool volume using the methods presented in Section 5.4.1, and Chapter 6. A basic wetpond requires a volume equal to or greater than the daily runoff volume at or below which 91 percent of the total runoff volume for the simulation period occurs, as determined using an approved continuous model. A large wetpond requires a volume at least 1.5 times the daily runoff volume at or below which 91 percent of the total runoff volume for the simulation period occurs.

Step 2 – Determine wetpool dimensions. Determine the wetpool dimensions satisfying the design criteria outlined below and illustrated in Figures 5.23a and 5.23b. A simple way to check the volume of each wetpool cell is to use the following equation:

$$V = \frac{h(A_1 + A_2)}{2}$$

Where V = wetpool volume (cf)
h = wetpool average depth (ft)
A_1 = water quality design surface area of wetpool (sf)
A_2 = bottom area of wetpool (sf).

Step 3 – Design the pond outlet pipe and determine the primary overflow water surface. The pond outlet pipe shall be placed on a reverse grade from the pond's wetpool to the outlet structure. Use the following procedure to design the pond outlet pipe and determine the primary overflow water surface elevation:

- Use the nomographs in Figures 5.24 and 5.25 to select a trial size for the pond outlet pipe sufficient to pass the on-line water quality design flow, Qwq indicated by an approved continuous runoff model
- Use Figure 5.26 to determine the critical depth dc at the outflow end of the pipe for Qwq
- Use Figure 5.27 to determine the flow area Ac at critical depth
- Calculate the flow velocity at critical depth using continuity equation (Vc = Qwq /Ac)
- Calculate the velocity head V_H ($V_H = Vc^2 /2g$, where g is the gravitational constant, 32.2 feet per second)
- Determine the primary overflow water surface elevation by adding the velocity head and critical depth to the invert elevation at the outflow end of the pond outlet pipe (i.e., overflow water surface elevation = outflow invert + dc + VH)
- Adjust outlet pipe diameter as needed and repeat Steps (a) through (e).

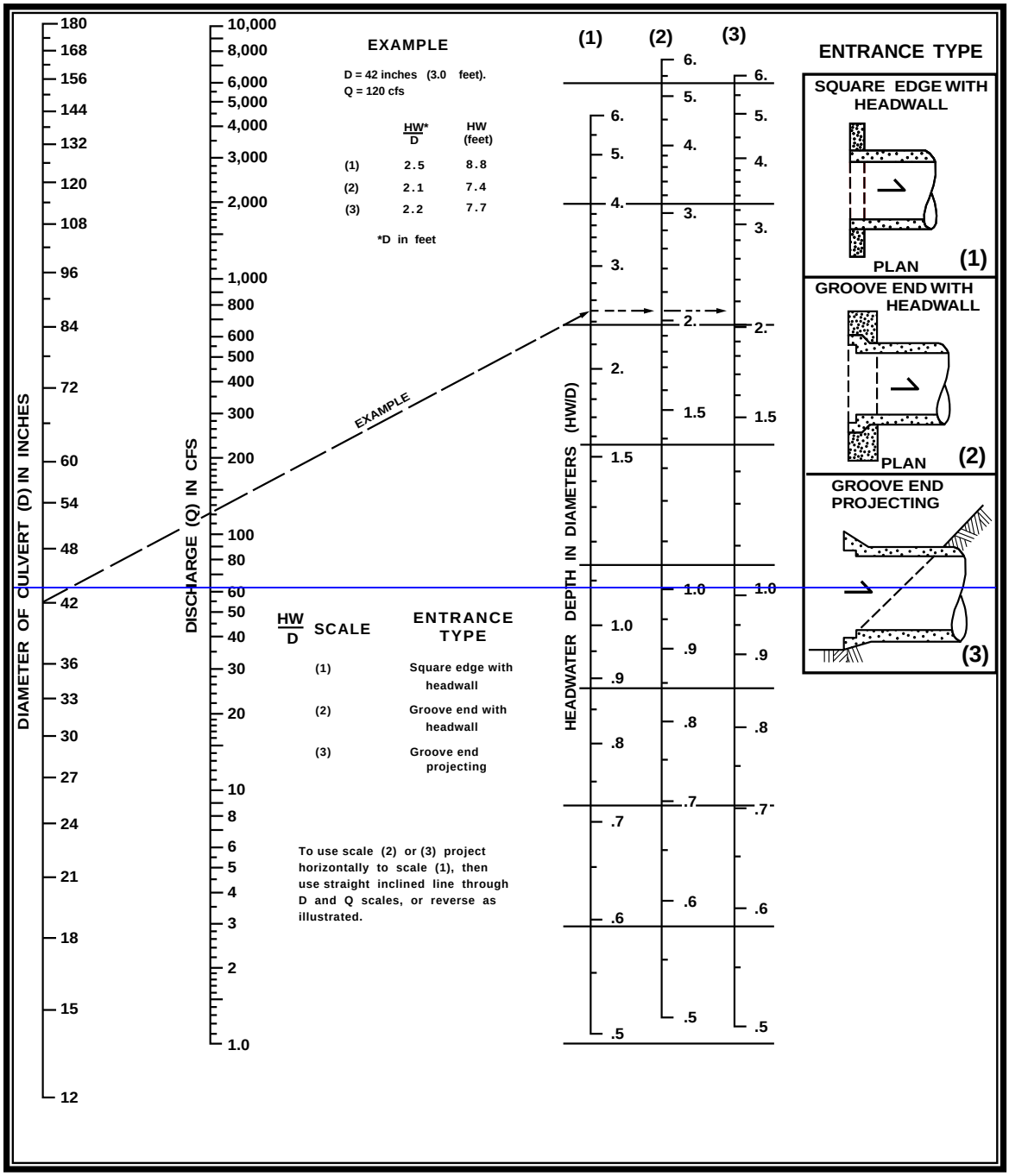


Figure 5.24. Headwater Depth for Smooth Interior Pipe Culverts with Inlet Control.

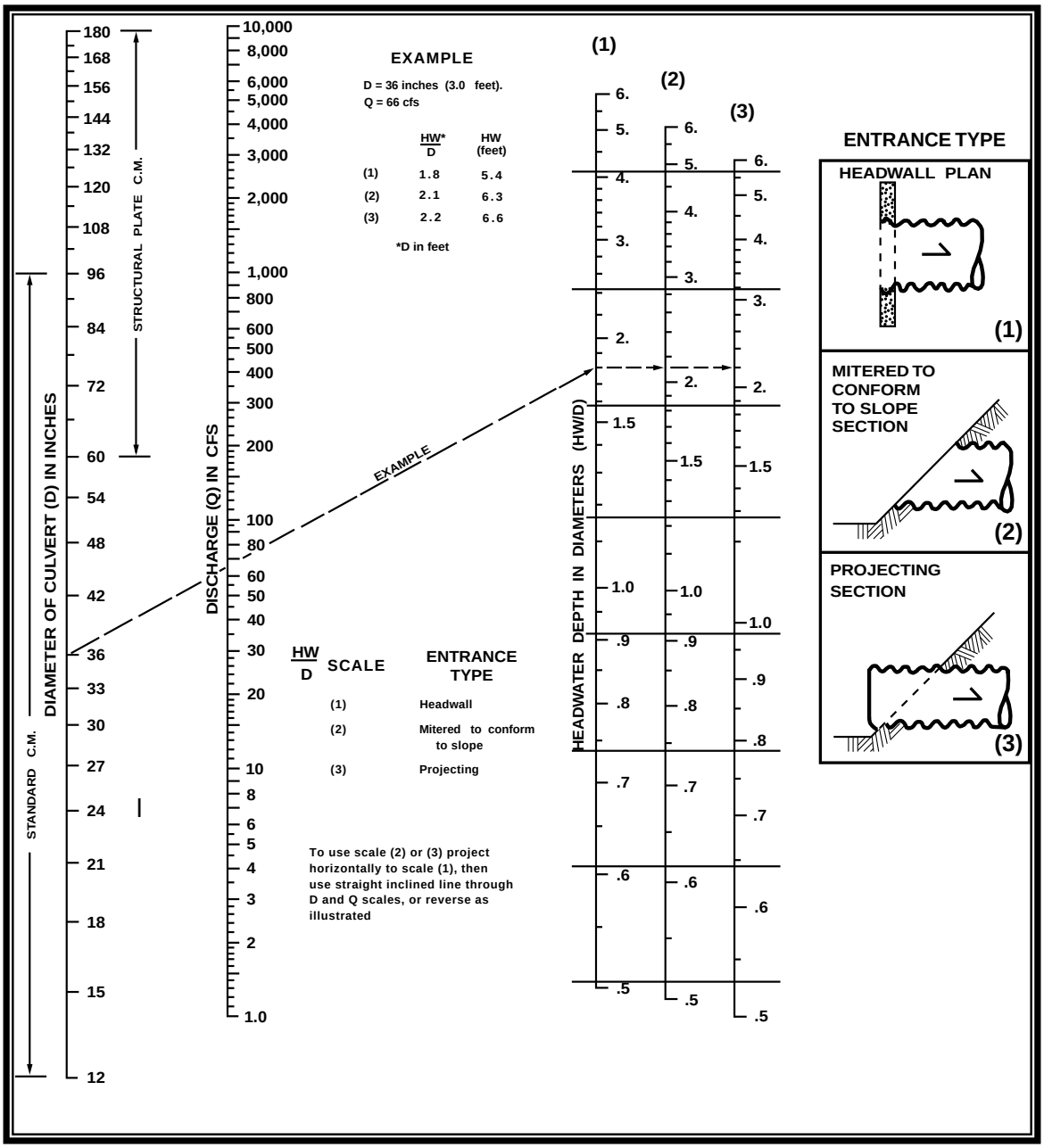


Figure 5.25. Headwater Depth for Corrugated Pipe Culverts with Inlet Control.

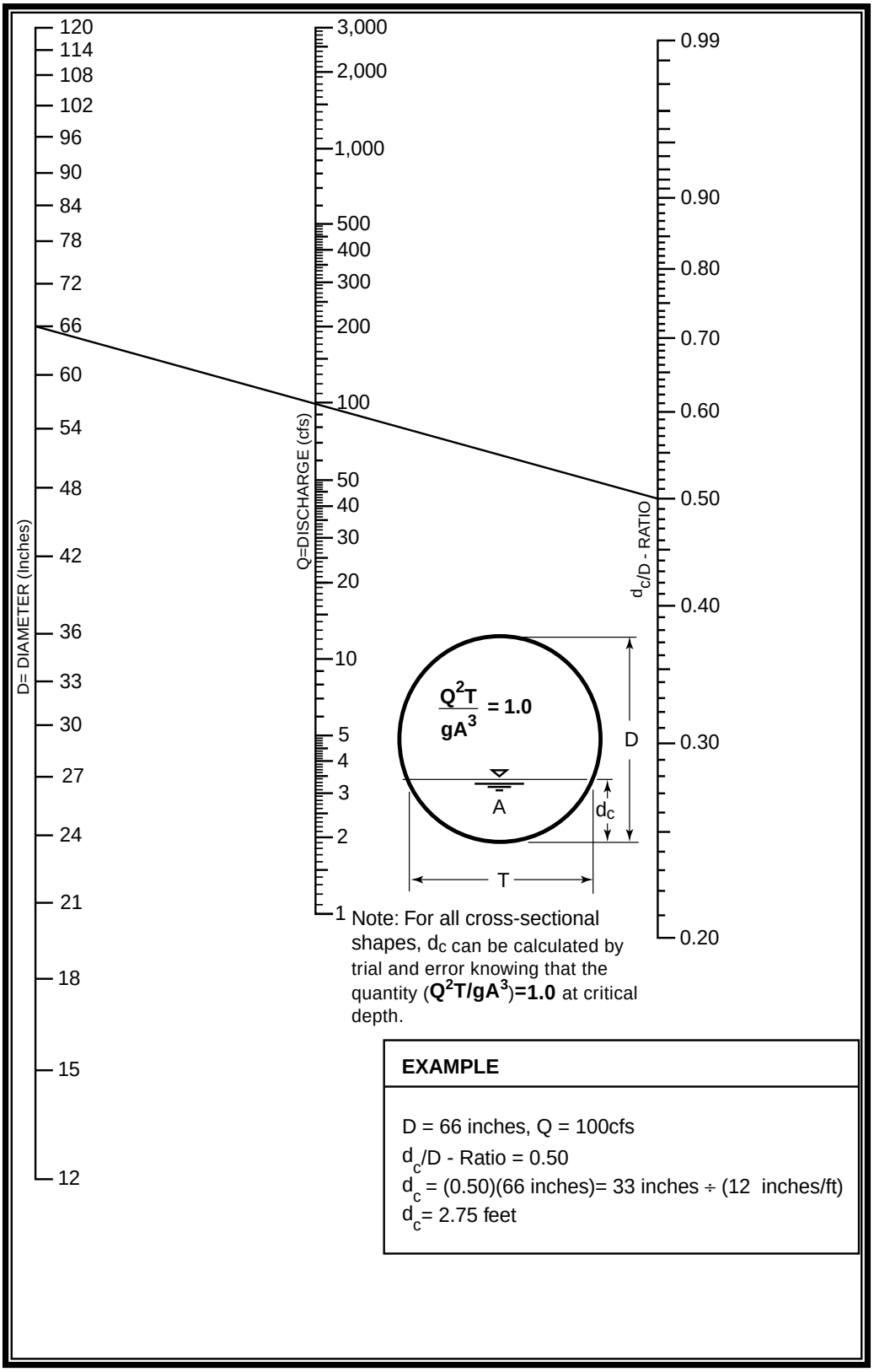


Figure 5.26. Critical Depth of Flow for Circular Culverts.

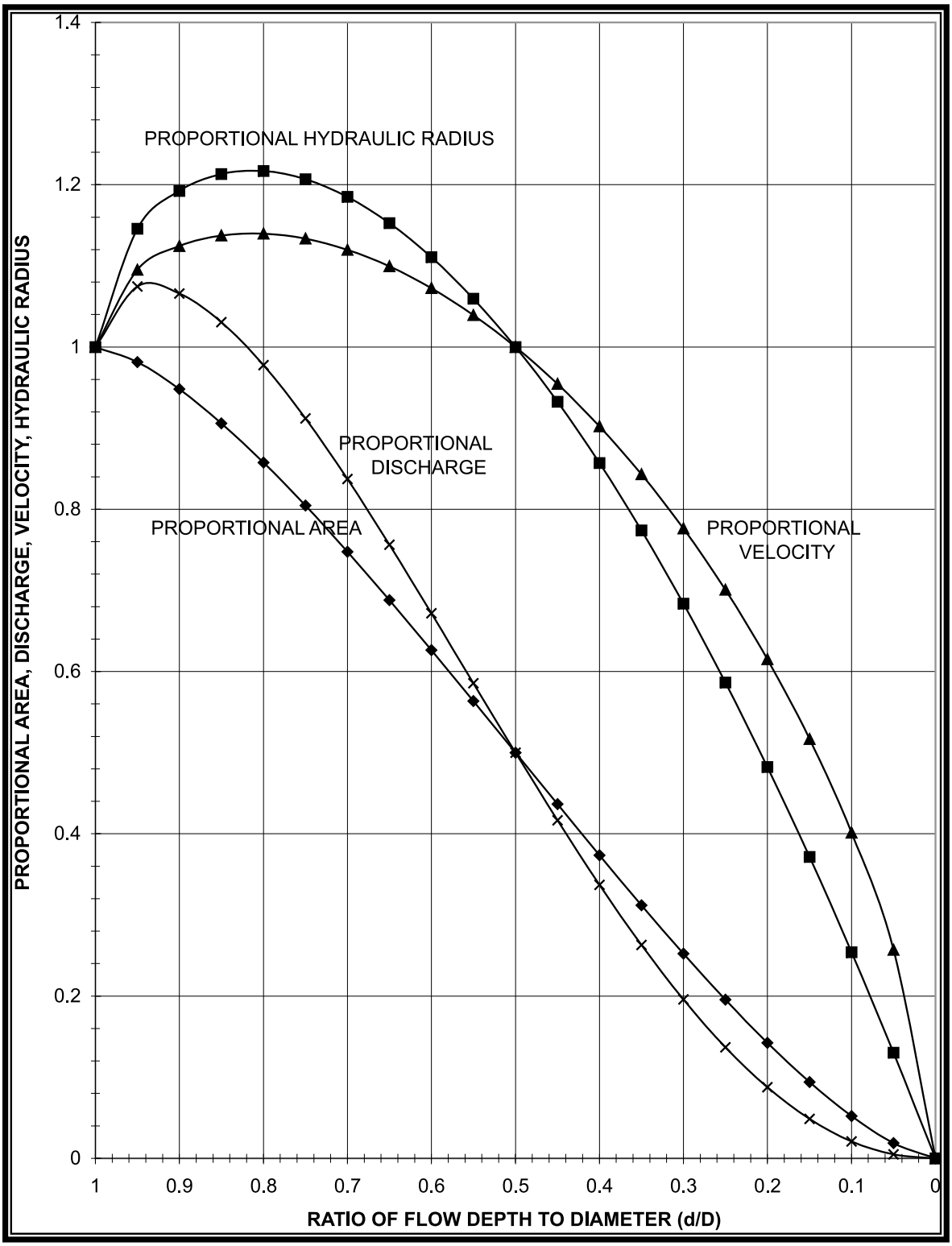


Figure 5.27. Circular Channel Ratios.

Step 4 – Determine wetpond dimensions. General wetpond design criteria and concepts are shown in Figures 5.23a and 5.23b.

Step 5 – Confirm that overflow meets the Overflow Structure requirements outlined below.

Wetpool Geometry

- The wetpool shall be divided into two cells separated by a baffle or berm. The first cell shall contain between 25 to 35 percent of the total wetpool volume. The baffle or berm volume shall not count as part of the total wetpool volume. The term baffle means a vertical divider placed across the entire width of the pond, stopping short of the bottom. A berm is a vertical divider typically built up from the bottom, or if in a vault, connects all the way to the bottom.

  Intent. The full-length berm or baffle promotes plug flow and enhances quiescence and laminar flow through as much of the entire water volume as possible. Alternative methods to the full-length berm or baffle that provide equivalent flow characteristics may be approved on a case-by-case basis by the City of Seattle.

- Sediment storage shall be provided in the first cell. The sediment storage shall have a minimum depth of 1 foot. A fixed sediment depth monitor shall be installed in the first cell to gauge sediment accumulation unless an alternative gauging method is proposed.

- The minimum depth of the first cell shall be 4 feet, exclusive of sediment storage requirements. The depth of the first cell may be greater than the depth of the second cell.

- The first and second cells must be lined in accordance with the liner requirements contained in Section 5.4.4.

- Maximum pond depth (excluding sediment storage) must not exceed 8 feet. Deep ponds (greater than 8 feet) may stratify during summer and create low oxygen conditions near the bottom resulting in re-release of phosphorus and other pollutants back into the water.

- The ratio of flowpath length to width from the inlet to the outlet shall be at least 3:1. The flowpath length is defined as the distance from the inlet to the outlet, as measured at mid-depth. The width at mid-depth can be found as follows: width = (average top width + average bottom width)/2.

- Wetponds with wetpool volumes less than or equal to 4,000 cubic feet may be single celled (i.e., no baffle or berm is required). However, it is especially important in this case that the flow path length be maximized. The ratio of flow path length to width shall be at least 4:1 in single celled wetponds, but should preferably be 5:1. In addition, a gravity drain for maintenance must be provided 12 to 18 inches from the pond bottom.

Berms and Baffles

A berm or baffle shall extend across the full width of the wetpool, and tie into the wetpond side slopes. If the berm embankments are greater than 4 feet in height, the berm must be constructed by excavating a key equal to 50 percent of the embankment cross-sectional height and width. This requirement may be waived if recommended by a geotechnical engineer for specific site conditions. The geotechnical analysis shall address situations in which one of the two cells is empty while the other remains full of water.

The top of the berm may extend to the water quality design water surface or be 1 foot below the water quality design water surface. If at the water quality design water surface, berm side slopes should be 3H:1V. Berm side slopes may be steeper (up to 2H:1V) if the berm is submerged 1 foot.

Intent. Submerging the berm is intended to enhance safety by discouraging pedestrian access when side slopes are steeper than 3H:1V. An alternative to the submerged berm design is the use of barrier planting to prevent easy access to the divider berm in an unfenced wetpond.

The interior berm or baffle may be a retaining wall provided that the design is prepared and stamped by a licensed civil engineer. If a baffle or retaining wall is used, it must be submerged 1 foot below the design water surface to discourage access by pedestrians.

Criteria for wetpond side slopes are included in Section 5.4.3.2, but generally should not exceed 3H:1V.

Berm embankments shall meet the following additional criteria:

- Pond berm embankments higher than 6 feet shall require design by a geotechnical engineer.
- For berm embankments 6 feet or less, the minimum top width shall be 6 feet, or as recommended by a geotechnical engineer.
- Pond berm embankments must be constructed on native consolidated soil (or adequately compacted and stable fill soils analyzed by a geotechnical engineer) free of loose surface soil materials, roots, and other organic debris.
- The berm embankment shall be constructed of compacted soil (95 percent minimum dry density, modified proctor method per ASTM D1557), placed in 6-inch lifts, with the following soil characteristics per the United States Department of Agriculture's Textural Triangle: a minimum of 20 percent silt and clay, a maximum of 60 percent sand, a maximum of 60 percent silt, with nominal gravel and cobble content. *Note: In general, excavated glacial till is well suited for berm embankment material.*

- Anti-seepage collars must be placed on outflow pipes in berm embankments impounding water greater than 8 feet in depth at the design water surface.

Inlet and Outlet

See Figures 5.26 and 5.27 for details on the following requirements:

- All inlets shall enter the first cell. The number of inlets to the facility should be limited; ideally there should be only one inlet. Inlets and outlets shall be placed to maximize the flowpath through the facility. If there are multiple inlets, the required length-to-width ratio (see geometry requirements above) shall be based on the average flowpath length for all inlets.

- The inlet to the wetpond shall be submerged with the inlet pipe invert a minimum of 2 feet from the pond bottom (not including sediment storage). The top of the inlet pipe should be submerged at least 1 foot, if possible.

  Intent. The inlet is submerged to dissipate energy of the incoming flow. The distance from the bottom is set to minimize resuspension of settled sediments. Alternative inlet designs that accomplish these objectives are acceptable.

- An outlet structure shall be provided. Either a Type 2 catch basin with a grated opening (jail house window) or a manhole with a cone grate (birdcage) may be used. No sump is required in the outlet structure for wetponds not providing detention storage. The outlet structure receives flow from the pond outlet pipe. The grate or birdcage openings provide an overflow route should the pond outlet pipe become clogged (see "Overflow Structure" requirements below).

- The pond outlet pipe (as opposed to the manhole or Type 2 catch basin outlet pipe) shall be back-sloped or have a turn-down elbow, and extend 1 foot below the water quality design water surface. At a minimum, the outlet pipe shall be sized to pass the water quality design flow. Note: The highest invert of the outlet pipe sets the water quality design water surface elevation in the pond. A floating outlet, set to draw water from 1 foot below the water surface, is also acceptable if vandalism concerns are adequately addressed.

  Intent. The inverted outlet pipe provides for trapping of oils and floatables in the wetpond.

- For online facilities, the outlet system (i.e., pipe exiting the outlet structure) shall be designed to pass the post-development peak flow rates with a 1 percent annual probability (100-year recurrence). For off-line facilities, the outlet system must be designed to pass the water quality design flow rate.

- A gravity drain for maintenance shall be provided if grade allows (see Figure 5.23b). Gravity drains are required in single celled ponds. The drain invert shall be at least 6 inches below the top elevation of the dividing berm or baffle. Deeper drains are encouraged where feasible, but must be no deeper than 18 inches above the pond bottom to prevent highly sediment-laden water from escaping the pond when drained for maintenance.

  The drain shall be at least 8 inches (minimum) diameter and shall be controlled by a valve (see below). Use of a shear gate is allowed only at the inlet end of a pipe located within an approved structure.

  It is anticipated that sediment removal will only be needed for the first cell in the majority of cases. The gravity drain is intended to temporarily reduce depth in the second cell, thereby providing capacity for water from the first cell to be pumped to the second cell.

- A bypass/shutoff valve is required to enable the pond to be taken off line for maintenance purposes. Operational access to the valve shall be provided to the finished ground surface. If the valve box is over 5 feet deep, an access manhole or vault is required. The valve location shall be accessible and well-marked with 1 foot of paving placed around the box. It must also be protected from damage and unauthorized operation.

- All control structures shall have round, solid locking lids with 5/8-inch diameter Allen head-cap screws.

- All inlet and outlet metal parts shall be corrosion-resistant. Galvanized materials shall not be used.

  Intent. Galvanized metal contributes zinc to stormwater, sometimes in very high concentrations.

Overflow Structure

For multi-purpose ponds (i.e., combined detention and wet pool facilities), see Section 4.6.3.2 of the flow control chapter for overflow requirements. The following criteria are required for single purpose wet ponds (i.e., treatment only):

- A primary overflow, either a grated inlet on the outlet structure or a bird cage structure located above the pond outlet structure (see also Figure 4.28), must be included in the outlet structure.

- For off-line facilities, the bottom of the grate opening in the outlet structure shall be set at or above the height needed to pass the water quality design flow through the pond outlet structure. For online facilities, the grate opening shall be sized and set at the height needed to pass the 100-year post development design flow through the pond outlet structure. Note: The grate invert elevation sets the overflow water surface elevation in the pond.

Access

The following guidelines for design of access roads may be used:

- Maintenance access road(s) shall be provided to the control structure and other drainage structures associated with the pond (e.g., inlet or bypass structures). Manhole and catch basin lids must be in or at the edge of the access road.
- Access roads must meet the requirements for design and construction of access roads specified below.
  - Maximum grade of access roads shall be 15 percent
  - Outside turning radius must be a minimum of 40 feet
  - Fence gates must be located only on straight sections of road
  - Access roads must be 15 feet in width on curves and 10 feet on straight sections
  - A paved apron must be provided where access roads connect to paved public roadways.
- If a fence is required, access shall be limited by a double-posted gate or by bollards – that is, two fixed bollards on each side of the access road and two removable bollards equally located between the fixed bollards.
- Unless all portions of the cell can be reached and sediment loaded from the top of the pond, an access ramp (7H minimum: 1V) is required for a truck and backhoe to access the bottom of the pond for maintenance and cleaning. The ramp must extend to the pond bottom if the bottom area is greater than 1,500 square feet (measured without the ramp). It may end at an elevation 4 feet above the pond bottom if the bottom area is less than 1,500 square feet.
- The internal berm of a wetpond or combined detention and wetpond may be used for maintenance access if the following conditions are met:
  - The berm is no more than 4 feet above the first wetpool cell
  - The first wetpool cell is less than 1,500 square feet (measured without the ramp)
  - The berm is designed to support a loaded truck, considering the berm is normally submerged and saturated
  - The berm is built to sustain loads of up to 80,000 pounds.

Signage

Signage discouraging feeding of waterfowl is recommended.

Signage meeting the specifications below shall be placed for maximum visibility from adjacent streets, sidewalks, and paths.

- Size: 48 inches by 24 inches
- Material: 0.125-gauge aluminum
- Face: Non-reflective vinyl or three coats of outdoor enamel (sprayed)
- Lettering: Silk screen enamel where possible, or vinyl letters
- Colors: Beige background with teal letters
- Type Face: Helvetica condensed (or similar). Title: 3 inches; Sub-Title: 1-1/2 inches; Text: 1 inch
- Installation: Secure to chain link fence if available. Otherwise, install on posts, mounted atop gravel bed, installed in 30-inch concrete filled post holes. Top of sign no higher than 42 inches from ground surface.
- Placement: Place sign in direction of primary visual or physical access. Do not block any access road. Do not place within 6 feet of structural facilities (e.g., maintenance holes, spillways, pipe inlets).
- Text: Title – "Stormwater Pond"
- Sub-Title – "This pond is in our care"
    - Text – "Runoff is held here after storms. It is released slowly or stored until the next storm, when it is replaced by incoming flows. This helps prevent downstream flooding and erosion, and helps clean the water. For more information or to report littering, vandalism or other problems, call [provide management name and contact phone number(s)]"
- Lining: If facility has a liner, the following note must be added to the face of the sign:
    - "This facility is lined to protect groundwater quality."
- In addition, the back of the sign shall include information indicating the extent of the lining, the liner material used, the liner thickness (if clay or till), and the type and distance of the marker above the line (if a geomembrane). This information need only be readable by someone standing at arms-length from the sign.

Planting Requirements

Planting requirements for detention ponds (Chapter 4, Section 4.6.3.2) also apply to wetponds. In addition:

- If the wet pond is intended for phosphorus control (i.e., a large wet pond), do not plant vegetation in cells below the water quality design surface because plants will release phosphorus in the winter when they die off.
- Exposed earth on the pond bottom and interior side slopes shall be sodded or seeded with an appropriate seed mixture. All remaining areas of the tract must be vegetated or stabilized before the pond is put online.

- If the second cell of a basic wetpond is 3 feet or shallower, the bottom area shall be planted with emergent wetland vegetation. See Table 5.24 for recommended emergent wetland plant species for wetponds.

  Intent. Planting of shallow pond areas helps to stabilize settled sediment and prevent resuspension.

- Cattails (Typha latifolia) are not recommended because they tend to crowd out other species, and the dead shoots need to be removed to prevent oxygen depletion in the wetpool.

Table 5.24. Emergent Wetland Plant Species Recommended for Wetponds.

Species	Common Name	Notes	Maximum Depth
Inundation to 1 foot			
Agrostis exarata[1]	Spike bent grass	Prairie to coast	to 2 feet
Carex stipata	Sawbeak sedge	Wet ground	
Eleocharis palustris	Spike rush	Margins of ponds, wet meadows	to 2 feet
Glyceria occidentalis	Western mannagrass	Marshes, pond margins	to 2 feet
Juncus tenuis	Slender rush	Wet soils, wetland margins	
Oenanthe sarmentosa	Water parsley	Shallow water along stream and pond margins; needs saturated soils all summer	
Scirpus atrocinctus (formerly S. cyperinus)	Woolgrass	Tolerates shallow water; tall clumps	
Scirpus microcarpus	Small-fruited bulrush	Wet ground to 18 inches depth	18 inches
Sagittaria latifolia	Arrowhead		
Inundation 1 to 2 feet			
Agrostis exarata[1]	Spike bent grass	Prairie to coast	
Alisma plantago-aquatica	Water plantain		
Eleocharis palustris	Spike rush	Margins of ponds, wet meadows	
Glyceria occidentalis	Western mannagrass	Marshes, pond margins	
Juncus effusus	Soft rush	Wet meadows, pastures, wetland margins	
Scirpus microcarpus	Small-fruited bulrush	Wet ground to 18 inches depth	18 inches
Sparganium emmersum	Bur reed	Shallow standing water, saturated soils	
Inundation 2 to 3 feet			
Carex obnupta	Slough sedge	Wet ground or standing water	1.5 to 3 feet
Beckmania syzigachne[1]	Western sloughgrass	Wet prairie to pond margins	
Scirpus acutus[2]	Hardstem bulrush	Single tall stems, not clumping	to 3 feet
Scirpus validus[2]	Softstem bulrush		
Inundation greater than 3 feet			
Nuphar polysepalum	Spatterdock	Deep water	3 to 7.5 feet
Nymphaea odorata[1]	White waterlily	Shallow to deep ponds	to 6 feet

[1] Non-native species. Beckmania syzigachne is native to Oregon. Native species are preferred.
[2] Scirpus tubers must be planted shallower for establishment, and protected from foraging waterfowl until established. Emerging aerial stems should project above water surface to allow oxygen transport to the roots.
Primary sources: Municipality of Metropolitan Seattle, Water Pollution Control Aspects of Aquatic Plants, 1990. Hortus Northwest, Wetland Plants for Western Oregon, Issue 2, 1991. Hitchcock and Cronquist, Flora of the Pacific Northwest, 1973.

Recommended Design Features

The following design features should be incorporated into the wet pond design as site conditions allow:

- For wetpool depths in excess of 6 feet, it is recommended that some form of recirculation be provided in the summer, such as a fountain or aerator, to prevent stagnation and low dissolved oxygen conditions.
- A small amount of base flow to the pond is desirable to maintain circulation and reduce the potential for low oxygen conditions during late summer.
- A tear-drop shape, with the inlet at the narrow end, rather than a rectangular pond is preferred since it minimizes dead zones caused by corners.

The following design features should be incorporated to enhance aesthetics where possible:

- Provide pedestrian access to shallow pool areas enhanced with emergent wetland vegetation. This allows the pond to be more accessible without incurring safety risks.
- Provide side slopes that are sufficiently gentle to avoid the need for fencing (3H:1V or flatter).
- Create flat areas overlooking or adjoining the pond for picnic tables or seating that can be used by residents. Walking or jogging trails around the pond are easily integrated into site design.
- Include fountains or integrated waterfall features for privately maintained facilities.
- Provide visual enhancement with clusters of trees and shrubs. On most pond sites, it is important to amend the soil before planting since ponds are typically placed well below the native soil horizon in very poor soils. Make sure dam safety restrictions against planting do not apply. See also the landscaping guidelines below.
- Orient the pond length along the direction of prevailing summer winds (typically west or southwest) to enhance wind mixing.

Landscaping Guidelines

Landscaping is encouraged, but not required for most stormwater tract areas (see below for areas not to be landscaped). However, if provided, landscaping must adhere to the criteria which follow so as not to hinder maintenance operations.

If landscaping is proposed, the following requirements shall apply:

- No trees or shrubs may be planted within 10 feet of inlet or outlet pipes or manmade drainage structures such as spillways or flow spreaders. Species with roots that seek water, such as willow or poplar, should be avoided within 50 feet of pipes or manmade structures.
- If the wetpond discharges to a phosphorus-sensitive lake or wetland, shrubs that form a dense cover should be planted on slopes above the water quality design water surface on at least three sides. The purpose of planting is to discourage waterfowl use of the pond and to provide shading. Some suitable trees and shrubs include vine maple (*Acer circinatum*), wild cherry (*Prunus emarginata*), red osier dogwood (*Cornus stolonifera*), California myrtle (*Myrica californica*), Indian plum (*Oemleria cerasiformis*), and Pacific yew (*Taxus brevifolia*) as well as numerous ornamental species.
- Planting is restricted on berms that impound water either permanently or temporarily during storms. *Note: This restriction does not apply to cut slopes that form pond banks, only to berms.*
    - Trees or shrubs may not be planted on portions of water-impounding berms taller than 4 feet high. Only grasses may be planted on berms taller than 4 feet.
    - Trees planted on portions of water-impounding berms less than 4 feet high must be small, not higher than 20 feet mature height, and have a fibrous root system. Table 5.25 gives some examples of trees with these characteristics.
    - Intent: These trees reduce the likelihood of blow-down trees, or the possibility of channeling or piping of water through the root systems, which may contribute to structural failure on berms that retain water.

Table 5.25. Small Trees and Shrubs with Fibrous Roots.

Small Trees/High Shrubs	Low Shrubs
Red twig dogwood (*Comus Stolonifera*)*	Snowberry (*Symphoricarpus albus*)*
Serviceberry (*Amelanchier ainifolia*)*	Salmonberry (*Rubus spectabillis*)*
Strawberry tree (*Arbutus unedo*)	Rosa rugosa (avoid spreading varieties)
Highbush cranberry (*Vaccinium opulus*)	Rock rose (*Cistus* spp.)
Blueberry (*Vaccinium* spp.)	Ceanothus spp. (choose hardier varieties)
Filbert (*Corylus comuta*, others)*	New Zealand flax (*Phormium penax*)
Fruit trees on dwarf rootstock	Ornamental grasses (e.g., *Miscanthis, Pennisetum*)

* Native Species

- All landscape material, including grass, must be planted in good topsoil. Native underlying soils may be suitable for planting if amended per requirements in Section 4.4.1.

- Soil in which trees or shrubs are planted may require additional enrichment or additional compost top-dressing. Consult a certified arborist for site-specific recommendations.
- For a naturalistic effect, as well as ease of maintenance, trees or shrubs should be planted in clumps to form "landscape islands" rather than evenly spaced.
- The landscaped islands must be a minimum of six feet apart, and if set back from fences or other barriers, the setback distance should also be a minimum of 6 feet. Where tree foliage extends low to the ground, the 6 feet of setback should be counted from the outer drip line of the trees (estimated at maturity).

  Intent: This setback allows a 6-foot wide mower to pass around and between clumps.
- Evergreen trees and other trees that produce relatively little leaf-fall (such as Oregon ash, mimosa, or locust) are preferred.
- Trees should be set back so that branches do not extend over the pond (to prevent leaf-drop into the water).
- Drought tolerant species are recommended.

5.10.3.3 Construction Specifications and Criteria

Sediment that has accumulated in the pond during construction must be removed once the site is stabilized (i.e., after all project improvements are completed and all exposed ground surfaces are stabilized by vegetation or landscaping).

If good vegetation cover is not established on the berm, erosion control measures must be used to prevent erosion of the berm back-slope when the pond is initially filled.

5.10.3.4 Operations and Maintenance Requirements

See Appendix D for specific operations and maintenance requirements for wet ponds.

5.10.4 Wetvaults

Wetvaults are similar to wet ponds, except the wet pool is constructed below grade in a concrete (or similar) vault. Being underground, the wetvault lacks the biological pollutant removal mechanisms, such as algae uptake, present in surface wetponds. Figure 5.28 shows a typical wetvault design.

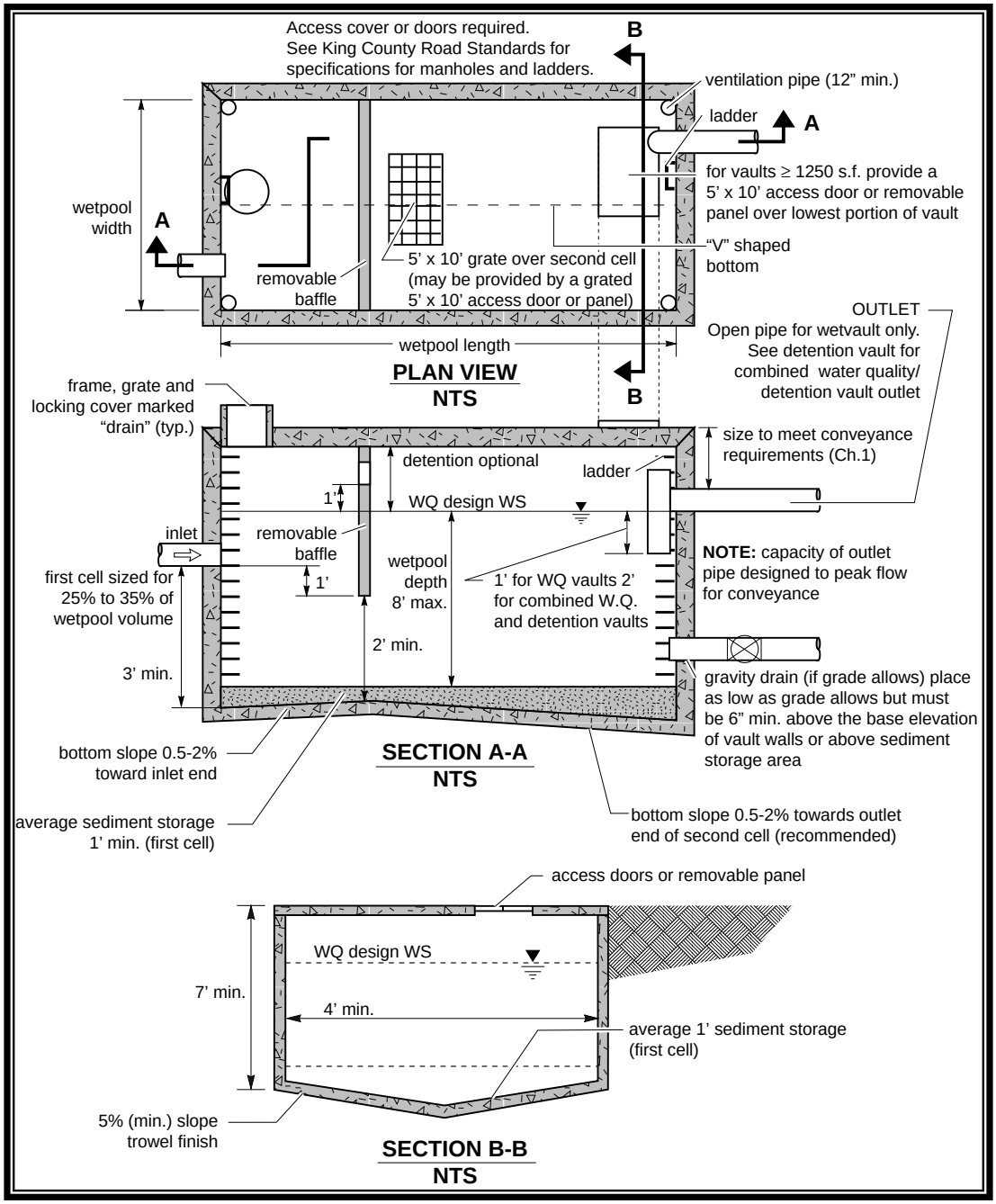


Figure 5.28. Typical Wetvault.

Wetvaults are believed to be ineffective in removing dissolved pollutants such as soluble phosphorus or metals. There is also concern that oxygen levels will decline in wetvaults, especially in warm summer months, because of limited contact with air and wind. However, the extent to which this potential problem occurs has not been documented.

If oil control is required for a project, a wetvault may be combined with an API oil/water separator.

5.10.4.1 Site Considerations

A wet vault may be used on any type or size of commercial, industrial, or roadway project if there are space limitations precluding the use of other treatment BMPs. However, vaults are most practical on small sites (less than 10 acres of contributing impervious area) that have high land values, because vaults are relatively expensive. Wet vaults are not permitted for single-family residential projects and are discouraged for multi-family projects, because underground structures are relatively difficult and expensive to maintain. In addition, the need for maintenance is not readily apparent to the homeowner/ property manager. As a result, routine maintenance often does not occur. Combined detention and wetvaults are allowed for single- or multi-family applications (see Section 5.10.6).

5.10.4.2 BMP Sizing and Design Criteria

As with wetponds, the primary design factor that determines the removal efficiency of a wetvault is the volume of the wetpool. The larger the volume, the higher the potential for pollutant removal. Performance is also improved by avoiding dead zones (like corners) where little exchange occurs, using large length-to-width ratios, dissipating energy at the inlet, and ensuring that flow rates are uniform to the extent possible and not increased between cells.

The methods for designing the wetvault are identical to the methods for designing wetponds. The wetpool volume for the wetvault shall be equal to or greater than the daily runoff volume at or below which 91 percent of the total runoff volume for the simulation period occurs, as determined using an approved continuous model. Typical design details and concepts for the wetvault are shown in Figure 5.28.

Wetvault Geometry

The minimum flow length-to-width ratio is 3:1. A greater ratio is desirable. The inlet and outlet should be at opposing corners of the vault to increase the flowpath, if possible.

Wetpool depths for vaults are the same as specified for wetponds except for the following two modifications:

1. The sediment storage in the first cell shall be an average of 1-foot. Because of the v-shaped bottom, the depth of sediment storage needed above the bottom of the side wall is roughly proportional to the vault width according to Table 5.26.
2. The second cell shall be a minimum of 3 feet deep since planting cannot be used to prevent resuspension of sediment in shallow water (as it can in open ponds).

Table 5.26. Sediment Depth Criteria for Wetvaults.

Vault Width	Sediment Depth (from bottom of side wall)
15 feet	10 inches
20 feet	9 inches
40 feet	6 inches
60 feet	4 inches

Wetvault Configuration

The vault shall be separated into two cells by a wall or a removable baffle. If a wall is used, a 5-foot by 10-foot removable maintenance access must be provided for both cells. If a removable baffle is used, the following criteria apply:

- The baffle shall extend from a minimum of 1 foot above the water quality design water surface to a minimum of 1 foot below the invert elevation of the inlet pipe.
- The lowest point of the baffle shall be a minimum of 2 feet from the bottom of the vault, and greater if feasible.

If the vault is less than 2,000 cubic feet (inside dimensions), or if the length-to-width ratio of the vault pool is 5:1 or greater, the baffle or wall may be omitted and the vault may be one-celled.

The two cells of a wetvault must not be divided into additional subcells by internal walls. If internal structural support is needed, it is preferred that post and pier construction be used to support the vault lid rather than walls. Any walls used within cells must be positioned so as to lengthen, rather than divide, the flowpath.

Intent: Treatment effectiveness in wetpool facilities is related to the extent to which plug flow is achieved and short-circuiting and dead zones are avoided. Structural walls placed within the cells can interfere with plug flow and create significant dead zones, reducing treatment effectiveness.

The bottom of the first cell shall be sloped toward the access opening. The slope must be between 0.5 percent (minimum) and 2.0 percent (maximum). The second cell should be sloped toward the outlet (0.5 to 2.0 percent), with a high point between the first and second cells. The intent of sloping the bottom is to

direct the sediment accumulation to the closest access point for maintenance purposes. The second cell should be sloped to the outlet, with a high point between the first and second cells (see Figure 5.28).

The vault bottom shall slope laterally a minimum of 5.0 percent from each side towards the center, forming a broad "v" to facilitate sediment removal. Note: More than one "v" may be used to minimize vault depth.

Exception: The vault bottom may be flat if removable panels are provided over the entire vault. Removable panels must be at grade, have stainless steel lifting eyes, and weigh no more than 5 tons per panel.

The highest point of a vault bottom must be at least 6 inches below the outlet elevation to provide for sediment storage over the entire bottom.

Inlet and Outlet

The number of inlets to the wetvault should be limited, and the flowpath length must be maximized from inlet to outlet for all inlets to the vault.

The inlet to the wetvault shall be submerged with the inlet pipe invert a minimum of 3 feet from the vault bottom (not including sediment storage). The top of the inlet pipe should be submerged at least 1 foot, if possible.

Intent. The submerged inlet is to dissipate energy of the incoming flow. The distance from the bottom is to minimize resuspension of settled sediments. Alternative inlet designs that accomplish these objectives are acceptable.

Unless designed as an off-line facility, the capacity of the outlet pipe and available head above the outlet pipe must be designed to convey the design flow for developed site conditions with a 1 percent annual probability (100-year recurrence) without overtopping the vault. The available head above the outlet pipe must be a minimum of 6 inches.

The outlet pipe shall be back-sloped or have a tee section, the lower arm of which must extend 1 foot below the water quality design water surface to provide for trapping of oils and floatables in the vault.

Where pipes enter and leave the vault below the water quality design water surface, they shall be sealed using a non-porous, non-shrinking grout.

Provision for passage of flows should the outlet become plugged shall be provided.

A gravity drain for maintenance shall be provided if grade allows.

- The gravity drain should be as low as the site situation allows; however, the invert shall be no lower than the average sediment storage depth. At a minimum, the invert shall be 6 inches above the base elevation of the vault side walls.

Intent: This placement prevents highly sediment-laden water from escaping when the vault is drained for maintenance. A lower placement is allowed than for wetponds since the v-shaped vault bottom will capture and retain additional sediments.

- The drain shall be 8 inches (minimum) diameter and shall be controlled by a valve. Use of a shear gate is allowed only at the inlet end of a pipe located within an approved structure.

  Intent: Shear gates often leak if water pressure pushes on the seal side of the gate. The gate must be situated so that water pressure pushes toward the seal.

- Operational access to the valve shall be provided to the finished ground surface. If not located in the vault, a valve box is allowed to a maximum depth of 5 feet without an access manhole. If the valve box is over 5 feet deep, an access manhole or vault is required. The valve location shall be accessible and well marked with 1 foot of paving placed around the box. It must also be protected from damage and unauthorized operation.

- All inlet and outlet metal parts shall be corrosion-resistant. Galvanized materials shall not be used.

  Intent. Galvanized metal contributes zinc to stormwater, sometimes in very high concentrations.

Modifications for Combining with a Baffle Oil/Water Separator

If the project site is a high-use site and a wetvault is proposed, the vault may be combined with a baffle oil/water separator to meet the water quality treatment requirements with one facility rather than two. Structural modifications and added design criteria are given below. However, the maintenance requirements for baffle oil/water separators must be adhered to, in addition to those for a regular wetvault. This will result in more frequent inspection and cleaning than for a wetvault. See Section 5.11.3 for information on maintenance of baffle oil/water separators.

- The sizing procedures for the baffle oil/water separator (Section 5.11.4) must be run as a check to ensure the vault is large enough. If the oil/water separator sizing procedures result in a larger vault size, increase the wetvault size to match.

- An oil retaining baffle shall be provided in the second cell near the vault outlet. The baffle must not contain a high-flow overflow, or else the retained oil will be washed out of the vault during large storms.

- The vault shall have a minimum length-to-width ratio of 5:1

- The vault shall have a design water depth-to-width ratio of between 1:3 to 1:2

- The vault shall be watertight and shall be coated to protect from corrosion
- Separator vaults shall have a shutoff mechanism on the outlet pipe to prevent oil discharges during maintenance and to provide emergency shut-off capability in case of a spill. A valve box and riser shall also be provided. See previous section for valve box requirements.
- Wetvaults used as oil/water separators must be off-line and must bypass flows greater than the off-line water quality design flow (i.e., the water quality design flow multiplied by the off-line factor of 3.0.

  Intent: This design minimizes the entrainment and/or emulsification of previously captured oil during very high flow events.

Access

- Access shall be provided over both the inlet pipe and outlet structure. Access openings shall be positioned a maximum of 50 feet from any location within the tank; additional access points may be required on large vaults. If more than one "v" is provided in the vault floor, access to each "v" must be provided.
- For vaults with greater than 1,250 square feet of floor area, a 5-foot by 10-foot removable panel shall be provided over the inlet pipe (instead of a standard frame, grate and solid cover). Alternatively, a separate access vault may be provided.
- For vaults under roadways, the removable panel must be located outside the travel lanes. Alternatively, multiple standard locking manhole covers may be provided. Ladders and hand-holds need only be provided at the outlet pipe and inlet pipe, and as needed to meet OSHA and WISHA confined space requirements. Vaults providing manhole access at 12-foot spacing need not provide corner ventilation pipes.
- All access openings, except those covered by removable panels, shall have round, solid locking lids, or 3-foot square, locking diamond plate covers.
- Vaults with widths 10 feet or less must have removable lids.
- The maximum depth from finished grade to the vault invert shall be 20 feet.
- Internal structural walls of large vaults shall be provided with separate access risers or openings sufficient for maintenance access between cells. The openings shall be sized and situated to allow access to the maintenance "v" in the floor.
  - The minimum internal height shall be 7 feet from the highest point of the vault floor (not sump), and the minimum width shall be 4 feet.

    Exceptions:

  - Concrete vaults may be a minimum 3 feet in height and width if used as tanks with access manholes at each end, and if the width is no larger than the height.
  - The minimum internal height requirements may be waived for any areas covered by removable panels.

- Vaults must comply with the OSHA and WISHA confined space requirements, which include clearly marking entrances to confined space areas. This may be accomplished by hanging a removable sign in the access riser(s) just under the access lid.
- A minimum of 50 square feet of grate shall be provided over the second cell. For vaults in which the surface area of the second cell is greater than 1,250 square feet, 4 percent of the top shall be grated. This requirement may be met by one grate or by many smaller grates distributed over the second cell area. *Note: a grated access door can be used to meet this requirement.*

  Intent: The grate allows air contact with the wetpool in order to minimize stagnant conditions which can result in oxygen depletion, especially in warm weather.

- All inlet and outlet metal parts shall be corrosion-resistant. Galvanized materials shall not be used.

  Intent. Galvanized metal contributes zinc to stormwater, sometimes in very high concentrations.

Structural Requirements

Wetvaults shall conform with the "Materials" and "Structural Stability" criteria specified for detention vaults in Chapter 4, Section 4.6.5.2.

Wetvaults may be constructed using arch culvert sections provided the top area at the water quality design water surface is, at a minimum, equal to that of a vault with vertical walls designed with an average depth of 6 feet.

Intent: To prevent decreasing the surface area available for oxygen exchange.

Recommended Design Features

The following design features should be incorporated into wetvaults where feasible, but they are not specifically required:

- Lockable grates instead of solid manhole covers are recommended to increase air contact with the wetpool.

5.10.4.3 Construction Specifications and Criteria

See the construction-related issues outlined above as part of the design criteria.

5.10.4.4 Operations and Maintenance Requirements

Wetvault BMP operations and maintenance requirements are provided in Appendix D.

5.10.5 Stormwater Treatment Wetlands

Stormwater treatment wetlands are similar to wet ponds, but also provide a shallow marsh area to allow the establishment of emergent wetland aquatic plants, which improves pollutant removal. See Figure 5.29 and Figure 5.30 for example stormwater wetland details.

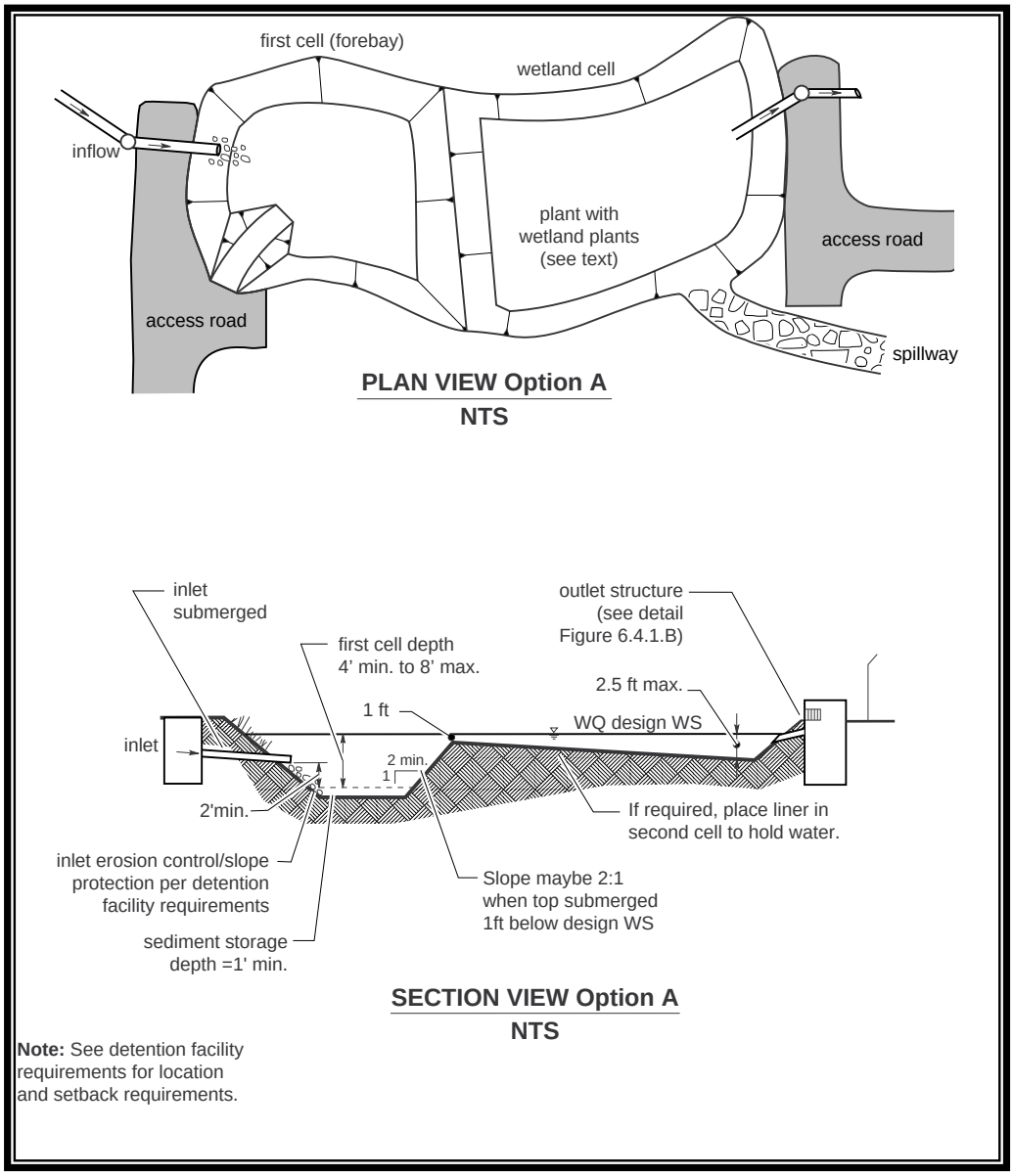


Figure 5.29. Stormwater Wetland — Option One.

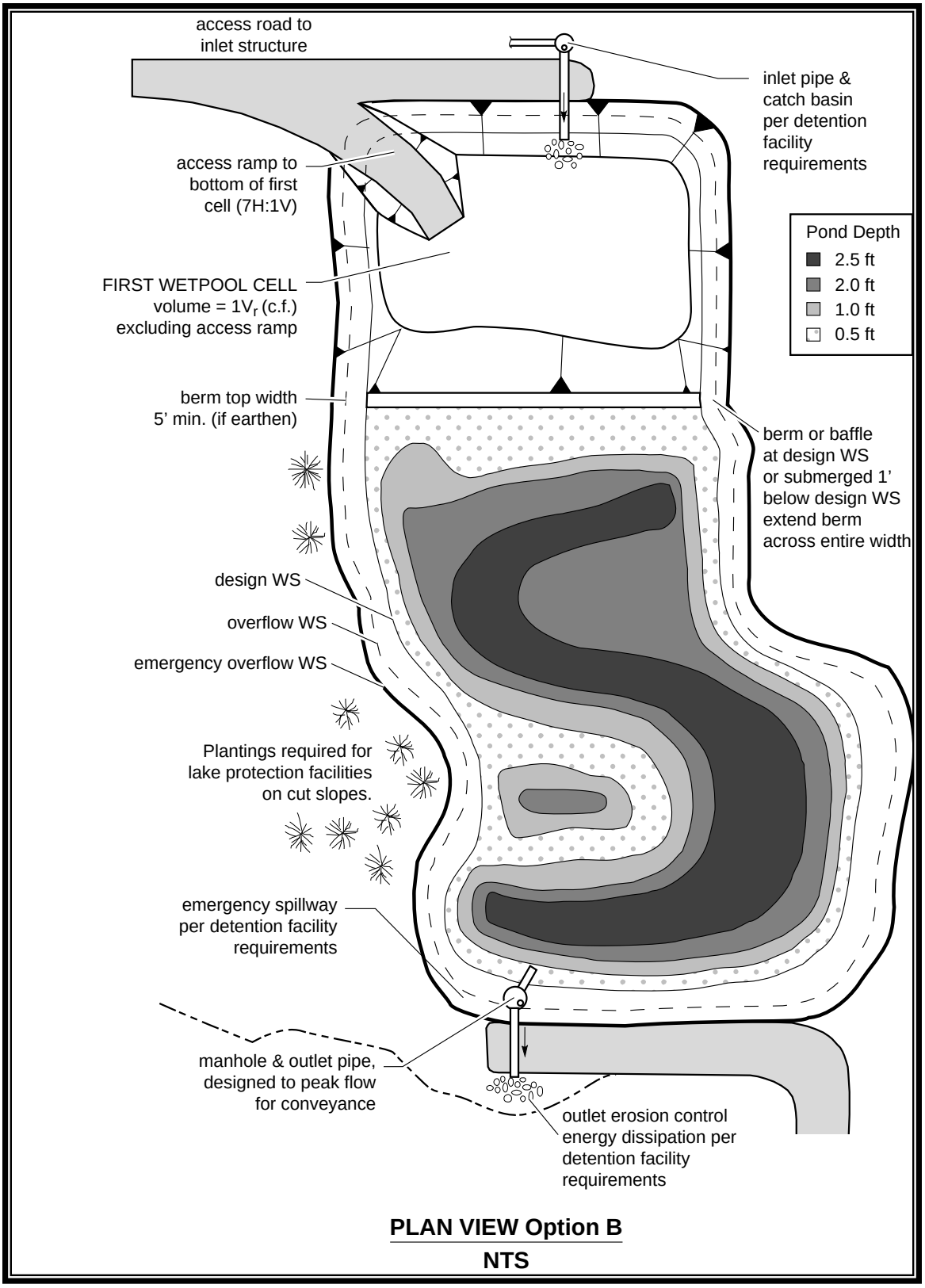


Figure 5.30. Stormwater Wetland — Option Two.

In land development situations, wetlands are usually constructed for two main reasons: to replace or mitigate impacts when natural wetlands are filled or impacted by development (mitigation wetlands) and to treat stormwater runoff (stormwater treatment wetlands). Mitigation wetlands may not be used as stormwater treatment facilities, because stormwater treatment functions are not compatible with normal wetland function. Mitigation wetlands are intended to function as full replacement habitat for fish and wildlife, providing the same functions and harboring the same species diversity and biotic richness as the wetlands they replace. Stormwater treatment wetlands are used to capture and transform pollutants, just as wetponds are, and over time pollutants will concentrate in the sediment. This is not a healthy environment for aquatic life. Stormwater treatment wetlands are used to capture pollutants in a managed environment so that they will not reach natural wetlands and other ecologically important habitats. In addition, vegetation must occasionally be harvested and sediment dredged in stormwater treatment wetlands, further interfering with use for wildlife habitat.

5.10.5.1 Site Considerations

Stormwater treatment wetlands occupy about the same surface area as wetponds, but have the potential to be better integrated aesthetically into a site because of the abundance of emergent aquatic vegetation. The most critical factor for a successful design is the provision of an adequate supply of water for most of the year. Careful planning is needed to be sure sufficient water will be retained to sustain good wetland plant growth. Since water depths are shallower than in wetponds, water loss by evaporation is an important concern. Stormwater wetlands are a good water quality facility choice in areas with high groundwater levels.

See also the general applications and limitations information on wet pool facilities provided at the beginning of this section.

Setbacks

Location of the stormwater wetland relative to site constraints (e.g., buildings, property lines, etc.) shall be the same as for wet ponds (see Section 5.10.3.1). See also Section 5.4.3 for general setback requirements for water quality facilities.

5.10.5.2 BMP Sizing and Design Criteria

When used for stormwater treatment, stormwater wetlands employ some of the same design features as wetponds. However, in addition to gravity settling, wetlands provide additional pollutant removal due to the presence of aquatic plants which aid in the settling process and may provide some pollutant uptake. The microbiological community associated with these aquatic plants also aids in removing dissolved pollutants, especially phosphorus. Thus, factors that affect

plant growth and biomass are critical components of wetland design. The following steps must be used for sizing stormwater treatment wetlands:

Step 1 – The volume of a basic wetpond is used as a template for sizing the stormwater wetland. The design volume is the daily runoff volume at or below which 91 percent of the total runoff volume for the simulation period occurs, as determined using an approved continuous model.

Step 2 – Calculate the surface area of the stormwater wetland. The surface area of the wetland shall be the same as the top area of a wetpond sized for the same site conditions. Calculate the surface area of the stormwater wetland by using the volume from Step 1 and dividing by the average water depth (use 3 feet).

Step 3 – Determine the surface area of the first cell of the stormwater wetland. Use the volume determined from Criterion 2 under "Wetland Geometry" below, and the actual depth of the first cell.

Step 4 – Determine the surface area of the wetland cell. Subtract the surface area of the first cell (Step 3) from the total surface area (Step 2).

Step 5 – Determine water depth distribution in the second cell. Decide if the top of the dividing berm will be at the surface or submerged (designer's choice). Adjust the distribution of water depths in the second cell according to Criterion 9 under "Wetland Geometry" below. Note: This will result in a facility that holds less volume than that determined in Step 1 above. This is acceptable.

Intent. The surface area of the stormwater wetland is set to be roughly equivalent to that of a wetpond designed for the same site so as not to discourage use of this option.

Step 6 – Choose plants. See Table 5.24 for a list of plants recommended for wetpond water depth zones, or consult a wetland scientist.

Wetland Geometry

1. Stormwater wetlands shall consist of two cells, a presettling cell and a wetland cell.
2. The presettling cell shall contain approximately 33 percent of the wetpool volume calculated in Step 1 above.
3. The depth of the presettling cell shall be between 4 feet (minimum) and 8 feet (maximum), excluding sediment storage.
4. One-foot of sediment storage shall be provided in the presettling cell.
5. The wetland cell shall have an average water depth of about 1.5 feet (plus or minus 3 inches).
6. The "berm" separating the two cells shall be shaped such that its downstream side gradually slopes to form the second shallow wetland

cell. See the section view in Figure 5.29. Alternatively, the second cell may be graded naturalistically from the top of the dividing berm (see Criterion 9 below).

7. The top of berm shall be either at the water quality design water surface or submerged 1 foot below the water quality design water surface, as with wetponds. Correspondingly, the side slopes of the berm must meet the following criteria:
   - If the top of berm is at the water quality design water surface, the berm side slopes shall be no steeper than 3H:1V.
   - If the top of berm is submerged 1 foot, the upstream side slope may be up to 2H:1V.
8. Inlets and outlets shall be placed to maximize the flowpath through the facility. The ratio of flowpath length to width from the inlet to the outlet shall be at least 3:1. The flowpath length is defined as the distance from the inlet to the outlet, as measured at mid-depth. The width at mid-depth can be found as follows: width = (average top width + average bottom width)/2.
9. Two examples are provided for grading the bottom of the wetland cell. One example is a shallow, evenly graded slope from the upstream to the downstream edge of the wetland cell (see Figure 5.29). The second example is a "naturalistic" alternative (Option 2), with the specified range of depths intermixed throughout the second cell (see Figure 5.30). A distribution of depths shall be provided in the wetland cell depending on whether the dividing berm is at the water surface or submerged. See Table 5.27 below). The maximum depth is 2.5 feet in either configuration. Other configurations within the wetland geometry constraints listed above may be approved by the Director.
10. To the extent possible create a complex microtopography within the wetland. Design the flow path to maximize sinuous flow between wetland cells.

Table 5.27. Distribution of Depths in Wetland Cell.

Dividing Berm at WQ Design Water Surface		Dividing Berm Submerged 1 Foot	
Depth Range (feet)	**Percent**	**Depth Range (feet)**	**Percent**
0.1 to 1	25	1 to 1.5	40
1 to 2	55	1.5 to 2	40
2 to 2.5	20	2 to 2.5	20

Lining Requirements

Many wetland plants can adapt to periods of summer drought, however the stormwater wetland design should maximize the duration of wet conditions to the

extent possible. Therefore, both cells of the stormwater wetland shall be lined with a low-permeability liner. The criteria for liners given in Section 5.4.4 must be observed. A minimum of 18 inches of native soil amended with good topsoil or compost (one part compost mixed with 3 parts native soil) must be placed over the liner. For geomembrane liners, a soil depth of 3 feet is recommended to prevent damage to the liner during planting. Hydric soils are not required.

Inlet and Outlet

Same as for wetponds.

Access

- Access and maintenance roads shall be provided and designed according to the requirements for wet ponds (see Section 5.10.3.2). Access and maintenance roads shall extend to both the wetland inlet and outlet structures. An access ramp (7H minimum:1V) shall be provided to the bottom of the first cell unless all portions of the cell can be reached and sediment loaded from the top of the wetland side slopes.
- If the dividing berm is also used for access, it must be built to sustain loads of up to 80,000 pounds.

Signage

Signage shall be provided according to the requirements for wetponds.

Planting Requirements

The wetland cell shall be planted with emergent wetland plants following the recommendations given in Table 5.24 or the recommendations of a wetland specialist. Note: Cattails (*Typha latifolia*) are not recommended. They tend to escape to natural wetlands and crowd out other species. In addition, the shoots die back each fall and will result in oxygen depletion in the wetpool unless they are removed.

5.10.5.3 Construction Specifications and Criteria

Construction considerations are the same as for basic wetponds.

Construction of the naturalistic alternative (Option 2) can be easily done by first excavating the entire area to the 1.5-foot average depth. Then soil subsequently excavated to form deeper areas can be deposited to raise other areas until the distribution of depths indicated in the design is achieved.

5.10.5.4 Operations and Maintenance Requirements

- Stormwater treatment wetland operations and maintenance requirements are provided in Appendix D.

5.10.6 Combined Detention and Wetpool Facilities

Combined detention and water quality wetpool facilities have the appearance of a detention facility but contain a permanent pool of water as well. The following design procedures, requirements, and recommendations cover differences in the design of the stand-alone water quality facility when combined with detention storage. Site considerations, setbacks, and other typical siting and design considerations for combined facilities are the same as specified for each individual facility, unless noted below. The following combined facilities are addressed:

- Detention/wetpond (basic and large)
- Detention/wetvault
- Detention/stormwater wetland.

There are two sizes of the combined wetpond, a basic and a large, but only a basic size for the combined wetvault and combined stormwater wetland. The facility sizes (basic and large) are related to the pollutant removal goals (see Section 5.3).

5.10.6.1 Applications and Limitations

Combined detention and water quality facilities are very efficient for sites that also have detention requirements. The water quality facility may often be placed beneath the detention facility without increasing the facility surface area. However, the fluctuating water surface of the live storage will create unique challenges for plant growth and for facility aesthetics.

The basis for pollutant removal in combined facilities is the same as in the stand-alone water quality facilities. However, in the combined facility, the detention function creates fluctuating water levels and added turbulence. For simplicity, the positive effect of the extra live storage volume and the negative effect of increased turbulence are assumed to balance, and are thus ignored when sizing the wetpool volume. For the combined detention/stormwater wetland, criteria that limit the extent of water level fluctuation are specified to better ensure survival of the wetland plants.

Unlike the wetpool volume, the live storage component of the facility must be provided above the seasonal high groundwater level.

5.10.6.2 Combined Detention and Wetpond (Basic and Large)

Typical design details and concepts for a combined detention and wetpond are shown in Figures 5.31 and 5.32. The detention portion of the facility shall meet the design criteria and sizing procedures set forth for detention ponds in Chapter 4, Section 4.6.3.

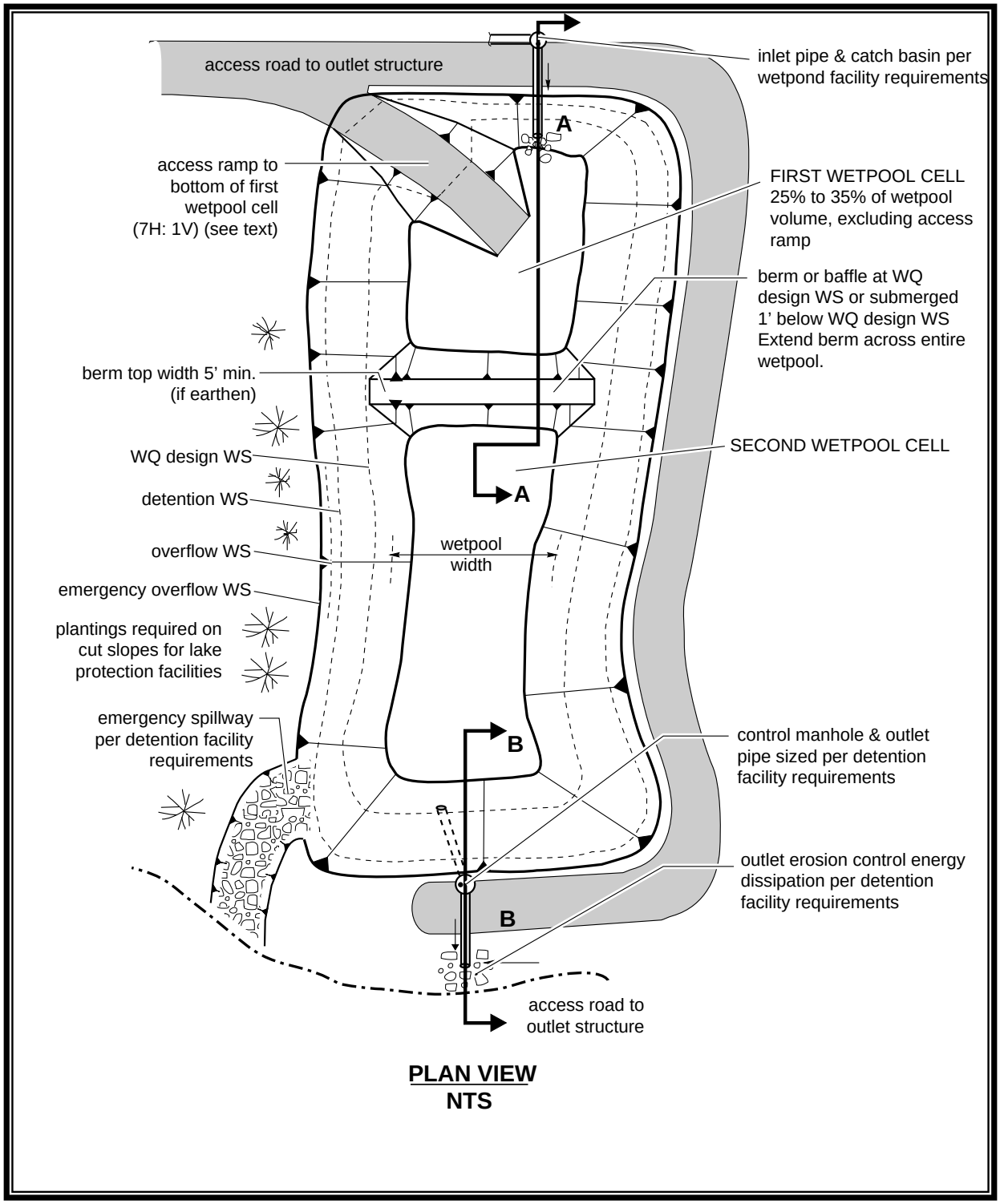


Figure 5.31. Typical Combined Detention and Wetpond.

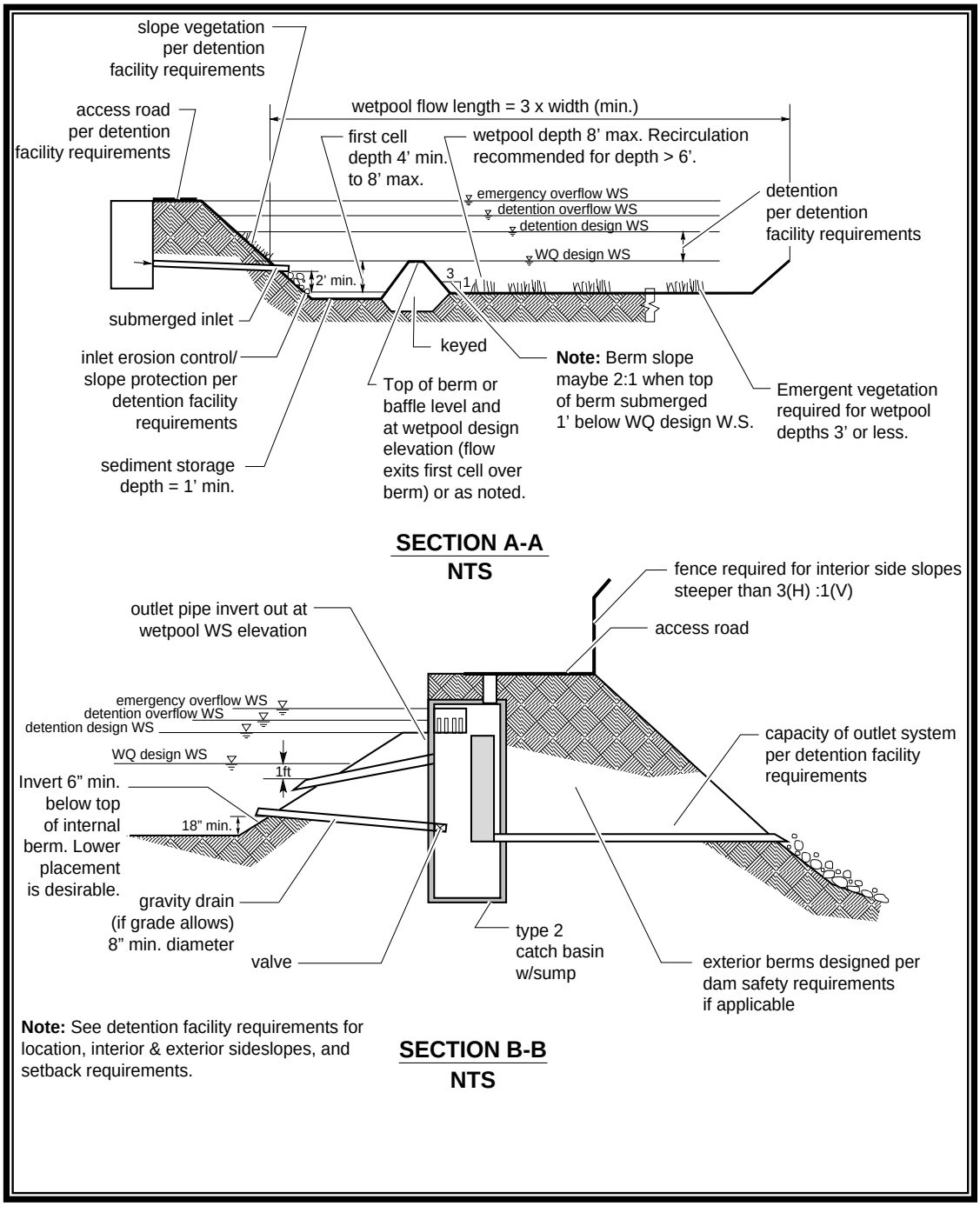


Figure 5.32. Typical Combined Detention and Wetpond (continued).

BMP Sizing and Design Criteria

The sizing procedure for combined detention and wetponds are identical to those outlined for wetponds and for detention facilities. The wetpool volume for a combined facility shall be equal to or greater than the daily runoff volume at or below which 91 percent of the total runoff volume for the simulation period occurs, as determined using an approved continuous model. Follow the standard procedure specified in Chapter 6 to size the detention portion of the pond.

Detention and Wetpool Geometry

- The wetpool and sediment storage volumes shall not be included in the required detention volume.
- The "Wetpool Geometry" criteria for wetponds (see Section 5.10.3.2) shall apply with the following modifications/clarifications:

  Criterion 1. The permanent pool may be made shallower to take up most of the pond bottom, or deeper and positioned to take up only a limited portion of the bottom. Note, however, that having the first wetpool cell at the inlet allows for more efficient sediment management than if the cell is moved away from the inlet. Wetpond criteria governing water depth must, however, still be met. See Figure 5.33 for two possibilities for wetpool cell placement.

  Intent. This flexibility in positioning cells is provided to allow for multiple use options, such as volleyball courts in live storage areas in the drier months.

  Criterion 2. The minimum sediment storage depth in the first cell is 1 foot. The 6 inches of sediment storage required for detention ponds does not need to be added to this, but 6 inches of sediment storage must be added to the second cell to comply with the detention sediment storage requirement.

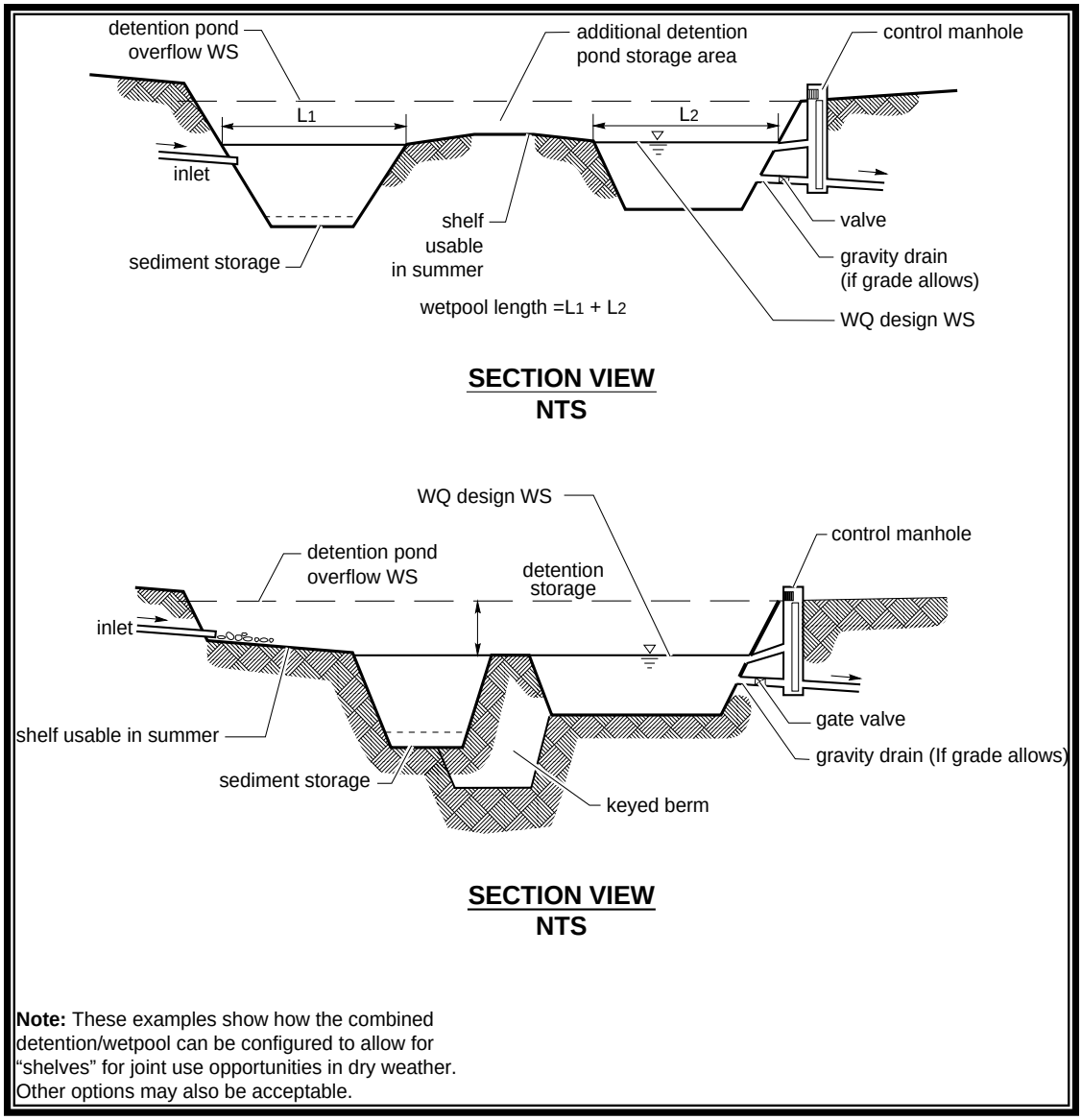
Berms, Baffles, and Slopes

Same as for wetponds.

Inlet and Outlet

The "Inlet and Outlet" criteria for wetponds (see Section 5.10.3.2) shall apply with the following modifications:

- A sump must be provided in the outlet structure of combined ponds.
- The detention flow restrictor and its outlet pipe shall be designed according to the requirements for detention ponds (see Chapter 4, Sections 4.6.3 and 4.6.8).



Note: These examples show how the combined detention/wetpool can be configured to allow for "shelves" for joint use opportunities in dry weather. Other options may also be acceptable.

Figure 5.33. Alternative Configurations of Detention and Wetpool Areas.

Access and Setbacks

Same as for wetponds.

Signage

Same as for wetponds.

Planting Requirements

Same as for wetponds.

5.10.6.3 Combined Detention and Wetvault

BMP Sizing and Design Criteria

The sizing procedure for combined detention and wetvaults is identical to those outlined for wetvaults and for detention facilities. The wetvault volume for a combined facility shall be equal to or greater than the daily runoff volume at or below which 91 percent of the total runoff volume for the simulation period occurs, as determined using an approved continuous model. Follow the standard procedure specified in Chapter 4, Section 4.6.5.3 to size the detention portion of the vault.

The design criteria for detention vaults and wetvaults must both be met, except for the following modifications or clarifications:

- The minimum sediment storage depth in the first cell shall average 1 foot. The 6 inches of sediment storage required for detention vaults does not need to be added to this, but 6 inches of sediment storage must be added to the second cell to comply with detention vault sediment storage requirements.
- The baffle shall extend a minimum of 2 feet below the water quality design water surface.

  Intent. The greater depth of the baffle in relation to the water quality design water surface compensates for the greater water level fluctuations experienced in the combined vault. The greater depth is deemed prudent to better ensure that separated oils remain within the vault, even during storm events.

Note: If a vault is used for detention as well as water quality control, the facility may not be modified to function as a baffle oil/water separator as allowed for wetvaults (see Section 5.10.4). This is because the added pool fluctuation in the combined vault does not allow for the quiescent conditions needed for oil separation.

5.10.6.4 Combined Detention and Stormwater Wetland

BMP Sizing and Design Criteria

The sizing procedure for combined detention and stormwater wetlands is identical to those outlined for stormwater wetlands and for detention facilities. Follow the procedure specified in Section 5.10.5 to determine the stormwater wetland size. Follow the standard procedure specified in Chapter 4, Section 4.6.3 to size the detention portion of the wetland.

The design criteria for detention ponds and stormwater wetlands must both be met, except for the following modifications or clarifications:

- The wetland geometry criteria for stormwater wetlands (Section 5.10.5.2) must be modified such that the minimum sediment storage depth in the first cell is 1 foot. The 6 inches of sediment storage required for detention ponds does not need to be added to this, nor does the 6 inches of sediment storage in the second cell of detention pond.

  Intent. Since emergent plants are limited to shallower water depths, the deeper water created before sediments accumulate is considered detrimental to robust emergent growth. Therefore, sediment storage is confined to the first cell which functions as a presettling cell.

- Water Level Fluctuation Restrictions: The difference between the water quality design water surface and the maximum water surface associated with the 2-year runoff shall not be greater than 3 feet. If this restriction cannot be met, the size of the stormwater wetland must be increased. The additional area may be placed in the first cell, second cell, or both. If placed in the second cell, the additional area need not be planted with wetland vegetation or counted in calculating the average depth.

  Intent: This criterion is designed to dampen the most extreme water level fluctuations expected in combined facilities to better ensure that fluctuation-tolerant wetland plants will be able to survive in the facility. It is not intended to protect native wetland plant communities and is not to be applied to natural wetlands.

- The "Inlet and Outlet" criteria for wetponds (see Section 5.10.5.2) shall apply with the following modifications:
  - A sump must be provided in the outlet structure of combined facilities.
  - The detention flow restrictor and its outlet pipe shall be designed according to the requirements for detention ponds (see Chapter 4, Sections 4.6.3 and 4.6.8).

- The "Planting Requirements" for stormwater wetlands are modified to use the following plants which are better adapted to water level fluctuations:
  - *Scirpus acutus* (hardstem bulrush) 2 - 6' depth
  - *Scirpus microcarpus* (small-fruited bulrush) 1 - 2.5' depth

- *Sparganium emersum* (burreed) 1 - 2' depth
- *Sparganium eurycarpum* (burreed) 1 - 2' depth
- *Veronica* sp. (marsh speedwell) 0 - 1' depth

In addition, the shrub Spirea douglasii (*Douglas spirea*) may be used in combined facilities.

5.11 Oil Control Facilities

Oil/water separators rely on passive mechanisms that take advantage of oil being lighter than water. Oil rises to the surface and can be periodically removed. Oil/water separators are designed to remove oil and total petroleum hydrocarbons (TPH) down to 15 mg/L at any time and 10 mg/L on a 24-hour average, and produce a discharge that does not cause an ongoing or recurring visible sheen in the stormwater discharge.

The two types of oil/water separators typically used for stormwater treatment are the baffle type or API (American Petroleum Institute) oil/water separator and the coalescing plate (CP) oil/water separator. See Figures 5.34 and 5.35. Linear sand filters are also approved for oil control (see Section 5.9.7). Spill control separators are often used as a source control BMP, but are not permitted as a stormwater treatment oil control BMP. See Volume 1, the Source Control Technical Requirements Manual for additional details on spill prevention and control.

Two BMPs are described in this section:

- Baffle type separator (API)
- Coalescing plate separator.

Baffle (API) oil/water separators use vaults that have multiple cells separated by baffles extending down from the top of the vault (see Figure 5.34). The baffles block oil flow out of the vault. Baffles are also commonly installed at the bottom of the vault to trap solids and sludge that accumulate over time. In many situations, simple floating or more sophisticated mechanical oil skimmers are installed to remove the oil once it has separated from the water.

Coalescing plate separators are typically manufactured units consisting of a baffled vault containing several inclined corrugated plates stacked and bundled together (see Figure 5.35). The plates are equally spaced (typical plate spacing ranges from 1/4 inch to 1 inch) and are made of a variety of materials, the most common being fiberglass and polypropylene. Efficient separation results because the plates reduce the vertical distance oil droplets must rise in order to separate from the stormwater. Once they reach the plate, oil droplets form a film on the plate surface. The film builds up over time until it becomes thick enough to migrate upward along the inclined plate. When the film reaches the edge of the plate, oil is released as large droplets which rise rapidly to the surface, where

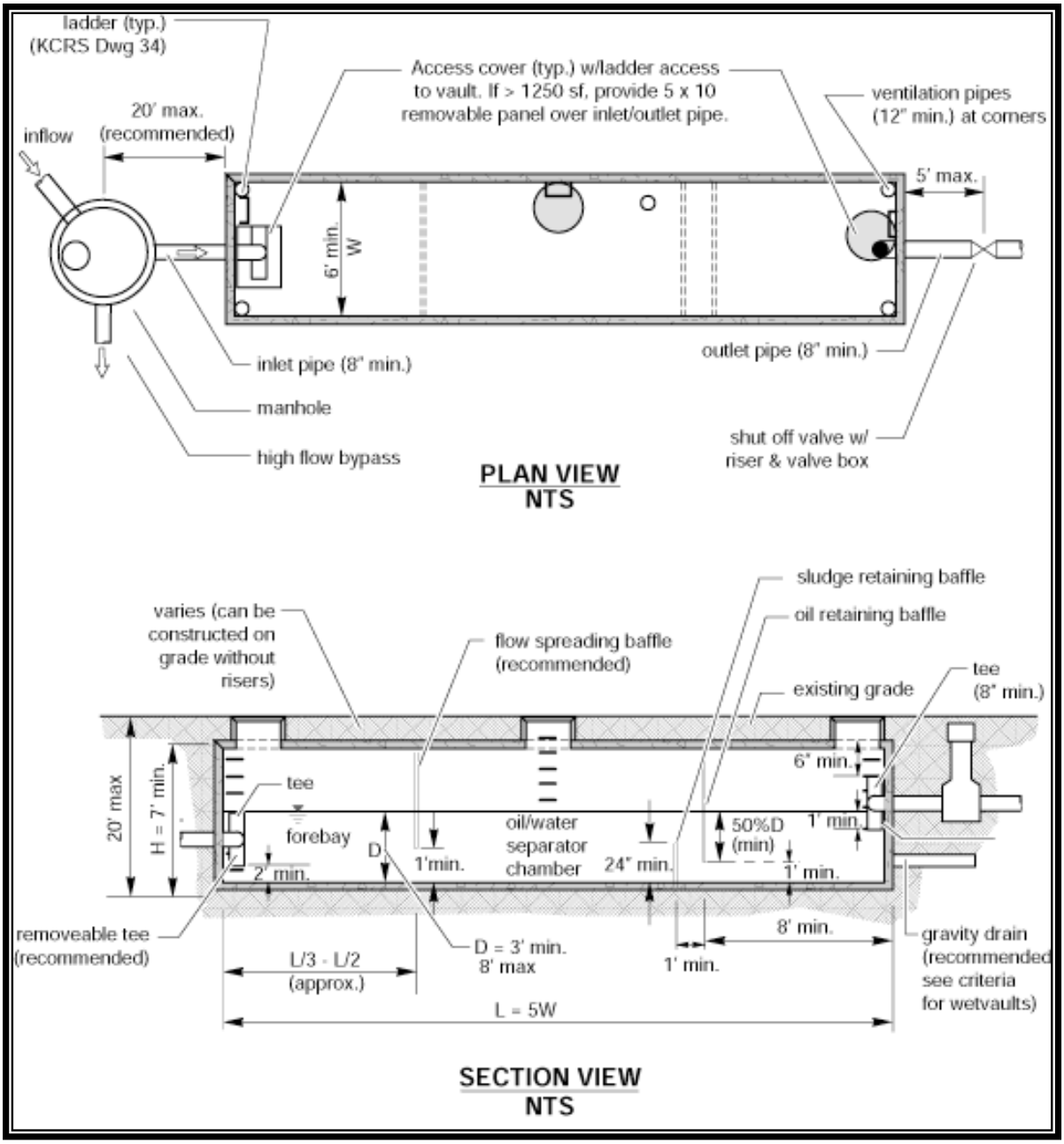


Figure 5.34. Typical API (Baffle Type) Separator.

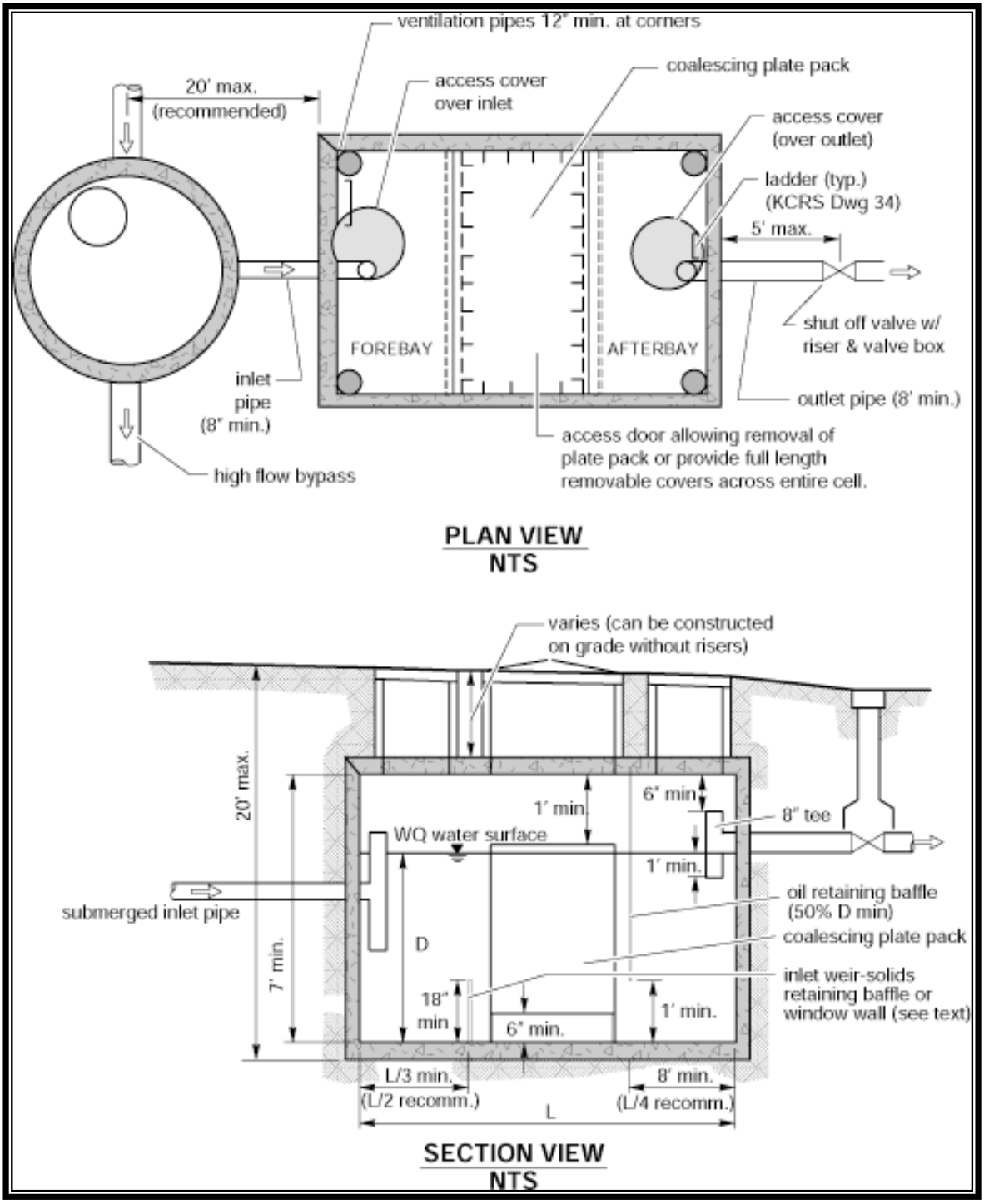


Figure 5.35. Typical Coalescing Plate Separator.

the oil accumulates until the unit is maintained. Because the plate pack increases treatment effectiveness significantly, coalescing plate separators can achieve a specified treatment level with a smaller vault size than a simple baffle separator.

5.11.1 Applications and Limitations

Oil/water separators are designed to remove free oil and are not generally effective in removing oil that has become either chemically or mechanically emulsified or dissolved in the stormwater. Therefore, separators should be installed upstream of facilities and conveyance structures that introduce turbulence that promotes emulsification. Emulsification of oil can also result if surfactants and detergents are used to wash parking areas that drain to the separator. Detergents should not be used in parking areas unless the wash water is collected and disposed of properly (usually to the sanitary sewer).

Oil/water separators are best located in areas where the contributing drainage area is nearly all impervious and a fairly high load of petroleum hydrocarbons is likely to be generated. They are not effective in removing low concentrations of oil and therefore are not recommended for use on sites with very dilute concentrations of petroleum hydrocarbons in the stormwater runoff. Other BMPs, such as sand filters, biofiltration swales, and some of the emerging technologies may be more applicable under these conditions.

Do not use oil/water separators for removing dissolved or emulsified materials such as coolants, soluble lubricants, glycols (anti-freeze), and alcohols.

Excluding unpaved areas helps to minimize the amount of sediment entering the vault, which reduces the need for maintenance. A unit that fails and ceases to function can release previously trapped oil to the downstream receiving water body, via release of oil sediment and from entrained surface oils. Pretreatment should be considered if the level of TSS in the inlet flow would cause clogging or otherwise impair the long-term efficiency of the separator.

The following are potential stormwater treatment applications of oil/water separators where free oil can be expected to be present at sufficient concentrations and where sediment is not likely to overwhelm the separator:

- Commercial and industrial areas including petroleum storage yards, vehicle maintenance facilities, manufacturing areas, airports, utility areas (water, electric, gas), and fueling stations.
- Facilities that would require oil control BMPs under the high-use site threshold described in Section 5.2.1 include parking lots at convenience stores, fast food restaurants, grocery stores, shopping malls, discount warehouse stores, banks, truck fleets, auto and truck dealerships, and delivery services.

- For inflows from small drainage areas (fueling stations, maintenance shops, etc.) a coalescing plate (CP) type separator is typically considered, due to space limitations. However, if plugging of the plates is likely, then a new design basis for the baffle type API separator may be considered on an experimental basis.

There is concern that oil/water separators used for stormwater treatment have not performed to expectations (Watershed Protection Techniques 1994; Schueler 1990). Therefore, emphasis must be given to proper application, design, O & M (particularly sludge and oil removal), and prevention of CP fouling and plugging (US Army Corps of Engineers 1994). Without intense maintenance, oil/water separators may not be sufficiently effective in achieving oil and TPH removal down to required levels. See Appendix D for additional information on maintenance requirements. Other treatment systems, such as sand filters and emerging technologies, should be considered for the removal of insoluble oil and TPH.

5.11.2 Construction Specifications and Criteria

Construction of oil/water separators shall follow the manufacturer's recommended construction procedures and installation instructions, as well as any applicable City requirements. Where flooding or high groundwater presents a risk of vault floatation, the vault shall be properly anchored in accordance with the manufacturer's or the design engineer's recommendations.

Upon completion of installation, the oil/water separator shall be thoroughly cleaned and flushed prior to operation.

Specify appropriate performance tests after installation and shakedown, and/or certification by a professional engineer that the separator is functioning in accordance with design objectives. Expeditious corrective actions must be taken if it is determined the separator is not achieving acceptable performance levels.

5.11.3 Operations and Maintenance Requirements

Operations and maintenance requirements for oil/water separators are provided in Appendix D.

5.11.4 API (Baffle type) Oil/Water Separator Bay

5.11.4.1 Site Considerations

API separators are designed for use on sites larger than 2 acres. The *Stormwater Management Manual for Western Washington* (Ecology 2005) described a design modification for using API separators on sites less than 2 acres (e.g., fueling stations, commercial parking lots). However, Ecology also requires the developer to perform detailed performance verification during at least one wet season when using the modified procedure for small sites. Given

this requirement, the City of Seattle has elected not to allow API separators to be used on sites smaller than 2 acres. The BMP sizing method below only applies to contributing drainage areas larger than 2 acres.

5.11.4.2 BMP Sizing

The API sizing method is based on the horizontal velocity of the bulk fluid (Vh), the oil rise rate (Vt), the residence time (tm), width, depth, and length considerations as follows:

1. Determine the oil rise rate, Vt, in cm/sec, using Stokes' Law (Water Pollution Control Federation 1985) or empirical determination. Stokes Law assumes that flow is laminar and that oil droplets are spherical shaped. Stokes Law equation for rise rate, Vt (ft/min):

   $$Vt = 1.97g(\sigma_w-\sigma_o)D^2 /18\eta_w)$$

   Where:

   1.97 = conversion factor (cm/sec to ft/min)
   g = gravitational constant (981 cm/sec²)
   D = diameter of the oil particle (cm)
   σ_w = water density in grams per cubic centimeter (gm/cc) at 32°F
   σ_o = oil density
   η_w = dynamic viscosity of water (gm/cm-sec) at water temperature of 32°F, (See American Petroleum Institute 1990)

   Use:

   g = 981 cm/sec²
   D = 60 microns (0.006 cm)
   σ_w = 0.999 gm/cc at 32°F
   σ_o = Select conservatively high oil density. For example, if diesel oil @ σ_o=0.85 gm/cc and motor oil @ σ_o = 0.90 gm/cc can be present then use σ_o=0.90 gm/cc
   η_w = 0.017921 gm/cm-sec

2. Determine Q:

   Q = the 15-minute Water Quality design flow rate in ft³/min multiplied by the off-line facility ratio of 3.0. Note that some continuous hydrologic models give the water quality design flow rate in ft³/sec. Multiply this flow rate by 60 to obtain the flow rate in ft³/min.

3. Calculate horizontal velocity of the bulk fluid, V_h (in ft/min) and water depth in separator (d) in feet.

   V_h = 15Vt
   d = $(Q/2V_h)^{1/2}$

   Note: Separator water depth (d) must be: $3 \leq d \leq 8$ feet to minimize turbulence (American Petroleum Institute 1990; US Army Corps of

Engineers 1994). If the calculated depth is less than 3 feet, an API separator is not appropriate for the site. If the calculated depth exceeds 8 feet, consider using two separators.

4. Calculate the minimum residence time (t_m), in minutes, of the separator at depth d:

$$t_m = d/V_t$$

5. Calculate the minimum length of the separator section, l(s):

$$l(s) = FQt_m/wd = F(V_h/V_t)d$$

Where:

F = 1.65
Use depth/width (d/w) ratio of 0.5 (American Petroleum Institute 1990)

For other dimensions, including the length of the forebay, the length of the afterbay, and the overall length, L; refer to Figure 5.34.

6. Calculate $V = l(s)wd = FQt_m$, and $A_h = wl(s)$

V = minimum hydraulic design volume, in cubic feet
A_h = minimum horizontal area of the separator, in square feet.

5.11.4.3 Design Criteria

Note: the following criteria apply to both API baffle and coalescing plate separators, unless otherwise specified.

General Siting

- Oil/water separators must be located off-line to avoid flushing oil and oil-laden sediment offsite during large storm events. When a separator is required (i.e., high use sites), it must be installed upstream of any other stormwater treatment facility. In addition, separators must be located upstream of any pumps to prevent oil present in the stormwater from becoming emulsified. If it is not possible to locate the separator off-line (e.g., roadway intersections), try to minimize the size of the area requiring oil control.
- When a separator is required, it shall precede other stormwater treatment facilities (except wet vaults). Separators may be located upstream or downstream of flow control facilities, since there are both advantages and disadvantages with either placement.
- In moderately pervious soils where seasonal groundwater may induce floatation, buoyancy shall be balanced by ballasting or other methods as appropriate (see also Section 5.11.2, Construction Specifications and Criteria).

- Any pumping devices shall be installed downstream of the separator to prevent the oil present in the stormwater from emulsifying.

Vault Structure-General

- Separator vaults shall be watertight. Pipes entering and exiting a vault below the water quality design water surface, shall be sealed using a non-porous, non-shrinking grout.
- Separator vaults shall have a shutoff mechanism on the outlet pipe to prevent oil discharges during maintenance and to provide emergency shutoff capability in the event of a spill. A valve box and riser shall be provided according to the design criteria for wet vaults (see Section 5.10.4).

Vault Structure and Geometry — API Separators

- API separators shall be divided into three compartments: a forebay, an oil separation cell, and an afterbay. The forebay is designed primarily to trap and collect sediment, support plug flow conditions, and reduce turbulence. The oil separation cell traps and holds oil as it rises from the water column, and it serves as a secondary sediment collection area. The afterbay provides a relatively oil-free cell before the outlet and provides a secondary oil separation area.
- The length of the forebay shall be approximately 1/3 to 1/2 the length of the vault. In addition, the surface area of the forebay must be at least 20 square feet per 10,000 square feet of impervious area draining to the separator.
- A removable flow-spreading baffle, extending from the surface to a depth of up to 1/2 the vault depth (D) is recommended to spread flows. Design guidelines for level spreaders are provided in Section 5.4.5.
- The removable bottom baffle (sediment-retaining baffle) shall be a minimum of 24 inches (see Figure 5.34), and located at least 1 foot from the oil-retaining baffle. A "window wall" baffle may be used, but the area of the window opening must be at least three times greater than the area of the inflow pipe.
- A removable oil retaining baffle shall be provided and located approximately 1/4 L from the outlet wall or a minimum of 8 feet, whichever is greater (the 8-foot minimum is for maintenance purposes). The oil-retaining baffle shall extend from the elevation of the water surface to a depth of at least 50 percent of the design water depth. Various configurations are possible, but the baffle shall be designed to minimize turbulence and entrainment of sediment.
- Baffles may be fixed rather than removable if additional entry ports and ladders are provided so that both sides of the baffle are accessible by maintenance crews.

- Baffle separator vaults shall have a minimum length-to-width ratio of 5:1.
- The design water depth (D) shall be no deeper than 8 feet unless approved by the Director. Depths greater than 8 feet may be permitted on a case-by-case basis, taking into consideration the potential for depletion of oxygen in the water during the warm summer months.
- Baffle separator vaults shall have a design water depth-to-width ratio of between 0.3 and 0.5.

Vault Structure and Geometry — Coalescing Plate Separators

In addition to the general criteria outlined previously, the following criteria apply specifically to coalescing plate separators:

- Coalescing plate separators shall be divided by baffles or berms into three compartments: a forebay, an oil separation cell which houses the plate pack, and an afterbay. The forebay controls turbulence and traps and collects debris. The oil separation cell captures and holds oil. The afterbay provides a relatively oil-free exit cell before the outlet.
- The length of the forebay shall be a minimum of 1/3 the length of the vault, L (but 1/2 L is recommended). In addition, it is recommended that the surface area of the forebay be at least 20 square feet per 10,000 square feet of tributary impervious area draining to the separator. In lieu of an attached forebay, a separate grit chamber, sized to be at least 20 square feet per 10,000 square feet of tributary impervious area, may precede the oil/water separator.
- An oil-retaining baffle shall be provided. For large units, a baffle position of 0.25 L from the outlet wall is recommended. The oil-retaining baffle shall extend from the water surface to a depth of at least 50 percent of the design water depth. Various configurations are possible, but the baffle shall be designed to minimize turbulence and entrainment of sediment.
- A bottom sediment-retaining baffle shall be provided upstream of the plate pack. The minimum height of the sludge-retaining baffle shall be 18 inches. Window walls may be used, but the window opening must be a minimum of three times greater than the area of the inflow pipe.
- It is recommended that entire space between the sides of the plate pack and the vault wall be filled with a solid but light-weight removable material such as a plastic or polyethylene foam to reduce short-circuiting around the plate pack. Rubber flaps are not effective for this purpose.
- The separator plates should meet the following requirements:
    - Plates shall be inclined at 45° to 60° from the horizontal. This range of angles exceeds the angle of repose of many solids and therefore provides more effective droplet separation while minimizing the accumulation of solids on the individual plates.

- Plates shall have a minimum plate spacing of 1/2-inch and have corrugations
- Plates shall be securely bundled in a plate pack for ease of removal and cleaning (with high-pressure rinse or equivalent)
- The plate pack shall be a minimum of 6 inches from the vault bottom for sediment storage
- There should be 1 foot of head space between the top of the plate pack and the bottom of the vault cover.

Inlet and Outlet

- The separator inlet shall be submerged. A tee section may be used to submerge the incoming flow and must be at least 2 feet from the bottom of the tank and extend above the water quality design water surface.

  Intent: The submerged inlet is to dissipate energy of the incoming flow. The distance from the bottom is to minimize resuspension of settled sediments. Extending the tee to the surface allows air to escape the flow, thus reducing turbulence. Alternative inlet designs that accomplish these objectives are acceptable.

- The vault outlet pipe shall be sized to pass the water quality design flow before overflow. The vault outlet pipe shall be back-sloped or have a tee extending 1 foot above and below the water quality design water surface to provide for secondary trapping of oils and floatables in the wetvault. *Note: The invert of the outlet pipe sets the water quality design water surface elevation.*

- For off-line separators, the high flow bypass must be designed so that all flows up to and including the water quality design flow rate are directed to the separator. Design guidelines for flow splitters are provided in Section 5.4.5. The water quality design flow rate is calculated by multiplying the design flow rate determined using an approved continuous simulation model by the off-line ratio of 3.0. For online separators, the water quality design flow rate is calculated by multiplying the flow rate determined using an approved continuous simulation model by the online ratio of 1.65. Separators must be designed as off-line facilities wherever possible.

Material Requirements

- All metal parts shall be corrosion-resistant. Zinc and galvanized materials are prohibited. Painting metal parts for corrosion resistance is not allowed due to lack of longevity.
- Vault baffles shall be concrete, stainless steel, fiberglass reinforced plastic, or other acceptable material and shall be securely fastened to the vault.
- Gate valves, if used, shall be designed for seating and unseating heads appropriate for the design conditions.

- For coalescing plate separators, plate packs shall be made of fiberglass, stainless steel, or polypropylene, unless otherwise recommended by the manufacturer and approved by the Director.

Access Requirements

Same as for wetvaults (see Section 5.10.4.2) except for the following modifications:

- Access to each compartment is required. If the length or width of any compartment exceeds 50 feet, an additional access point for each 50 feet is required.
- Access points for the forebay and afterbay shall be positioned partially over the inlet or outlet tee to allow visual inspection as well as physical access to the bottom of the vault.
- For coalescing plate separators, the following also apply:
  - Access to the compartment containing the plate pack shall be a removable panel or other access able to be opened wide enough to remove the entire coalescing plate bundle from the cell for cleaning or replacement. Doors or panels shall have stainless steel lifting eyes, and panels shall weigh no more than 5 tons per panel.
  - A parking area or access pad (25-foot by 15-foot minimum) shall be provided near the coalescing plate bundles to allow for their removal from the vault by a truck-mounted crane or backhoe, and to allow for extracting accumulated solids and oils from the vault using a vactor truck.

Recommended Design Features

The following recommended design features should be applied:

- A gravity drain for maintenance is recommended if grade allows. The drain invert should be at a depth equal to the depth of the oil retaining baffle. Deeper drains are encouraged where feasible.
- If large amounts of oil are likely to be captured, a bleed-off pipe and separate waste oil tank can be located adjacent to the vault to channel separated oils into the tank. This improves the overall effectiveness of the facility, especially if maintenance is only annually. It also improves the quality of the waste oil recovered from the facility.
- If practicable, determine oil/grease (or TPH) and TSS concentrations, lowest temperature, pH, and empirical oil rise rates in the runoff, and the viscosity, and specific gravity of the oil. Also determine whether the oil is emulsified or dissolved.
- Oil retaining baffles (top baffles) should be located at least at 1/4 of the total separator length from the outlet and should extend down at least 50 percent of the water depth and at least 1 foot from the separator bottom.

- Baffle height to water depth ratios should be 0.85 for top baffles and 0.15 for bottom baffles.
- To collect floatables and settleable solids, design the surface area of the forebay at ≥ 20 ft² per 10,000 ft² of area draining to the separator. The length of the forebay should be 1/3 to 1/2 of the length of the entire separator. Include roughing screens for the forebay or upstream of the separator to remove debris, if needed. Screen openings should be about 3/4 inch.
- Use absorbents and/or skimmers in the afterbay as needed.

5.11.5 Coalescing Plate (CP) Oil/Water Separator Bay

5.11.5.1 Site Considerations

Coalescing plate separators (CPS) are typically smaller than API separators and are suitable for sites where space is limited. CPS units are required for sites smaller than 2 acres that are required to provide oil control, where API separators are not allowed.

CPS designs may be required to add pretreatment for TSS that could cause clogging of the CP separator, or otherwise impair the long-term effectiveness of the separator.

5.11.5.2 BMP Sizing

CPS units are sized the same as API separators. See oil rise rate (V_t) calculations under Section 5.11.4.2. Calculate the projected (horizontal) surface area of plates needed using the following equation:

$$A_p = Q/V_t = Q/0.00386(\sigma_w - \sigma_o/\eta_w)$$

$$A_p = A_a(\text{cosine } b)$$

Where:

A_p = projected surface area of the plate in ft²; .00386 is unit conversion constant
Q = the online (1.65) or off-line (3.0) adjustment factor x the 15-minute water quality design flow rate, ft³/min
V_t = Rise rate of 0.033 ft/min, or empirical determination, or Stokes Law based
σ_w = density of water at 32ºF
σ_o = density of oil at 32ºF
A_a = actual plate area in ft² (one side only)
b = angle of the plates with the horizontal in degrees (usually varies from 45 to 60 degrees)
η_w = viscosity of water at 32ºF.

5.11.5.3 Design Criteria

Design criteria for coalescing plate separators are outlined in Section 5.11.4.3 above.

5.12 Emerging Technologies

This manual describes City requirements for stormwater treatment based on the existing knowledge of stormwater management principles and existing treatment technologies that have been approved by Ecology at the time this manual was written. The City recognizes that advances in stormwater treatment will occur in the future and does not want to preclude the use of these new/emerging technologies should they prove to provide equivalent treatment effectiveness and applicability to Seattle's urban environment than the technologies currently described in this manual. This section describes how the City of Seattle will evaluate the use of new stormwater treatment technologies that may be developed in the future and that are not covered in this manual.

5.12.1 Ecology's Evaluation of Emerging Technologies

To receive Ecology approval for use in stormwater applications in Washington, new technologies must be evaluated following Ecology's technology assessment protocols (TAPE and CTAPE), which establish guidelines for evaluating the performance of stormwater BMPs. The evaluation process requires that manufacturers develop and implement a quality assurance project plan to field-test the performance of new stormwater treatment technologies. After the successful completion of field testing, the vendor then submits a technology evaluation report to Ecology for review and approval. Ecology has formed two technical review committees to evaluate and approve new technologies:

- TRC for ultra-urban treatment technologies and those treatment technologies that do not have a chemical component for treatment
- CTRC (chemical technology review committee) for construction site treatment technologies and any technology that uses a chemical component for treatment.

The technical review committees consist of representatives from local governments from eastern and western Washington that act in an advisory capacity to provide recommendations to Ecology on the level of development of each technology. Information about Ecology's evaluation process can be downloaded (http://ecy.wa.gov/programs/wq/stormwater/newtech/index.html).

Under the technology assessment process, Ecology assigns "Use Level Designations" to emerging technologies based on the results of the TAPE and CTAPE analysis. These designations are described below:

- **GULD - General Use Level Designation**
  A General Use Level Designation (GULD) is assigned to technologies for which the performance test results demonstrate with a sufficient degree of

confidence, that the technology is expected to achieve Ecology's performance goals.

- **CULD - Conditional Use Level Designation**
  A Conditional Use Level Designation (CULD) is assigned to technologies that are in widespread use in Washington or are considered equivalent to approved technologies, and they are considered likely to attain a General Use Level provided that testing following the established protocols is completed within a specified time period. Not all CULD technologies are required to conduct field testing. Units that are in place do not have to be removed after the specified time period. Use is subject to the use level designation conditions.

- **PLD - Pilot Level Designation**
  A Pilot Use Level Designation (PULD) is assigned to promising new technologies prior to completion of performance testing. The PULD allows limited use of the technology to allow field testing to be conducted. PULD technologies may be installed provided that the vendor and/or developer agree to conduct field testing based on the TAPE at all installations.

Note: As of late 2009, Ecology has put the TAPE process on hold and is not evaluating new technologies until further notice. See Ecology's website for further updates (http://ecy.wa.gov/programs/wq/stormwater/newtech/index.html).

5.12.2 Applicability and Restrictions

The City will rely on the results of Ecology's stormwater treatment technology evaluation process to identify BMPs that can be used on new and redevelopment projects in Seattle. The Director will accept technologies approved by Ecology as described below:

- GULD technologies will be accepted subject to the conditions of use established by Ecology/TRC.
- CULD technologies will also be accepted provided that the project owner signs a maintenance agreement with the City stating that the owner will maintain the structure in accordance with manufacturers' and/or Ecology's guidelines at all times and that the owner will modify/upgrade the system in accordance with any conditions that Ecology/TRC may require as part of the final GULD designation following field testing. The owner must also file annual reports as outlined for the specific CUD BMP by the City.
- PULD technologies will be accepted on a limited basis to enable manufacturers to obtain data to help fulfill the requirements of the testing protocol of the Technical Review Committee. These projects must be approved in advance by the Director of Seattle Public Utilities, have an approved monitoring plan from the Technical Review Committee or

Department of Ecology, and provide a financial bond to provide clean-up and replacement in the event of failure.

5.12.3 Operations and Maintenance Requirements

Refer to the Department of Ecology's web site (below) and the manufacturer for facility-specific maintenance requirements (http://www.ecy.wa.gov/programs/WQ/stormwater/newtech/technologies.html).

5.13 References

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Chapter 6 - Hydrologic Analysis and Design

6.1 Introduction

This chapter presents hydrologic modeling concepts to support the design of stormwater management BMPs to meet the minimum requirements in the Stormwater Code and in Chapter 2 of this volume. It includes descriptions of acceptable methods for estimating the quantity and hydrologic characteristics of stormwater runoff, and the assumptions and data requirements of these methods. Specifically, hydrologic tools and methods are presented for the following tasks:

- Calculating runoff hydrographs and time series using single-event and continuous rainfall runoff models
- Calculating peak flows for conveyance, peak flow detention and retention, and water quality rate treatment BMPs
- Calculating volumes for detention and retention and water quality volume treatment BMPs
- Calculating flow durations for flow duration detention and retention based requirements.

Flow control and water quality performance standards are presented in Chapter 2. BMP design requirements and specifical modeling methods are provided in Chapters 4 and 5. Any request for alternative calculation methods shall follow the principles laid out in this chapter and be approved by the Directors.

6.2 Applicability of Hydrologic Analysis Methods

The choice of a hydrologic analysis method depends on the type of facility being designed (conveyance, detention, or water quality) and the required performance standard. The size of the tributary area and watershed characteristics, including backwater effects should also be considered.

Hydrologic analysis approaches may be grouped into three categories;

- Rational method
- Single-event rainfall-runoff models
- Continuous rainfall-runoff models.

The applicability of each method is summarized in Table 6.1 and discussed in the following sections.

Table 6.1. Hydrologic Analysis Method Applicability

Type of Computation	Constraints	Peak Flow Conveyance Sizing	Flow Control BMPs	Water Quality Treatment BMPs	TESC Design Flows
Rational Method	<10 acres (measured to individual conveyance elements) Upstream of storage routing and backwater effects	Acceptable	NA	NA	Acceptable
Single Event Methods	NA	Acceptable	NA	NA	Acceptable
Continuous Flow Modeling	See Table 6.21 for time step requirements	Acceptable	Acceptable	Acceptable	Acceptable

NA – Not Applicable

6.2.1 Rational Method

The rational method is appropriate for designing conveyance systems that receive runoff from small quickly responding areas (less than 10 acres) where short intense storms generate the highest peak flow. The method only produces a flow peak discharge rate and routing effects are not included. Advantages of this method are that it is easy to apply and generally produces conservative results. For larger, more complex basins, routing and timing of the flood peaks from various sub-areas becomes more important and the rational method can produce overly conservative results. For complex basins, single-event or continuous rainfall-runoff methods are required.

6.2.2 Single-Event Rainfall-Runoff Methods

Single-event models simulate rainfall-runoff for a single storm typically 2 hours to 72 hours in length, usually of a specified exceedance probability (recurrence interval). Single event methods are applicable for sizing conveyance facilities. Acceptable single-event hydrologic models include:

- Natural Resources Conservation Service (formerly the Soil Conservation Service (SCS)) TR-55
- Santa Barbara Urban Hydrograph (SBUH) method
- StormShed
- Corps of Engineers HMS and HEC-1
- SWMM, PCSWM and XP-SWMM
- Other peer-reviewed hydrograph models approved by the Directors.

6.2.3 Continuous Rainfall-Runoff Simulation Methods

Continuous rainfall-runoff models use multi-decade precipitation and evaporation time series as input to produce a corresponding multi-decade time series of runoff.

Continuous models are to be used to size stormwater management facilities to meet peak and/or flow duration performance standards, as well as water quality treatment requirements. Discharge rates computed with continuous models may also be used to size conveyance facilities. For continuous modeling, the time step of the precipitation data input and the computational time should be 5 minutes, unless specified otherwise. See also Section 6.5.4.1.

Approved continuous hydrologic models for use in designing flow control and water quality facilities in accordance with the minimum requirements outlined in Chapter 2 include:

- USEPA Hydrologic Simulation Program Fortran (HSPF)
- Washington State Department of Ecology Western Washington Hydrology Model (WWHM)
- MGSFlood.

Because the City accepts flow control and water quality treatment design calculations from multiple models, Seattle requires the use of a post-processing software called SPUHydroStats. HydroStats is a statistical post-processor that reads output from any approved continuous simulation hydrologic models listed above and creates a standardized project report for design review submittal. The HydroStats program and User's Manual are available for free download from the DPD website.

In addition, the following continuous hydrologic models may be used for project specific situations not directly related to compliance with the minimum requirements outlined in Chapter 2:

- InfoWorks
- Mouse
- ModFlowHMS
- Other models approved by the Directors.

6.3 Rational Method

6.3.1 Rational Method Equation

The rational method is based on the assumption that rainfall intensity for any given duration is uniform over the entire tributary watershed. The rational

formula relates peak discharge from the site of interest to rainfall intensity times a coefficient:

$$Q = CiA \quad (1)$$

Where:
- Q = Peak discharge from the site of interest
- C = Dimensionless runoff coefficient
- i = Rainfall Intensity for a given recurrence interval (inches/hour)
- A = Tributary area (acres)

The method only produces a peak discharge rate (not a hydrograph) and is therefore useful for applications where the flow peak is of interest, such as for conveyance sizing.

The rainfall intensity (i) is determined from Figure 6.1 or Table 6.2 for the precipitation recurrence interval of interest and duration corresponding to the time of concentration computed per Section 6.3.4.

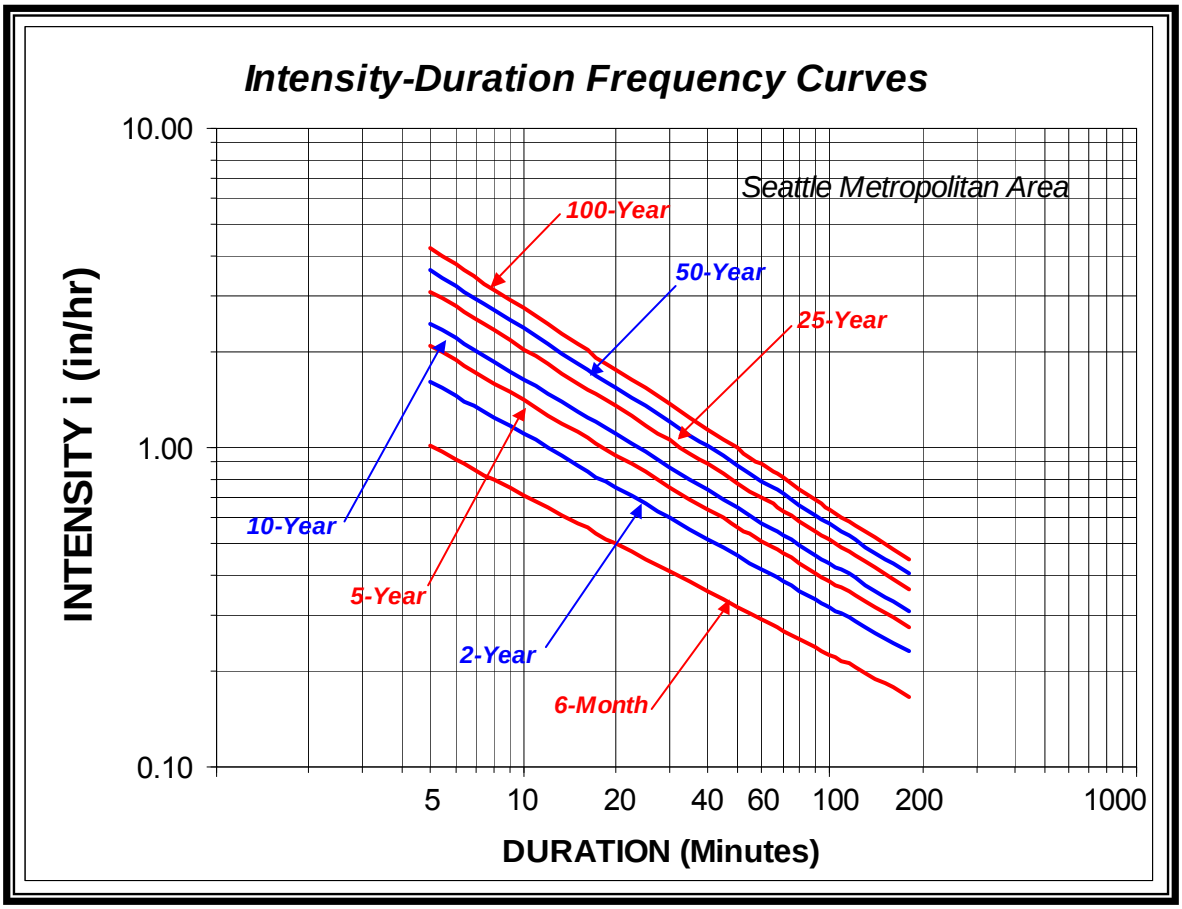

Figure 6.1. Intensity-Duration-Frequency Curves, City of Seattle.

Table 6.2. Intensity-Duration-Frequency Values for Durations from 5-Minutes through 180-Minutes for Selected Recurrence Intervals for the Seattle Metropolitan Area.

Duration (minutes)	Precipitation Intensities (in/hr)							
	Recurrence Interval (years)							
	6-Mo	2-yr	5-yr	10-yr	20-yr	25-yr	50-yr	100-yr
5	1.01	1.60	2.08	2.45	2.92	3.08	3.61	4.20
6	0.92	1.45	1.87	2.21	2.62	2.76	3.23	3.75
8	0.80	1.24	1.59	1.87	2.21	2.32	2.71	3.13
10	0.71	1.10	1.40	1.64	1.93	2.03	2.36	2.72
12	0.65	1.00	1.27	1.48	1.74	1.82	2.11	2.43
15	0.58	0.88	1.12	1.30	1.52	1.60	1.84	2.11
20	0.50	0.75	0.95	1.10	1.28	1.34	1.54	1.76
25	0.45	0.67	0.84	0.97	1.12	1.18	1.35	1.53
30	0.41	0.61	0.76	0.87	1.01	1.05	1.21	1.37
35	0.38	0.56	0.69	0.80	0.92	0.96	1.10	1.24
40	0.35	0.52	0.64	0.74	0.85	0.89	1.01	1.14
45	0.33	0.49	0.60	0.69	0.79	0.83	0.94	1.06
50	0.32	0.46	0.57	0.65	0.74	0.78	0.88	0.99
55	0.30	0.44	0.54	0.61	0.70	0.73	0.83	0.94
60	0.29	0.42	0.51	0.58	0.67	0.70	0.79	0.89
65	0.28	0.40	0.49	0.56	0.64	0.66	0.75	0.84
70	0.27	0.38	0.47	0.53	0.61	0.64	0.72	0.80
80	0.25	0.36	0.43	0.49	0.56	0.59	0.66	0.74
90	0.24	0.33	0.41	0.46	0.52	0.55	0.62	0.69
100	0.22	0.32	0.38	0.43	0.49	0.51	0.58	0.64
120	0.20	0.29	0.35	0.39	0.44	0.46	0.52	0.57
140	0.19	0.26	0.32	0.36	0.40	0.42	0.47	0.52
160	0.18	0.24	0.29	0.33	0.37	0.39	0.43	0.48
180	0.17	0.23	0.27	0.31	0.35	0.36	0.40	0.45

6.3.2 Peak Rainfall Intensity Duration Frequency (IDF Curves)

Rainfall intensity-duration-frequency (IDF) curves allow calculation of the average design rainfall intensity for a given exceedance probability (recurrence interval) over a range of durations. Analyses of precipitation-frequency statistics presented in this chapter were conducted using data from the City's 17-gauge precipitation measurement network within the City of Seattle, and the national NOAA cooperative gage network 13. Durations of 5 minutes, 10 minutes,

15 minutes, 20 minutes, 30 minutes, 45 minutes, 60 minutes, 2 hours, 3 hours, 6 hours, 12 hours, 24 hours, 48 hours, 72 hours, and 7 days were analyzed to develop the IDF curves. IDF curves for storm durations up to 3 hours and applicable to sites within the City of Seattle are shown in Figure 6.1.

6.3.3 Runoff Coefficients

Runoff coefficients vary with the tributary land cover and to a certain extent, the total depth and intensity of the rainfall. The storm depth and intensity is typically neglected and the runoff coefficient is based on land cover only (Table 6.3). For watersheds containing several land cover types, an aggregate runoff coefficient can be developed by computing the area weighted average from all cover types present (equation 2):

$$C_c = (C_1A_1 + C_2A_2 + C_3A_3 + \ldots + C_nA_n)/A_t \qquad (2)$$

Where:
C_c = Composite runoff coefficient for the site
$C_{1,2,,\ldots n}$ = The runoff coefficient for each land cover type
$A_{1,2,,\ldots n}$ = Area of each land cover type (acres)
A_t = Total tributary area (acres).

Table 6.3. Rational Equation Runoff Coefficients (C).

Land Cover	Runoff Coefficient (C)
Dense Forest	0.10
Light Forest	0.15
Pasture	0.20
Lawns	0.25
Gravel Areas	0.80
Pavement and Roofs	0.90
Open Water (Ponds Lakes and Wetlands)	1.00

6.3.4 Time of Concentration Estimation

Time of concentration (T_c) is defined as the time it takes for runoff to travel from the hydraulically most distant point of the drainage area to the outlet. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

$$T_c = T_1 + T_2 + T_3 + \ldots T_n \qquad (3)$$

Where:
T_c = The time of concentration in minutes
$T_{1,2,3,\ldots n}$ = is the time for consecutive flow path segments with different land cover categories or flow path slope.

Travel time for each segment is computed using the following equation:

$$T_t = \frac{L}{60V} \tag{4}$$

Where: T_t = Travel time (minutes)
L = Length of flow across a given segment (feet)
V = average velocity (ft/sec) across the land segment.

$$V = k_r \sqrt{S_o} \tag{5}$$

Where: k_r = Velocity factor (Table 6.4)
S_0 = Slope of flow path (feet/feet).

Table 6.4. Coefficients for Average Velocity Equation (k_r).

Land Cover	Velocity Factor (k_r)
Forest with Heavy Ground Cover and Meadow	2.5
Grass, Pasture, and Lawns	7.0
Nearly Bare Ground	10.1
Grassed Swale or Channel	15.0
Paved Areas	20.0

6.4 Single-Event Rainfall-Runoff Methods

6.4.1 Introduction

Single-event models simulate rainfall-runoff processes for a single storm typically 2 hours to 72 hours in length, usually of a specified exceedance probability. Because the primary interest is the flood hydrograph, calculation of evapotranspiration, soil moisture changes between storms, and base flow processes are typically not needed. This is in contrast to continuous rainfall-runoff models (Section 6.5) where multi-decade precipitation and evaporation time series are used as input to produce a corresponding multi-decade time series of runoff.

Precipitation input to single-event models can include either historic data recorded from a rain gage or a synthetic design storm hyetograph. This section describes the use of both types of precipitation input.

6.4.2 Design Storm Hyetographs

Design storm hyetographs were developed using noteworthy storms that were recorded by the City of Seattle gauging network. Statistical analyses were conducted for the storm characteristics and dimensionless design storms were developed for short, intermediate, and long-duration storm events (MGS 2004). The short, intermediate, and long-duration design storms can be scaled to any

site-specific recurrence interval using precipitation magnitudes at the 2-hour, 6-hour, and 24-hour duration.

The choice of a design storm hyetograph depends on the characteristics of the watershed being analyzed. Short duration storms have high intensity but limited volume and are generally the controlling storm for small watersheds with little hydrologic storage. Intermediate and long duration storms have progressively lower intensities but higher volume. These types of storms often control the design of conveyance facilities in larger watersheds and control the design of volume-dependent structures.

Table 6.5 summarizes the applicability of the four City design storms. If multiple storm types are listed for a particular application, then all applicable storm types should be considered candidates and used in the hydrologic model. The candidate storm that produces the most severe hydrologic loading and most conservative design is then adopted as the design storm. Note that this table does not override the modeling requirements for specific facilities outlined in Chapters 2, 4, and 5 or Table 6.1. Table 6.5 is for general guidance and applicability only.

Table 6.5. Applicability of Storm Types for Hydrologic Design Applications.

Storm Type	Design Applicability	Total Storm Duration	Precipitation from SPU Rain Gages
Short	Conveyance Flow Control	3 Hours	2 Hours
Intermediate	Conveyance Flow Control	18 Hours	6 Hour
Seattle 24-Hour	Volume Based BMPs	24 Hour	24 Hour
Long – Front Loaded	Flow Control	64 Hours	24 Hour
Long – Back Loaded	Flow Control	64 Hours	24 Hour

6.4.2.1 Short Duration Storm (3-hour)

Short duration storms typically occur in the late spring through early-fall seasons and are characterized by high intensities for short periods of time over localized areas. These types of storms can produce high rates of runoff and flash flooding in urban areas.

Short-duration design storms are used for design situations where peak discharge is of primary interest. Common applications include design of storm drains, ditches, and culverts, and other hydraulic structures for conveyance. The short-duration storm hyetograph is 3 hours in duration. The storm temporal pattern is shown in Figure 6.2 as a dimensionless unit hyetograph. Tabular values for this hyetograph are listed in Appendix B. The total storm precipitation is 1.06 times the 2-hour precipitation amount.

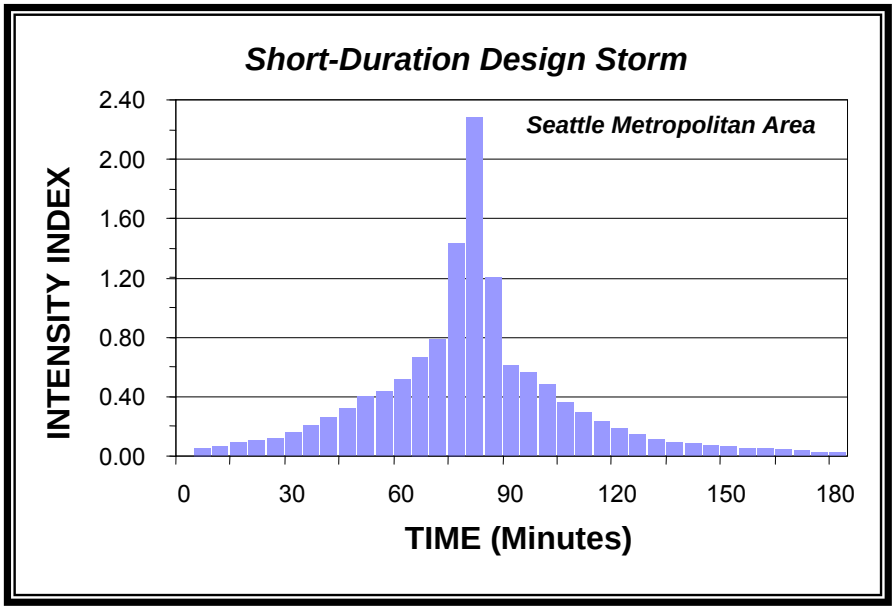

Figure 6.2. Dimensionless Short-Duration (3-Hour) Design Storm, Seattle Metropolitan Area.

The following steps describe how to utilize this storm in hydrologic analyses.

Step 1 – Obtain the 2-hour precipitation amount for the recurrence interval of interest for the watershed (refer to Appendix C and the DPD-SPU Stormwater webpage for modeling resources).

Step 2 – Multiply the 5-minute incremental ordinates of the dimensionless short-duration design storm (Appendix B, Table B-1) by the 2-hour value from Step 1. Note that the resulting storm has a duration of 3 hours and total storm amount will be 1.06 times the volume of the 2-hour precipitation (refer to the DPD-SPU Stormwater webpage for modeling resources).

Step 3 – Input the resulting storm hyetograph into the hydrologic model. The resultant incremental precipitation ordinates have units of inches. To obtain the corresponding intensities (inch/hour), multiply the precipitation increments by 12.

6.4.2.2 Intermediate Duration Storm (18-hour)

Intermediate-duration design storms are used in design applications where both peak discharge and runoff volume are important considerations and there is a need for a runoff hydrograph. Intermediate duration storms generally occur in the fall to early winter.

The intermediate-duration storm hyetograph is 18 hours in duration. The storm temporal pattern is shown in Figure 6.3 as a dimensionless unit hyetograph. Tabular values for this hyetograph are listed in Appendix B. The total storm precipitation is 1.51 times the 6-hour precipitation amount.

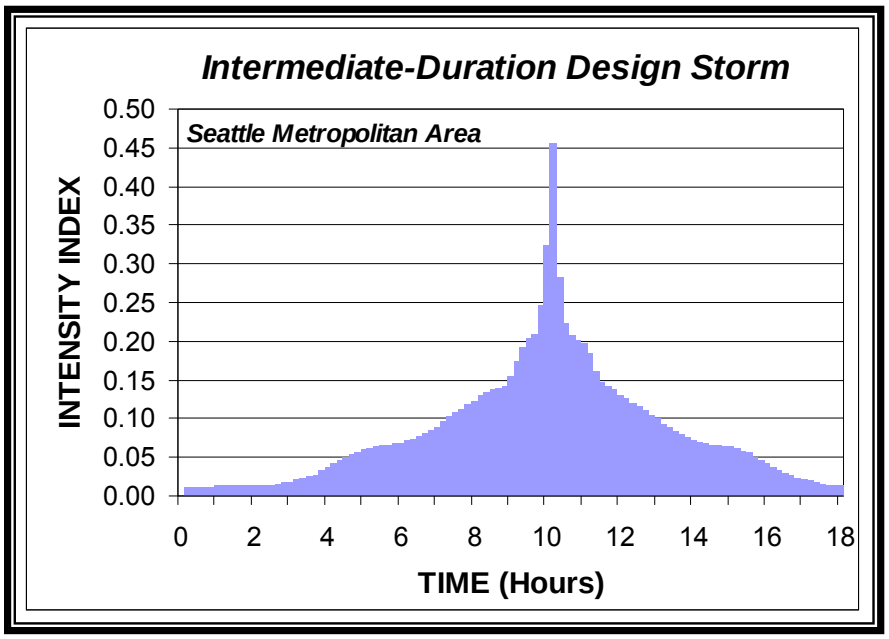

Figure 6.3. Dimensionless Intermediate-Duration (18-Hour) Design Storm, Seattle Metropolitan Area.

The following steps describe how to utilize this storm in hydrologic analyses.

Step 1 – Obtain the 6-hour precipitation amount for the recurrence interval of interest for the watershed (refer to Appendix C and the DPD-SPU Stormwater webpage for modeling resources).

Step 2 – Multiply the 10-minute incremental ordinates of the dimensionless intermediate-duration design storm (Appendix B, Table B-2) by the 6-hour value from Step 1. Note that the resulting storm has a duration of 18 hours and the total storm amount will be 1.51 times the volume of the 6-hour precipitation (refer to the DPD-SPU Stormwater webpage for modeling resources).

Step 3 – Input the resulting storm hyetograph into the hydrologic model. The resultant incremental precipitation ordinates have units of inches. To obtain the corresponding intensities (inch/hour), multiply the precipitation increments by 6.

6.4.2.3 Long Duration Storm (64-hour)

Long-duration design storms are primarily used in design of stormwater detention facilities and other projects where runoff volume is a primary consideration. Long duration storms occur primarily in the late fall into early spring.

Two long-duration dimensionless design storms are provided: a front-loaded design storm with the highest intensities at the beginning of the storm; and a back-loaded storm with the higher intensities nearer the end of the storm period. Characteristics of the front-loaded design storm have been observed more frequently, and this storm would be expected to produce more “typical” runoff

conditions. The back-loaded storm occurs less often and is typically a more conservative event for drainage control facility design.

The long-duration storm hyetographs are 64 hours in duration. The storm temporal patterns for the front loaded and back loaded storms are shown in Figures 6.4 and 6.5 respectively. Tabular values for these storms are listed in Appendix B. The total storm precipitation is 1.29 times the 24-hour precipitation amount for both the front and back loaded long-duration storm.

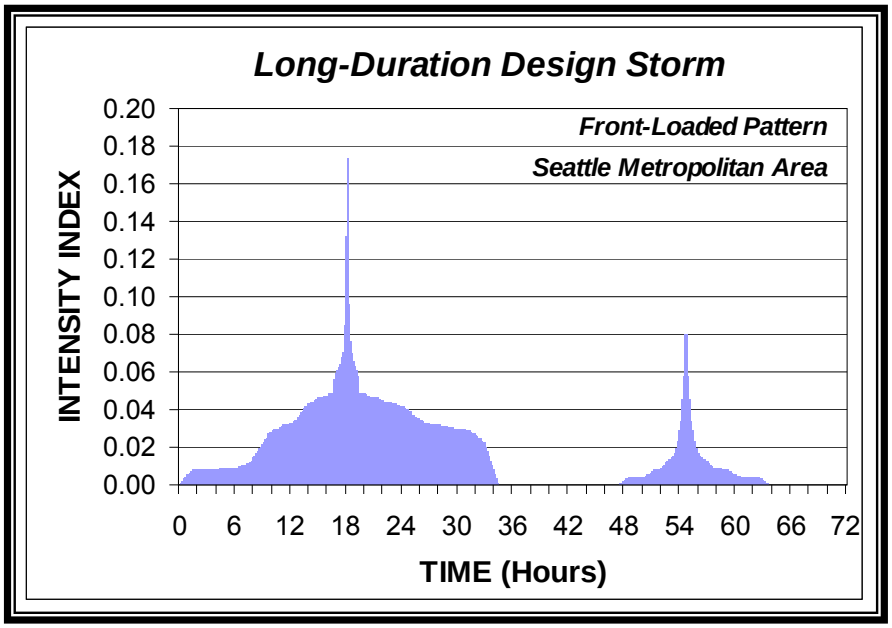

Figure 6.4. Dimensionless Front-Loaded Long-Duration (64-Hour) Design Storm for the Seattle Metropolitan Area.

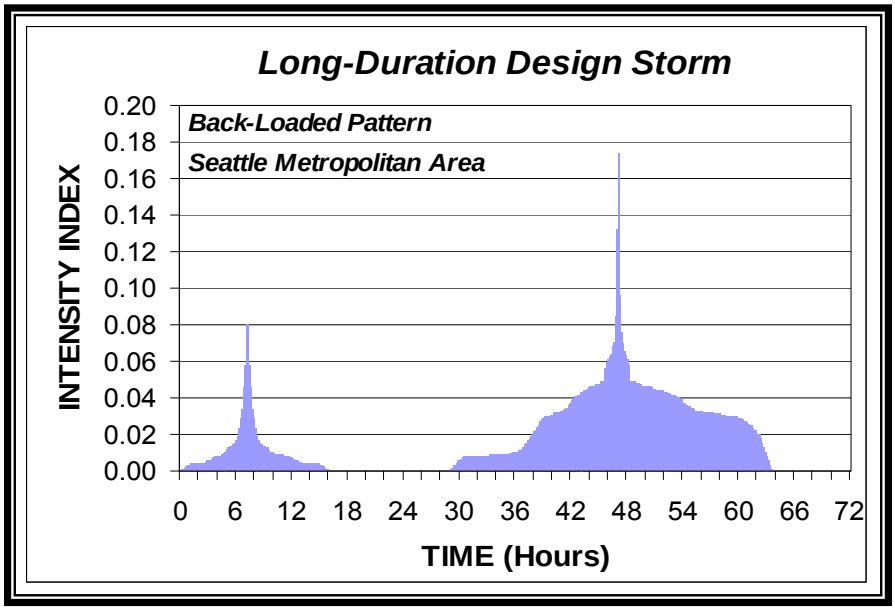

Figure 6.5. Dimensionless Back-Loaded Long-Duration (64-Hour) Design Storm for the Seattle Metropolitan Area.

The following steps describe how to utilize the long-duration storm in hydrologic analyses.

Step 1 – Obtain the 24-hour precipitation amount for the recurrence interval of interest for the watershed (refer to Appendix C and the DPD-SPU Stormwater webpage for modeling resources).

Step 2 – Multiply the 10 minute incremental ordinates of the dimensionless long-duration design storm (Appendix B, Table B-3 or B-4) by the 24-hour value from Step 1. Note that the resulting storm has a duration of 64 hours and total storm amount 1.29 times the volume of the 6-hour precipitation (refer to the DPD-SPU Stormwater webpage for modeling resources).

Step 3 – Input the resulting storm hyetograph into the hydrologic model. The resultant incremental precipitation ordinates have units of inches. To obtain the corresponding intensities (inches/hours), multiply the precipitation increments by 6.

6.4.2.4 24-Hour Dimensionless Design Storm

Some specific stormwater applications require or allow the use of a 24-hour design storm. To meet this need, the 24-hour dimensionless design storm was developed based on the maximum 24-hour period of precipitation within the long-duration design storm. In should be noted that the 24-hour dimensionless design storm has the same temporal shape and ordinates as the period of maximum 24-hour precipitation within the front-loaded and back-loaded long-duration dimensionless design storms. The City of Seattle 24-hour design storm is shown in Figure 6.6.

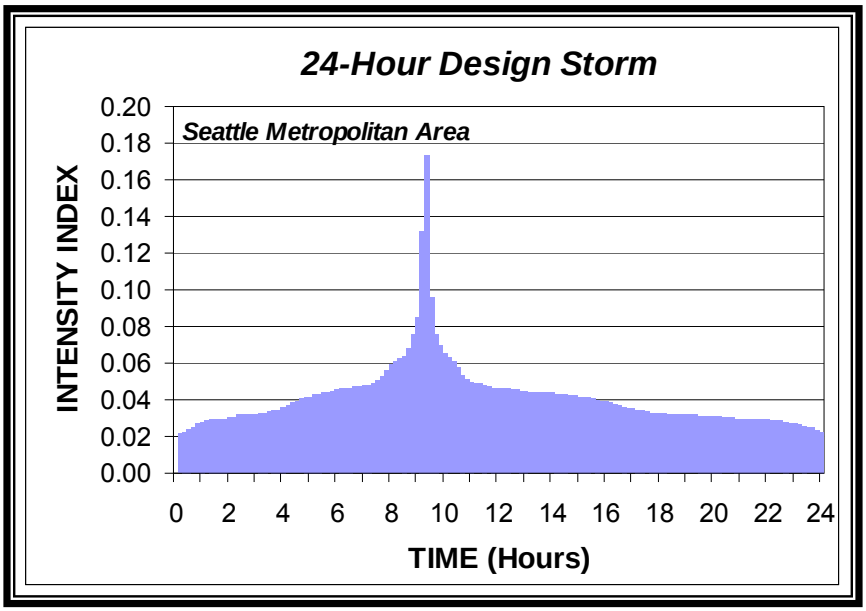

Figure 6.6. Dimensionless 24-Hour Design Storm for Seattle Metropolitan Area.

The following steps describe how to utilize the long-duration storm in hydrologic analyses.

Step 1 – Obtain the 24-hour precipitation amount for the recurrence interval of interest (refer to Appendix C and the DPD-SPU Stormwater webpage for modeling resources).

Step 2 – Multiply the 10 minute incremental ordinates of the dimensionless long-duration design storm (Appendix B, Table B-5) by the 24-hour value from Step 1 (refer to the DPD-SPU Stormwater webpage for modeling resources).

Step 3 – Input the resulting storm hyetograph into the hydrologic model. The resultant incremental precipitation ordinates have units of inches. To obtain the corresponding intensities (inches/hours), multiply the precipitation increments by 6.

6.4.2.5 City of Seattle Precipitation

National Oceanic and Atmospheric Administration (NOAA) Atlas 2 precipitation-frequency (isopluvial) maps published in the early 1970s have historically been used in hydrologic analysis and design. These maps are replaced in this manual by precipitation magnitude-frequency estimates more specific to the City of Seattle.

These estimates are based on a regional analysis using data from the SPU Rain Gage Network and gages from the NOAA national cooperative gaging network in western Washington. The most recent analysis included data from 1940 to 2003. Figure 6.7 shows the SPU Rain Gage Network as of 2003 and Appendix C gives the precipitation data based on this network. Changes to the SPU Rain Gage Network have been made since 2003 and updates to data analysis are ongoing. Updated information can be obtained from the SPU Rain Gage Network Data Steward or the DPD-SPU Stormwater webpage for modeling resources.

6.4.3 Use of Historic Storms in Analysis

This section includes a catalog of the storms used to derive the design storm patterns described in the previous section. These historic storms can be used in rainfall runoff models to aid in the design process by replicating past floods. For example, an engineer could use the historic storms to demonstrate that a proposed conveyance system design would have adequate capacity to pass a large historic flood that occurred in the watershed. The storms could also be used for calibrating the hydrologic model to recorded flow data. Use of these historic storms to confirm a facility design is recommended but is not required for the design of stormwater facilities.

Tables 6.6, 6.7, and 6.8 summarize historic storms recorded at City gauges for durations of 2 hours, 6 hours, and 24 hours respectively. Included in each table is the date when the storm ended, storm recurrence interval, and total precipitation for the duration of interest. The gage locations are shown in Figure 6.7. Electronic data for each storm is available in tabular form from Seattle Public Utilities (refer to the DPD-SPU Stormwater webpage for modeling resources).

Figure 6.7. City Rain Gage Network Stations.

Table 6.6. Catalog of Short-Duration (2-Hour) Storms at City Rain Gages.

Station ID	Station Name	Storm End-Date	Storm Recurrence Interval (years)	2-hour Precipitation (in)
45-S002	Mathews Beach Pump Stn	06/14/1978	16	0.86
45-S003	UW Hydraulics Lab	11/03/1978	10	0.79
45-S009	Woodland Park Zoo	08/17/1980	20	0.89
45-S008	Ballard Locks	08/28/1980	20	0.89
45-S002	Mathews Beach Pump Stn	05/29/1985	7	0.74
45-S014	West Seattle High School	10/26/1986	15	0.85
45-S020	TT Minor Elementary	10/04/1990	18	0.88
45-S009	Woodland Park Zoo	08/09/1991	6	0.72
45-S008	Ballard Locks	09/23/1992	45	1.02
45-S003	UW Hydraulics Lab	11/23/1997	9	0.77
45-S011	Metro-KC Denny Regulating	02/17/1998	14	0.84
45-S016	Metro-KC E Marginal Way	07/15/2001	6	0.71
45-S012	Catherine Blaine Jr	08/23/2001	14	0.84
45-S020	TT Minor Elementary	05/28/2002	4	0.83
45-S009	Woodland Park Zoo	09/03/2002	10	0.79
45-S004	Maple Leaf Reservoir	10/20/2003	18	0.88
45-S003	UW Hydraulics Lab	12/14/2006	13	0.83

Table 6.7. Catalog of Intermediate-Duration (6-Hour) Storms at City Rain Gages.

Station ID	Station Name	Storm End Date	Storm Recurrence Interval (years)	6-Hour Precip (inches)
45-S016	Metro-KC E Marginal Way	9/22/1978	32	1.61
45-S001	Haller Lake Shop	11/04/1978	70	1.74
45-S003	UW Hydraulics Lab	12/03/1982	92	1.82
45-S001	Haller Lake Shop	09/05/1984	5	1.21
45-S020	TT Minor Elementary	01/18/1986	>100	2.27
45-S010	Rainier Ave Elementary	01/09/1990	88	1.83
45-S003	UW Hydraulics Lab	12/29/1996	16	1.45
45-S004	Maple Leaf Reservoir	06/24/1999	7	1.28
45-S004	Maple Leaf Reservoir	10/20/2003	>100	1.96
45-S003	UW Hydraulics Lab	12/14/2006	36	1.62

Table 6.8. Catalog of Long-Duration (24-Hour) Storms at City Rain Gages.

Station ID	Station Name	Storm End Date	Storm Recurrence Interval (years)	24-Hour Precip (inches)
45-S008	Ballard Locks	12/17/1979	4	2.40
45-S009	Woodland Park Zoo	10/06/1981	24	3.07
45-S004	Maple Leaf Reservoir	11/01/1984	3	2.11
45-S001	Haller Lake Shop	01/18/1986	96	3.69
45-S016	Metro-KC E Marginal Way	11/23/1986	9	2.70
45-S003	UW Hydraulics Lab	11/24/1990	17	2.91
45-S002	Mathews Beach Pump Stn	04/04/1991	4	2.15
45-S020	TT Minor Elementary	02/08/1996	>100	5.07
45-S020	TT Minor Elementary	04/23/1996	8	2.56
45-S003	UW Hydraulics Lab	03/18/1997	7	2.53
45-S004	Maple Leaf Reservoir	11/25/1998	11	2.68
45-S010	Rainier Ave Elementary	11/14/2001	34	3.31
45-S004	Maple Leaf Reservoir	10/20/2003	>100	4.05

When using historic data from the City rain gage network for model calibration, storms should be selected from stations as close as possible to the center of the watershed tributary to the project site. This will help ensure that the recorded data is representative of precipitation that fell in the watershed for storm of interest. In general, the shorter duration storms typically have smaller areal coverage and greater spatial variability than the longer duration storms. Thus, greater simulation errors would be expected if gage data outside the watershed is used to simulate short duration storms.

6.4.4 Watershed Characterization

Prior to conducting any detailed stormwater runoff calculations, the overall relationship between the proposed project site and upstream and downstream off-site areas must be considered. The general hydrologic characteristics of the project site dictate the amount of runoff that will occur and where stormwater facilities can be placed. It is important to identify the stormwater destination point, including potential backwater effects. Drainage patterns and contributing areas can be determined from preliminary surveys of the area, available topographic contour maps, and Seattle Public Utilities drainage system maps. Note that the drainage systems often cross topographic divides within the City of Seattle. Maps can be obtained through the Seattle Public Utilities' GIS map counter (http://www.ci.seattle.wa.us/GIS/docs/mapctr.htm).

6.4.4.1 Calculation of Total Impervious Area

Impervious coverage for proposed development must be estimated. Impervious coverage of streets, sidewalks, hard surface trails, etc., shall be taken from plans of the site. See Chapter 2 and the Stormwater Code for definitions and descriptions of all surfaces that must be considered. Impervious coverage for off-site areas contributing flow to the site can be estimated from orthophotos available through GIS.

6.4.4.2 Calculation of Effective Impervious Area

Effective impervious surface is the fraction of impervious surface connected to a drainage system and is used in hydrologic simulations to estimate runoff. The effective impervious area is the total impervious area multiplied by the effective impervious fraction. Non-effective impervious surface is assumed to have the same hydrologic response as the immediately surrounding pervious area. Typically, the total impervious surface shall be assumed connected. For the existing condition modeling, off-site areas with unconnected rooftops may be estimated from visual survey as approved by the Directors.

6.4.5 Infiltration Equation

When computing runoff using the short or intermediate duration design storms, an infiltration soil loss method should be used. Examples of infiltration methods include the Green-Ampt (Rawls et al. 1993), Philip (Rawls et al. 1993), and Holtan (Holtan 1961) methods. The City recommends the use of Green Ampt method. These methods are incorporated into several commonly available computer programs including StormShed, PCSWMM, HEC HMS, and HEC-1.

6.4.5.1 Green Ampt Equation

The Green-Ampt model calculates cumulative infiltration by assuming water flow into a vertical soil profile like a piston flow.

$$f_t = K(\frac{\psi\Delta\theta}{F_t} + 1) \tag{6}$$

$$F_{t+\Delta t} = F_t + K\Delta t + \psi\Delta\theta \ln[\frac{F_{t+\Delta t} + \psi\Delta\theta}{F_t + \psi\Delta\theta}] \tag{7}$$

Where f_t is infiltration rate, mm/hr or inches/hr ; ψ is the initial matric potential of the soil, mm or inches; $\Delta\theta$ is the difference of soil water content after infiltration with initial water content, K is hydraulic conductivity, mm/hr or in/hr; F_t is the cumulative infiltration at time t, mm or inches; $F_{t+\Delta t}$ is the cumulative infiltration at time t+Δt , mm or inches; Δt is the time incremental in hours.

Equation (6) is used for determining ponding situation and (7) is used for calculating the cumulative infiltration after ponding. Trial and error method is the

most popular method to solve equation (7) (Chow et al. 1988). Parameters ψ, $\Delta\theta$, K were tabulated by Chow et al. (1988) for all soil classes. Green-Ampt model is simple to use. Chow et al. (1988) developed a procedure to solve infiltration with changing rainfall intensity by Green-Ampt method in a table. However, since it simplifies the water movement as a piston flow, the wetting front is distorted.

Typical values suggested by Rawls, Brakensiek, and Miller (as reflected in Chow 1988) are shown in Table 6.9 below.

Table 6.9. Green – Ampt Infiltration Parameters.

USDA Soil Classification	Suction Head ψ		Hydraulic Conductivity K		Porosity η	Effective Porosity θ_e
	(mm)	(in/hr)	(mm/hr)	(in/hr)		
Sand	49.5	1.95	117.8	4.64	0.437	0.417
Loamy Sand	61.3	2.42	29.9	1.18	0.437	0.401
Sandy Loam	110.1	4.34	10.9	0.43	0.453	0.412
Loam	88.9	3.50	3.4	0.13	0.463	0.434
Silt Loam	166.8	6.57	6.5	0.26	0.501	0.486
Sandy Clay Loam	218.5	8.61	1.5	0.06	0.398	0.330
Clay Loam	208.8	8.23	1.0	0.04	0.464	0.309
Silty Clay Loam	273.0	10.76	1.0	0.04	0.471	0.432
Sandy Clay	239.0	9.42	0.6	0.02	0.430	0.321
Silty Clay	292.2	11.51	0.5	0.02	0.479	0.423
Clay	316.3	12.46	0.3	0.01	0.475	0.385

6.4.5.2 Holtan's Equation

The empirical infiltration equation devised by Holtan (1961) is explicitly dependent on soil water conditions in the form of available pore space for moisture storage:

$$F = (GI)(AH)\ SMD^{IEXP} + FC \qquad (8)$$

Where:
- F = surface infiltration rate at a given time (inches/hour)
- GI = Growth Index representing the relative maturity of the ground cover (0 for newly planted, 1 for mature cover)
- AH = constant as specified below
- SMD = soil moisture deficit at a given time (inches)
- IEXP = infiltration exponent, default value is 1.4
- FC = minimum surface infiltration rate (inches/hour) and occurs when SMD equals zero.

Parameters GI, AH, FC, and the initial soil moisture deficit (SMD0) are the principal input parameters and can be determined as follows:

- GI is typically set to 1.0 to represent mature ground cover
- AH can be determined from Table 6.10.

Table 6.10. Estimates of Holtan AH.

Land Use or Cover	Base Area Rating [1]	
	Poor Condition	Good condition
Fallow [2]	0.10	0.30
Row crops	0.10	0.20
Small grains	0.20	0.30
Hay (legumes)	0.20	0.40
Hay (sod)	0.40	0.60
Pasture (bunchgrass)	0.20	0.40
Temporary pasture (sod)	0.40	0.60
Permanent pasture (sod)	0.80	1.00
Woods and forests	0.80	1.00

[1] Adjustments needed for "weeds" and "grazing.
[2] For fallow land only, "poor condition" means "after row crop," and "good condition" means "after sod."
Source: Holton et al. (1975)

FC can be approximated from Table 6.11 or by using the saturated hydraulic conductivity, which is available from soil survey reports.

Table 6.11. Estimates of Holtan FC Values.

SCS Hydrologic Soil Group	Minimum Infiltration Rates FC (inches/hour)
A	0.30-0.45
B	0.15-0.30
C	0.05-0.15
D	<0.05

Source: Musgrave (1955)

This equation has been found to be suitable for inclusion in catchment models because of soil water dependence, and satisfactory progress has been reported for runoff predictions (Dunin 1976).

6.4.5.3 Kostiakov's Equation

Kostiakov (1932) proposed the following equation for estimating infiltration

$$i(t) = \alpha t^{\beta} \tag{9}$$

where i is the infiltration rate at time t, and $\alpha(\alpha > 0)\, and\, \beta(0 < \beta < 1)$ are empirical constants.

Upon integration from 0 to t, equation 9 yields equation 10, which is the expression for cumulative infiltration, I(t):

$$I(t) = \frac{\alpha}{1-\beta} t^{(1-\beta)} \tag{10}$$

The constants α and β can be determined by curve-fitting equation 10 to experimental data for cumulative infiltration, I(t). Since infiltration rate, i, becomes zero as $t \to \infty$, rather than approach a constant non-zero value, Kostiakov proposed that equations 9 and 10 be used only for $t < t_{\max}$ where $t_{\max}$ is equal to $(\alpha / K_s)^{(1/\beta)}$, and K_s is the saturated hydraulic conductivity of the soil. Kostiakov's equation describes the infiltration quite well at smaller times, but becomes less accurate at larger times (Philip 1957a and 1957b; Parlange and Haverkamp, 1989).

6.4.5.4 Horton's Equation

Horton (1940) proposed to estimate infiltration in the following manner,

$$i(t) = i_f + (i_0 - i_f)e^{-\gamma t} \tag{11}$$

and

$$I(t) = i_f t + \frac{1}{\gamma}(i_0 - i_f)(1 - e^{-\gamma t}) \tag{12}$$

where $i_0\ and\ i_f$ are the presumed initial and final infiltration rates, and γ is an empirical constant. It is readily seen that i(t) is non-zero as t approaches infinity, unlike Kostiakov's equation. It does not, however, adequately represent the rapid decrease of i from very high values at small t (Philip 1957a and 1957b). It also requires an additional parameter over the Kostiakov equation. Parlange and Haverkamp (1989), in their comparison study of various empirical infiltration equations, found the performance of Horton's equation to be inferior to that of Kostiakov's equation.

6.4.5.5 Mezencev's Equation

In order to overcome the limitations of Kostiakov's equation for large times, Mezencev (Philip 1957a and 1957b) proposed the following as modifications to equations 9 and 10. Mezencev proposed infiltration estimated by

$$i(t) = i_f + \alpha t^{-\beta} \tag{13}$$

and

$$I(t) = i_f t + \frac{\alpha}{1-\beta} t^{(1-\beta)} \tag{14}$$

where i_f is the final infiltration rate at steady state.

6.4.5.6 SCS Equation

The USDA Soil Conservation Service developed an equation for rainfall-runoff relationship based on daily rainfall data as input:

$$R = \frac{(P - 0.2F_w)^2}{P + 0.8F_w} \tag{15}$$

where P is the daily rainfall, R is the runoff, and F_w is a statistically derived parameter bearing some resemblance to the initial soil moisture deficit. Infiltration is calculated as the excess of rainfall over runoff:

I = P – R

The SCS Curve Number loss method may be used when computing runoff using the Long Duration storms (24 hours or 66 hours in length). The Natural Resources Conservation Service (NRCS) developed relationships between land use, soil type, vegetation cover, interception, infiltration, surface storage, and runoff. These relationships have been characterized by a single runoff coefficient called a “curve number” (CN). The National Engineering Handbook – Part 630: Hydrology (NRCS 1997) contains a detailed description of the development and use of the curve number method.

The CN is related to the runoff potential of a watershed according to equations 16 and 17.

$$Q_d = \frac{(P - 0.2\,SMD_{MAX})^2}{(P + 0.8\,SMD_{MAX})} \tag{16}$$

$$SMD_{MAX} = \frac{1000}{CN} - 10 \tag{17}$$

Where: Qd =runoff depth in inches over the subbasin,
P =precipitation depth in inches over the subbasin,
SMD_{MAX} =maximum soil moisture deficit (inches),
CN =SCS Curve Number for the soil (Table 6.12).

Table 6.12. SCS Western Washington Runoff Curve Numbers.

Land Use Description		Curve Numbers by Hydrologic Soil Group			
Land Cover	**Condition**	**A**	**B**	**C**	**D**
Cultivated land	Winter condition	86	91	94	95
Mountain open areas	Low growing brush and grasslands	74	82	89	92
Meadow or pasture		65	78	85	89
Wood or forest land Wood or forest land Orchard	Undisturbed young second growth or brush with cover crop	42 55 81	64 81 88	76 72 92	81 86 94
Open spaces, lawns, parks, golf courses, cemeteries, landscaping	Good: grass cover on ≥75 percent of the area	68	80	86	90
	Fair: grass cover on 50-75 percent of the area	77	85	90	92
Gravel roads and parking lots Dirt roads and parking lots		76 72	85 82	89 87	91 89
Impervious surfaces, pavement, roofs etc., open water bodies: lakes, wetlands, ponds		98 100	98 100	98 100	98 100

The CN is a combination of a hydrologic soil group and land cover with higher CNs resulting in higher runoff. CN values for combinations of land cover and hydrologic soil group are listed in Table 6.12. Hydrologic soil groups for common soil types in the Seattle area are listed in Table 6.13.

6.4.6 Time of Concentration Estimation

The time of concentration for the various surfaces and conveyances should be computed using the following methods, which are based on NRCS publication 210-VI-TR-552, Chapter 3.

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

Table 6.13. SCS Hydrologic Soil Group Definition for Common Soils in King County.

SOIL GROUP	HYDROLOGIC GROUP *	SOIL GROUP	HYDROLOGIC GROUP *
Alderwood	C	Orcas Peat	D
Arents, Alderwood Material	C	Oridia	D
Arents, Everett Material	B	Ovalt	C
Beausite	C	Pilchuck	C
Bellingham	D	Puget	D
Briscot	D	Puyallup	B
Buckley	D	Ragnar	B
Coastal Beaches	Variable	Renton	D
Earlmont Silt Loam	D	Riverwash	Variable
Edgewick	C	Salal	C
Everett	A	Sammamish	D
Indianola	A	Seattle	D
Kitsap	C	Shacar	D
Klaus	C	Si Silt	C
Mixed Alluvial Lan	Variable	Snohomish	D
Nellton	A	Sultan	C
Newberg	B	Tukwila	D
Nooksack	C	Urban	Variable
Normal Sandy Loam	D	Woodinville	D

HYDROLOGIC SOIL GROUP CLASSIFICATIONS

A. (Low runoff potential). Soils having high infiltration rates, even when thoroughly wetted, and consisting chiefly of deep, well-to-excessively drained sands or gravels. These soils have a high rate of water transmission

B. (Moderately low runoff potential). Soils having moderate infiltration rates when thoroughly wetted, and consisting chiefly of moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

C. (Moderately high runoff potential). Soils having slow infiltration rates when thoroughly wetted, and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine textures. These soils have a slow rate of water transmission.

D. (High runoff potential). Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a hardpan or clay later at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

* From SCS, TR-55, Second Edition, June 1986, Exhibit A-1. Revisions made from SCS, Soil Interpretation Record, Form #5, September 1988.

Water is assumed to move through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is best determined by field inspection. The time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments.

$$T_c = T_1+T_2+T_3+\ldots T_n \qquad (18)$$

Where: T_c = the time of concentration in minutes
$T_{1,2,3,n}$ = the time for consecutive flow path segments with different land cover categories or flow path slope

Travel time for each segment is computed using the following equation:

$$T_t = \frac{L}{60V} \qquad (19)$$

Where: T_t = Travel time (minutes)
L = Length of flow across a given segment (feet)
V = average velocity (ft/sec) across the land segment

Sheet Flow: Sheet flow is flow over plane surfaces. Sheet flow travel time is computed using equation 20. This equation is applicable for relatively impervious areas with shallow flow depths up to about 0.1 foot and for travel lengths up to 300 feet. Modified Manning's effective roughness coefficients (n_s) are summarized in Table 6.14. These ns values are applicable for shallow flow depths up to about 0.1 foot and for travel lengths up to 300 feet.

$$T_t = 0.42 * (ns * L)0.8 / ((P_{24})0.5 * (So)0.4) \qquad (20)$$

where: T_t = the travel time, in minutes
n_s = the sheet flow Manning's effective roughness coefficient from Table 5.12
L = the overland flow length in feet
P_{24} = the 2-year, 24-hour rainfall in inches
So = the slope of hydraulic grade line or land slope, in feet per feet.

Shallow Concentrated Flow: After a maximum of 300 feet, sheet flow is assumed to become shallow concentrated flow. The average velocity for this flow can be calculated using the ks values from Table 6.14 in which average velocity is a function of watercourse slope and type of channel. After computing the average velocity using the Velocity Equation 21 below, the travel time (T_t) for the shallow concentrated flow segment can be computed using the travel time equation 19 described above.

Velocity Equation: A commonly used method of computing average velocity of flow, once it has measurable depth, is the following equation:

$$V = k_s\sqrt{S_o} \qquad (21)$$

Where: k_s = Velocity factor (Table 6.14)
S_0 = Slope of flow path (feet/feet)

Table 6.14. Values of "n" and "k" for use in computing Time of Concentration.

FOR SHEET FLOW	n_8
Smoot surfaces (concrete, asphalt, gravel, or bare hard soil)	0.011
Fallow fields of loose soil surface (no vegetal residue)	0.05
Cultivated soil with crop residue (slope < 0.20 ft/ft)	0.06
Cultivated soil with crop residue (slope > 0.20 ft/ft)	0.17
Short prairie grass and lawns	0.15
Dense grass	0.24
Bermuda grass	0.41
Range, natural	0.13
Woods or forest, poor cover	0.40
Woods or forest, good cover	0.80
FOR SHALLOW, CONCENTRATED FLOW	k_8
Forest with heavy ground litter and meadows (n=0.10)	3
Brushy ground with some trees (n=0.06)	5
Fallow or minimum tillage cultivation (n=0.04)	8
High grass (n=0.035)	9
Short grass, pasture and lawns (n=0.04)	11
Newly-bare ground (n=0.025)	13
Paved and gravel areas (n=0.012)	27
CHANNEL FLOW (INTERMITTENT, R = 0.2)	k_c
Forested swale with heavy ground litter (n=0.10)	5
Forested drainage course/ravine with defined channel bed (n=0.050)	10
Rock-lined waterway (n=0.035)	15
Grassed waterway (n=0.030)	17
Earth-lined waterway (n=0.025)	20
CMP pipe (n=0.024)	21
Concrete pipe (n=0.012)	42
Other waterways and pipes	0.508/n
CHANNEL FLOW (CONTINUOUS STREAM, R = 0.4)	k_c
Meandering stream with some pools (n=0.040)	20
Rock-lined stream (n=0.035)	23
Grassed stream (n=0.030)	27
Other streams, man-made channels and pipe	0.807/n

"k" values in Table 6.14 have been computed for various land covers and channel characteristics with assumptions made for hydraulic radius using the following rearrangement of Manning's equation:

$$k = (1.49\ (R)\ 0.667)/n \qquad (22)$$

Where: R = An assumed hydraulic radius
n = Manning's roughness coefficient for open channel flow, from Tables 6.14 or 6.15.

Open Channel Flow: Open channels are assumed to begin where flow enters ditches or pipes, where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where lines indicating streams appear (in blue) on United States Geological Survey (USGS) quadrangle sheets. The k_c values from Table 6.14 used in the Velocity Equation 21 above or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full conditions. The travel time (T_t) for the channel segment can be computed using the Travel Time Equation 19 above.

Lakes or Wetlands: Sometimes it is necessary to estimate the velocity of flow through a lake or wetland at the outlet of a watershed. This travel time is normally very small and can be assumed as zero. Where significant attenuation may occur due to storage effects, the flows should be routed using the "level-pool routing" technique described in Section 6.4.7.3.

Limitations: The following limitations apply in estimating travel time (T_t):

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and other surfaces, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or non-pressure flow.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. A hydrograph should be developed to this point and the "level pool routing" technique should be used to determine the outflow rating curve through the culvert or bridge.

6.4.7 Single-Event Routing Methods Overview

In the United States, the majority of single-event models for computation of runoff hydrographs are based on unit-hydrographs. Most commercial software packages utilize unit-hydrographs for making the transformation from computation of runoff volume to generation of the runoff hydrograph. This may require direct input of the ordinates of the unit-hydrograph or the unit-hydrograph

Table 6.15. Other values of the roughness coefficient "n" for channel flow.

Type of Channel and Description	Manning's "n"*
A. Constructed Channels	
a. Earth, straight and uniform	
1. Clean, recently completed	0.018
2. Gravel, uniform selection, clean	0.025
3. With short grass, few weeds	0.027
b. Earth, winding and sluggish	
1. No vegetation	0.025
2. Grass, some weeds	0.030
3. Dense weeds or aquatic plants in deep channels	0.035
4. Earth bottom and rubble sides	0.030
5. Stony bottom and weedy banks	0.035
6. Cobble bottom and clean sides	0.040
c. Rock lined	
1. Smooth and uniform	0.035
2. Jagged and irregular	0.040
d. Channels not maintained, weeds and brush uncut	
1. Dense weeds, high as flow depth	0.080
2. Clean bottom, brush on sides	0.050
3. Same, highest stage of flow	0.070
4. Dense brush, high stage	0.100
B. Natural Streams	
B-1 Minor streams (top width at flood stage < 100 ft.)	
a. Streams on plain	
1. Clean, straight, full stage no rifts or deep pools	0.030
2. Same as above, but more stones and weeds	0.035
3. clean, winding, some pools and shoals	0.040
4. Same as above, but some weeds	0.040
5. Same as 4, but more stones	0.050
6. Sluggish reaches, weedy deep pools	0.070
7. Very weedy reaches, deep pools, or floodways with heavy stands of timber and underbrush	0.100
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages	
1. Bottom: gravel, cobbles, and few boulders	0.040
2. Bottom: cobbles with large boulders	0.050
B-2 Flood plains	
a. Pasture, no brush	
1. Short grass	0.030
2. High grass	0.035
b. Cultivated areas	
1. No crop	0.030
2. Mature row crops	0.035
3. Mature field crops	0.040
c. Brush	
1. Scattered brush, heavy weeds	0.050
2. Light brush and trees	0.060
3. Medium to dense brush	0.070
4. Heavy, dense brush	0.100
d. Trees	
1. Dense willows, straight	0.150
2. Cleared land with tree stumps, no sprouts	0.040
3. Same as above, but with heavy growth of sprouts	0.060
4. Heavy stand of timber, a Few down trees, little undergrowth, flood stage below branches	0.100
5. Same as above, but with flood stage reaching branches	0.120

may be computed internally based on watershed characteristics provided by the user. Notable exceptions include event-based models that utilize linear reservoir concepts, such as the Santa Barbara Urban Hydrograph model (SBUH), event-based models that utilize kinematic wave approaches, and continuous flow simulation models such as HSPF.

Section 5.4.7.1 describes rainfall-runoff modeling based on unit-hydrograph concepts. The reader is referred to any standard hydrology textbook (e.g., Gray 1961; Linsley et al. 1975; Pilgrim and Cordery 1993; Viessman et al. 1977) for a detailed discussion of unit-hydrograph theory. Section 6.4.7.2 includes a discussion of runoff hydrographs developed using the SBUH model. Section 5.4.7.3 provides a discussion on the level-pool method, which is appropriate for routing hydrographs through lakes, wetlands, and other areas of standing water.

6.4.7.1 Unit-Hydrograph Routing Methods

The unit-hydrograph is defined as the time-distribution of runoff (Figure 6.8) measured at the watershed outlet as produced by 1 inch of runoff uniformly generated over the watershed during a specified period of time. Thus, a 10-minute unit-hydrograph would be the runoff hydrograph (cfs) observed at the watershed outlet as generated by 1 inch of runoff uniformly produced over the watershed in a 10-minute period.

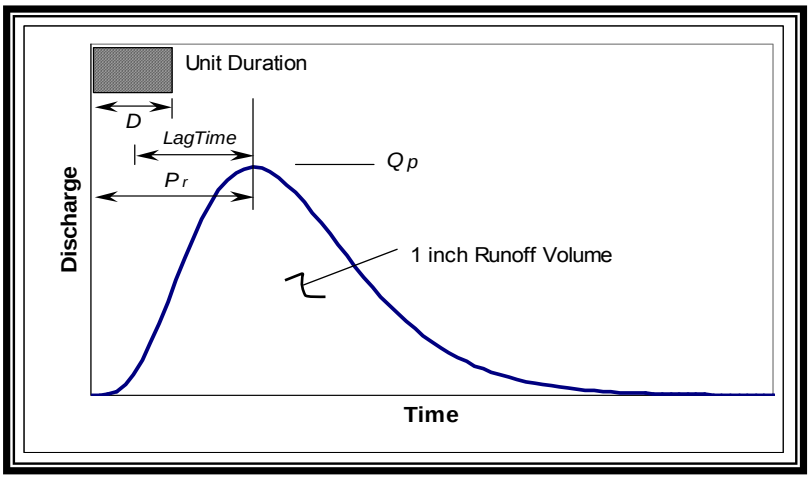

Figure 6.8. Characteristics of Unit Hydrographs.

In computation of the runoff hydrograph, the unit-hydrograph is scaled by the runoff in each D-minute period, and the resultant hydrographs for each D-minute period are added by superposition to yield the runoff hydrograph from the watershed.

<u>Relationship of Computational Time step to Time Lag (LagTime)</u>. As indicated above, the ordinates of the unit-hydrograph are specified on intervals equal to the computational time step. Recognizing that the time step and unit duration are equal (Δt=D), the unit duration must be chosen small enough to allow reasonable

definition of the rising limb of the unit-hydrograph. This is required to provide for adequate definition of the resultant runoff hydrograph in the vicinity of the runoff peak discharge. In addition, the value of D should be an integer multiple of the period of rise P_r so that the computational time step (Δt) falls on the peak discharge of the unit hydrograph.

Selection of Time Step (Δt) Based on Time of Concentration (T_c). The time-of concentration of the watershed (T_c) is often taken to be the elapsed time from the end of the unit duration (D) to the inflection point on the recession limb of the unit-hydrograph (NRCS 1997). When the runoff hydrograph is computed based on unit-hydrograph concepts utilizing time of concentration, the computational time step should be:

$$\Delta t < T_c/5 \tag{23}$$

To enhance compatibility with the City of Seattle design storms, the computational time step for runoff computations should be a multiple of the time step used to describe the design storm. The short-duration design storm is described in 5-minute intervals and the intermediate and long-duration design storms are described in 10-minute intervals. Therefore, the following additional criteria are required for selection of the time step for use with the short-duration design storm:

$$\Delta t = 5/n \tag{24}$$

and for use with the intermediate and long-duration design storms:

$$\Delta t = 10/n \tag{25}$$

where: n is an integer greater than or equal to one.

The above information should be particularly helpful for use with computer software that allows output of the runoff hydrograph on a time interval other than that used for internal computation of the runoff hydrograph. For those cases, the user may be unaware of the unit duration (D) and internal time step (Δt) being used by the computer program.

6.4.7.2 Santa Barbara Urban Hydrograph Routing Method

The Santa Barbara Urban Hydrograph (SBUH) method is an adaptation of standard hydrologic routing methods that employ the principle of conservation of mass. The routing equation for the SBUH method may be derived from linear reservoir concepts (Linsley et al. 1975; Fread 1993) where storage is taken to be a linear function of discharge.

The SBUH method uses two steps to synthesize the runoff hydrograph:

Step 1 – Compute the instantaneous hydrograph, and

Step 2 – Compute the runoff hydrograph.

The instantaneous hydrograph is computed as follows:

$$I(t) = 60.5\ R(t)\ A/\Delta t \tag{26}$$

Where: $I(t)$ = is the instantaneous hydrograph at each time step (Δt) in cfs,
$R(t)$ = is the total runoff depth (both impervious and pervious) at time increment Δt in inches
A = the area in acres
Δt = the computational time step in minutes.

The runoff hydrograph is then obtained by routing the instantaneous hydrograph through an imaginary reservoir with a time delay equal to the time of concentration of the drainage basin. The following equation estimates the routed flow:

$$Q(t+1) = Q(t) + w[I(t) + I(t+1) - 2Q(t)] \tag{27}$$

$$w = \Delta t /(2T_c + \Delta t) \tag{28}$$

Where: $Q(t)$ = the runoff hydrograph or routed flow in cfs
T_c = the time of concentration in minutes
Δt = the computational time step in minutes.

<u>Selection of Time Step (Δt) Based on Time of Concentration (T_c)</u>. Equation 27 requires that the computational time step be sufficiently short that the change in inflow, outflow, and storage during the time step can be treated as linear. For the case of very small urban watersheds, the low to moderate intensities in the long-duration design storm would typically generate runoff over a longer period than the time of concentration of the watershed. As a result, the elapsed time of the rising limb of the runoff hydrograph (T_r) would likewise be much longer than the time of concentration of the watershed. In addition, the computational time step for routing should be a multiple of the time step used to describe the design storm. Therefore, for intermediate and long duration storms, the computational time step should satisfy equations 29 and 30:

$$\Delta t < T_c \tag{29}$$

$$\Delta t = 10/n \tag{30}$$

Where: Δt = the computational time step in minutes
T_c = the time of concentration in minutes
n = an integer greater than or equal to one.

For short-duration design storms, the flood peak of the runoff hydrograph may be quite flashy and produced by high-intensity precipitation during a limited portion of the storm. For this case, the elapsed time for the rising limb of the runoff hydrograph may be similar in magnitude to that of the time-of-concentration of the watershed. In this situation, the time step should be smaller than the time of concentration. In addition, the computational time step for routing should be a

multiple of the time step used to describe the design storm. Therefore, for the short duration storm, the computational time step should satisfy equations 31 and 32:

$$\Delta t < T_c/5 \quad (31)$$

$$\Delta t = 5/n \quad (32)$$

Where: Δt = the computational time step in minutes
T_c = the time of concentration in minutes
n = an integer greater than or equal to one.

6.4.7.3 Level-Pool Routing Method

This section presents a general description of the methodology for routing a hydrograph through a retention/detention facility, closed depression, or wetland. Note that the City of Seattle does not allow the use of single-event models for retention/detention facility design. The information presented in this section is for informational purposes only. The level pool routing technique (Fread 1993) is based on the continuity equation:

Inflow-outflow=change in storage

$$\left[\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2}\right] = \frac{\Delta S}{\Delta t} = S_2 - S_1 \quad (33)$$

rearranging:

$$I_1 + I_2 + 2S_1 - O_1 = O_2 + 2S_2 \quad (34)$$

Where: I = the inflow at time 1 and time 2
O = the outflow at time 1 and time 2
S = the storage at time 1 and time 2
Δt = the computational time step in minutes

The time step (Δt) must be consistent with the time interval used in developing the inflow hydrograph.

The following summarizes the steps required in performing level-pool hydrograph routing:

- Develop stage-storage-discharge relationship, which is a function of pond/wetland geometry and outflow
- Route the inflow hydrograph through the structure by applying the equation 34 at each time step, where the inflow hydrograph supplies values of I, the stage-storage relationship supplies values of S, and the stage discharge relationship provides values of O.

Commercially available hydrologic computer models perform these calculations automatically.

6.4.8 Modeling Guidance

The following sections present the general process involved in conducting a hydrologic analysis using single-event hydrograph methods to evaluate or design stormwater conveyance systems. Information is also included for addressing outfalls that are affected by downstream backwater conditions. Applicability of single-event methods and design standard requirements are discussed in Section 6.2 of this chapter.

6.4.8.1 Steps for Hydrologic Design Using Single-Event Methods

The following summarizes the process for conducting hydrologic analyses using single-event models. Information specific to the type of facility under consideration is included in later sections.

- Review all minimum requirements that apply to the proposed project (Chapter 2)
- Review applicable site definition and mapping requirements (Chapters 2 and 3)
- Identify and delineate the overall drainage basin for each discharge point from the development site under existing conditions.
    - Identify existing land use
    - Identify existing soil types using on-site evaluation, NRCS soil survey, or mapping performed by the University of Washington (http://geomapnw.ess.washington.edu/)
    - Identify existing drainage features such as streams, conveyance systems, detention facilities, ponding areas, depressions, etc.
- Select and delineate pertinent subbasins based on existing conditions
    - Select homogeneous subbasin areas
    - Select separate subbasin areas for on-site and off-site drainage
    - Select separate subbasin areas for major drainage features.

Additional design steps specific to conveyance facility design are described below.

6.4.8.2 Stormwater Conveyance

Existing and proposed stormwater conveyance facilities may be analyzed and designed using peak flows from hydrographs derived from single event approaches described in this chapter. In addition to the steps listed in Section 6.4.8.1, the following steps should be followed for designing/analyzing conveyance facilities:

- Determine runoff parameters for each subbasin

- Identify pervious and impervious areas per Section 6.4.4
  - The short or intermediate duration design storm generally governs the design of conveyance facilities. Both storm durations should be treated as candidate design storms and the one that produces the more conservative design (higher peak discharge rates) used as the design storm (see Section 6.4.2).
  - Select runoff parameters per Section 6.4.5
  - Compute time of concentration per Section 6.4.6.
- Identify downstream hydraulic controls, such as outfalls, known flooding locations, receiving pipe HGL, pump station, regulator station, weirs or orifice. Determine if backwater calculations or a dynamic hydraulic routing model are required.
- Compute runoff for the drainage system and determine peak discharge at the outlet of each subbasin for the design storm of interest
- For capacity analysis, verify that there is sufficient capacity in the public drainage system. For conveyance design, size the pipe based on the designated level of service. It is recommended that a factor of safety be applied to account for capacity losses due to changing downstream hydraulics and pipe or channel deformation or clogging.
- Information on outfall design is discussed in Section 6.6.

6.5 Continuous Simulation Methods

6.5.1 Introduction

Continuous rainfall-runoff models use multi-decade precipitation and evaporation time series as input to produce a corresponding multi-decade time series of runoff. Continuous models are well suited to accounting for the climatological conditions in the lowland Puget Sound area. Continuous models include algorithms that maintain a continuous water balance to account for soil moisture and hydraulic conditions antecedent to each storm.

An approved continuous simulation hydrologic model is required for the design of stormwater flow control and water quality treatment facilities. Continuous models may also be used to size conveyance facilities. When used to design stormwater BMPs, the City of Seattle Design Time Series must be used (Section 6.5.2.1) at the computational time step specified in Table 6.21. Approved continuous flow models are listed in Section 6.2.3 of this chapter. Other peer-reviewed continuous models may be utilized if approved by the Directors.

6.5.2 Precipitation Input

Continuous hydrologic models use multi-year inputs of precipitation and evaporation to compute a multi-year time series of runoff from the site. Using

precipitation input that is representative of the site under consideration is critical for the accurate computation of runoff and the design of stormwater facilities.

Two types of precipitation and evaporation data are available for stormwater analysis. The first type is a design precipitation and evaporation time series. The design time series are appropriate for design and analysis of stormwater facilities and were developed by combining and scaling records from distant precipitation stations. The second type of time series is historic precipitation and evaporation time series. The historic time series consist of data collected from the City gage network. Because the record length of the historic precipitation and evaporation is relatively short, this data should be used for model calibration and not for design. Both time series types and their applicability are discussed in the following sections.

6.5.2.1 Design Precipitation Time Series

The City of Seattle Design Time Series consists of a precipitation and evaporation time series that are representative of the climatic conditions in the City of Seattle. The design precipitation time series was developed by combining and scaling precipitation records from widely separated stations to produce an "extended precipitation time series" with a 158-year record length. The precipitation scaling was performed such that the scaled precipitation record would possess the regional statistics at durations of 5-minutes, 10-minutes, 15-min, 20-min, 30-min, 45-min, 60-min, 2-hour, 6-hours, 24-hours, 3-days, 10-days, 30-days, 90-days, 6-months, and annual (refer to the DPD website for modeling resources).

The evaporation time series were developed using a stochastic evaporation generating approach whereby daily evaporation was generated in a manner to preserve the daily and seasonal variability and accounting for differences observed on rainy versus rain-free days. The evaporation time series were developed from data collected at the Puyallup 2 West Experimental Station (station number 45-6803). Refer to the DPD website for modeling resources).

The City of Seattle Design Time Series should be used for flow control and water quality treatment facility analysis and design. The precipitation time series was developed at a time step of 5-minutes and the evaporation time series has a time step of 1-hour. Selection of an appropriate computational time step for simulations is described in Section 6.5.4.1.

6.5.2.2 Historic Precipitation Time Series

Data collected from the SPU rain gage network may be used in rainfall runoff models to aid in the design process by replicating past floods. For example, an engineer could use data from a gage near the project site to demonstrate that a conveyance system design under consideration would have adequate capacity to pass a large historic flood that occurred in the watershed. The time series could

also be used for model calibration or to investigate anecdotal flood information. Use of the historic time series is recommended, but is not required for the design of stormwater facilities.

Continuous historic precipitation data are available from 17 gages from January 1978 through the present at a time step of 5 minutes. Gage name and locations are summarized in Table 6.16 and Figure 6.7 respectively (see Section 6.4.3).

Table 6.16. City of Seattle Continuous Precipitation Rain Gage Stations.

Station ID	Station Name
45-S001	Haller Lake Shop
45-S002	Mathews Beach Pump Station
45-S003	UW Hydraulics Lab
45-S004	Maple Leaf Reservoir
45-S005	Fauntleroy Ferry Dock
45-S007	Whitman Middle School
45-S008	Ballard Locks
45-S009	Woodland Park Zoo
45-S010	Rainier View Elementary
45-S011	Metro-KC Denny Regulating
45-S012	Catherine Blaine Jr.
45-S014	West Seattle High School
45-S015	Metro-KC Diagonal Pump
45-S016	Metro-KC E Marginal Way
45-S017	West Seattle Engineering Shop
45-S018	Hillman Engineering Shop
45-S020	TT Minor Elementary

6.5.3 Watershed Characterization

Prior to conducting any detailed stormwater runoff calculations, the overall relationship between the proposed project site and upstream and downstream off-site areas must be considered. The general hydrologic characteristics of the project site dictate the amount of runoff that will occur and where stormwater facilities can be placed. It is important to identify the stormwater destination point, including potential backwater effects. Drainage patterns and contributing areas can be determined from preliminary surveys of the area, available topographic contour maps, and Seattle Public Utilities drainage system maps. Note that the drainage systems often cross topographic divides within the City of Seattle. Maps can be obtained through the City GIS map counter (http://www.ci.seattle.wa.us/GIS/docs/mapctr.htm).

6.5.3.1 Land Cover Categorization

Currently approved continuous flow models based on HSPF include five land cover types: forest, pasture, grass, wetland, and impervious. These cover types shall be applied as specified in Table 6.17.

Table 6.17. Continuous Hydrologic Cover Groups and Areas of Application.

Continuous Model Land Cover	Application	
	Predeveloped	**Post Developed**
Forest	All forest/shrub cover, irrespective of age	All permanent (e.g., protected by covenant) onsite forest/shrub cover, irrespective of age planted at densities sufficient to ensure 80%± canopy cover within 5 years
Pasture	All grassland, pasture land, lawns, and cultivated or cleared area except for lawns in redevelopment areas with predevelopment densities greater than 4 DU/GA	Unprotected forest in rural residential development shall be considered half pasture, half grass
Grass / Landscape	Lawns in redevelopment areas with predevelopment densities greater than 4 DU/GA	All post-development grassland and landscaping and all onsite forested land not protected by covenant. This includes all disturbed areas required to meet the post construction soil quality and depth requirements (see also Chapter 2 and Chapter 4, Section 4.4.1).
Wetland	All delineated wetland areas	All delineated wetland areas
Impervious	All impervious surfaces, including heavily compacted gravel and dirt roads, parking areas, etc., and open water bodies (ponds and lakes)	All impervious surfaces, including heavily compacted gravel and dirt roads, parking areas, etc., and open water bodies including onsite detention and water quality ponds

6.5.3.2 Calculation of Total Impervious Area

See Section 6.4.4.1 of this chapter.

6.5.3.3 Calculation of Effective Impervious Area

See Section 6.4.4.2 of this chapter.

6.5.3.4 Soil Type Categorization

For a general discussion of infiltration equations see Section 6.4.5. The following discussion on HSPF also applies to use of MGSFlood and WWHM.

HSPF Runoff Parameters

Default model parameters that define interception, infiltration, and movement of moisture through the soil, are based on work by the USGS (Dinicola 1990, Dinicola 2001) and King County (1998). Pervious areas have been grouped into three land cover categories; forest, pasture, and lawn, and three soil/geologic

categories; till, outwash, and saturated/wetland soil for a total of seven cover/soil type combinations as shown in Table 6.18. The combinations of soil type and land cover are called pervious land segments or PERLNDS. Default runoff parameters for each PERLND are summarized in Table 6.19. These parameter values are used automatically by WWHM and MGSFlood programs for each land use type. Additional model parameters and modeling approaches for green stormwater infrastructure are included in Chapter 4.

Table 6.18. Pervious Land Soil Type/Cover Combinations used with HSPF Model Parameters.

Pervious Land Soil Type/Cover Combinations	
1.	Till/Forest
2.	Till/Pasture
3.	Till/Lawn
4.	Outwash/Forest
5.	Outwash/Pasture
6.	Outwash/Lawn
7.	Saturated Soil/All Cover Groups

Table 6.19. Default Runoff Parameters for Each Pervious Land Segment (PERLND).

	Pervious Land Segment (PERLND)						
	Till Soil			Outwash Soil			Saturated Soil
Parameter	Forest	Pasture	Lawn	Forest	Pasture	Lawn	Forest/Pasture/or Lawn
LZSN	4.5	4.5	4.5	5.0	5.0	5.0	4.0
INFILT	0.08	0.06	0.03	2.0	1.6	0.8	2.0
LSUR	400	400	400	400	400	400	100
SLSUR	0.1	0.1	0.1	0.05	0.05	0.05	0.001
KVARY	0.5	0.5	0.5	0.3	0.3	0.3	0.5
AGWRC	0.996	0.996	0.996	0.996	0.996	0.996	0.996
INFEXP	2.0	2.0	2.0	2.0	2.0	2.0	10.0
INFILD	2.0	2.0	2.0	2.0	2.0	2.0	2.0
BASETP	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AGWETP	0.0	0.0	0.0	0.0	0.0	0.0	0.7
CEPSC	0.2	0.15	0.1	0.2	0.15	0.1	0.1
UZSN	0.5	0.4	0.25	0.5	0.5	0.5	3.0
NSUR	0.35	0.3	0.25	0.35	0.3	0.25	0.5
INTFW	6.0	6.0	6.0	0.0	0.0	0.0	1.0
IRC	0.5	0.5	0.5	0.7	0.7	0.7	0.7
LZETP	0.7	0.4	0.25	0.7	0.4	0.25	0.8

PERLND parameter definitions:

- LZSN = lower zone storage nominal (inches)
- INFILT = infiltration capacity (inches/hour)
- LSUR = length of surface overland flow plane (feet)
- SLSUR = slope of surface overland flow plane (feet/feet)
- KVARY = groundwater exponent variable (inch $^{-1}$)
- AGWRC = active groundwater recession constant (day $^{-1}$)
- INFEXP = infiltration exponent
- INFILD = ratio of maximum to mean infiltration
- BASETP = base flow evapotranspiration (fraction)
- AGWETP = active groundwater evapotranspiration (fraction)
- CEPSC = Interception storage (inches)
- UZSN = upper zone storage nominal (inches)
- NSUR = roughness of surface overland flow plane (Manning 's n)
- INTFW = interflow index
- IRC = interflow recession constant (day $^{-1}$)
- LZETP = lower zone evapotranspiration (fraction).

A complete description of the PERLND parameters can be found in the HSPF User Manual (US EPA 2001).

Default PERLND Parameters used in the model were developed for the Puget Sound Lowlands by the US Geological Survey (Dinicola 1990, Dinicola 2001).

Soils formed in areas with glacial till are underlain at shallow depths by relatively impermeable glacial till. Glacial till deposits contain large percentages of silt or clay and have low percolation rates. Only a small fraction of infiltrated precipitation reaches the groundwater table through the till. The rest moves laterally through the thin surface soil above the till deposit as interflow. Shallow soils over bedrock should also be classified as till soils because the hydrologic response from these areas is similar to till.

Soils formed in areas with glacial outwash deposits consist of sand and gravels that have high infiltration rates. The majority of rainfall is infiltrated and percolates to the groundwater table in these areas. Creeks draining outwash deposits often intersect the groundwater table and receive most of their flow from groundwater discharge. Site developments in outwash areas are typically located higher in the watershed and groundwater discharge is not present. Thus,

groundwater is typically not included in runoff calculations in outwash (or till) areas. Developments located in areas with high percentages of outwash are generally conducive to infiltration methods for mitigating stormwater runoff.

Wetland soils remain saturated throughout much of the year. The hydrologic response from wetlands is variable depending on the underlying geology, the proximity of the wetland to the regional groundwater table, and the bathymetry of the wetland. Generally, wetlands provide some base flow to streams in the summer months and attenuate storm flows via temporary storage and slow release in the winter.

Mapping of soil types by the Soil Conservation Service (SCS, now the National Resource Conservation Service (NRCS)), or alternatively mapping performed by the University of Washington (http://geomapnw.ess.washington.edu/) may be used as a source of soil/geologic information for use in hydrologic modeling for drainage control facility design. The interactive online geologic maps for the Seattle area developed by the University of Washington generally provide a higher degree of resolution and better characterization of underlying soil geology. If using SCS maps, each soil type defined by the SCS has been classified into one of four hydrologic soil groups; A, B, C, and D. Table 6.13 shows SCS hydrologic soil groups for common soil types in King County. As is common practice in hydrologic modeling in western Washington, the soil groups used in the model generally correspond to the SCS hydrologic soil groups as shown in Table 6.20.

Table 6.20. Relationship Between SCS Hydrologic Soil Group and Continuous Model Soil Group.

SCS	Model Soil Group
A	Outwash
B	Till or Outwash
C	Till
D	Wetland

SCS Type B soils can be classified as either glacial till or outwash depending on the type of soil under consideration. Type B soils underlain by glacial till or bedrock, or have a seasonally high water table would be classified at till. Conversely, well-drained B type soils would be classified as outwash.

Note that neither the University of Washington nor SCS maps may be used for determining infiltration capacity or design infiltration rate.

6.5.4 Modeling Guidance

6.5.4.1 Selection of Computational Time Step

An appropriate computational time step for continuous hydrologic models depends on the type of facility under consideration and the characteristics of the

tributary watershed. In general, the design of facilities dependent on peak discharge require a shorter time step than facilities dependent on runoff volume. A longer time step is generally desirable to reduce the overall simulation time provided that computational accuracy is not sacrificed. Table 6.21 summarizes the allowable computational time steps for various hydrologic design applications.

Table 6.21. Required Continuous Simulation Model Computational Time Step for Various Stormwater Facilities.

Type of Analysis	Maximum Time Step
Peak Flow Conveyance Sizing, Off-Site	5 minutes[1]
Peak Flow Conveyance Sizing On-Site Upstream of Stormwater Control Facility, TESC Design Flows	5 minutes[1]
Peak Flow Conveyance Sizing On-Site Downstream of Stormwater Control Facility, TESC Design Flows	15 minutes
Downstream Analysis, Off-Site	5 minutes[1]
Flow Control (detention and/or infiltration) Facility Sizing	5 minutes[1]
Water Quality Design Flow Rate	15 minutes
Water Quality Design Flow Volumes/Pollutant Loading	1 hour

[1]A 15-minute time step may be used if the time of concentration computed (see Section 6.4.6) is 30-minutes or more.

6.5.4.2 Steps for Hydrologic Design Using Continuous Simulation Models

This section presents the general process involved in conducting hydrologic analyses using continuous simulation models. The actual design process will vary considerably depending on the project scenario, the applicable requirements, the facility being designed, and the environmental conditions. However, the general process for all continuous modeling is as follows:

- Review all minimum requirements that apply to the proposed project (Chapter 2)
- Review applicable site assessment requirements (Chapters 2 and 3)
- Identify and delineate the overall drainage basin for each discharge point from the development site under existing conditions
    - Identify existing land use
    - Identify existing soil types using on-site evaluation, NRCS soil survey, or mapping performed by the University of Washington (http://geomapnw.ess.washington.edu/) as discussed previously
    - Convert SCS soil types to HSPF soil classifications (till, outwash, or wetland)

  - Identify existing drainage features such as streams, conveyance systems, detention facilities, ponding areas, depressions, etc.

- Select and delineate pertinent subbasins based on existing conditions
  - Select homogeneous subbasin areas
  - Select separate subbasin areas for on-site and off-site drainage
  - Select separate subbasin areas for major drainage features.
- Determine hydrologic parameters for each subbasin under existing conditions, if required
  - Determine appropriate rainfall time series. For most design applications, the City of Seattle Design Time Series will be required.
  - Categorize soil types and land cover according to Section 6.5.3
  - Determine total and effective impervious areas within each subbasin
  - Determine areas for each soil/cover type in each subbasin
  - Select the required computational time step according to Table 6.21.
- Compute runoff for the predeveloped condition. The continuous hydrologic model will utilize the selected precipitation time series, compute runoff from each subbasin, and route the runoff through the defined network. Flood-frequency and flow duration statistics will subsequently be computed at points of interest in the study area by the model.
- Determine hydrologic parameters for each subbasin under developed conditions
  - Utilize rainfall time series selected for existing conditions
  - Categorize soil types and land cover according to Section 6.5.3
  - Determine total and effective impervious areas within each subbasin
  - Determine areas for each soil/cover type in each subbasin
  - Utilize computational time step selected for existing conditions.
- Compute runoff for the developed condition. The continuous hydrologic model will utilize the selected precipitation time series, compute runoff from each subbasin, and route the runoff through the defined network. Flood-frequency and flow duration statistics will subsequently be computed at points of interest in the study area by the model.

Additional design steps specific to flow control and water quality facility design are described below.

6.5.4.3 Flow Control Facility Design

Peak Standard

Peak flow control-based standards require that the stormwater facilities be designed such that the post-development runoff peak discharge rate is controlled to one or more discharge rates, usually at specified recurrence intervals. An example of this type of standard is the Peak Flow Control Standard.

Flood-frequency analysis seeks to determine the flood flow or water surface elevation with a probability (p) of being equaled or exceeded in any given year. Return period (T_r) or recurrence interval is often used in lieu of probability to describe the frequency of exceedance of a flood of a given magnitude. Return period and annual exceedance probability are reciprocals (equation 35). Flood-frequency analysis is most commonly conducted for flood peak discharge and peak water surface elevation but can also be computed for maximum or minimum values for various durations. Flood-frequency analysis as used here refers to analysis of flood peak discharge or peak water surface elevation.

$$\mathrm{Tr} = \frac{1}{\mathrm{p}} \tag{35}$$

Where: T_r = average recurrence interval in years, and
p = the annual exceedance probability.

The annual exceedance probability of flow (or water surface elevation) may be estimated using the Gringorten (1963) plotting position formula (equation 36), which is a non-parametric approach.

$$Tr = \frac{N + 0.12}{i - 0.44} \tag{36}$$

Where: T_r = the recurrence interval of the peak flow or peak elevation in years,
i = the rank of the annual maxima peak flow, ranked from highest to lowest, and
N = the total number of years simulated.

A probability distribution, such as the Generalized Extreme Value or Log-Pearson III (Interagency Advisory Committee on Water Data 1981), is not recommended for estimating the frequency characteristics because these and other three-parameter distributions typically do a very poor job of fitting annual maxima flows regulated by stormwater ponds and infiltration facilities and can produce grossly inaccurate estimates of the flow for rare recurrence intervals.

Flood frequency analyses are used in continuous flow simulations to determine the effect of land use change and assess the effectiveness of flow control facilities. Flow control facilities are designed such that the post developed peak discharge rate is at or below a target predeveloped peak discharge rate at one or

more recurrence intervals. For example, Figure 6.9 shows predeveloped and post-developed flood frequency curves for a stormwater pond designed to control peak discharges at the 2-year and 10-year recurrence intervals. Currently approved continuous simulation hydrologic models perform the frequency calculations and present the results in graphical and tabular form.

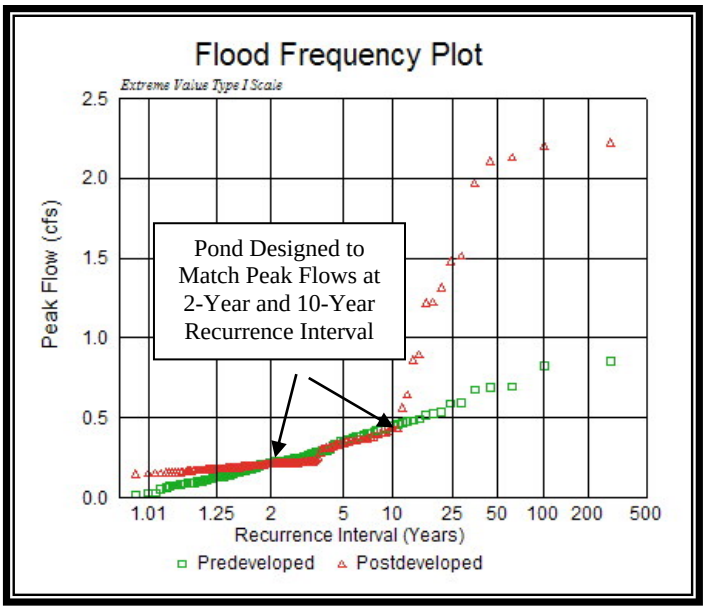

Figure 6.9. Example Flood-Frequency Curves for Stormwater Pond Designed to Control Post-Developed Peak Discharge Rates to Predeveloped Levels at the 2-year and 10-year Recurrence Interval.

Flow Duration Standard

Flow duration statistics provide a convenient tool for characterizing stormwwater runoff computed with a continuous hydrologic model. Example of this type of standard are the Pre-developed Forest Standard and the Pre-developed Pasture Standard. Duration statistics are computed by tracking the fraction of total simulation time that a specified flow rate is equaled or exceeded. Continuous runoff programs do this by dividing the range of flows simulated into discrete increments, and then tracking the fraction of time that each flow is equaled or exceeded. For example, Figure 6.10 shows a 1-year flow time series computed at hourly time steps from a 10-acre forested site and Figure 6.11 shows the flow duration curve computed from this time series.

The fraction of time is termed *exceedance probability* because it represents the probability that a particular flow rate will be equaled or exceeded. It should be noted that exceedance probability for duration statistics is different from the *annual exceedance probability* associated with flood frequency statistics and there is no practical way of converting/relating annual exceedance probability statistics to flow duration statistics.

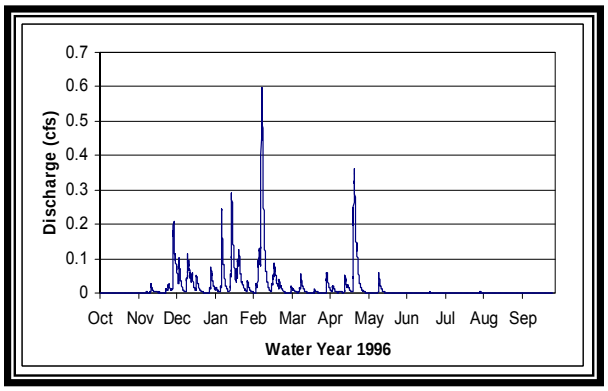

Figure 6.10. Runoff from 10-Acre Forested Site.

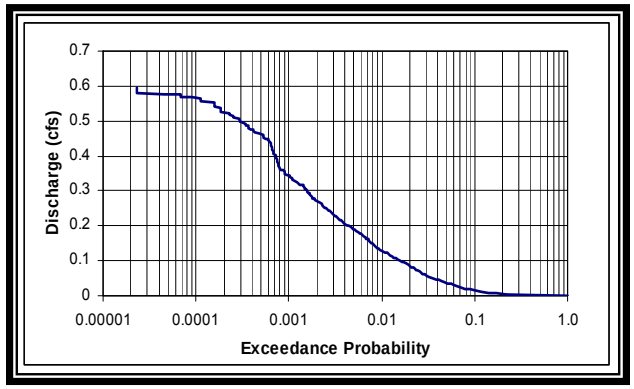

Figure 6.11. Flow Duration Curve Computed Using Time series in Figure 5.10.

The flow duration standard can be viewed graphically as shown in Figure 6.12. The flow duration curve for the site under predeveloped conditions is computed and is the target to which the postdeveloped flow duration curve is compared. The flow duration curve for the pond discharge must match the applicable predeveloped curve between 1/2 of the predeveloped 2-year (1/2 Q_2) and an upper limit, either the 2-year (Q_2) or the 50-year (Q_{50}) depending on the flow duration design standard for the facility.

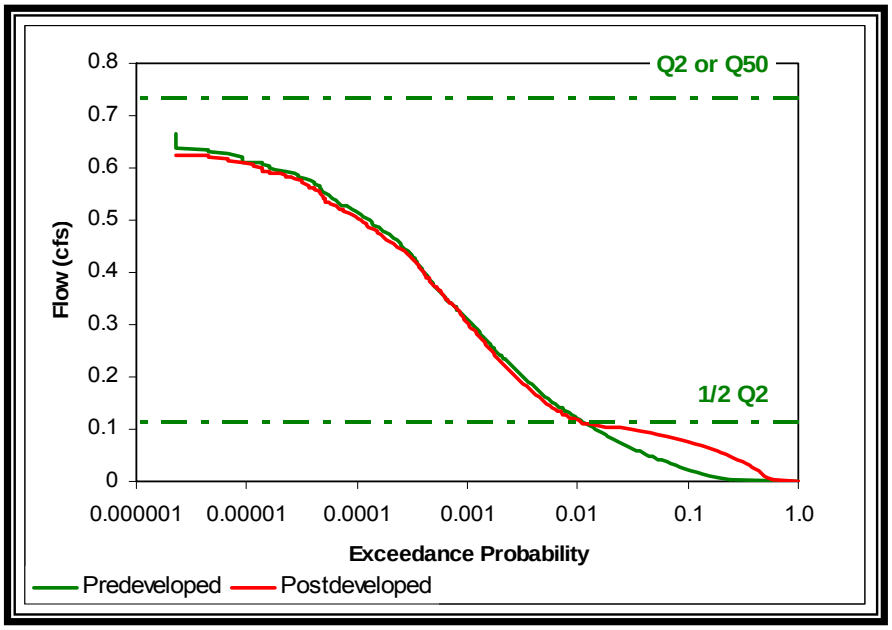

Figure 6.12. Comparison of Predeveloped and Postdeveloped Flow Duration Curves.

General guidance for adjusting the geometry and outlet works of stormwater ponds to meet the duration standard were developed by King County (1999) and are summarized in Figure 6.13 and described below. Refinements should be made in small increments with one refinement at a time. See Chapter 4 for complete facility design and sizing requirements.

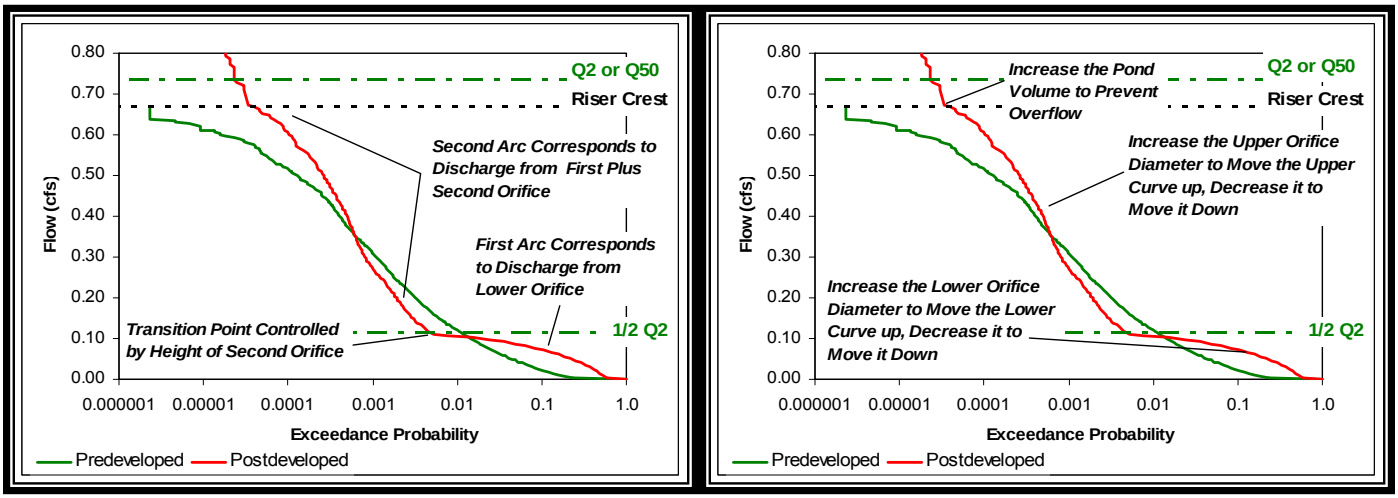

Figure 6.13. General Guidance for Adjusting Pond Performance.

- *Bottom Orifice Size* – Adjust the bottom orifice to control the lowest arc of the postdeveloped flow duration curve. Increase the orifice size to raise the arc, decrease it to lower the arc.
- *Height of Second Orifice* – The invert elevation of the second orifice affects the point on the flow duration curve where the transition (break in slope) occurs from the curve produced by the low-level orifice. Lower the invert elevation of the second orifice to move the transition point to the right on the lower arc. Raise the height of the second orifice to move the transition point to the left on the lower arc.
- *Second Orifice Size* – Adjust to control the arc of the curve for postdeveloped conditions. Increase the size to raise the arc, decrease it to lower the arc.
- *Pond Volume* – Adjust the pond volume to control the upper end of the duration curve. Increase the volume to prevent overflow, decrease the volume if the duration curve is substantially below the overflow level.
- Analyze the duration curve from bottom to top, and adjust orifices from bottom to top.
- The bottom arc corresponds with the discharge from the bottom orifice. Reducing the bottom orifice discharge lowers and shortens the bottom arc while increasing the bottom orifice raises and lengthens the bottom arc.
- Inflection points in the outflow duration curve occur when additional structures (orifices, notches, overflows) become active.
- Lowering the upper orifice moves the transition right on the lower arc and raising the upper orifice moves the breakpoint left of the lower arc.
- The upper arc represents the combined discharge of both orifices. Adjustments are made to the second orifice as described above for the bottom orifice.

- Increasing the facility volume moves the entire curve down and to the left. This is done to control riser overflow conditions. Decreasing facility volume moves the entire curve up and to the right. This is done to ensure that the outflow duration curve extends up to riser overflow.

6.5.4.4 *Water Quality Volume Treatment BMPs*

The water quality design volume for sizing wet ponds is computed as the daily runoff volume at or below which 91 percent of the total runoff volume for the simulation period occurs, as determined using an approved continuous model. The continuous model develops a daily runoff time series from the hourly pond inflow time series and scans the computed daily time series to determine the 24-hour volume that is greater than or equal to 91 percent of all daily values in the time series. This value is then used as the volume for a “Basic Wet Pond” and 1.5 times this value is used for sizing a “Large Wet Pond."

The water quality design volume is defined as the daily runoff volume at which 91 percent of the total runoff volume is produced by smaller daily volumes. The procedure can be visualized using Figure 6.14 below. The bars on the graph represent daily inflow volume for the entire simulation. The time span along the x-axis in Figure 6.14 is for 105 days, but in practice, this would include the entire simulated inflow time series (typically 60 to 158 years).

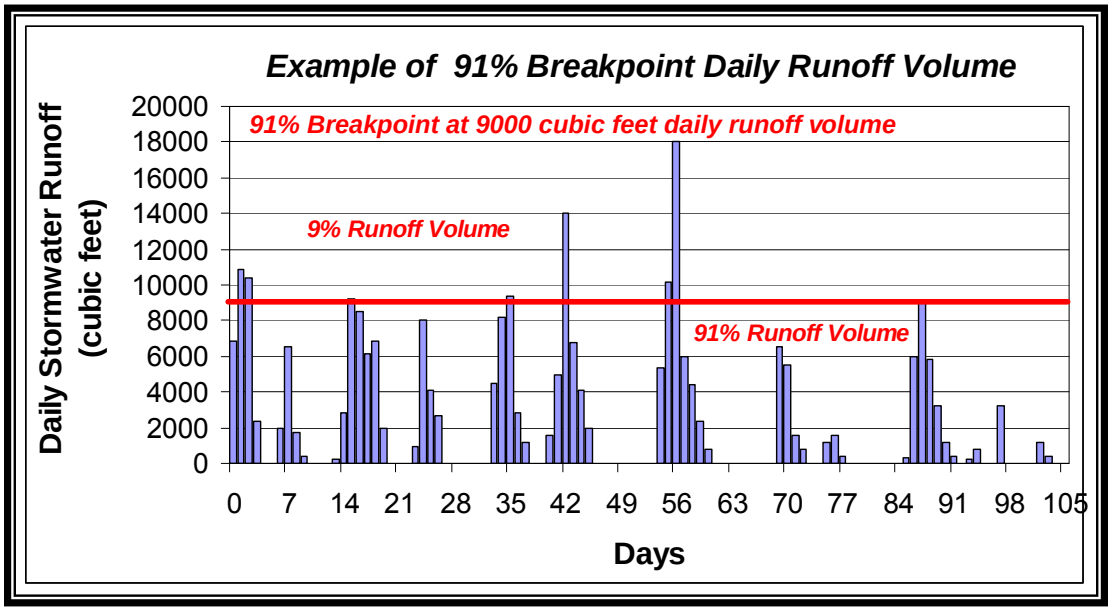

Figure 6.14. Example of Portion of Time-Series of Daily Runoff Volume and Depiction of Water Quality Design Volume.

The horizontal line represents the water quality design volume. Its value is calculated such that 91 percent of the total daily runoff volume for the entire simulation resides below this line and 9 percent of the total daily runoff volume for the entire simulation exceeds the water quality design volume. Stated

another way, if you total the daily runoff volumes that exceed the 9,000 cubic foot water quality design volume, they represent 9 percent of the total runoff volume.

The process for computing this volume may vary among continuous simulation models. An example of a typical algorithm (as used in MGSFlood) to compute the water quality design volume is summarized below:

1. Daily volume is computed using the inflow time series to the pond. The daily inflow volume to the pond is computed by converting the inflow rate to inflow volume on a midnight to midnight basis. The time step of the inflow time series may be 1 hour or less depending on the computational time step used in the model to compute the runoff.
2. The total inflow volume is then computed by summing all of the daily inflow volume values for the entire simulation.
3. A breakpoint value is computed by multiplying the total runoff volume computed in Step 2 by 9 percent.
4. The daily runoff values computed in Step 1 are sorted in descending order (highest to lowest).
5. The sorted daily volume values are summed until the total equals the 9 percent breakpoint. That is, the largest volume is added to the second largest, which is added to the third largest, etc. until the total equals the 9 percent breakpoint.
6. The last daily value added to match the 9 percent breakpoint is defined as the water quality design volume.

6.5.4.5 Water Quality Rate Treatment BMPs

The flow rate used to design flow rate dependent treatment facilities depends on whether or not the treatment is located upstream of a stormwater detention facility and whether it is an on-line or off-line facility (Figure 6.15).

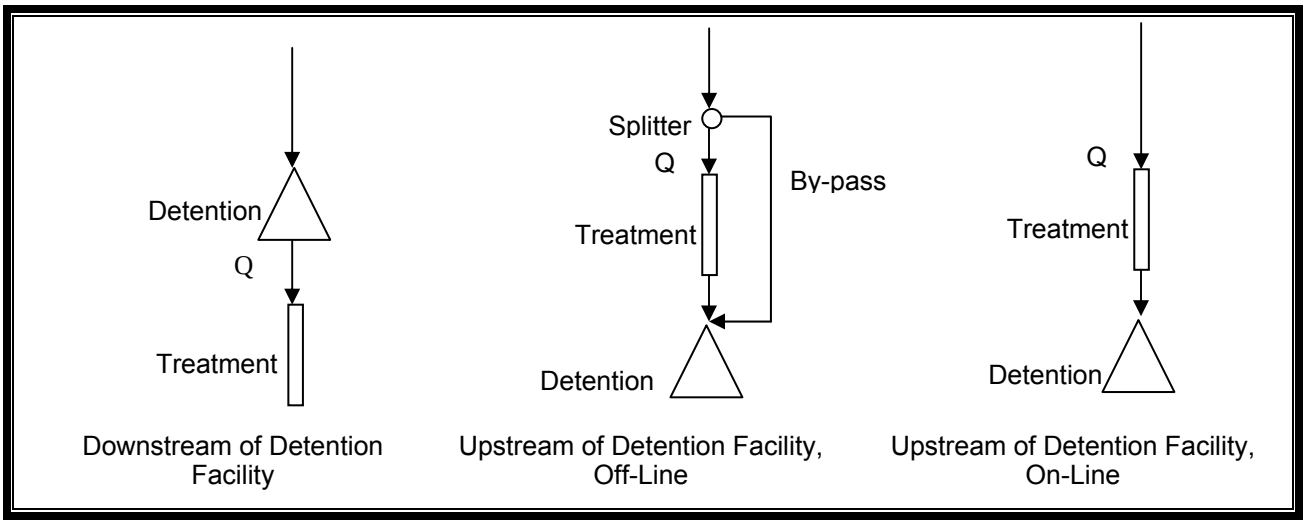

Figure 6.15. Water Quality Treatment and Detention Definition.

Downstream of Detention Facilities. If the treatment facility is located downstream of a stormwater detention facility, then the design flow rate is the release rate from the detention facility that has a 50 percent annual probability of occurring in any given year (2-year recurrence interval).

Upstream of Detention Facilities, Off-Line. Off-line water quality treatment located upstream of the detention facility includes a high-flow by-pass that routes the incremental flow in excess of the water quality design rate around the treatment facility. It is assumed that flows from the bypass enter the system downstream of the treatment facility but upstream of the detention facility. The continuous model determines the water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91 percent of the runoff volumes (Figure 6.16). If runoff is computed using the City of Seattle Design Time Series with a computational time step of 15 minutes or less, then no time step adjustment factors are need for the water quality design discharge.

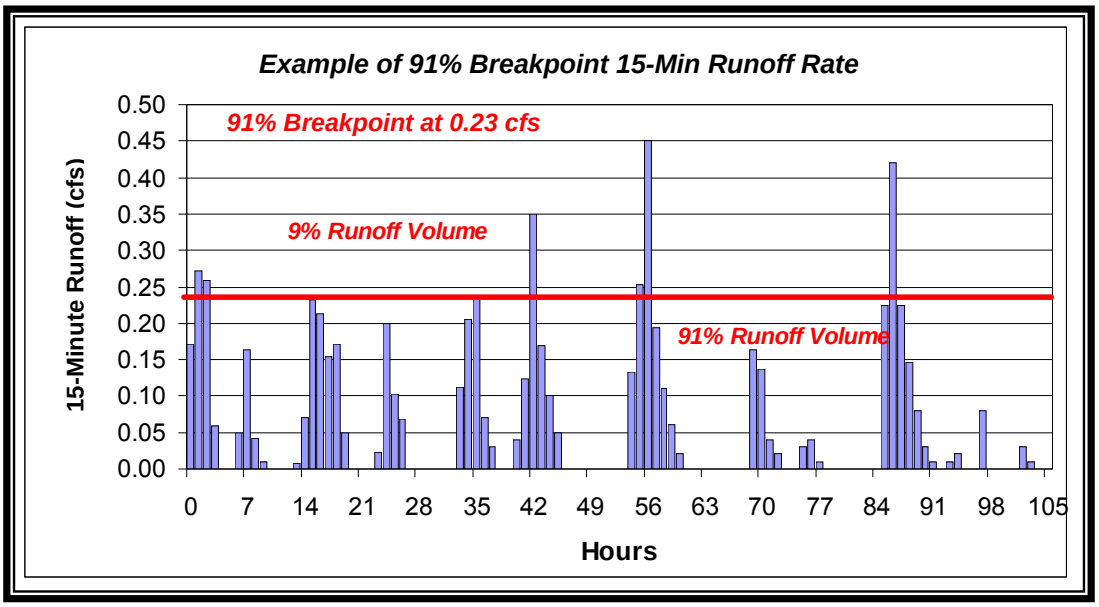

Figure 6.16. Example showing calculation of Off-Line Water Quality Treatment Discharge Off-line 15-minute Discharge of 0.23 cfs (in this case).

Upstream of Detention Facilities, On-Line. On-line water quality treatment does not include a high-flow bypass for flows in excess of the water quality design flow rate and all runoff is routed through the facility. The continuous model determines the water quality treatment design flow rate as the rate corresponding to the runoff volume that is greater than or equal to 91 percent of the 15-minute runoff volume entering the treatment facility. However, those flows that exceed the water quality design flow are not included in the calculation (Figure 6.17). Thus, the design flow rate for on-line facilities is higher than for off-line facilities. If runoff is computed using the City of Seattle Design Time Series with a

computational time step of 15 minutes or less, then no time step adjustment factors are need for the water quality design discharge.

Infiltration Facilities Providing Water quality Treatment: Infiltration facilities designed to for water quality treatment must infiltrate 91 percent of the total runoff volume as determined using an approved continuous runoff model. The procedure is the same as for designing infiltration for flow control (see Chapter 4, Section 4.5), except that the target is to infiltrate 91 percent of the runoff file without overflow. In addition, to prevent the onset of anaerobic conditions, an infiltration facility designed for water quality treatment purposes must be designed to drain the water quality design treatment flow within 48 hours.

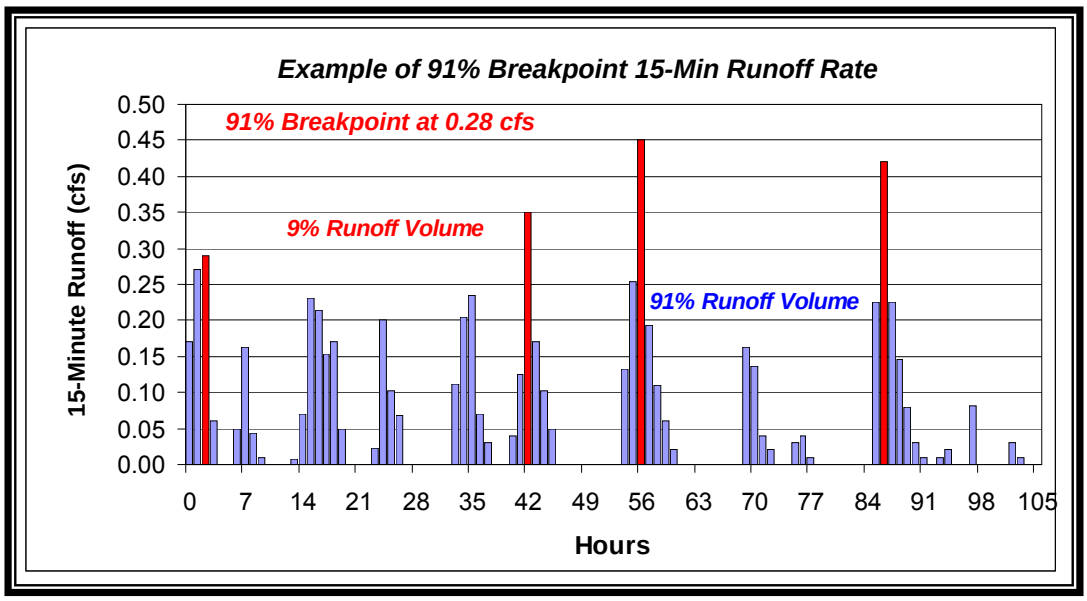

Figure 6.17. Example showing calculation of On-Line Water Quality Treatment Discharge On-line 15-minute Discharge of 0.28 cfs (in this case).

6.5.4.6 Water Quality Infiltration BMPs

Water quality infitlration BMPs are designed to infiltrate 91 percent of the total runoff volume for the simulation period through soil meeting the treatment soil requirements outlined in Section 5.8.4. In addition, an infiltration facility designed for water quality treatment must drain the water quality design volume (see Section 6.5.4.4) within 48 hours. Drain time can be calculated by using a horizontal projection of the infiltration basin mid-depth dimensions and the estimated long-term infiltration rate.

6.5.4.7 Stormwater Conveyance

Storms that produce the highest rates of runoff from developed areas are typically shorter in duration and are characterized by brief periods of high intensity rainfall. A 5-minute computational time step (see also Table 6.21) is required to adequately simulate the runoff peak discharge and hydrograph shape

resulting from these high intensity storms. A 15-minute time step may be used if the time of concentration computed is 30 minutes or more. Follow the modeling steps outlined in Section 6.5.4.2, and for conveyance-specific designs also perform the following:

- Identify downstream hydraulic controls, such as outfalls, known flooding locations, receiving pipe HGL, pump station, regulator station, weirs, or orifices. Determine if backwater calculations or a dynamic hydraulic routing model are required.
- Analyze flood frequencies and select the flows representing the level of conveyance service and/or flood protection required
- Utilize the peak flows to size or assess the capacity of pipe systems, culverts, channels, spillways and overflow structures
- For capacity analysis, verify that there is sufficient capacity in the public drainage system. For conveyance design, size the pipe to convey the selected peak flows. See SMC 22.805.020.J for full requirements, but in general, capacity analysis for discharges to the public drainage system shall be based on peak flows with a 4 percent annual probability (25-year recurrence interval); and capacity analysis for discharges to the public combined sewer shall be based on peak flows with a 20 percent annual probability (5-year recurrence interval). It is recommended that a factor of safety be applied to account for capacity losses due to changing downstream hydraulics and pipe or channel deformation or clogging
- Information on outfalls is discussed in Section 6.6.

6.5.4.8 Models *Using Continuous Simulation Hydrographs with Dynamic Routing*

Continuous hydrologic models based on the HSPF program utilize hydrologic (also known as lumped) routing routines to determine the time and magnitude of flow on a watercourse. Because of this, these models cannot simulate complex hydraulics such as where the flow reverses direction or where a downstream channel or pipe influences another upstream in a time dependent way.

For simulation of complex hydraulics in pipe systems or tidally influenced boundaries, a dynamic routing hydraulic program, such as the Storm Water Management Model (SWMM) Extran routine, may be necessary to accurately determine the discharge rate and the water surface elevation or hydraulic grade line. Flows simulated using the continuous hydrologic model may be exported and used as input to the dynamic routing hydraulic model.

Dynamic routing models solve the full unsteady flow equations using numeric approximation methods. These methods typically require computational time steps that are relatively short to maintain numerical stability, and it may not be practical to attempt routing of multi-year sequences of runoff produced by the

continuous hydrologic model. To reduce the simulation time, flow hydrographs from specific storms of interest computed using the continuous flow model may be used rather than the entire simulated flow time series.

To utilize a dynamic routing model to route hydrographs computed with the continuous flow model, the procedure described in Section 6.5.4.2 Steps for Designing Facilities Using Continuous Simulation Models should be followed to create the runoff time series. The following additional steps should be followed to identify storms of a particular recurrence interval, export them from the continuous model, and import them into SWMM (or other dynamic routing model):

- The watershed should be delineated with subbasin outlets (runoff collection points) corresponding to the main inflows to the pipe system to be analyzed with the dynamic routing model
- The routing effects of the pipe or other conveyance system to be analyzed system should not be included in the continuous hydrologic model
- Run the continuous hydrologic model for the full period of record. For most design applications, the City of Seattle Design Time Series should be used.
- Use flood peak discharge statistics computed by the continuous model to identify when floods of various recurrence intervals occur in the simulated time series. Hydrographs with peak discharge rates corresponding to desired recurrence intervals may then be exported in a format that can be read by the hydraulic model.

For example, Table 6.22 shows flood peak discharge-frequency results for a subbasin. If hydrographs corresponding to the 100-year, 25-year, and 10-year recurrence intervals were needed for conveyance design purposes, then simulated hydrographs with recurrence intervals closest to those required would be exported from the continuous hydrologic model as indicated in the right column of the table. The hydrograph duration would include a period antecedent to the flood peak (typically several days to a week) and several days following the flood peak.

Table 6.22. Example Simulated Peak Discharge Frequency Table and Hydrographs Exported to SWMM or other Hydraulic Model for Desired Recurrence Intervals.

Flood Peak Recurrence Interval (years)	Date of Peak*	Peak Discharge Rate (cfs)	Desired Recurrence Interval for Analysis
282	06/10/2010	7.62	
101	**11/04/1998**	**6.11**	**100-Year**
62	06/29/1952	6.06	
44	02/03/2062	5.38	
35	07/18/2043	4.71	
28	10/06/1981	4.64	
24	**03/03/1950**	**4.54**	**25-Year**
21	01/09/1990	4.40	
18	09/30/2011	4.40	
17	11/24/1990	4.27	
15	08/24/2077	4.25	
14	05/03/2002	4.25	
13	10/27/2054	4.15	
12	10/26/1986	4.03	
11	09/01/2061	3.93	
10	**01/20/2013**	**3.92**	**10-Year**
9.6	08/23/1968	3.92	
9.0	01/14/2040	3.76	

*Note: Simulation was performed using SPU Design Time Series, which is 158 years in length, and has dates spanning 10/1/1939 - 9/30/2097.

6.6 Outfalls

An outfall is defined as a point where collected and concentrated surface and stormwater runoff or combined sewage is discharged into an open drainage feature. These drainage features include streams, rivers, lakes, or Puget Sound.

6.6.1 Outfalls to Lakes and the Ship Canal

Single event hydraulic analysis of outfalls that discharge to lakes and the Ship Canal should be performed using high water from the observed record. This assumption may lead to conservative results and it is recommended that the designer consider using continuous simulation with a varying receiving water level. Table 6.23 shows the maximum observed water levels in Seattle lakes.

Table 6.23. Physical Characteristics of Seattle Lakes. [1]

	Bitter Lake	Haller Lake	Green Lake	Lake Union	Lake Washington
Water surface elevation (ft, NAVD88) [2]	434.4	376.9	164.3	17.0	17.0
Max depth (ft)	31.0	36.0	30.0	50.0	214.0
Mean depth (ft)	16.0	16.0	13.0	34.0	108.0
Area (ac)	19.0	14.9	255.3	580.0	21,500

[1] Sources: Washington State Dept of Game (1949) and King County (1995).
[2] Seattle Public Utilities Engineering Support Division - Survey Field Books, measurements were all converted to NAVD88 from the old City of Seattle Vertical Datum based on a conversion factor of 9.7 feet.
Note: Water Levels may vary from year to year by as much as 3 feet.

For continuous simulations, the designer may choose to use the historic record or the highest observed elevations. Lake Washington and associated waters are controlled at the Hiram M. Chittenden Locks by the U.S. Army Corps of Engineers. Refer to http://www.nwdwc.usace.army.mil/nws/hh/basins/lwscsh.html for Lake Washington Ship Canal data and note that elevations given are in USACE datum and should be converted to NAVD88 before use.

6.6.2 Tidal Influence/Sea Level Rise

Single event hydraulic analysis of outfalls that discharge to the tidally influenced Duwamish River or Puget Sound should be performed using the highest observed tide from the observed record. The peak rainfall intensity should be matched to a tide cycle simulation with a peak of 12.14 (NAVD88). This assumption may lead to conservative results and it is recommended that the designer consider using continuous simulation with a varying receiving water level.

For continuous simulations, the designer should match, by time, the historic tidal record to the historic rainfall record. For rainfall simulations where there is no observed tidal elevation, use of a tide predictor is recommended. Tidal information is available from National Oceanic and Atmospheric Administration (http://tidesandcurrents.noaa.gov/index.shtml) and from the U.S. Army Corps of Engineer's (http://www.nws.usace.army.mil/PublicMenu/Documents/Reg/applications/tides/tides.cfm). The tidal boundary is simulated as a water surface elevation time series computed using astronomical tide theory (NOAA 1995).

Sea level is rising, and for both continuous and single event modeling, the designer should evaluate the risks depending on the project design life and objectives. The observed trend from 1898 to 1999 was a rise of 2.11 mm per year (0.69 feet total). The effect of climate change on predicted sea level rise is expected to exceed that rate. A report by the University of Washington Climate Impacts Group (Mote et al. 2008) has provided low, medium, and high estimates of local sea level rise as shown in Figure 6.18.

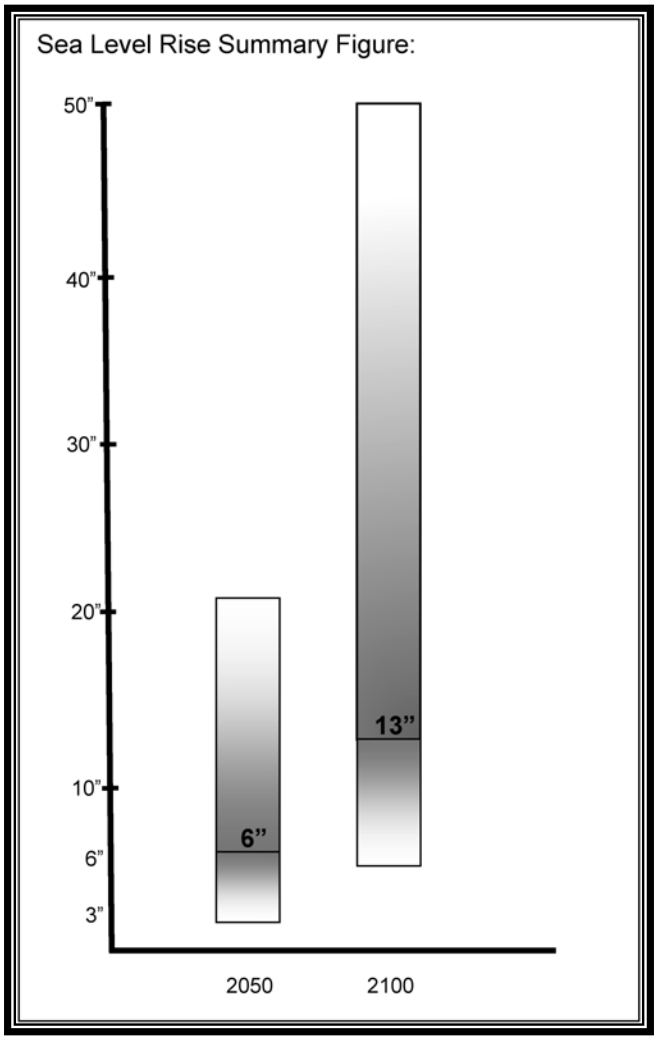

Figure 6.18. Projected Sea Level Rise in Washington's Waters Relative to 1980-99, in Inches (shading roughly indicates likelihood).

For Puget Sound, the medium estimate of sea level rise is 6 inches by 2050 and rising by 13 inches by 2100. The low-probability high-impact estimate is for a rise of 22 inches by 2050 and up to 50 inches by 2100.

Considerable uncertainty still exists over the timing and severity of sea level rise attributable to climate change. It is recommended that designers analyze risk by adjusting the tidal record upwards by 1 to 4 feet, depending on the design life and risk tolerance of the project. Likewise, designers should look to further mitigate risk by considering current design adjustments or identifying possible future modifications. For design of facilities where water level elevation at the outfall is critical, the City recommends that the designer consider storm surge due to low atmospheric pressure and/or wind and wave action.

6.7 Risk-Based Hydrologic Design Concepts

Risk-based concepts and analytical approaches are being used more frequently in hydrologic design. Application of a risk-based approach focuses on evaluating the two components of risk; the probability of failure and the consequences of failure. Failure may be defined in a broad context ranging from failure to meet a project goal, failure to meet a regulatory requirement, or the physical failure of a project element. Consequences of failure vary with the project type and project features and may include consequences that can be generally described as economic, life-safety, environmental, and political. Risk can then be described qualitatively or quantitatively.

Qualitative risk is often expressed, for example, as low, moderate, high, or very-high based on various combinations of the probability of failure and the consequences of failure. Quantitative risk assessment requires more detailed analysis to provide numerical measures of the probability of failure and consequences of failure. Units of measure for risk include loss-of-life per year for life-safety risk and dollars per year for all other consequences that can be expressed in economic terms.

Risk concepts are often used in design wherein the design target, level-of-service, etc. is based on the consequences of failure or upon some adopted level of qualitative or quantitative risk. The design targets and level of conservatism of design are typically set based on the tolerable level of risk for a given project type and/or consideration of the regulatory requirements.

The role of engineers and hydrologists in application of a risk-based approach is primarily in evaluating the probability of failure (probability of being in-compliance) and may include an assessment of how and which uncertainties affect the probability of failure (probability of being in-compliance). Application of hydrologic computer models and detailed numerical descriptions of hydrologic/hydraulic system components are an integral part of assessing the probability of being in-compliance.

Historically, uncertainty in hydrologic simulation analyses and the consequences of these uncertainties on results are rarely quantified as part of stormwater engineering design. Factors of safety have typically been applied at the end of a hydrologic analysis to account for uncertainties in the analysis. The same factor of safety is typically used regardless of the level of uncertainty or the confidence in the hydrologic model's ability to realistically simulate runoff. For many projects, the fixed safety factor approach is adequate. However, for projects where the consequences of failure (an erroneous design) are large, quantifying the analysis uncertainty and risk of not meeting the design standard may be beneficial in selecting an appropriate level of design conservatism.

An analysis approach called Generalized Likelihood Uncertainty Estimation (GLUE) by Beven and Binley (1992) addresses model input and parameter uncertainty, and provides a method for developing uncertainty bounds on model predictions. The basic premise of the GLUE approach is that "there is no reason

to expect that any one set of parameters will represent a true parameter set (within some particular model structure) to be found by some calibration procedures". The search for the "true" parameter set is obstructed by the errors and uncertainties that exist in the flood measurements, hydrometeorological inputs, and the model structure and algorithms.

Recognizing these limitations, their approach is to identify those combinations of parameter values (parameter sets) that can reasonably replicate the observed runoff hydrograph(s) and then assign a likelihood to each parameter set based on some objective measure of the goodness-of-fit between observed and simulated discharges. The model is then run numerous (hundreds or thousands) of times selecting parameter sets using the likelihood functions. Best-estimate results can then be displayed along with uncertainty bounds that represent the various uncertainties in the analysis.

The information provided by a GLUE analysis may be used to make a more informed selection of a single parameter set for use in project design/analysis in a manner consistent with traditional practice. However, it has its greatest advantage in developing a best estimate and uncertainty bounds for outputs from hydrologic/hydraulic models such as simulated discharges and volumes.

For example, Figure 6.19 shows a flood frequency curve for post-development conditions for a stormwater conveyance design project. If the design level of service were the 10-year recurrence interval, then the pipe would be designed to accommodate a discharge corresponding to the 10-year recurrence interval, or 4 cfs in this case.

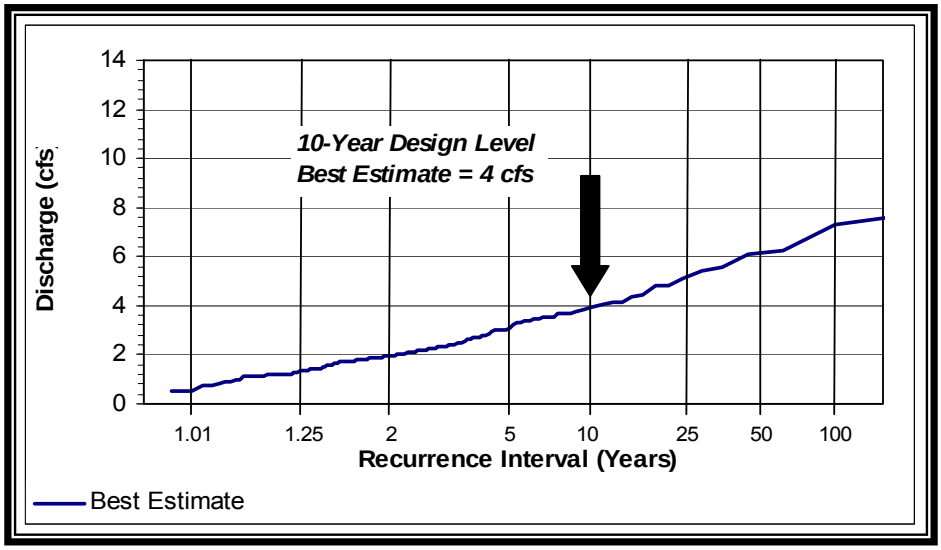

Figure 6.19. Best-Estimate Flood-Frequency Curve Developed Using Continuous Flow Model for Conveyance Design Project.

Figure 6.20 shows the same best-estimate flood frequency curve, but with 90 percent uncertainty bounds that reflect a (hypothetical) uncertainty in the model accuracy, precipitation/climate input, global warming, and other factors

that contribute to the overall uncertainty. Note that in this example the 10-year discharge rate ranges from 2 cfs to 6 cfs for the 90 percent uncertainty bounds.

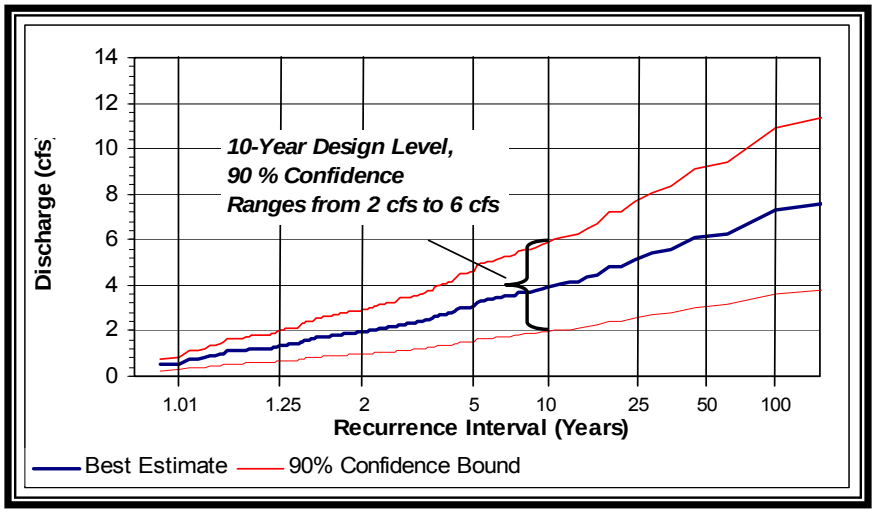

Figure 6.20. Best-Estimate Flood-Frequency Curve with 90 percent Confidence Bounds Developed Using Continuous Flow Model for Conveyance Design Project.

Using this information, the probability of being in compliance with the 10-year design discharge standard may be plotted against the required discharge capacity (Figure 6.21). Note that if the best estimate were used to design the conveyance facility without a safety factor, then there would be a 35 percent chance that the facility would be under designed. If a safety factor of 2 were applied, the design discharge would be 8 cfs, and the probability of not meeting the design target would be near zero.

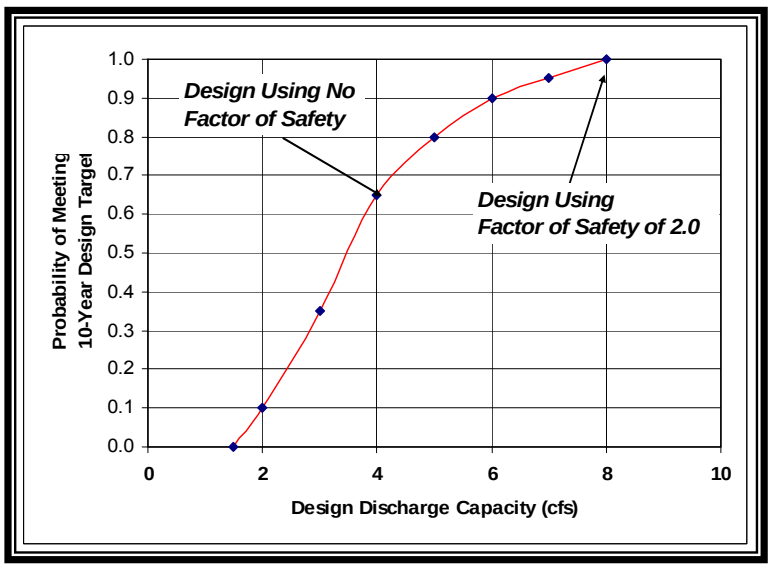

Figure 6.21. Performance Curve for Selecting Pipe Conveyance Discharge and Level of Reliability (a Similar Curve Could be Developed for Stormwater Detention Projects).

The facility size may also be expressed in economic terms by plotting the probability of compliance versus the cost associated with various sizes of a facility. This figure would provide specific information as to the level of protection offered by various levels of conservatism and the cost associated with the conservatism.

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